English Summary Edition of the Japan Society of Civil Engineers (JSCE) Standard Specifications for Concrete Structures

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Japan Society of Civil Engineers

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On the Publication of the Japan Society of Civil Engineers (JSCE) Standard Specifications for Concrete Structures (English Summary Edition)

Introduction

The Japan Society of Civil Engineers (JSCE) Standard Specifications for Concrete Structures are technical standards for concrete structures published by JSCE, the primary academic society in the field of civil engineering in Japan. In response to changes in social circumstances and technological advances in the field of concrete, revisions have been made to the specifications approximately every five years. The original text was intended primarily for use in Japan, and, thus, is authored in Japanese. However, as overseas construction projects also make reference to the standards at times, JSCE has decided to publish a summary English edition for the purpose of disseminating information internationally.

History of the JSCE Standard Specifications for Concrete Structures

The original JSCE Standard Specifications for Concrete Structures was based on the Standard Specifications for Reinforced Concrete Structures established by JSCE in 1931.

The standards underwent minor revision five years later in 1936. Further revisions were made in 1940, 1949, and 1956, under committee chairman Dr. Tokujiro Yoshida. Today, Dr. Yoshida's name is commemorated by ‘JSCE Yoshida Award, which is conferred on persons who have demonstrated outstanding achievements in the field of concrete engineering. The Standard Specifications for Concrete Structures at the time of the noted revisions consisted of "Standard Specifications for Plain Concrete," "Standard Specifications for Reinforced Concrete Structures," "Standard Specifications for Concrete Pavement," "Standard Specifications for Dam Concrete," and " JSCE Standards."

Revisions were again carried out in 1974, 1977, and 1980, under committee chairman Dr. Masatane Kokubu.

The design method was fully revised in 1986, based on the limit state design approach. This revision can be considered to have detailed the form leading to the current Standard Specifications. In addition, the "Design" and "Construction" volumes of the 1986 edition of the Standard Specifications were translated into English. Further revision was carried out in 1991, although without major changes.

In the 1996 revision, the "Design" volume was divided into a "Design" volume covering general structural design and a "Seismic Design" volume focused on seismic design. This change was motivated by the severe damage that occurred to structures during the Great Hanshin earthquake of January 1995.

In 1999, the "Construction and Durability Verification" volume was added. This work addresses aspects of the durability of structures, including verification of steel corrosion in concrete due to chloride ion penetration.

In 2001, the "Maintenance" volume was added to the scheme of the Standard Specifications, which until then had covered only the construction of new structures. This was added to address the growing importance of repair, strengthening, and other maintenance issues in existing structures.

Reaffirming that the purpose of design, construction work, and maintenance is to ensure the required performance of structures over their design service life, revisions were carried out in 2002 based on a performance verification-based approach aimed at the rational confirmation of required performance. This performance verification-based approach has been carried forward in the Standard Specifications. An English translation of the 2002 Standard Specifications was also published.

Revisions in 2007 preserved the characteristics of performance verification-based design that recognizes wide latitude in methods as long as assurance of required performance can be confirmed, and successfully married this approach with standard methods for achieving structures through current technologies. The "Design," "Construction," and "Maintenance" volumes all consist of "Main Text" that describes principles and rules that are not dependent on methods, and "Standards" that describe standard methods based on current technologies. Beginning with the 2007 Standard Specifications, "Design" also covers verification of initial cracking (e.g., thermal cracking caused by hydration of cement) and verification of durability (e.g., steel corrosion in concrete caused by chloride ion penetration). The 2007 Standard Specifications have also been translated into English.

Revisions in 2012 added a "Basic Principles" volume describing common matters and outlining the scheme of the Standard Specifications, in order to improve the clarity of the increasingly voluminous and complex specifications overall. While the structure of the overall scheme was not changed in the revisions, the content was upgraded.

The “Design” and “Construction” volumes were revised in March 2018, followed by the “Maintenance” volume in October 2018. This new English translation is based on those editions.

Features of the Standard Specifications for Concrete Structures

The JSCE Standard Specifications for Concrete Structures differ significantly from other technical standards in that, from their inception to the present, they have been set forth by a public academic society, not by administrative bodies, business operators, or private organizations. The Concrete Committee of JSCE, representing engineers and researchers from universities, companies, and business operators, has remained involved in the establishment and revision of the Standard Specifications. These specifications can be considered technical documents created through the cooperative work of people from diverse organizations, for the purpose of developing technologies of benefit to society. Accordingly, these Standard Specifications have not only seen application in the design, construction, and maintenance of concrete structures but have also served as guidelines for research and technological development, even influencing education and human resource development.

Another feature of the JSCE Standard Specifications for Concrete Structures is coverage of all concrete structural objects in fields of civil engineering, not only road, railway, and harbor structural objects. In Japan today, separate standards have been established for roads, railways, ports, and so on, but these standards make reference to the JSCE Standard Specifications for Concrete Structures for the design, construction, and maintenance of such structures.

Finally, the Standard Specifications differ from other domestic and foreign standards for concrete structural objects in their broad coverage of not only structural design but also construction and maintenance.

About the English Summary Edition

This marks the first English edition of the Standard Specifications in approximately 15 years, following the full translation of the 2007 Standard Specifications. The Standard Specifications have undergone revision every five years and have expanded considerably in content. The standards reached their current form out of necessity as domestically oriented technical standards; it is not necessary to include the full content of the standards within English text disseminated internationally. Moreover, while the current Standard Specifications are divided into separate publications, collection into a single book is preferable as a means of overviewing the standards. Therefore, a partial translation covering selected portions has been adopted as the format for this English translation.

The structure of the content is based on the following policy:

1. This work covers "Basic Principles" from 2012, "Design" and "Construction" from 2017, and "Maintenance" from 2018.
2. Portions to be translated into English have been selected on a case-by-case basis. In general, "Main Text" volumes have been translated in their entirety while "Standards" volumes have been translated selectively.
3. Portions to be translated have been selected on the basis of the suitability of their content for international information dissemination. Portions closely tied to circumstances peculiar to Japan have been omitted.

The Japanese edition of the Standard Specifications for Concrete Structures takes precedent over other editions. In the event of a discrepancy in content between the English edition and the Japanese edition, or for additional details not covered in the English edition, readers are asked to refer to the Japanese edition.

August 2023

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General Principles

Standard Specifications for Concrete Structures -2012

"General Principles"

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Chapter 1 General Rules

1.1 General

(1) The Standard Specifications for Concrete Structures are intended to present technical standards for the creation of concrete structures that possess the required performance to achieve the desired functions over the design service life of the structures.

(2) The Standard Specifications for Concrete Structures are composed of the parts "General Principles", "Design", "Construction", "Maintenance", "Dam Concrete", and "Criteria".

Commentary: Regarding (1) The Standard Specifications for Concrete Structures are a compilation of techniques related to the materials design of structural concrete and to the design, construction, and maintenance of concrete structures. Since the establishment of the Standard Specifications for Reinforced Concrete Structures in 1931, they have been subjected to minor revisions every five years and major reviews every ten years to reflect the latest concrete-related technologies in Japan and overseas, eventually reaching their current form. Amid ongoing technological developments related to the design, construction, and maintenance of concrete and concrete structures, the functions and performance demanded of structures have diversified and become more advanced as society has evolved. In response, the content of the Standard Specifications for Concrete Structures has also transitioned from a prescriptive approach to a performance verification approach based on the limit state design method.

Regarding (2) The "Design" and "Construction" parts established in 2012 and the "Dam Concrete" part established in 2013 both consist of "Main Text" and "Standards". "Main Text" covers methods of setting the required performance of structures and of the performance verification. "Standards" covers standard verification methods for ensuring required performance of structures under certain conditions and methods for satisfying required performance without conducting detailed verification, while taking into account the efficiency and convenience of design and construction. However, the applicability of the standard methods for given conditions must be carefully confirmed. When creating a new standard method involving structures, regions, *etc.* that differ from the standard conditions, t the content of the standards serves as a basis.

The "Maintenance" part established in 2013 consists of "Main Text" as well as "Standards", "Deterioration and its Mechanisms", and "Examples of Maintenance". In “Main Text”, the basic approach to maintenance is described in terms of concepts in the "General Principles" and "Design" parts. “Standards” presents basic and specific methods for the maintenance of structures based on the approach of the Main Text. "Deterioration and its Mechanisms” presents more detailed and specific maintenance methods for each deterioration mechanism that can be applied to cases in which deterioration mechanisms can be estimated from damage or deterioration appearing on the surfaces of structures. Maintenance procedures are introduced in "Examples of Maintenance" for use as reference.

The "Criteria" part contains Japanese Industrial Standards, Japan Society of Civil Engineers (JSCE) standards, and other standards related to the test methods described in "Design", "Construction", "Maintenance", and "Dam Concrete"

1.2 Scope of application

(1) The Standard Specifications for Concrete Structures apply to the design, construction, and maintenance of concrete structures.

(2) In the Standard Specifications for Concrete Structures, the "General Principles" part presents basic approaches related to it as a whole, its purpose and structure, the role of each part, etc.

(3) The "Design" part applies to the design of concrete structures using reinforced concrete, prestressed concrete, etc.

(4) The "Construction" part applies to the construction of concrete structures specified in design documents.

(5) The "Maintenance" part applies to maintenance performed to ensure the performance of concrete structures during their service life.

(6) The "Dam Concrete" part applies to concrete used in concrete dams.

(7) When special materials, structures, and construction and maintenance methods are adopted that are not sufficiently addressed in the Standard Specifications for Concrete Structures, guidelines, manuals, etc. published by JSCE may be consulted.

Commentary: Regarding (1) The Standard Specifications for Concrete Structures may be applied as practical technical standards to design, construction, and maintenance for the construction of concrete structures for civil infrastructure. Practical work involving concrete structures is generally performed based on technical standards issued by various organizations. Within such technical standards, the Standard Specifications for Concrete Structures are referred to as technical standards.

Regarding (2) The "General Principles" part presents the purpose of the Standard Specifications for Concrete Structures, along with the role, layout, and structure of each of its parts. **Section 2** outlines the purpose of the Standard Specifications for Concrete Structures, followed by their composition and scope of application. **Section 3** describes the roles of concrete structures, the performance demanded of concrete structures to fulfill the roles, basic matters related to the examination of performance, and the approaches for ensuring performance. **Section 4** presents general principles concerning the responsibilities and roles of engineers in ensuring the performance of concrete structures. **Section 5** describes the environmental performance of concrete structures in terms of the global environment, local environment, work environment, landscapes, and other matters that shall be considered in the short term and for the life cycles of concrete structures, with the aim of creating a sustainable society.

Regarding (3) "Design" presents standard methods of performance verification for concrete structures and specifies assumptions and structural details concerning verification. Although unreinforced concrete does not fall within the scope of application, the design values of required materials and other information may be applied *mutatis mutandis* to unreinforced concrete designs.

Note that the provisions on required performance stipulated in the "Design" part are certified as conforming to "ISO 19338: Performance and assessment requirements for design standards on structural concrete" from the International Organization for Standardization (ISO).

Regarding (4) The "Construction" part presents general principles regarding the construction of concrete structures. At the construction stage, construction methods are determined from design documents and from construction constraints, and placing performance is set for the concrete. The mix design of concrete is determined so as to meet conditions set at the design stage, including water content per unit of concrete, cement volume, and cement type. Then, a construction plan is drafted, covering a chain of construction procedures including concrete production, transport, placing, compacting, finishing, curing, and the assembly of reinforcing bars and formwork. Appropriate methods are used to confirm that the drafted construction plan satisfies construction requirements and required performance of the structure; if it does not satisfy these, then reworking of the construction methods, modification of the mix design, *etc.* are performed, within the scope of conditions set at the design stage. If a construction plan cannot be properly established even through such methods, then it is necessary to return to the design stage, rework aspects such as structural details and maintenance conditions, and conduct performance verification for the structure again. Construction is generally carried out according to this construction plan.

Regarding (5) Design and construction performed based on the verification system within the Standard Specifications for Concrete Structures are intended to ensure that the performance necessary for the structure to fulfill its intended functions over its design service life will not be impaired. However, design conditions are merely assumptions made at the time of design; some conditions, such as variable actions and environmental actions, cannot be assumed with certainty over the design service life. Moreover, at current levels of technology, it is difficult to verify through direct inspection that all specifications have been satisfied in a completed structure, meaning that an absolute guarantee of quality cannot be assumed. Accordingly, in order to ensure appropriate service in civil engineering structures, appropriate maintenance that includes diagnostics to verify the performance of structures is necessary.

The "Maintenance" part presents general principles regarding the maintenance of concrete structures. Work must be performed efficiently and rationally at the maintenance stage, making full use of documents including design specification documents handed over from the design stage, maintenance categories, construction plan documents handed over from the construction stage, as-built drawings, construction records, and inspection reports. When the use and functions of a structure change as a result of social changes, the performance of the structure must be verified against the newly derived required performance, and if said required performance cannot be ensured, the implementation of remedial measures such as repairs or retrofitting must be examined.

Regarding (6) The "Dam Concrete" part prescribes the performance and quality required for dam concrete and presents the performance verification methods and general principles of design, construction, and maintenance. Note that design and construction in "Dam Concrete" involve factors peculiar to dam concrete, such as the unreinforced concrete used in the structural body and the low or zero slump in the concrete used; on many points, the content differs from that presented in "Design" and "Construction". Accordingly, design and construction content specific to dam concrete has been collected in "Dam Concrete". Content that is shared with general concrete or that can be applied *mutatis mutandis* follows "Design" and "Construction" as appropriate.

In the discussion of maintenance, the text mentions three deterioration mechanisms specific to dam concrete (freezing and thawing, abrasion, and thermal shrinkage). It subsequently specifies conformance with the "Maintenance" part regarding methods and schedules for diagnosis of these mechanisms, or determination of methods and schedules taking into account the structure of the dam body, the installation environment, *etc.* The "Maintenance" part should be referred to in the event that a deterioration mechanism other than the above three becomes a problem.

Regarding (7) JSCE has established various design and construction guidelines for special materials and construction methods that are not sufficiently addressed in the Standard Specifications for Concrete Structures. These guidelines complement the Standard Specifications and JSCE will consider the establishment of further guidelines as needed. When parts of the Standard Specifications make reference to various standards published by other academic societies, these should be used to complement the content of the Standard Specifications after thorough confirmation of their preconditions and content.

In order to incorporate new research findings, technologies, and knowledge into the Standard Specifications when revisions are made, matters described in the standards are revised or abolished following judgment of the reliability, actual usage conditions, *etc.* of the technologies. Techniques involving design methods, structures, materials, and construction methods that were deleted in line with revisions are not rejected; when alternatives to the use of deleted techniques are not available, reference may be made to older editions of the Standard Specifications that describe the techniques. However, when using older editions of the Standard Specifications, responsible engineers are to use their judgment in adapting to newer stipulated matters, based on full understanding of technical progress and knowledge following the publication of the older Standard Specifications.

1.3 The roles of concrete structures

Concrete structures must be equipped with necessary functions to support daily life, to protect life and property from disaster, and to enable the sustainable development of society through land conservation.

1.4 Definitions of terms

Terms used in the Standard Specifications are defined as follows:

Civil engineer: An engineer who possesses basic knowledge concerning civil engineering structures.

Senior engineer: A civil engineer who possesses advanced knowledge and extensive practical experience concerning civil engineering structures.

Responsible engineer: An engineer who possesses the authority and bears the responsibility to execute the work that must be performed at all stages in the planning, design, construction, and maintenance of civil engineering structures. In principle, this engineer must also be a senior engineer.

Concrete engineer: An engineer who possesses basic knowledge concerning concrete structures and who is involved in the planning, design, construction, and maintenance of these.

Concrete specialist engineer: An engineer who possesses wide-ranging, advanced knowledge and extensive practical experience concerning concrete structures and who is able to make appropriate technical judgments in the planning, design, construction, and maintenance of these.

Required performance: The performance demanded of a structure according to its purpose and functions.

Verification: The act of determining whether a structure meets required performance, through confirmation experiments using full-scale specimens, analytical methods with empirical and theoretical evidence, *etc.*

Durability: The resistance of a structure to decline in performance over time due to deterioration of the materials in the structure.

Safety: The performance of a structure that prevents threats to the lives or property of the structure's users and surrounding people.

Serviceability: Performance that ensures that people can use a structure comfortably and that surrounding people do not experience discomfort due to the structure, and performance that properly ensures other functions required of the structure.

Restorability: Performance that enables the restoration of degraded performance in a structure and continued use of the structure.

Environmental performance: Compatibility with the global environment, regional environment, local environment, work environment, and landscape.

Constructability: The safety, certainty, and ease of construction work during the fabrication and erection of a structure.

Service life: The period during which a structure is in service.

Scheduled service life: The scheduled period of service of a structure.

Design service life: The period during which a structure or its structural members must fully perform the functions intended at the time of design.

Verification metrics: Required performance expressed as physical quantities that can be quantitatively evaluated.

Maintenance category: The level of maintenance set based on the basic approach to maintenance of the structure.

Maintenance thresholds: The performance, degree of deterioration, *etc.* set as thresholds for maintenance.

Commentary: Regarding senior engineers The abilities of senior engineers must be guaranteed through qualification by an official accreditation body. For example, JSCE certifies four levels of qualifications: Executive Professional Civil Engineer, Senior Professional Civil Engineer, Professional Civil Engineer, and Associate Professional Civil Engineer. An Executive Professional Civil Engineer and a Senior Professional Civil Engineer are considered senior engineers in the Standard Specifications. The Professional Engineer qualification certified by the national government is also an example of this.

Regarding responsible engineers Planning, design, construction, and maintenance demand not only the ability to execute work under the Standard Specifications but also appropriate technical judgment. For that reason, each project must be assigned responsible engineers who possess the required capabilities and experience and have been granted the required authority. The scope of authority and responsibilities must also be made clear. Note that responsible engineers are in principle senior engineers, although this need not be the case for simple work.

Regarding concrete specialist engineers Concrete specialist engineers are to possess the technical capabilities guaranteed by qualifications issued by official accreditation bodies, in accordance with the scale and degree of importance of the construction work, the content of the work, *etc.* An overview of the categories of engineers is shown in **Figure C1.4.1**. Concrete specialist engineers are not necessarily senior engineers; conversely, concrete engineers may be senior engineers.

Regarding scheduled service life and design service life Scheduled service life is a matter determined by the project plan, *etc.*, and is a given condition in structural plans. Design service life may differ from scheduled service life as a result of structural planning and design. Its determination may be changed at the design stage, such as by setting the design service life to 100 years with respect to a scheduled service life of 100 years, or by setting the design life of some structural members to 20 years and replacing the members at that interval.

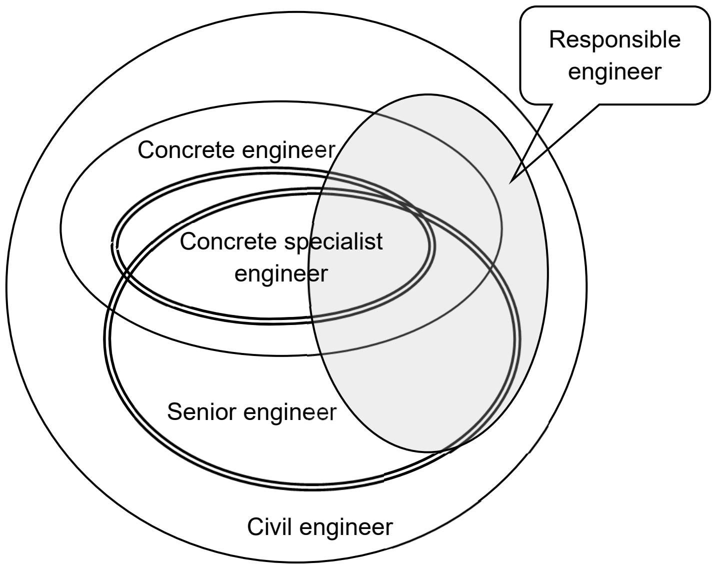


Figure C1.4.1 Overview of the categories of engineers.

Chapter 2 Structure of the Standard Specifications for Concrete Structures and the Relationships between its Parts

2.1 General

This Section presents the structure of the Standard Specifications for Concrete Structures, the relationships between the parts "Design", "Construction", and "Maintenance" and the role of each, and the workflow required to achieve reliable concrete structures.

Commentary: "Reliable concrete structures" refers to structures that fully exhibit a variety of functions in accordance with the structures' roles and their use as civil infrastructure over the required service life. To achieve such reliable concrete structures, it is important to understand the relevance of each part in the Standard Specifications for Concrete Structures, to fully comprehend the content of each part, and, having made the authority and loci of responsibility clear, to proceed with work through proper procedures at each of the stages of planning, design, construction, and maintenance of the concrete structures. **Figure C2.1.1** presents the relationships between the parts "Design", "Construction", and "Maintenance" in the Standard Specifications, and the scope addressed by the Sections in this part "General Principles".

To achieve reliable concrete structures, work is to proceed in such a way that ensures prescribed performance at each of the stages of design, construction, and maintenance, based on policies set forth in the plans for the structures. In order to ensure the performance of the structures, it is important to carry out tasks collaboratively while unfailingly communicating information at every stage. Principles for ensuring performance through this series of tasks are described in detail in **Section 3**.

In order to proceed with tasks without trouble or reworking at each stage, it is necessary to make the loci of responsibility of engineers clear at every stage, and to appropriately assign engineers who have been granted authority in line with their responsibilities. **Section 4** presents the specific roles of engineers at each of the stages of planning, design, construction, and maintenance of concrete structures.

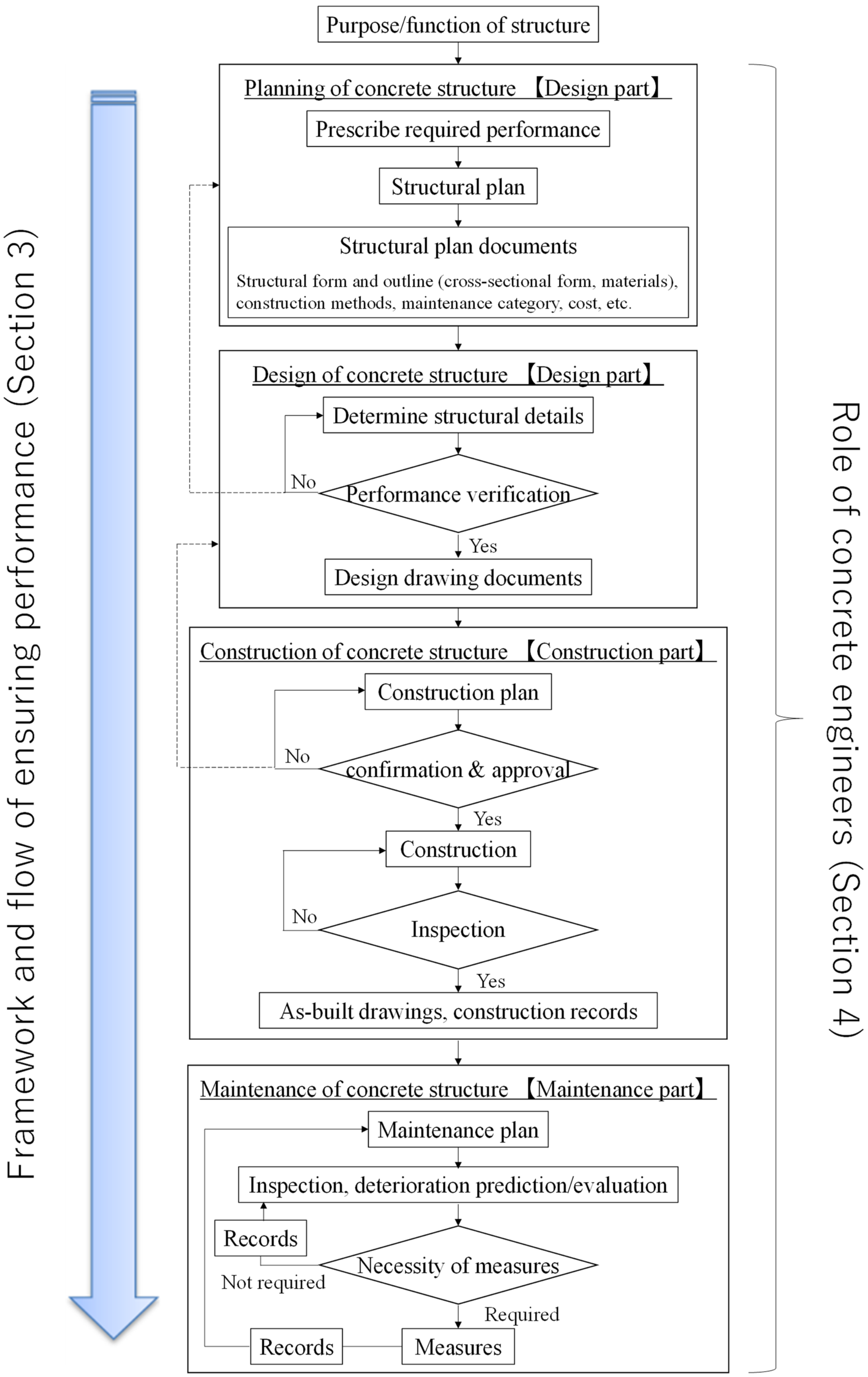


Figure C2.1.1 Relationships between the "Design", "Construction", and "Maintenance" parts in the Standard Specifications for Concrete Structures and the layout of each Section

2.2 Tasks and relationships at each stage

(1) At the planning stage of a concrete structure, the required performance must be in line with the purpose and function of the structure. An appropriate structural plan must be created that takes into consideration such factors as materials used, construction method, maintenance category, environmental performance, and economic performance.
(2) At each of the stages of design, construction, and maintenance, work must be performed in compliance with the structural plan, and matters that were decided at each stage must be reliably communicated to the next stage.
(3) At each of the stages of design, construction, and maintenance, appropriate technical decisions must be made to ensure the performance of concrete structures, and stages must be linked to each other as needed.
(4) For an existing structure, after an appropriate maintenance category has been set and a maintenance plan has been formulated, the assurance of required performance over the service life must be verified, and appropriate measures must be taken as needed.

Commentary: Regarding (1) As shown in **Figure C2.2.1**, in the construction of a structure, planning conditions are set based on survey results, laws and design standards, and plans including the National Spatial Strategies and urban plans. Structural types such as bridge structures, tunnel structures, and earthworks are determined by site planning. Once the structural type has been determined, the structural form is examined with a focus on factors such as differences in materials used. Specifically, the appropriate required performance is set based on the planning conditions, after which materials, primary dimensions, construction methods, basic policies for maintenance, environmental performance, economic performance, and other matters are comprehensively considered, and the structural form (steel structure, hybrid structure, concrete structure, *etc.*) and other matters are selected.

When a concrete structure has been selected as the structural form following the above examination, the required performance is first set at the planning stage of the structure in line with its purpose, functions, usage, and importance. Next, to satisfy the required performance, a structural plan is formulated with consideration of all factors including materials used, construction method, maintenance category, environmental performance, and economic performance, and the structural form, primary cross-sectional form, and other features of the structural outline are determined.

Appropriate structural planning is vital in achieving reliable concrete structures. As changing the structural form and the primary dimensions set in the structural plan is extremely difficult after beginning the stage of determining structural details, it is important in terms of design rationality to create a structural plan that avoids changes to the structural form and the primary dimensions as a result of verification of required performance. Similarly, as reliably performing construction so as to satisfy the conditions indicated in the design drawings, *etc.* is a prerequisite for ensuring the performance of concrete structures, structural planning must be carried out with due consideration of constraints on construction. For example, in the case of a bridge constructed over a steep valley in a mountainous area, the structural form may be determined from the construction method.

Moreover, as overlooking the progress of deterioration of structures during service may result in the need for remedial measures such as large-scale repairs, retrofit, or replacement and the incurring of enormous costs, performing appropriate maintenance is also vital. Here, the maintenance category varies greatly with the

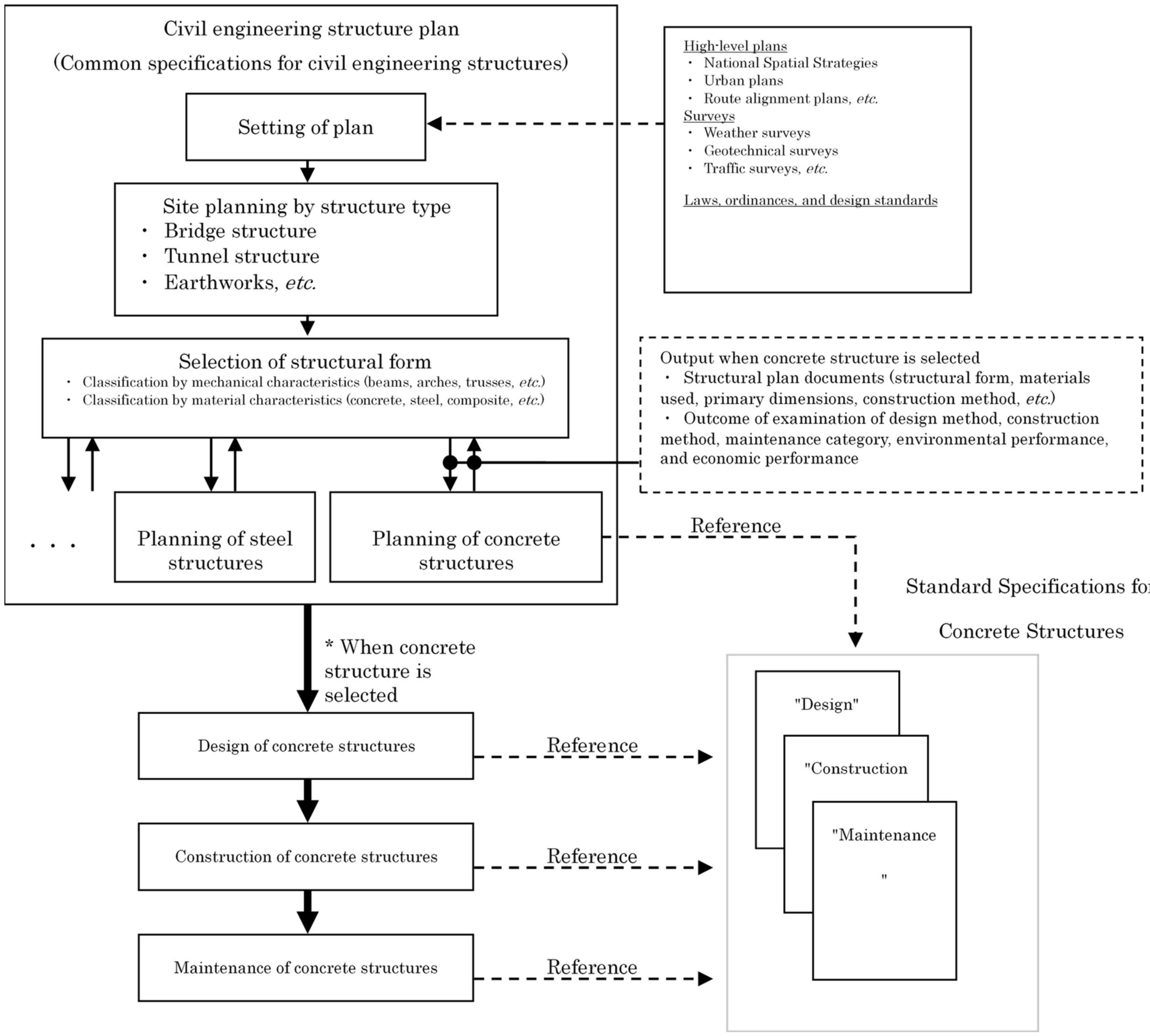


Figure C2.2.1 Layout of structural plans for concrete structures.

importance, required performance, scheduled service life, *etc.* of the structure; the difficulty of diagnosis and of countermeasures also differs by structure. Accordingly, in the structural plan, the maintenance category should be determined with various conditions taken into account. In terms of efficiently and effectively carrying out maintenance during the service of the structure, it is also important to examine factors such as the structural form and materials used and whether monitoring equipment will be used for inspection, and to indicate basic policies for these. In the structural plan, not only an outline of the costs required for construction but also costs required for future maintenance are roughly determined.

As shown above, structural planning for concrete structures is an extremely important process that affects actions and work at every subsequent stage. For that reason, formulation of the structural plan must be carried out by engineers who possess high-level expertise, comprehensive knowledge, and extensive experience concerning concrete structures, led by the ordering entity of the concrete construction work.

Regarding (2) During the long period between the planning of a concrete structure and the end of service, many organizations are involved in work on the structure. In addition to the proper performance of work at each of the stages of design, construction, and maintenance of structures, as indicated in **Figure C2.2.2**, ensuring the performance of concrete structures requires that necessary information is carried forward without fail at each stage. First, at the planning stage of the concrete structure, the information that should be carried forward to each successive stage should be made clear and specific, and at every stage of design and construction, information should be organized and communicated on the assumption that it will be used in the next stage. Each organization overseeing work must establish systems for communicating information in bulk. Here, it is particularly important to organize specific information in an easily understood manner to facilitate objective decisions regardless of the subjectivity of individual people in charge of each work even without consulting a person in charge.

Throughout the processes of construction, maintenance, and demolition of a structure, verifying information on design, construction, and maintenance from objective and scientific standpoints, and feeding back the knowledge thus obtained into newly built and existing structures, are extremely important for the improvement of concrete technologies and the achievement of highly reliable structures.

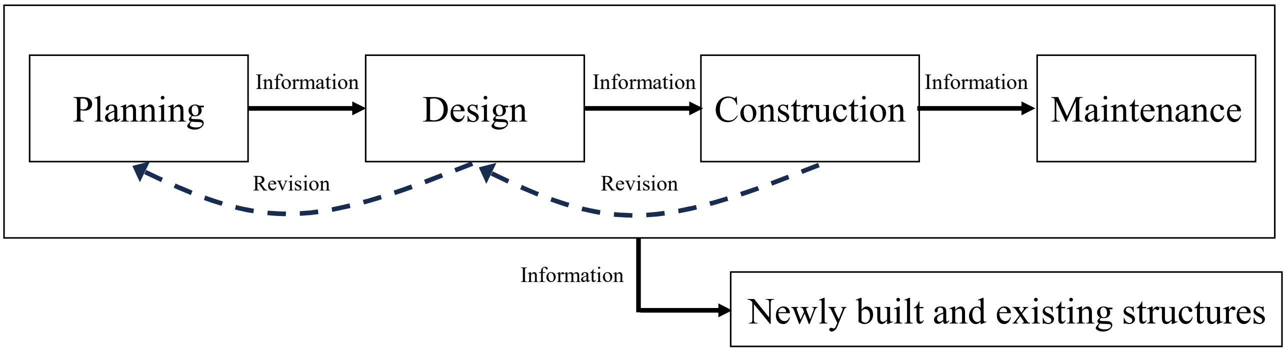


Figure C2.2.2 Flow of information between the stages of planning, design, construction, and maintenance of concrete structures.

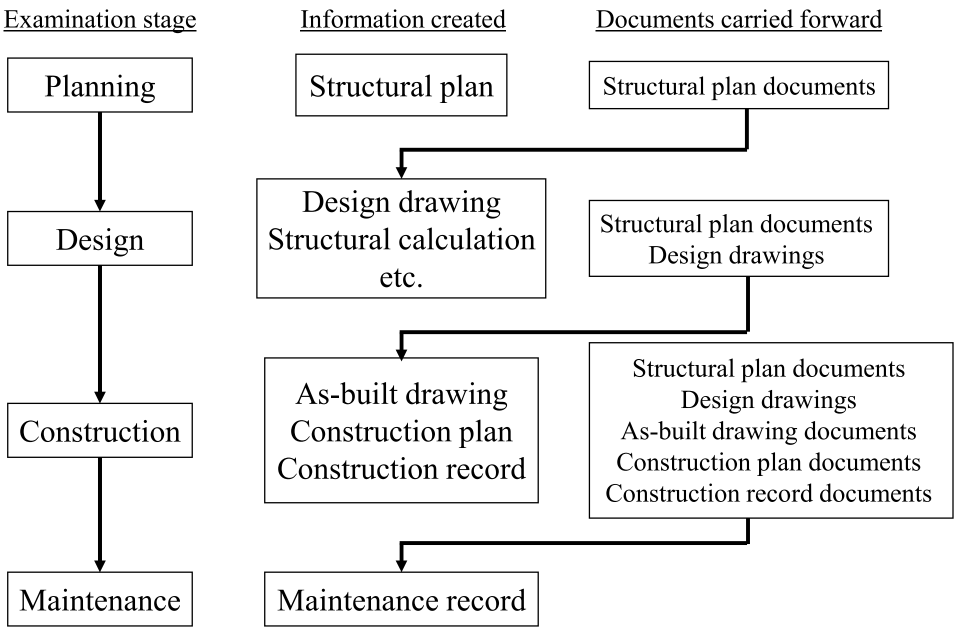


Figure C2.2.3 Communication of information at each stage.

An example of information to be carried forward from stage to stage is shown in **Figure C2.2.3**.

At the planning stage of a concrete structure, basic policies and methods for design, construction, and maintenance are defined and are collected into a structural plan document that is communicated to the design stage.

At the design stage, structural details such as reinforcing bar arrangement and cross-sectional forms/dimensions are set and durability, safety, serviceability, restorability, and other aspects of performance are verified. At this point, overall specifications regarding construction methods, maintenance methods, *etc.* are roughly determined. This information is included in design drawing documents, *etc.*, and is carried forward to the next stage without fail. Structural plan documents and design drawing documents are communicated from the design stage to the construction stage.

Construction records created at the construction stage constitute important information for implementing inspections, deterioration prediction, and countermeasures at the maintenance stage. It is important that accurate construction records are carried forward by the maintenance manager without fail. Construction plan documents and all manner of inspection reports must also be carried forward as needed. In this way, structural plan documents, design drawing documents, as-built drawings, construction plan documents, construction records, and other documents are communicated from the construction stage to the maintenance stage.

Taking verification of both concrete durability and initial cracking as an example, the importance of accurately understanding the relationships between the parts is shown in detail below through the example of the relationship between the "Design" part and the "Construction" part.

Regarding the durability of concrete, the carbonation rate coefficient, chloride ion diffusion coefficient, relative dynamic modulus of elasticity in freeze-thaw testing, shrinkage strain, and other characteristic values are set at the design stage, based on methods for setting and verifying structural details indicated in "Design". At the time of construction, the mix design of concrete is determined in line with "Construction" so as to satisfy the characteristic values contained in the design drawings. In the design drawing documents, the use of a previously used mix design is noted; the type of cement, maximum size of coarse aggregate, cement content per unit volume of concrete, slump or slump flow, water-to-cement ratio, and other information are also noted as reference values, with consideration given to the convenience of mix design at the time of construction. The design drawing documents that describe the characteristic values of these materials and the reference values for mix design are the sole means by which the intent of the designer is communicated to persons in charge of construction and to maintenance managers.

Regarding initial cracking, verification is performed for cracks caused by hydration of cement and cracks associated with shrinkage, following the "Design" part. Based on the outcome of this verification, characteristic values and reference values for materials used and the mix design are included in the design drawing documents. Accordingly, as is essentially the case with regard to durability, construction can be carried out in line with the "Construction" part at the construction stage, based on the reference values carried forward from the design stages. However, the occurrence of cracking may be related to factors including actual environmental conditions and construction methods, in addition to the performance of the concrete itself (which is determined by materials, mix design, *etc.*). Accordingly, "Construction" clarifies the importance of reviewing the characteristic values and reference values indicated in the design at the construction stage, and of enacting appropriate countermeasures as needed.

Regarding (3) Even if the Standard Specifications for Concrete Structures are observed and information is communicated reliably and appropriately at every stage, cases may occur in which the performance of the concrete structure cannot be fully ensured. Depending on actual work conditions and construction conditions, unexpected problems may occur, requiring appropriate and high-level technical judgment in each case. To deal with such conditions, concrete specialist engineers who possess the required technical capabilities are to be assigned in accordance with the difficulty of the work, and are to make appropriate technical judgments concerning problems. The performance of concrete structures can be ensured when three factors are all present: compliance with the Standard Specifications, reliable communication of information, and apt conditional judgments by concrete specialist engineers. Note that the concrete specialist engineers who bear responsibility for technical judgments must be granted authority in line with their degree of responsibility for the work.

Depending on factors such as the environmental conditions in which the structure is placed, information carried forward from the previous stage may not always be rational. Particularly at the construction stage, problems that cannot be solved solely by engineers in charge of construction may occur. In such cases, it may be possible to solve problems more rationally by holding discussions at the design stage with the involvement of concrete specialist engineers, and reexamining the design or reviewing the maintenance category as needed. In this way, to construct structures that satisfy the required performance and that exhibit functionality according to the use and purpose of the structure over its design service life, it is important that the engineers who perform work at each stage understand the conditions at all stages and uncover the best technical solutions that can be accommodated at each point in time, regardless of individual expertise or positions. Moreover, it is the responsibility of the ordering entity to establish opportunities and structures that enable engineers at every stage to share the same information and discuss solutions.

The roles of engineers at every stage are described in detail in **Section 4** for use as a reference.

Regarding (4) It is important that appropriate maintenance is carried out to ensure the performance of existing structures. Toward that end, maintenance plans must be formulated based on the maintenance category set out in the structural plan. When drafting a maintenance plan for an existing structure for which a maintenance category has not been set, a maintenance category is first set in line with the importance, the scheduled service life, conditions, and retained performance of the structure, after which maintenance thresholds for managing the structure are set based on that category.

In diagnosis performed based on the maintenance plan, inspection is first performed, after which data is acquired to evaluate the conditions and the retained performance of the structure. Next, based on the inspection results, deterioration mechanisms are estimated and the degree of deterioration is evaluated in order to determine whether the existing performance of the structure and the predicted future performance of the remaining service life satisfy the required performance. As a result of this evaluation and judgment, remedial measures may be required to ensure performance, such as repair and retrofit. Details of the inspection, estimation of deterioration mechanisms, prediction, evaluation, judgment, and countermeasures are described in the "Maintenance" part.

Even if deterioration or damage is not observed in the structure and deterioration in performance is almost non-existent, it is still possible to envision cases in which the performance of the structure does not satisfy the required performance, following a review of values of actions (loads), verification methods, threshold values for verification, *etc.* and following changes to the use of the structure during the service life, changes to demanded functions, or revision or changes to the Standard Specifications for Concrete Structures or other design standards. Measures for the enhancement of the performance of existing structures, which are necessary in such cases, are indicated in the "Maintenance" part.

Chapter 3 Ensuring the Performance of Concrete Structures

3.1 General

(1) In order for a concrete structure to satisfy the functions demanded of it, the planning, design, construction, and maintenance must be performed so as to satisfy all aspects of required performance.
(2) At the planning stage of concrete structures, conditions are comprehensively considered and a structural plan is formulated so that work is performed appropriately at each of the stages of design, construction, and maintenance.
(3) At the design stage, appropriate design work is performed based on the information carried forward from the structural plan, and design verification is performed by multiple organizations.
(4) At the construction stage, a construction plan is drafted based on the structural plan and on information carried forward from the design. Reliable construction is carried out according to the construction plan, and inspections are performed at appropriate times.
(5) At the maintenance stage, a maintenance plan is formulated based on information carried forward from the structural plan, design, and construction. Diagnosis is performed in line with the maintenance plan, and remedial measures such as repairs and retrofits are performed as needed.

Commentary: Regarding (1) **Figure C3.1.1** indicates basic concepts for ensuring the performance of structures. Every concrete structure has its own purpose and has a role that is expected from it in society. Functions are demanded of the structure so as to fulfill that purpose and role. The processes of building and maintaining a structure that possesses these functions equate to planning, design, construction, and maintenance.

At the planning and design stages of a concrete structure, specific performance items required of the structure are selected and their levels are set. For example, if the role of safely carrying vehicles and pedestrians on a high-traffic road is demanded of a bridge, then the specific required performance demanded of the structure, such as strength, deflection limit, seismic resistance, wind resistance, vibration resistance, and visibility, must be set on the basis of laws and standards.

The work of setting the required performance and giving form to a structure that satisfies said performance in drawings equates to design, while the work of confirming that required performance is satisfied by the design equates to verification. Because some aspects of required performance may not be necessary depending on conditions, and because it may be possible to omit verification depending on materials and structural form, it is also important to identify the minimum required aspects of performance.

Construction consists of work performed to realize the structure envisioned in the design, essentially following the principle of constructing the structure envisioned in the design documents. Maintenance consists of work performed so that the finished structure retains required performance during its service life. Construction and maintenance, like design, constitute important work for ensuring the performance of a structure.

Regarding (2) As the planning stage of a concrete structure consists of work performed at the first stage in building the structure, it greatly impacts all subsequent

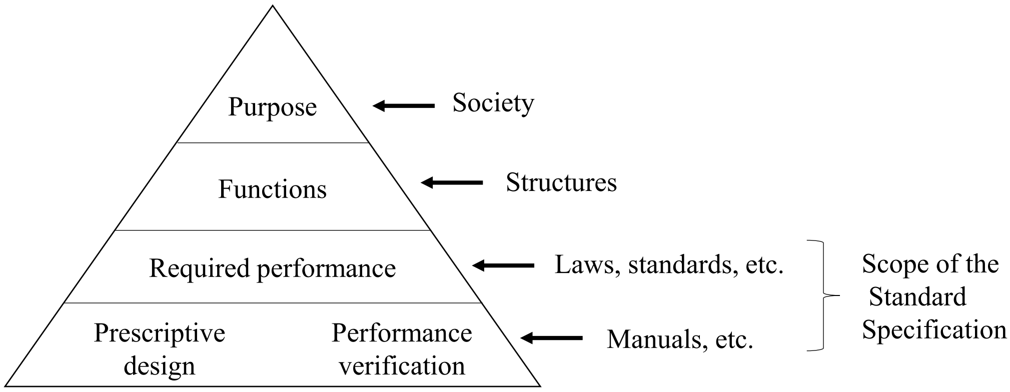


Figure C3.1.1 Basic concepts for ensuring performance in concrete structures.

stages of design, construction, and maintenance. Specifically, the construction method and maintenance method of the concrete structure are comprehensively examined with consideration of environmental performance and economic performance, and the layout of the structural type, the structural form, and other matters are selected. Accordingly, the structural plan for a concrete structure must be drafted by a concrete specialist engineer who possesses sufficient knowledge and experience concerning not only design but also construction methods and maintenance methods. Furthermore, as already indicated in **2.2**, it is important that conditions used at the time of planning the structure are communicated reliably to each stage and are reflected in work at each stage.

In addition, to draft a rational structural plan, it is necessary to obtain information on the environment around the planned construction site and to carry out field surveys as appropriate to the importance, scale, *etc.* of the concrete structure.

Regarding (3) Design work is performed primarily by a concrete specialist engineer working under a responsible engineer, based on policies and items formulated in structural plan documents. Design verification must be carried out with respect to the specifications of the designed structure. Here, "design verification" refers to the act of confirming the design conditions, the content and quality of design results, *etc.* In principle, design verification is performed twice. For example, it may be performed by design work checkers or by the ordering entity, in addition to by the design work contractor. (For specifics, see the roles of engineers at the design stage as indicated in **4.2**, and see **Figure C4.2.1**). The adoption of such methods guarantees the certainty of the design work. The design drawings used in the construction contract should be signed by specialist engineers and concrete specialist engineers of the two organizations responsible for the design and for confirmation work.

At the design stage, the characteristic values used in the design and the reference values that are affected by construction conditions must be clearly indicated in the design documents and must be carried forward to the construction stage. For example, load characteristic values and material strength characteristic values are set with reasonable variability taken into consideration. Conversely, reference values are values that change according to construction conditions, environmental conditions, and other conditions, such as the conditions for the examination of cracks caused by cement hydration.

In addition, information necessary for the performance of maintenance must be described in the design documents. Using measures to combat chloride attack as an example, corresponding information is information that affects subsequent maintenance plans, such as increased cover depth, the use of epoxy resin-coated reinforcing bars, the use of high-durability permanent formwork, and other matters selected at the design stage.

Regarding (4) At the construction stage, a construction plan is created that reflects the characteristic values, the construction conditions, *etc.* described in design documents. Reference values used for the examination of cracks, *etc.* at the design stage are reexamined with respect to construction conditions, and the results are reflected in the construction plan.

Construction work is performed based on the construction plan formulated under responsible engineers and concrete specialist engineers, with quality ensured through quality control performed by the contractor and inspections performed by the ordering entity. In order to confirm that the completed structure has been achieved through construction in accordance with design specifications, items that essentially can be inspected are inspected in completed form to the degree possible. Matters that cannot be confirmed in completed form are inspected during construction (*i.e.*, are subjected to process inspection).

Generally, the adoption of a highly reliable method enables a reduction in the effort required for quality control and inspection. Otherwise, the contractor must raise the level of quality control or the ordering entity must increase the frequency of inspection, or else it is necessary to use advanced inspection techniques to confirm that the specifications of the designed structure have been reliably achieved. As inspection items and their acceptance/rejection criteria affect the quality and construction costs of the structure, it is important to determine these in detail through consultation between the ordering entity and the contractor at the time of contracting.

Information concerning construction conditions must be collected and described in construction records and as-built drawing documents to enable the efficient performance of maintenance.

Regarding (5) Structural plans, design drawing documents, as-built drawings, construction plans, and construction records are communicated as information to the maintenance stage. Based on these, a detailed maintenance plan is formulated under responsible engineers and concrete specialist engineers. Note that initial defects may be detected in the concrete structure or deterioration over time may become apparent, owing to the occurrence of events not envisioned in the design. In such cases, the maintenance category set out in the structure plan is reviewed, a comprehensive judgment is made based on design documents, construction records, as-built documents, and other information, and a new maintenance plan is drafted on the basis of the new maintenance thresholds that are set.

At the maintenance stage, regular inspections are important to ensure the performance of the structure and guarantee its safety. If anomalies are found by inspections, then the causes and deterioration mechanisms are estimated. Following this, the performance of the structure is evaluated, and, if the determination is made that required performance is not satisfied, measures are taken as needed.

3.2 Required performance

3.2.1 Setting of required performance

For concrete structures, all aspects of performance required for conformance with the intended use of the structure during construction and within the design service life must be set. In general, required performance should be set in terms of durability, safety, serviceability, restorability, environmental performance, *etc.*

Commentary: The specific aspects of the required performance that the structure should satisfy during construction and during its design service life are generally set at the time of design, taking landscape, constructability, ease of maintenance, and economic performance into consideration at the structural planning stage. Note that, in order for the new structure to satisfy safety, serviceability, and restorability throughout its design service life, it is necessary to prevent material deterioration and anomalies due to environmental actions that would present obstacles to these aspects of performance during service.

"Durability" refers to the resistance of a structure to decline in performance over time, caused by deterioration of materials under assumed actions.

"Safety" refers to performance that prevents a structure from posing threats to the lives and property of users and surrounding persons under all assumed actions.

"Serviceability" refers to performance by which users of a structure and surrounding persons use the structure comfortably under expected actions, as well as the water-tightness, water permeability, soundproofing, moisture resistance, and other performance required of the structure.

"Restorability" refers to performance that enables continuous use of the structure during restoration following deterioration due to unforeseen actions from earthquakes, *etc.* Restorability is set with consideration of reparability, *i.e.*, the difficulty of repairing damage to the structure, and all factors that affect reparability.

"Environmental performance" refers to performance that indicates compatibility with the global environment, local environment, work environment, and landscape, with details as presented in **Section 5**.

3.2.2 Design service life

When designing a concrete structure, the design service life of the structure and of its structural components/members must be stipulated, while taking into account the importance, required performance, scheduled service life, environmental conditions, maintenance method, economic performance, *etc.* of the structure.

Commentary: In general, the design service life of a structure and its components/members is set longer than the scheduled service life, and maintenance must be properly carried out throughout the scheduled service life. Normally, when a concrete structure has been designed and constructed in accordance with appropriate standards such as the "Design" and "Construction" parts of the Standard Specifications for Concrete Structures, the emergence of deterioration during the design service life, with accompanying major impacts on the required performance of the structure, is unlikely. In such cases, routine inspection or periodic inspection, primarily via visual inspection, is sufficient for maintenance during the design service life.

Conversely, for structures built in environments of harsh deterioration, setting a design service life for all structural members that is longer than the scheduled service life will result in the need for excessive corrosion protection, which may not be economical. In such cases, the maintenance plan may be set on the assumption that specific members will be replaced, repaired, or reinforced during the scheduled service life of the structure. Even in this case, simple inspection at the visual level can be considered sufficient for routine inspections, *etc.* in maintenance during the design service life, but if service is to be continued beyond the design service life, it is necessary to perform a detailed investigation and evaluate the performance.

Maintenance plans set at the design stage may undergo revision for reasons including measured values of deterioration at the maintenance stage during service that differ from the predicted values at the design stage. In this case, it may be necessary to review the service life of the structure or structural parts/members based on diagnosis results. Performance case (1) in **Figure C3.2.1** is a normal case in which the design service life is not reworked. Performance case (2) is a case in which a repair or retrofit plan to secure the scheduled service life is set, and service life is reworked.

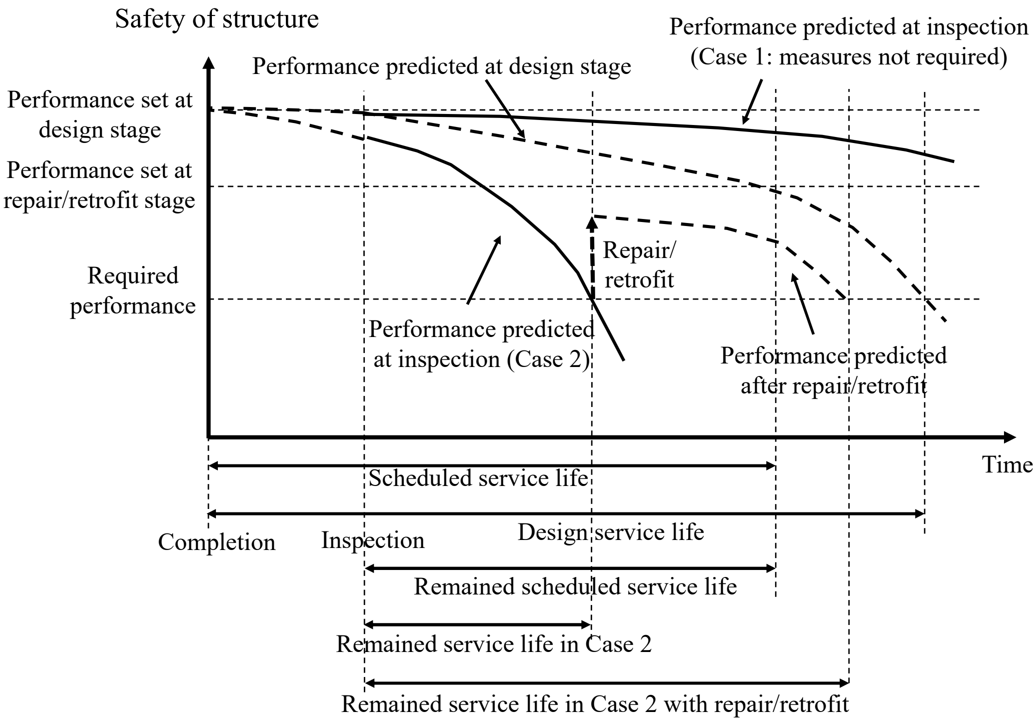


Figure C3.2.1 Conceptual diagram for reworking service life at the maintenance stage.

3.2.3 Basics of performance verification

(1) When quantitative verification of performance is performed on concrete structures, appropriate verification metrics and their threshold values are in principle established with respect to the set performance. Verification through experiments, mathematical methods based on mechanisms of mechanics, or other methods backed by an extensive track record are used to calculate responses in order to confirm that the structure satisfies the required performance during construction and during the design service life.

(2) Of the performance aspects required of concrete structures, those for which quantitative verification metrics and associated threshold values cannot be set must be considered appropriately.

Commentary: Regarding (1) In order to conduct performance verification of structures with greater clarity, verification metrics able to quantitatively express the performance demanded of structures, and the threshold values of these, should be established. It should further be confirmed that the response values of the structure do not exceed the threshold values. The method for calculating response values is generally verification by experimental or mathematical methods based on material mechanics and structural mechanics, but other methods with an extensive track record may also be used. The "Design" part presents a method for verifying the required performance of a structure with regard to durability, safety, serviceability, and restorability.

By its nature, performance verification must be carried out by comparing threshold values and response values using quantitative verification metrics. However, at current research and technical levels, there are aspects of performance that have not yet been quantified, or that can be expressed quantitatively but for which appropriate threshold values are difficult to set. For example, among aspects of performance for which verification metrics are considered to be clear, there are some (such as safety and serviceability) for which the setting of appropriate verification metrics is difficult, and for which verification of performance is thus replaced by verification of other closely related aspects of performance, in what is called deemed verification. Taking durability as an example, performance verification methods for chloride attack and carbonation are clearly shown in "Design". However, even given the same degree of chloride attack, at the current level of technology it is difficult to properly evaluate the damage when it occurs in an environment in which deicing salts are used. Accordingly, "Design" presents a situation in which chloride attack must be addressed by eliminating factors behind its occurrence through considerations such as waterproofing work and proper installation of drainage ditches.

Regarding (2) This presents the basic approach to the examination of environmental performance, indicated in **5.2**, as one of the aspects of performance to be considered. As with economic performance, the examination of environmental performance over the design service life is considered important. For example, circumstances are possible in which the introduction of special materials and techniques to enhance the performance of a structure increases the environmental impact at the construction stage, but the enhancement of performance suppresses the environmental impact at the maintenance stage, resulting in a reduction of the total environmental impact over the design service life. In such circumstances, assuming that a structure satisfies requirements for durability, safety, serviceability, *etc.*, it is important to consider the balance between constructability, ease of maintenance, social circumstances, and economic performance, and to consider the reduction of environmental impacts and enhancement of environmental benefits to the extent possible at all the stages of planning, design, construction, and maintenance.

3.3 Approaches to ensuring the performance of existing structures

(1) Through a maintenance plan based on the maintenance category of a structure, an existing structure must retain the functions and performance demanded of it during its service life.

(2) Regular diagnosis should be performed on the target structure to confirm its function and performance.

(3) If diagnosis determines that the function or performance of a structure has deteriorated, then the need for countermeasures must be examined, taking into account the degree and trend of deterioration, the importance of the structure, urgency, economic performance, social rationality, *etc.* If countermeasures are deemed necessary, then these are to be enacted based on the diagnosis results.

(4) If higher functionality is demanded during the service life, or if the performance of the structure no longer satisfies

the required performance, owing to causes such as the revision of design standards, then retrofitting or other measures must be undertaken.

(5) When carrying out countermeasures, the latest Standard Specifications at the time are to be used.

Commentary: Regarding (1) Asmentioned in Section **2.2**, to ensure the performance of concrete structures, the stages of design, construction, and maintenance must all be properly linked. Accordingly, for newly built structures, the maintenance category is set at the planning stage of the structure and an appropriate maintenance plan is formulated based on the maintenance category, after which the functions and required performance demanded during the service life must be secured. This applies equally to existing structures and new structures alike. If a maintenance category has not been set, then, based on information from initial inspections, *etc.* performed during the service life, it is necessary to set a maintenance category for the remaining scheduled service life and to formulate a maintenance plan based on this, and, at the appropriate time, to review the maintenance plan.

Regarding (2) and (3) Concrete structures are subjected to various actions during their service life. These actions might not have been anticipated at the time of construction or their durations differed from assumptions, which may result in the functions and performance of a structure falling below the prescribed standard. In order to perform maintenance more effectively during a structure's service life, diagnoses are carried out periodically to check for anomalies of the structure or deterioration in its performance. If these have occurred, then they should be detected and countermeasures should be enacted as quickly as possible. More effective countermeasures can be enacted by accurately estimating and identifying the mechanisms of deterioration that underlie the anomalies or the deterioration of performance.

Even if diagnosis results reveal deterioration in functions or performance, immediate countermeasures are not necessarily required if the degree of deterioration does not exceed prescribed reference values. However, depending on the cause and the rate of the deterioration, functions and performance can be secured through more economical means during the service period by enacting countermeasures at an early stage. When determining the necessity of countermeasures, it is also necessary to consider urgency according to the importance of the structure and the degree of deterioration, budgetary measures related to countermeasure expenses, and aspects of social responsibility, including renewal and disposal. Any countermeasures deemed necessary should be as economical and effective as possible, taking remaining scheduled service life into account.

Regarding (4) and (5) Even when deterioration or anomaly of the structure has not occurred and a decline in initially demanded performance is not apparent, if the use of the structure will change during the service life, if higher performance is demanded, or if the calculation methods for response values or the judgment criteria for threshold values have been revised because of factors such as new knowledge and technological advancement or changes in anticipated actions, then the performance of the existing structure must be reassessed using new methods. If the calculation method and judgment criteria for response values at that time are revised in line with revisions to the Standard Specifications for Concrete Structures or to other guidelines, then the performance of existing structures must be confirmed and assessed based on the new regulations. Then, as necessary, appropriate countermeasures to improve performance must be enacted, taking into account the importance of the structure, urgency, economic performance, social rationality, *etc.* A typical example in which these types of countermeasures are demanded is seismic retrofit resulting from revisions to approaches in seismic standards. In such cases, the importance of the structure, urgency, economic performance, *etc.*, are comprehensively considered in order to judge the necessity of countermeasures, and appropriate repairs, retrofit, renewal, *etc.* are carried out as needed. In the design of repairs and retrofit, the required performance must be examined with the remaining scheduled service life of the structure, its manner of use, life cycle cost during its service life, *etc.* taken into account. Furthermore, the design of repairs and retrofit must be performed in accordance with the most recent editions of the "Design" and "Maintenance" parts.

Chapter 4 The Roles of Engineers

4.1 General

(1) Engineers involved in the planning, design, construction, and maintenance of concrete structures must fulfill their roles and responsibilities in accordance with their respective positions.
(2) In principle, in the planning, design, construction, and maintenance of concrete structures, responsible engineers who possess appropriate technical capabilities and who have been granted responsibility and authority in line with the difficulty level of the work are to be assigned to each of the ordering entity, contractor, and design work checker or construction supervisor.
(3) In principle, in the planning, design, construction, and maintenance of concrete structures, engineers who possess technical capabilities concerning concrete in line with the importance and scale of the target structure, the degree of difficulty of the construction, *etc.* are to be assigned to each of the ordering entity, contractor, and design work checker or construction supervisor.

Commentary: <u>Regarding (1)</u> To achieve reliable concrete structures, engineers must fulfill their roles and responsibilities in accordance with their respective positions at each of the stages of design, construction, and maintenance. Engineers are divided into those who have wide-ranging and advanced knowledge and extensive practical experience concerning civil engineering structures, such as senior engineers, and those who possess the ability to make expert technical judgments primarily in practical aspects, such as concrete specialist engineers and concrete engineers. See Section **1.4** for categories and definitions of terms related to these engineers.

<u>Regarding (2)</u> Responsible engineers possess the authority and responsibility to make judgments regarding contracts at each of the stages of planning, design, construction, and maintenance of concrete structures. In general, responsible engineers are assigned to each of the ordering entity, contractor, and design work checker or construction supervisor, including the facility manager, *etc.* Responsible engineers are not limited to engineers specializing in concrete; in cases such as a facility consisting of multiple steel structures, soil structures, *etc.*; it is necessary to appoint appropriate responsible engineers according to the target structures.

Responsible engineers involved in the planning, design, construction, and maintenance of concrete structures must take a long-term and broad perspective at each of the stages of planning, design, construction, and maintenance, and must possess capabilities for high-level and comprehensive judgment. In principle, engineers possessing such technical capabilities correspond to senior engineers. Appropriate qualified engineers should be assigned according to the scale of planning, design, construction, and maintenance, and to the difficulty level of the judgment.

<u>Regarding (3)</u> In the planning, design, construction, and maintenance of concrete structures, concrete specialist engineers who possess technical capabilities in line with the importance, scale, degree of difficulty, *etc.* of the target facility are to be assigned in principle; concrete engineers may be assigned as needed. Concrete

specialist engineers bear the duty and responsibility to identify issues at each stage and to find, propose, and implement solutions. If the target facility is a concrete structure, then the responsible engineer may also be a concrete specialist engineer.

The role of a concrete specialist engineer who possesses specialized knowledge and experience is particularly important when special materials, structural forms, construction methods, and maintenance methods are adopted.

4.2 The roles of engineers at the design stage

(1) In the design of concrete structures, engineers who possess technical capabilities related to concrete must be assigned as appropriate in accordance with the importance, scale, degree of difficulty, *etc.* of the target structures.
(2) At the design stage, it is conventional to assign responsible engineers to both the ordering entity and the design work contractor, and, separately, to also assign a responsible engineer to the design work checker. These responsible engineers must hold independent and equal positions.
(3) The responsible engineer in the ordering entity must clearly present the content of the design and the required performance, *etc.* of the target structure, and must determine an appropriate design service life for the structure, while taking into account factors including the service life demanded of the structure, maintenance methods, environmental conditions, and economic performance.
(4) The responsible engineer in the design work contractor must perform design after setting the structural form and structural details so as to enable the structure to satisfy the required performance throughout the set design service life, and must ensure the outcomes, construction period, and quality required by the contract.
(5) The responsible engineer acting as the design work checker must verify whether the set structural form and the structural details satisfy aspects of the required performance including durability, safety, serviceability, and restorability over the design service life of the structure.
(6) At the design stage, concrete specialist engineers are generally assigned to each of the ordering entity, design work contractor, and design work checker, and must make technical judgments regarding the design and carry out design verification, with the responsible engineer in each position taking on part of the scope of responsibility and authority.

Commentary: <u>Regarding (1) and (2)</u> The standard organizational relationships and the assignment of engineers within them at the design stage of concrete structures are shown in **Figure C4.2.1**. Here, communication lines indicate the relationships between the engineers in each organization who exchange information required for work. In general, the design of concrete structures is undertaken by contract agreement, with responsible engineers acting as the supervision engineers or verification engineers for design and possessing all responsibility and authority needed for design at the design stage. When the scale of a design is large or when there are many types of construction in the design, engineers with expertise equivalent to or greater than that of a concrete specialist engineer are assigned as responsible engineers, under which concrete engineers may be assigned as needed. In other words, in addition to cases in which responsible engineers perform design directly, there are also cases in which multiple concrete engineers perform design while receiving guidance under the responsibility and authority of the responsible engineers.

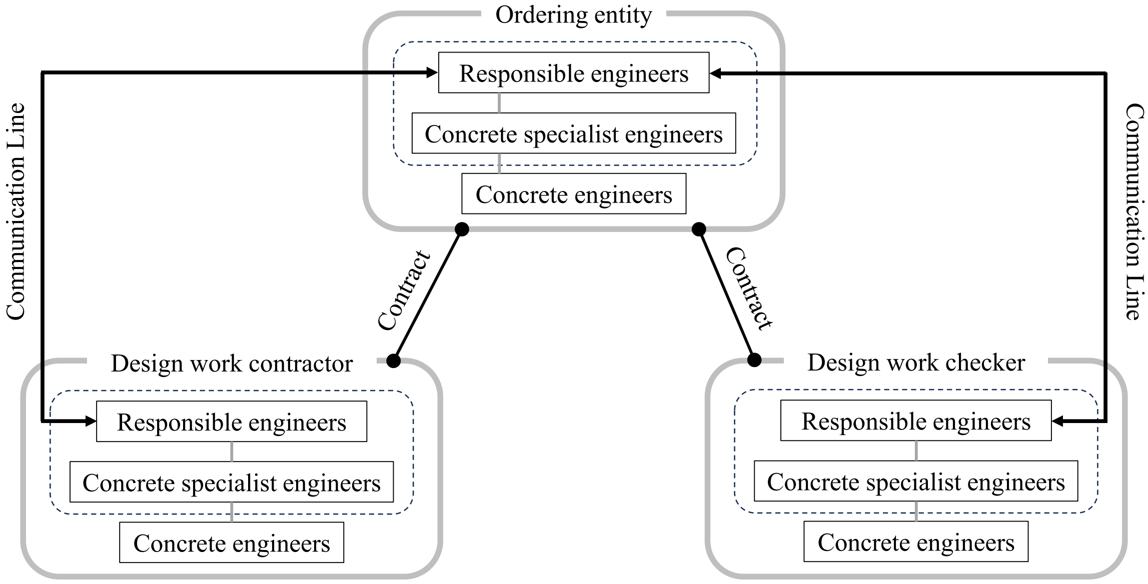


Figure C4.2.1 Standard organizational relationships at the design stage of concrete structures and the relationships between assigned engineers.

In the design of a concrete structure, it is conventional to assign responsible engineers who possess sufficient knowledge and experience to both the ordering entity and the design work contractor in accordance with the importance and degree of difficulty of the target structure, and to assign engineers in charge of design verification. Here, "design verification" refers to the act of confirming the design conditions, the content and quality of design results, *etc.* In design work based on a contract agreement, the assignment of a verification engineer is commonly specified as a contract condition; in many cases, an obligation to provide notification of the verification engineer is stated in the general specifications.

At the same time, separate from engineers in charge of design verification assigned to the ordering entity and the design work contractor as noted, an engineer assigned to confirm that design is performed properly so as to protect the public interest is intended to serve as the design work checker, as described in **Figure C4.2.1**; a responsible engineer must be assigned here as well. Note that a design work checker may not be required if a sufficient design verification system can be established within the ordering entity.

In order to satisfy the required performance of structures and to plan and design highly reliable structures, responsible engineers in each of the organizations involved must possess the capabilities and the high ethical standards required to execute the work, and relationships of independent and equal positions must be established and maintained among these engineers. Performing verification of designs through independent organizations is fundamental to ensuring reliability.

Qualification requirements for responsible engineers include, among JSCE Executive Professional Civil Engineers, Senior Professional Civil Engineers, and Consultant Engineers ("Engineering Management" or "Civil Engineering"), those engineers who specialize in concrete, RCCM "Materials & Structures", or qualified engineers who have equivalent or greater technical capabilities, in accordance with the importance, degree of difficulty, *etc.* of the structure. Appropriate qualified engineers should be assigned according to the scale, importance, and degree of difficulty of the target structure.

<u>Regarding (3) and (4)</u> The responsible engineer in each organization, possessing the required technical capabilities and responsibility, has the duty to construct

structures of sufficient quality economically and within the prescribed construction period, and to provide these to society. Toward that end, it is important to clarify the division of roles and the mutual relationships between the responsible engineers in the ordering entity and the design work contractor.

In planning the concrete structure prior to design, the responsible engineer in the ordering entity must set the required performance of the structure with consideration of natural conditions, social conditions, constructability, environmental performance, economic performance, *etc.* Together with this, it is necessary to determine the design service life of the structure with consideration of the service life required for the target structure or its structural components/members, as well as maintenance methods, environmental performance, economic performance, *etc.*

The responsible engineer in the design work contractor must set the structural form and structural details (dimensions of members, reinforcement arrangement, materials used, *etc.*) so as to enable the structure to satisfy the required performance set in the plan for the structure over its prescribed design service life, and must ensure the outcomes, construction period, and quality required by the contract. If defects associated with the design are confirmed through checks of the design work, through design verification during construction, *etc.*, then the responsible engineer bears the responsibility to take appropriate measures based on the contract.

Regarding (5) The responsible engineer who is acting as the design work checker must perform verification to confirm that the structural form and structural details set at the planning stage satisfy aspects of required performance including durability, safety, serviceability, and restorability over the design service life of the structure. As shown in (2), in effect, design verification is performed twice by both the design work checker or ordering entity and by the design work contractor in order to guarantee its performance is correct.

Regarding (6): It is important to assign concrete specialist engineers as needed under the responsible engineers in the ordering entity, design work contractor, and design work checker, and to enable prompt and appropriate handling of design verification and of all manner of issues that occur at the design stage by sharing scope of authority and responsibility among the responsible engineers in each position.

4.3 The roles of engineers at the construction stage

(1) In the construction of concrete structures, engineers who possess technical capabilities related to concrete must be assigned as appropriate in accordance with the importance, scale, degree of difficulty, *etc.* of the target structures.

(2) In principle, responsible engineers at the construction stage should be assigned to the ordering entity, construction contractor, and construction supervisor. The responsible engineers must hold independent and equal positions.

(3) Responsible engineers at the construction stage must make technical judgments regarding the construction so as to satisfy the quality, construction period, *etc.* of the structure.

(4) At the construction stage, concrete specialist engineers must be assigned to each of the ordering entity, construction contractor, and construction supervisor. While sharing the scope of authority and responsibilities with responsible engineers in each position, these concrete specialist engineers must make technical judgments regarding the concrete construction, perform construction instruction, and supervise construction.

Commentary: Regarding (1), (2), and (3) The standard organizational relationships and the assignment of engineers within them at the construction stage of concrete structures are shown in **Figure C4.3.1**. At the construction stage, responsible engineers must be assigned to each of the ordering entity, construction contractor, and construction supervisor. Acting under authority and responsibilities based on the contract, these responsible engineers must make technical judgments regarding the construction so as to satisfy the outcomes, construction period, and quality required by the contract. If there is no appropriate engineer to serve as the responsible engineer within the organization of the ordering entity, then the ordering entity may separately conclude a delegation contract with an agent and may entrust technical judgments to the agent as the responsible engineer in the ordering entity.

The responsible engineer in the ordering entity is required to present specifications necessary for construction and to make technical judgments. The responsible engineers in the construction contractor and in the construction supervisor are required to have technical judgment capabilities for each process, to ensure quality throughout the construction overall.

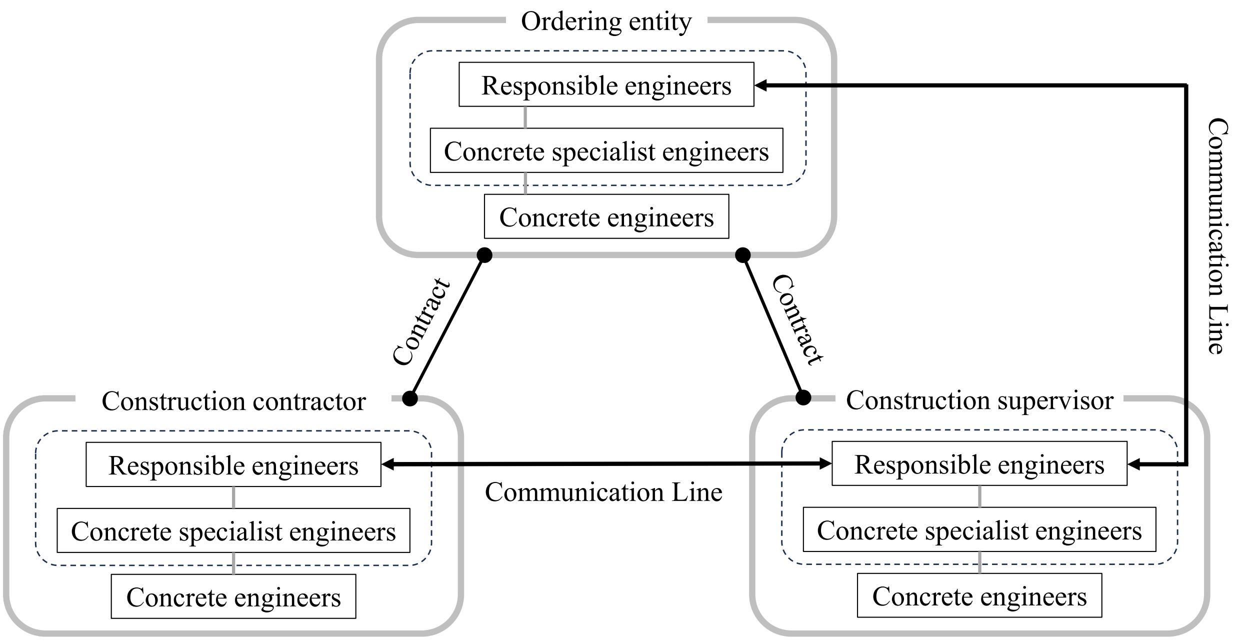


Figure C4.3.1 Standard organizational relationships at the construction stage of concrete structures and the relationships between assigned engineers.

In order to satisfy the required performance of structures and to construct highly reliable structures, responsible engineers in each of the organizations at the construction stage must possess the capabilities and the high ethical standards required to execute the work. Relationships of independent and equal positions must be established and maintained among these engineers.

The responsible engineer in the ordering entity has the duty to clearly present the content of the construction, required performance, *etc.* to the responsible engineers in the construction contractor and in the construction supervisor.

The responsible engineer in the construction contractor bears responsibility for the construction overall. In order to satisfy the quality of the structure and to complete the construction within the prescribed construction period, the responsible engineer makes apt technical judgments with regard to all manner of issues that occur in work stages, and bears responsibility for the results.

The responsible engineer in the construction supervisor must perform appropriate inspections at the appropriate times to satisfy the performance and quality of the structure.

Regarding (4) Throughout the construction, the ordering entity, the construction contractor, and the construction supervisor must assign concrete specialist engineers who possess appropriate technical capabilities in their respective positions. These concrete specialist engineers must make technical judgments, carry out instructions for construction, and supervise construction based on their knowledge and experience concerning concrete. Concrete engineers should be appropriately assigned under the concrete specialist engineers in accordance with the scale and the degree of difficulty of the construction.

It is important that responsible engineers at the construction stage clarify the scope of responsibility and authority of concrete specialist engineers or concrete engineers in order to enable prompt and appropriate responses to all manner of issues that occur in work stages, and that these engineers take on a portion of the responsibility and authority of responsible engineers.

The concrete specialist engineer in the ordering entity bears the duty of confirming that the structure has been completed according to design drawing documents, and of drafting an appropriate inspection plan that enables judgment of the validity of wide-ranging matters implemented at each stage of construction. Moreover, concrete specialist engineers who supervise construction must judge whether the inspection plan is appropriate.

The concrete specialist engineer in the construction contractor, who comprehensively takes into account work safety, compliance with relevant laws, economic performance, construction period, and environmental impact, bears the responsibility of drafting a construction plan capable to guaranteeing the performance of the structure indicated in the design drawing documents and of setting appropriate concrete construction methods and constructability. Moreover, concrete specialist engineers who supervise construction must judge whether the construction plan is appropriate.

The quality of construction is greatly affected by human factors such as the experience and aptitude of the engineers involved in the construction. For this reason, it is extremely important to have the construction contractor station concrete specialist engineers with extensive knowledge and experience regarding concrete construction at the site, and to perform concrete construction at the site, and to perform construction under the instruction of the specialist engineers.

The concrete specialist engineer in the construction supervisor bears the duty of making technical judgments on whether construction by the construction contractor is performed according to the contract, and, if not, bears the duty of instructing the construction contractor to take appropriate measures, within the scope of its authority and responsibility.

In actual construction, situations not anticipated at the time of formulating the construction plan often occur, and situations in which it is difficult to comply with the formulated construction plan can occur. In such cases, it is important that the concrete specialist engineer in the construction contractor reviews the construction plan and enacts appropriate and prompt measures to ensure the required performance, and that the concrete specialist engineer in the construction supervisor judges whether said measures are appropriate, and, based on given authority and responsibility, instructs the construction contractor to act as needed.

Similarly, when signs of increasing variability in quality are recognized through quality control, the concrete specialist engineer in the construction contractor must investigate the causes and must enact countermeasures so that quality falls within the prescribed tolerance range.

4.4 The roles of engineers at the maintenance stage

(1) In the maintenance of concrete structures, engineers who possess technical capabilities related to concrete must be assigned as appropriate in accordance with the importance, scale, degree of difficulty, *etc.* of the target structures.
(2) In principle, responsible engineers at the maintenance stage should be assigned to both the facility manager, *etc.* and the maintenance contractor. It is advisable to establish a relationship of independent and equal positions for these responsible engineers.
(3) The responsible engineer in the facility manager, *etc.* must formulate a maintenance plan such that, throughout the scheduled service life, the performance of the structure does not fall below the maintenance thresholds set based on the maintenance category, and, following the establishment of the required maintenance system, must perform proper maintenance of the structure.
(4) The responsible engineer in the maintenance contractor diagnoses the performance of the structure based on the maintenance plan, appropriately records the results, and drafts countermeasures as needed.
(5) The concrete specialist engineer at the maintenance stage must perform maintenance of the structure, sharing the scope of authority and responsibility of responsible engineers between the facility manager, *etc.* and the maintenance contractor.
(6) When design and construction are required in maintenance, the roles of engineers are according to Sections **4.2** and **4.3**.

Commentary: Regarding (1), (2), (3), and (4) **Figure C4.4.1** shows the standard two-party relationship and engineers at the maintenance stage of concrete structures. Unlike the design stage and construction stage in which a three-party relationship is common, the maintenance stage generally consists of a two-party relationship between the facility manager, *etc.* and the maintenance contractor. Accordingly, at each of the stages of maintenance planning, diagnosis, countermeasures, and recording, responsible engineers are in principle assigned to the facility manager, *etc.* and to the maintenance contractor, in accordance with the existence of a contract, details of the contract, the scale of maintenance, *etc.* Note that if the facility manager, *etc.* performs maintenance through an in-house engineer, a responsible engineer should be assigned to the facility manager, *etc.* There is also a two-party relationship by which, when the facility manager, *etc.* outsources the construction of the maintenance structure and the formulation of the maintenance plan to an agent, a responsible engineer is assigned to the agent, and all other technical activities are outsourced to the maintenance contractor. Note that responsible engineers are not limited to engineers specializing in concrete; in some cases, such as a facility consisting of multiple steel structures, soil structures, *etc.*, it is necessary to appoint appropriate responsible engineers in accordance with the target facility or structure.

During the scheduled service period, the performance of the structure must not fall below prescribed maintenance thresholds. Toward this end, at the maintenance stage of the structure, the responsible engineer in the facility manager must formulate a maintenance plan in advance, and then must prepare a system for properly executing the series of maintenance activities, including diagnosis, countermeasures, and recording, consisting of inspection, deterioration prediction, evaluation, judgment, *etc.* At the same time,

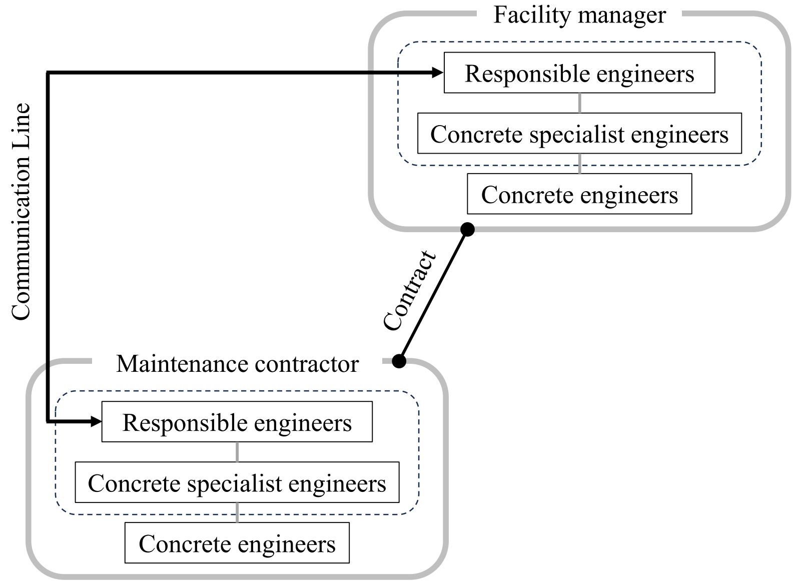


Figure C4.4.1 Standard organizational relationships at the maintenance stage of concrete structures and the relationships between assigned engineers.

the responsible engineer in the maintenance contractor must execute maintenance activities including inspection, judgment of inspection results, determination of measures, and recording, in accordance with the scope of the contract.

Appropriate qualified engineers should be assigned as responsible engineers, in accordance with the scale, importance, *etc.* of the construction.

<u>Regarding (5)</u> In the maintenance of concrete structures, concrete specialist engineers who possess the necessary technical capabilities at each of the stages of maintenance planning, diagnosis, countermeasures, and recording are in principle assigned to the facility manager, *etc.* and to the maintenance contractor, while concrete engineers are assigned as needed. However, if the facility manager, *etc.* performs maintenance through an in-house engineer, then a concrete specialist engineer should be assigned only to the facility manager. A concrete specialist engineer at the maintenance stage is an engineer who possesses wide-ranging, advanced knowledge and extensive experience regarding concrete structures, and who is able to make appropriate technical judgments regarding the maintenance of these.

Concrete specialist engineers at the maintenance stage bear the duty to identify issues in the maintenance of concrete structures, and to find, propose, and implement solutions. If the target facilities are limited to concrete structures, then the responsible engineers may also serve as concrete specialist engineers, and if special measurement/monitoring, repair/retrofit materials, and construction methods are adopted in maintenance, then the roles and responsibilities of concrete specialist engineers who possess specialized knowledge and experience are of particular importance.

It is advisable to assign appropriate qualified engineers as concrete specialist engineers, in accordance with the scale, importance, *etc.* of the work. In general, it is often difficult to have engineers who possess advanced knowledge always handle the maintenance of all structures, including simple maintenance performed on a daily basis. Accordingly, it may be rational to change the engineer in charge in accordance with the level of maintenance inspections (including the evaluation and judgment of inspection results). For example, there may be cases in which sufficient judgments can be made by engineers capable of doing so using basic knowledge in simple daily inspections, and not only by responsible engineers who make highly difficult comprehensive

judgments regarding inspection results or by concrete specialist engineers who possess specialized knowledge and technical capabilities for the selection of inspection investigation methods and the evaluation of results. In such cases, a concrete engineer can take charge of inspections and perform these based on manuals, *etc.* in which methods for investigation and for evaluation of results are described in detail. In other words, in performing maintenance of concrete structures, it is important that responsible engineers who possess responsibility and authority with respect to the maintenance work are appropriately assigned, and that under these, concrete specialist engineers and concrete engineers fulfill their individual roles.

4.5 Coordination of engineers to ensure performance

To ensure the performance of concrete structures, appropriate technical judgments must be made at each of the stages of design, construction, and maintenance, and, as needed, discussions must be held with concrete specialist engineers in charge of other stages.

Commentary: Work on concrete structures is performed at each of the stages of construction, service, and maintenance based on basic policy set forth in the structural plan. The service life until the end of the lifespan of the structure is long; in many cases, it is not possible to perform examination with all necessary conditions finalized. In such cases, concrete specialist engineers and concrete engineers must fully cooperate with each other at each of the stages of design, construction, and maintenance, and, as needed, must return to the work of the previous stage and perform reviews of or make changes to the set conditions.

Typical examples that show the importance of coordination in design and construction include verification of and countermeasures for thermal cracking, structural conditions for cross-sectional dimensions and rebar arrangement, workability of concrete, and curing and drying shrinkage.

The examination of mass concrete is carried out based on conditions specified in the structural plan at the design stage. The conditions obtained as a result, such as materials used and their mix proportions, are carried forward to the construction stage via design drawing documents. In many cases, however, the construction conditions and work procedures will not have been finalized at the design stage. In the examination of mass concrete, the result of which is greatly affected by construction conditions, it may be necessary to perform examination again when the actual construction method has been finalized. For example, while cold-weather concreting may be planned at the design stage, hot-weather concreting may be required in construction, and it may become necessary to change the initial assumption of a concrete pouring temperature of approximately 10 °C to a temperature of 30 °C or higher, and, as a result, to thoroughly review cracking countermeasures. In such cases, it may be possible to address the situation by using only countermeasures that solve the issue through changes at the construction stage without a return to the design stage, such as through pre-cooling or a change in cement type. However, it is often advantageous in terms of cost and performance to include measures that involve a return to the design stage and reexamination, such as the addition of reinforcing bars, installation of crack-control joints, or a change in the quality-control age for concrete strength. Accordingly, it is extremely important that, in design and construction, concrete specialist engineers coordinate to conduct a multifaceted examination and proceed with work.

Issues related to concrete placing during construction include cases in which concrete placing is not possible at the location assumed at the design stage because of constraints in the surrounding environment, cases in which the rebar arrangement is dense and the prescribed placing location cannot be secured, and cases in which compacting work is difficult. Even when such problems occur, rather than only performing an examination focused on countermeasures at the construction stage, such as the application of high-fluidity concrete or reworking of slump to improve concrete workability, it may be more rational to rework the design itself, such as by eliminating high-density rebar arrangements through the adoption of mechanical joints or mechanical anchorage, the use of refinements to the rebar arrangement at concrete pouring openings, or the use of large-diameter reinforcing bars. However, in order to avoid such reworking, it is also important to duly consider construction-related constraints in planning and design at the stages of structural planning and design.

With regard to drying shrinkage, too, it is often difficult to sufficiently reflect actual construction conditions at the design stage. For this reason, conditions may arise in which, in order to satisfy the tolerance for the amount of concrete drying shrinkage set in the design at the construction stage, material or mix design improvements entailing significant cost increases become necessary, or the construction period lengthens because of extension of the curing period. Moreover, reports exist of drying shrinkage cracking occurring several years after the completion of construction, depending on the quality of the aggregate or other materials used. In such cases, at the construction stage or the maintenance stage, the concrete specialist engineers at the design stage, construction stage, and maintenance stage must coordinate and hold discussions, and must enact optimum countermeasures that consider the structure as a whole.

Note that to achieve highly reliable structures that satisfy performance requirements, it is necessary that engineers at each stage understand conditions at all stages and strive to uncover the best technical solutions to varied problems, regardless of their individual positions. Together with this, it is also effective for the ordering entity to take the lead in organizing a council that gathers specialist engineers and concrete specialist engineers at each stage, and to organize a structure by which engineers at each stage can share the same information and coordinate with each other.

Chapter 5 Environmental Performance of Concrete Structures

5.1 General

This Section addresses the environmental performance to be considered in the planning, design, construction, and maintenance of concrete structures.

Commentary: Civil infrastructure consists of systems developed to support the sustainable development of societies in which people lead safe, secure, and prosperous lives. It covers management and preservation of national land, transportation and transport systems, disaster prevention, information infrastructure, and other wide-ranging aspects. The development of the facilities that make up civil infrastructure—roads, railroads, ports, airports, dams, riverways, water and sewage systems, electricity, gas, *etc.*—create extremely high value. At the same time, as the development of civil infrastructure also impacts a variety of environments, it is vital to proceed with development while remaining mindful of the sustainability that has come to be emphasized in recent years. Sustainability is a concept that expresses the sustainability of human activities in terms of three aspects: environment, economy, and society.

As concrete is a key construction material that is used in large quantities, the field of concrete bears significant social responsibility. Accordingly, it is necessary to always consider the environment when engaging in planning, design, construction, and maintenance of concrete structures. While the environmental performance of a project overall naturally cannot be addressed through matters related to concrete alone, it is important to enhance environmental performance in concrete-related projects, taking information, standards, *etc.* from other fields into account as needed. Accordingly, this Section presents basic approaches to this.

With regard to the life cycle of concrete structures, not only legal compliance but also activities that contribute to the formation of a cyclical society, such as resource conservation, energy conservation, and the 3Rs (Reduce, Reuse, Recycle), have been actively carried out. For example, Japan's technological levels of energy efficiency and effective use of waste and by-products in cement manufacturing are among the highest in the world. Moreover, systems for performing construction and maintenance of high-durability concrete structures achieve resource conservation over the life cycle of civil infrastructure. In order to provide easily understood explanations of forward-looking efforts in the development and deployment of concrete-related research and technology to citizens who are both users of and investors in civil infrastructure, it is important to build a shared framework that considers the environmental performance of concrete structures.

Table C5.1.1 summarizes the categories of environment to be considered in the planning, design, construction, and maintenance of concrete structures, the significance of examination, influencing factors, and basic approaches to examination. Environments to consider for concrete structures and their life cycles include the global environment, regional environment, work environment, and landscapes. Of these, "global environment" refers to the environment that is subject to global warming, depletion of the ozone layer, and other impacts on a global scale, and primarily involves greenhouse gas emissions and resource consumption.

"Regional environments" refers to environments that are subject to air pollution, soil contamination, water pollution, waste treatment, and other impacts in areas where activities related to concrete structures are carried out, and primarily involve the emission of air, soil, and water pollutants and wastes, the use of by-products, *etc.*

Table C5.1.1 Relationship between environmental categories and the examination stages of concrete structures.

Environmental category	Significance of examination	Cause	Examination stage	Examination method
Global environment	Maintain and conserve the global environment and guarantee the healthy and permanent existence of humankind on Earth.	・Greenhouse gases ・Resource consumption ・Energy consumption *etc.*	・Planning ・Design ・Construction ・Maintenance	Control greenhouse gas emissions, resource consumption, and energy consumption to the smallest degree possible throughout the life cycle of the structure. If required values are specified by contract or if societally accepted values exist, mandate compliance.
Regional environments	Maintain and conserve local environments, and guarantee healthy and comfortable lifestyles for local communities. With biodiversity in mind, work toward conservation of plants and animals to guarantee the preservation of species.	・Air pollutants ・Water pollutants ・Soil contaminants ・Wastes ・Recycling ・Use of by-products ・Biodiversity, *etc.*	・Planning ・Design ・Construction ・Maintenance	Make construction environments conform with environmental standards and emissions standards. Mandate compliance with the Construction Material Recycling Act for demolition waste. Mandate the enactment of measures to preserve animal and plant species.
Work environment	Maintain and conserve work environments and guarantee that construction workers can work in health and comfort.	・Noise ・Vibration ・Dust ・Soot, *etc.*	・Construction ・Maintenance	Make construction environments conform with environmental standards and emissions standards. Make work environments conform with occupational safety and health regulations.
Landscapes	Ensure that natural landscapes are not disturbed.	・Landscapes ・Aesthetics, *etc.*	・Planning ・Maintenance	Strive for harmonization with the surrounding natural environment.

"Work environments" refers to the environments surrounding persons engaged in the construction and maintenance of concrete structures. It primarily involves emissions of substances harmful to humans and the generation of noise and vibration during work, *etc.* at the stages of concrete construction.

"Landscapes" refers to environments that should be considered to achieve harmonization with the natural environment without disturbance of surrounding scenery by structures, and involves landscapes at the planning stage of concrete structures and aesthetics at the maintenance stage.

Items to be examined include greenhouse gases and other negative impacts on the environment, social contribution such as the use of wastes, and the effects of water purification, sound insulation, thermal storage, *etc.* (environmental benefits). Use of the large volumes of debris created by the March 11, 2011 Tohoku earthquake and tsunami can also be regarded as an environmental benefit.

Consideration of the environmental performance of concrete structures is positioned as an evaluation of the overall impacts on sustainability of a project, including its structures. Moreover, in the same way that improvements in the safety, durability, *etc.* of structures lead to improvements in the environmental performance of concrete structures over their life cycles, environmental performance is also closely related to other aspects of structures' required performance. Accordingly, consideration of environmental performance means an examination of ways to reduce negative environmental impacts and enhance environmental benefits, taking into account the balance of economic performance and other aspects of required performance, while observing limit values stipulated by laws. While environmental performance is important in the lives of people, objective methods of evaluation are not necessarily sufficient, and there are still considerable differences of opinion regarding how much consideration should be given to environmental performance. For such reasons, further examination of the handling of environmental performance is needed.

5.2 Consideration of environmental performance

In order to reduce negative environmental impacts and enhance environmental benefits, consideration must be given to environmental performance at each of the stages of planning, design, construction, and maintenance of concrete structures, while taking into consideration required durability, safety, serviceability, restorability, constructability, ease of maintenance, social conditions, economic performance, *etc.*

Commentary: Many government ordinances, regulations, and other environment-related legal systems have been established in connection with environmental conservation to ensure healthy and cultured lifestyles for citizenry now and into the future, and to contribute to the welfare of humankind. Many related laws and standards have also been established regarding the development of civil infrastructure. As such, it is only natural that these laws, regulations, *etc.* be observed in the consideration of concrete structures' environmental performance, as well. As such laws, regulations, *etc.* may be revised in the future, it is advisable to always obtain up-to-date information.

If environmental performance has been set as an aspect of the required performance of a concrete structure, then it is necessary to comprehensively take into consideration required durability, safety, serviceability, and restorability, as well as constructability, ease of maintenance, social conditions, economic performance, *etc.* over the design service life of the structure, and to consider the environmental performance required at each of the stages of planning, design, construction, and maintenance. Note that, just as it is important to examine the economic performance of a concrete structure in terms of life cycle cost, it is also important at that time to examine environmental performance in terms of the entire life cycle. For example, even if initial environmental impacts increase due to the enhancement of performance during construction, this may enable a reduction of environmental impacts over the entire life cycle by suppressing environmental impacts in maintenance.

An extremely large number of organizations are involved in the life cycle of concrete structures, including design consultants at the planning and design stages, cement manufacturers at the materials manufacturing stage, construction companies and concrete product manufacturing companies at the construction stage, repair companies at the maintenance stage, and dismantling companies at the dismantling stage. For that reason, the manager of the structure first determines basic policy for the consideration of environmental performance over the life cycle of the concrete structure, and every organization and every individual at each stage examines and implements specific methods to reduce environmental impacts and enhance environmental benefits to the degree possible.

Stipulated matters regarding environmental performance at the project planning stage must be examined. In addition, if there are any items specified by the ordering entity or items proposed by the contractor at the bidding stage, then examination is performed based on these items and on specific targets.

In the same way that life cycle costs are reduced through high durability, preventive maintenance, *etc.*, in environmental performance, too, environmental impacts can be lessened significantly in some cases through the selection of construction methods and techniques that ensure high durability in structures. For that reason, while taking the effects of high durability and other environmental performance issues into consideration at the planning stage during which basic policies for the concrete structure are examined, it is also important to formulate a structural plan that enables reduction of greenhouse gases, resource and energy conservation, low pollution, *etc.* At the same time, as detailed construction conditions, materials to be used, and other matters may not have been finalized at the planning stage, outcomes of examinations of environmental performance are not always highly accurate. Accordingly, it is necessary to accumulate information calculated and measured at each of the stages of design, construction, and maintenance to enable rough estimation of environmental performance at the planning stage, as is done in examination of economic performance.

Note that the targets addressed here are not necessarily specific quantitative targets, but may include qualitative targets such as making maximum use of industrial wastes. For landscapes and other items that cannot be expressed using clear numerical values, methods appropriate to the items should be selected and examined.

Environmental performance can be examined under certain assumptions at the design stage, but as detailed information may change at the construction stage, when performing examinations, it is important to make a clear distinction between ascertained conditions and assumed conditions. Even in cases in which specific targets have been set, it is necessary to compare the outcomes of examinations to targets with attention given to assumptions made for various conditions, and to undertake redesign or review of the targets as needed.

At the construction stage, selected items are generally examined through objectively valid methods. In cases in which specific targets have been set, whether these targets are satisfied is checked. Regarding items for which specific values can be obtained through measurement at the construction work stage, it is advisable to confirm the validity of values calculated at the planning or design stage, and to perform correction of calculated values and review of related examination methods as needed. In cases in which targets have not been set at the planning stage, the construction plan should be examined through comparison with information examined at the design stage, and through comparison with the construction plan when performing construction.

At the maintenance stage, selected items are generally examined through objectively valid methods. In cases in which specific targets have been set, whether these targets are satisfied is checked. Note that if the maintenance plan has been changed, then examination is to be based on the changed maintenance plan. As the maintenance of structures is an activity that extends to the long term, there is a possibility that technologies will be developed that are better able to economically reduce environmental impacts and enhance environmental benefits while satisfying required performance. Accordingly, it is also important to construct a maintenance plan that can flexibly adapt to such changes.

Further development of environment-related legal systems and evaluation techniques for environmental impact assessment is anticipated, as is further accumulation of data concerning inventory. With regard to examination of the environmental performance of concrete structures, there is a need to prepare for future re-assessments related to their environmental performance and to keep records of examinations of their environmental performance for disclosure of information on the structures and as important materials that will contribute to the development of future examination methods.

Design

Standard Specifications for Concrete Structures -2018

"Design"

CONTENTS

General Requirements

Standards

General Requirements

Chapter 1 General

1.1 Scope of application

The "Design" volume of the Standard Specifications for Concrete Structures of the Japan Society of Civil Engineers (hereinafter "Standard Specifications") presents principles of structural planning and verification with respect to the required performance of structures consisting of general reinforced concrete structures and prestressed concrete structures, and further stipulates standard verification methods, assumptions regarding reinforced concrete, and structural specifications.

Commentary: In concrete structures, work proceeds according to the sequence of structural planning, design, construction planning, manufacturing work, construction work, and maintenance. Basic approaches concerning the Standard Specifications for Concrete Structures as a whole, its purpose, the roles and relationships of its Volumes, and so on are presented in "Basic Principles."

The Standard Specifications consist of two parts: the Main Volume and the Standards. The Main Volume presents principles for the structural planning of reinforced concrete and prestressed concrete structures at the design stage, and for the verification of durability, safety, usability, and recoverability. The Standards present standard methods for the verification of durability, safety, usability, and seismic resistance, as well as structural specifications and assumptions concerning reinforced concrete. The standard performance verification methods presented in the Standard Specifications apply to concrete with a characteristic value of compressive strength of no more than 80 N/mm^2. The applicable steel materials are PC steel and rebar with a characteristic value of tensile yield strength of up to 685 N/mm^2.

The Standard Specifications establish key principles for the clarification of performance required for the durability, safety, usability, and recoverability of structures in the design stage, and verification of these by reliable and rational methods. The limit state design method, which has an extensive track record, is used for verification of required performance. Basic approaches concerning environmental performance that should be considered for concrete structures are as shown in "Basic Principles."

1.2 Basics of design

(1) In design, the required performance is set to fulfill the purpose and function of the structure, the structural planning and structural details of the structure are set to meet that required performance, and the satisfaction of the required performance throughout the design service life is verified.

(2) In design, all aspects of required performance are to be set in accordance with Chapter 2 during construction and during the design service life.

(3) In structural planning, the type of structure and other matters are set in accordance with Chapter 3 to satisfy the required performance, after considering structural properties, materials, construction work, maintenance, environmental performance, economic performance, and other factors.

(4) In the setting of structural details, the form and dimensions, reinforcing bar arrangement, and other structural details, as well as the rough mixture of concrete, type of steel material, and other properties of materials to be used, are to be set for the type of structure that was set in structural planning, according to structural specifications and other matters in the Standard Specifications.

(5) In the verification of required performance, the satisfaction of required performance by the structure in terms of durability, safety, usability, recoverability, *etc*. throughout the design service life is to be verified in accordance with Chapter 4.

(6) In design, the degree of satisfaction with respect to required performance must be determined using appropriate metrics, and evaluated based on engineering standards.

Commentary: In the design of structures, it is necessary to work toward more rational structures so that the specified performance is demonstrated to be in line with specific design objectives and with consideration of natural conditions, social conditions, construction conditions, environmental performance, economic efficiency, *etc*. The safety, usability, and recoverability of structures specified in these Standard Specifications are strongly influenced by the specifics of form, dimensions, and arrangement of reinforcement and by the physical properties of materials. Therefore, many of these specifications can be determined by these Standard Specifications. However, the form, dimensions, arrangement of reinforcement, and other structural details are deeply connected with workability, as presented in "Construction." These Standard Specifications require that these be taken into consideration in advance to ensure the design as a whole is rational, so as to avoid rejection on the grounds of workability. Once a concrete structure has been built, its subsequent repair, reinforcement, or improvement may be difficult. Therefore, surveys must be performed in the initial stage of design to accurately predict events that could occur during the service period, and the type of structure and specifications must be determined with consideration of ease of maintenance. Basics of maintenance are presented in "Maintenance."

Regarding (1): The design of a structure consists of individual tasks of setting of required performance, structural planning, setting of structural details, and verification of required performance, all of which must be conducted in an integrated fashion (see **Commentary Figure 1.2.1**).

Regarding (2): The required performance of a structure must be set appropriately with consideration of the purpose, functions, importance, and other aspects of the structure.

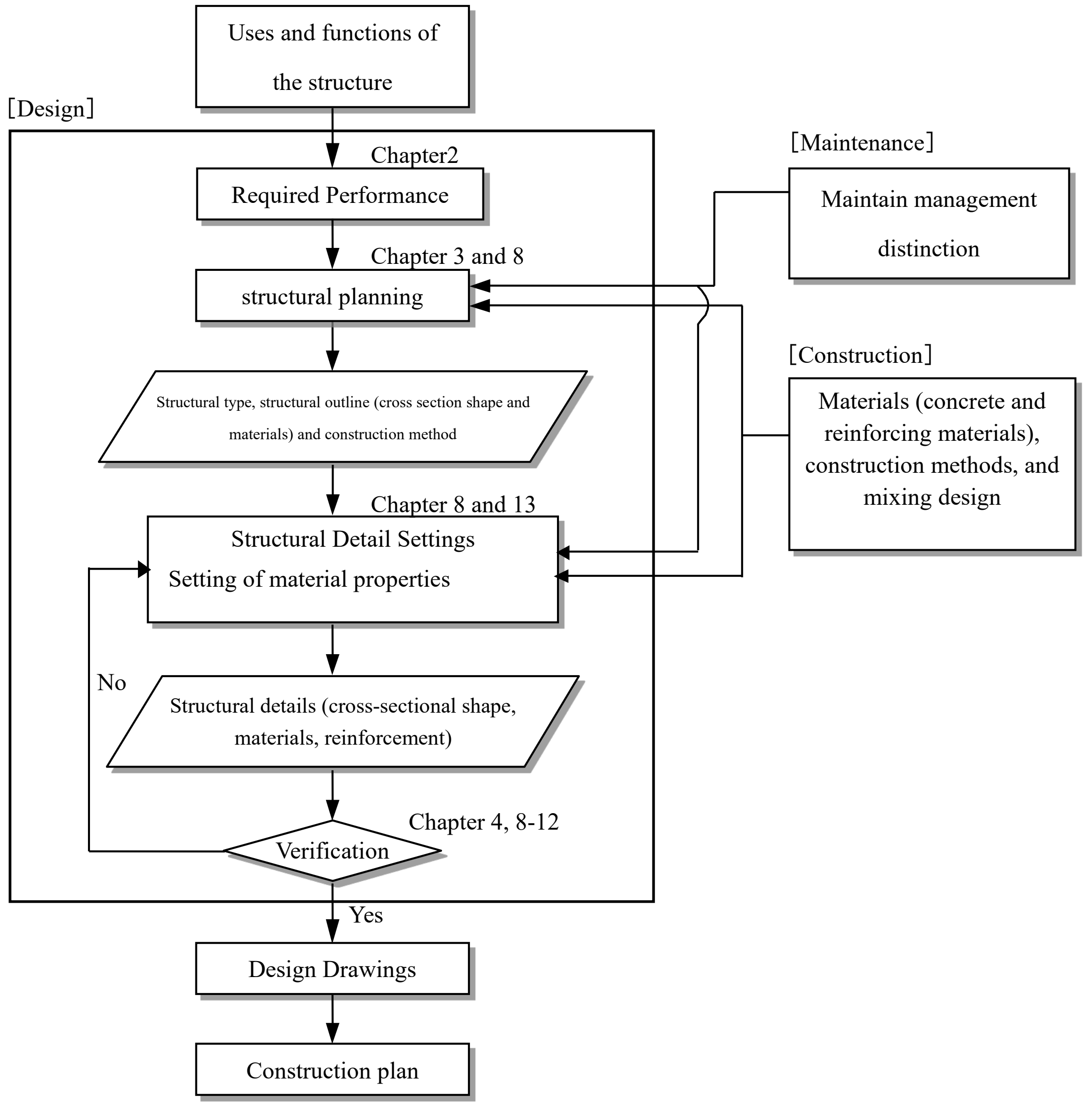
Regarding (3): In structural planning, the structural type and other matters must be determined with consideration of all factors including the structural properties, materials used, construction, maintenance, environmental performance, and economic performance of the structure, so as to satisfy the required performance that was set. Because verification of performance is performed under set conditions, it is necessary to recognize that circumstances exceeding the set conditions may occur. Therefore, under current design structures, it is necessary in the structural planning stage to address events that exceed set conditions. In structural planning, redundancy and robustness with respect to events that exceed the settings of performance verification should be kept in mind to create structures and systems containing structures that possess critical tolerance to avoid sudden catastrophic states. From the standpoint of rationality of design, it is important to conduct structural planning for the prevention of changes, *etc*. to the type of structure to the extent possible, based on the outcomes of required performance verification.

It is also necessary in the structural planning stage to consider matters that cannot be specifically verified in these Standard Specifications.

Regarding (4): In the setting of structural details, information required for verification of the performance of structural member dimensions, reinforcing bar arrangement, materials used, and so on is set for the type of structure that was set in structural planning. Here, it is necessary to appropriately consider structural specifications presented in these Standard Specifications, design manuals, details of similar structures, and information gained through experience. At this stage, structural details must be set with consideration of openings, passages, stairs, and other maintenance facilities used in inspections related to maintenance of the structure. For concrete, it is necessary to roughly set the aggregate, cement, admixtures, water-to-cement ratio, and other aspects of the mixture to determine the properties of the concrete for the purpose of surveying resistance to deterioration, permeation by substances, *etc*.

Regarding (5): Verification of the set required performance is conducted according to these Standard Specifications using information on the structural details that were set. Performance verification is the work of using appropriate methods to confirm that a structure designed with consideration of natural conditions, social conditions, workability, environmental performance, economic performance, and other factors possesses the required performance. "Appropriate methods" here include actual experiments, model experiments, and numerical analysis for which accuracy has been demonstrated. For cases in which the use of these methods is difficult, these Standard Specifications use verification methods based on the limit state design method. In those cases, it is vital that the assumptions for verification indicated in these Standard Specifications are satisfied.

Regarding (6): In general, there are many designs that satisfy required performance. To evaluate the validity of a design with satisfaction of required performance taken into consideration, engineering standards must be established at the beginning of the design stage.



Commentary Figure 1.2.1 Flow of structure design

1.3 Definitions

The following terms are defined for general use in this Specification.

Design – Ac act consisting of the determination of the required performance of a structure, structural planning, structural detailing and performance verification

Required performance – Performance required of a structure according its purpose and function

Verification – The act of checking whether a structure has the required performance, for example by a verification experiment on full-scale specimens or an empirically and theoretically proven analysis method

Durability – The resistance of a structure to performance degradation over time due to the deterioration of materials in the structure caused by expected deteriorating agencies

Safety – The performance of a structure in preventing risks to users and other people in the vicinity

Serviceability – The performance of a structure that enables users to use the structure comfortably or prevents other people in the vicinity from feeling uncomfortable because of the structure, and the performance that ensure other functions required of the structure are performed appropriately

Restorability – Performance that enables functional restoration and continue use of a deteriorated structure

Earthquake resistance – Earthquake-related performance encompassing safety during an earthquake and post-quake serviceability and restorability

Service Life – Period during which a structure is in service

Design life – Period specified in design during which the structure or the structural members shall maintain the service requirements

Verification index – Physically quantifiable index for the required performance

Limit state – The state in which a structure begins to fail to meet the performance requirements

Load – Any action which causes variations in stresses or deformations in structures or members

Permanent load – Load that acts continuously, or is such that the variations in the magnitude are rare or negligible

Variable load – Load that varies frequently or continuously and is such that the variations in the magnitude cannot be neglected compared with the permanent load.

Accidental load – Load that occurs rarely during the design life, but has serious consequences when it occurs.

Design load – Value of load that is obtained by multiplying each characteristic value of load by a corresponding load factor.

Characteristic value of load – Value of load determined based on variations, limit states concerned, and combinations of loads, for all loads during construction period and throughout design life of the structure.

Specified value of load – Value of load specified by other design codes or specifications, apart from the characteristic value of load.

Nominal value of load – Value of load not specified in available relevant codes, but is commonly used in practice.

Characteristic value of material strength – Value of material strength which guarantees that the probability of tested strength being below this value is within the specified limit based on statistical interpretation of test results.

Specified value of material strength – Value of material strength that is specified by other design codes or specifications in a manner other than characteristic value of material strength.

Design basic strength – Basic strength for design. For concrete, the characteristic compressive strength may usually be taken as the design basic strength.

Design strength – Value obtained by dividing the characteristic value of material strength by a material factor.

Material factor – Safety factor to consider the unfavorable deviations of material strengths from the characteristic values, differences in material properties between test specimens and actual structures, effect of material properties on the specific limit states, and time dependent variations of material properties.

Load factor – Safety factor to consider the unfavorable deviations of loads from the characteristic values, uncertainty in evaluation of loads, effect of nature of loads on the limit states, and variations of environmental actions.

Structure factor – safety factor to account for the relative importance of the structure, as determined by the social impact of the structure failing or reaching a particular limit state.

Structural analysis factor – Safety factor to consider the uncertainty of computational accuracy in determination of member forces through structural analysis.

Member factor – Safety factor to consider the uncertainties in computation of capacity of the member, differences in the design and actual size of the member, and the importance of the member which reflects the influence on the overall structure when it reaches a certain limit state.

Load modification factor – Safety factor to transform specified or nominal values of loads into characteristic values of loads.

Material modification factor – Safety factor to transform specified values of material strength into characteristic values of material strength.

Design response value – A value obtained by multiplying a response value under the design load by a structural analysis factor.

Design sectional force – A value obtained by multiplying the sectional force due to a combination of design load by a structural analysis factor and a design response value obtained by using force as a verification index.

Design limit value – A value obtained by dividing the performance of a member or structure calculated by using a material-specific design value by a member factor and a verification limit value set according to the required performance.

Design sectional capacity – A value obtained by dividing the sectional capacity calculated by using the design strength of a material by a member factor and a design limit value obtained by using force as a verification index.

Liner analysis – Structural analysis based on elastic primary theory assuming a linear stress-strain relationships for the materials used, and neglecting secondary effects of deformations.

Primary reinforcement – Reinforcement calculated and arranged to fulfill requirements of each limit state.

Positive moment reinforcement - Primary reinforcement arranged to resist the tensile force due to positive flexural moment.

Negative moment reinforcement – Primary reinforcement arranged to resist the tensile force due to negative flexural moment.

Distribution reinforcement – Reinforcement that is usually placed at right angles to the primary reinforcement for the purpose of appropriate distribution of stresses.

Shear reinforcement – Reinforcement arranged to resist the shear force.

Stirrup – Transverse reinforcement placed perpendicular or nearly perpendicular to the axis of member enclosing positive and / or negative moment reinforcement.

Bent bar – Reinforcement formed by bending up or down positive or negative moment reinforcement.

Tie - Transverse reinforcement enclosing longitudinal reinforcement at given intervals.

Hoop – Within the tie, the reinforcement enclosing longitudinal reinforcement circularly or elliptically.

Intermediate tie – Transverse reinforcement arranged to intersect the cross section.

Spiral reinforcement – Spirally wound continuous transverse reinforcement enclosing longitudinal reinforcement.

Additional reinforcement – Auxiliary reinforcement provided as a precaution against concentrated stress caused by loads, or to guard against cracking due to variation of temperature or shrinkage.

Prestressing steel – High strength steel mainly used for prestressing.

Tendon – Single or a bundle of prestressing steel.

Sheath – Tube to form voids in concrete for accommodation of tendons in pos-tensioned prestressed concrete members.

Anchorage – Device used to anchor tendons in concrete member.

Coupler – Device to connect one tendon with another.

Fretting fatigue – Accumulated fatigue in wire of tendon due to rubbing or pressing by loading in portions where they are in contact with one another.

Pre-tensioning system – Prestressing system in which tendons are tensioned before concrete is placed. Tensile stress in tendon is transferred to hardened concrete by bond between tendon and concrete.

Post-tensioning system – Prestressing system in which tendons are tensioned after concrete has hardened. Tensile stress in tendon is transferred to concrete by anchorage at ends.

Effective depth – Distance measured from extreme compression fiber to centroid of positive or negative moment reinforcement.

Tension reinforcement ratio – Ratio of cross-sectional area of primary tension reinforcement to effective cross-sectional area of concrete. The effective cross-sectional area of concrete is defined as the product of effective depth and flange width.

Compression reinforcement ratio – Ratio of cross-sectional area of primary compression reinforcement to effective cross-sectional area of concrete.

Balanced reinforcement ratio – Tension reinforcement ratio where both tensile stress of primary tension reinforcement and compressive strain of concrete at extreme fiber reach the design yield strength and the ultimate compressive strain simultaneously.

Development length of reinforcement – Length of embedded reinforcement required to develop the design tensile stress of reinforcement at the critical section.

Clear distance – Face to face distance between adjacent reinforcing bars, tendons or sheaths.

Concrete cover – Minimum thickness of concrete between concrete surface and surface of reinforcing bars, tendons or sheaths.

Clear span – Distance from face to face of supports for beams or slabs.

One-way slab – Rectangular slab supported by two opposite sides.

Two-way slab – Rectangular slab supported by four sides.

Deep beam – Beam of which depth is relatively greater than a span length.

Corbel – Cantilever attached to column with span to depth ratio being less than or equal to one.

Column – Member oriented vertical or nearly vertical, and the length of which is greater than three times of the least transverse dimension.

Beam element model – A model where a structure is represented as made up of a combination of one-dimensional beam elements.

Finite element model – A model where a structure is represented as made up of a combination of two or three-dimensional finite elements.

Nonlinear hysteric model – A model that gives nonlinear hysteresis in the stress-strain relationship of constitutive materials, or in the load-displacement relationship of a member or a structure under revered cyclic loading.

1.4 Symbols

In these Standard Specifications, symbols used in design calculations for structures are as follows.

A : cross-sectional area

A_a : area subject to bearing pressure

A_c : cross-sectional area of concrete

A_m : torsional effective cross-sectional area

A_s : cross-sectional area of arranged rebar, or cross-sectional area of tensile-side steel

A_{sc} : cross-sectional area of rebar required for calculation

A_{tl} : cross-sectional area of longitudinal rebar that effectively acts as a torsional reinforcing bar

A_{tw} : cross-sectional area of single transverse rebar that effectively acts as a torsional reinforcing bar

Aw : cross-sectional area of single shear reinforcing bar

a_v : distance from loading point to bearing frontal face

b : width of structural member

b_e : effective width

b_0 : length of short leg of transverse rebar

b_w : width of structural member web

C'_d : design diagonal compressive force per unit width acting on concrete

c : concrete cover

Δc_e : construction error in cover

cs : center-to-center distance of steel

d : effective depth

d_0 : diameter of concrete cross section enclosed by a transverse rebar if it has a circular cross section, or the length of the long leg of a transverse rebar if it has a rectangular cross section

E_c : Young's modulus of concrete

E_p : Young's modulus of PC steel

E_s : Young's modulus of rebar

F : action

F_k : characteristic value of action

F_n : specification value of action

F_p : permanent action

F_r : variable action

f : material strength

f'_a : bearing pressure strength of concrete

f_b : flexural strength of concrete, flexural cracking strength

f_{bo} : bond strength between concrete and rebar

f'_c : compressive strength of concrete

f'_{ck} : characteristic value of compressive strength of concrete, design basic strength
f_k : characteristic value of material strength
f_{ly} : yield strength of longitudinal torsional reinforcing bars
f_n : specification value of material strength
f_{pu} : tensile strength of PC steel
f_{py} : yield strength of PC steel
f_r : fatigue strength
f_t : tensile strength of concrete
f_u : tensile strength of steel
f_{vy} : shear yield strength of steel
f_{wy} : yield strength of shear reinforcing bars or transverse torsional reinforcing bars
f_y : tensile yield strength of steel
f'_y : compressive yield strength of steel
h : depth of cross section
I_e : converted second moment of area
I_g : cross-sectional second moment of gross cross-sectional area
K_t : torsion coefficient
k_c : coefficient to express effects of cover and transverse rebar on anchoring of rebar
l_o : development length of rebar
l_d : basic development length of rebar
l_s : length of shift of calculated position of development length of longitudinal tensile rebar of flexural structural members in the direction of smaller flexural moment
M : flexural moment
M_{cr} : flexural moment of limit at which cracking occurs in cross section
M_t : torsional moment
M_{tc} : pure torsional capacity when no torsional reinforcing bars are present
M_{tcu} : diagonal compression fracture load capacity with respect to torsion in web concrete
M_{tu} : torsional capacity
M_{ty} : torsional capacity determined by yield of torsional reinforcing bars
M_u : flexural capacity
N : fatigue life, or equivalent number of cycles of fatigue action
N' : longitudinal compressive force
N_1, N_2 : main in-plane forces acting on plane members, where N_1 is tensile force, not less than N_2
P_e : effective tensioning force in tendon
p : ratio of tensile rebar
p' : ratio of compressive rebar
p_w : ratio of shear reinforcing bars
R : limit value or cross-sectional load capacity
R_r : fatigue capacity

r : inside radius of bend
S : response value or cross-sectional force
S_p : cross-sectional force due to permanent action
S_r : cross-sectional force due to variable action
s : spacing of shear reinforcing bars, torsional reinforcing bars, or transverse rebar
T_c : total tensile force occurring in concrete
T_x : tensile force acting on rebar in x direction, per unit width of structural member
T_y : tensile force acting on rebar in y direction, per unit width of structural member
u : peripheral length of rebar cross section or of loaded surface
u_p : effective peripheral length for resisting punching shear in slab, *i.e.*, πd added to peripheral length of loaded area of concentrated action or concentrated reaction (where d is effective depth)
V : shear force
V_c : shear capacity of structural member when no shear reinforcement steel is present
V_{cw} : shear transfer load capacity at shear plane
V_h : parallel component of shear force caused by change in structural member depth
V_p : shear force due to permanent action
V_{pc} : punching shear capacity of plane member
V_{pe} : parallel component of shear force of effective tensile force in longitudinal tendon
V_r : shear force due to variable action
V_{rp} : punching shear fatigue capacity of plane member when no shear reinforcing bars are present
V_s : shear capacity provided by shear reinforcement steel
V_{wc} : diagonal compression fracture load capacity with respect to shear in web concrete
V_y : shear capacity
w : crack width
w_a : limit value of crack width
z : distance from resultant compressive stress to the centroid of tensile steel cross section
α : coefficient to express effects of concrete cover and transverse rebar on anchoring of rebar
α_c : angle between compressive edge and axis of a structural member
α_p : angle between tendon and structural member axis
α_s : angle between shear reinforcing bars and structural member axis
α_t : angle between tensile steel and structural member axis
β_d : coefficient for effective depth of shear capacity
β_n : coefficient for axial force of shear capacity
β_{nt} : coefficient for axial force of torsional capacity
β_p : coefficient for longitudinal rebar ratio of shear capacity
γ : rate of apparent relaxation of PC steel
γa : structural analysis coefficient
γ_b : structural member coefficient
γc : material coefficient of concrete

γ_f : action coefficient

γi : structure coefficient

γ_m : material coefficient

γ_s : material coefficient of steel

δ : coefficient of variation

δ_y : yield displacement

ε'_c : compressive strain of concrete

ε'_{cc} : compressive creep strain of concrete

ε'_{cu} : ultimate compressive strain of concrete

ε'_{cs} : shrinkage strain of concrete

ε'_{csd}: value of compressive strain for evaluation of increase in crack width due to shrinkage and creep in concrete

ρ_f : action correction coefficient

ρ_m : material correction coefficient

σ : standard deviation

σ'_{cp} : compressive stress intensity in concrete due to permanent action

σ'_n : average compressive force intensity due to longitudinal compressive force

σ_{pe} : amount of increase in PC steel stress intensity for examination of crack width

σ_{pp} : amount of increase in PC steel stress intensity due to permanent action

σ_{pw} : tensile stress intensity in tendons for shear reinforcement during yielding of shear reinforcing bar

σ_r : variable stress intensity

σ_{se} : amount of increase in rebar stress intensity for examination of crack width

σ_{sp} : amount of increase in rebar stress intensity due to permanent action

σ_w : stress intensity in shear reinforcing bars

σ_{wpe}: effective tensile stress intensity in tendons for shear reinforcement

σ_{wr} : stress intensity in shear reinforcing bars due to variable action

σ_x : normal stress intensity

σ_y : stress intensity in direction orthogonal to that of σ_x

σ_I : diagonal tensile stress intensity

τ : shear stress intensity due to shear force or torsional moment

ϕ : diameter of steel, diameter of duct, nominal diameter of rebar, creep coefficient of concrete

Commentary: Because using identical symbols with different meanings creates confusion, symbols should be unified to the extent possible. However, because listing all symbols would be complicated, only commonly used symbols are shown. Identical symbols with different meanings, or symbols not shown in this section, may be used in some locations, but explanations are given in the respective sections.

The meanings of the main symbols are as follows:

A : cross-sectional area

b : width

c : concrete cover

d : effective depth

E : Young's modulus

F : action

f :material strength

I : second moment of area
l :span, development length
M : moment
N : cycle, axial force
P : tensioning force in tendon
s : spacing
u : peripheral length
V : shear force
w : crack width
x : distance from support
α : angle with structural member axis
β : coefficient for shear capacity
γ : safety coefficient, rate of relaxation
δ : coefficient of variation, displacement
ε : strain
ρ : correction coefficient
σ : stress intensity
p : ratio of rebar
R : limit value or sectional load capacity
ϕ : diameter, creep coefficient
S : response value or sectional force

Subscripts have the following meanings:

a : bearing pressure, structural analysis
b : structural member, equilibrium, flexing
bo : bond
c : concrete, compression, creep
cr : crack
d : design value
e : effective, transformed
f : action
g : gross cross section
k : characteristic value
l : longitudinal
m : material, mean
n :specification value, standard, longitudinal
p :prestress, PC steel, permanent, punching
r : variation
s : steel, rebar
t : tension, torsion, transverse
u : ultimate
v : shear
w : structural member web
y : yield

When the characteristic value of an action or material property is meant, the subscript k is added. When the design value of a response (cross-sectional force, *etc*.) or a limit value (cross-sectional yield strength, *etc*.) is meant, the subscript d is added. However, when the distinction between characteristic value and design value is clear, the subscripts k and d are omitted. Stress intensity and strain are positive for tension and negative for compression. However, when a prime symbol ( ) is added to the upper right of the symbol, this indicates compression, and compression is set to positive.

Chapter 2 Required Performance

2.1 General

(1) The design service life of a structure is to be decided with consideration of the service period, maintenance method, durability, environmental performance, economic performance, and other factors required for the structure.
(2) The required performance of a structure is to be set to fulfill the purpose and function of the structure during its construction and design service life. The durability, safety, usability, and restorability are generally to be set within the required performance.

Commentary: Regarding (1): When designing a structure, its design service life must be set in advance. Setting a long design service life generally demands that the structure have high durability.

Regarding (2): The required performance, which is set to fulfill the purpose and function of the structure, must be satisfied in full during the design service life.

As discussed here, the "required performance" of a structure refers to the performance of the structure as a whole. In general, the performance of the structure bears a close relationship to the performance of the structural elements that compose the structure. Therefore, to achieve the required performance for the structure as a whole, it is necessary to give full consideration to its relationship with the performance of individual structural elements, and to set their performance accordingly.

2.2 Durability

"Durability" refers to performance by which a structure maintains safety, usability, and restorability over its design service life.

Commentary: Durability in a structure is the assurance of safety, usability, restorability, and other elements of required performance over the entirety of the design service life. Therefore, the structure must possess resistance to changes in performance over time. Toward that end, it is necessary to evaluate safety, usability, restorability, and other elements of performance as a function of time, taking into consideration changes in performance over time. At present, however, performing a detailed evaluation of change in performance over time is difficult, and not always practical in the design stage. Therefore, these Standard Specifications generally conduct verification of the safety, usability, restorability, and other elements of required performance of the structure over the design service life, after ensuring factors including resistance to permeation by substances and resistance to deterioration of materials due to environmental actions. Doing this indirectly guarantees that the structure will satisfy all elements of required performance over the design service life. However, with respect to increased displacement/deformation, decreased yield strength, and other changes in performance over time due to recurring actions originating in variable actions or other external forces, the decision was made to consider resistance to these changes in terms of safety and usability.

2.3 Safety

"Safety" refers to performance that prevents a structure from posing a threat to the lives and property of users and surrounding persons under all envisioned situations.

Commentary: Safety in a structure is performance determined by the mechanics of the structure, including failure and collapse due to variable actions and accidental actions such as earthquakes. Performance prevents events that threaten the lives or property of users, third parties, etc., such as falls of ancillary equipment, peeling of covering, or leakage of hazardous content from containers as a result of damage.

2.4 Usability

"Usability" refers to performance that enables a structure to be used normally under expected actions during normal use.

Commentary: Usability is performance that enables the normal use of a structure, as well as its comfortable use, and its performance with respect to varied functions under normal conditions.

For "comfortable use," factors such as ease of driving or walking, external appearance, noise, and vibration can generally be set. For "performance with respect to varied functions," water-tightness, permeability, soundproofing, moisture resistance, cold resistance, heat resistance, and other material shielding or permeability factors, and performance that prevents unsuitability for use owing to damage caused by variable actions, environmental actions, accidental actions, and other causes can generally be set.

2.5 Restorability

"Restorability" refers to performance that enables continuous use of a structure through restoration of its performance when it has deteriorated as a result of accidental actions, *etc.*, such as earthquakes.

Commentary: Restorability is performance that indicates ease of recovery when the performance of a structure has deteriorated as a result of accidental action, *etc.*, such as earthquakes. Civil engineering structures are generally of a highly public nature; as such, trouble-free assurance and maintenance of the purposes and functions of them has a considerable impact on people's lifestyles and on community and industrial activities. Restorability greatly depends on not only the ease of repairing damage to a structure, *i.e.*, the repairability of the structure, but also the presence or absence of tangible aspects such as the ease of inspection following a disaster, the ability to secure materials for restoration, and the enhancement of restoration technologies, along with intangible aspects such as systems for restoration of structures. In these Standard Specifications, it was decided to set mechanically required performance with respect to the repairability of concrete structures based on the assumption that factors included in restorability other than repairability will be considered separately. Earthquakes, wind, fire, *etc.* are envisioned as accidental actions that cause damage to structures, but it was decided that, in general, the effects of earthquakes should be examined. For the repairability of concrete structures, the level of required performance should generally be set in accordance with the scale of actions, keeping in mind situations in which use is possible without repair, and situations that require repair to an extent that enables restoration of use and of functions in a short period of time.

2.6 Environmental performance

"Environmental performance" refers to performance related to compatibility with the natural environment and social environment.

Commentary: When planning a concrete structure, for its environmental performance, it is necessary to take into consideration its compatibility with the global environment, local environment, work environment, etc., along with its compatibility with the landscape and other elements of the social environment, as well as its other impacts on the environment.

With regard to consideration for the environment, there are cases in which standard values and target values for items stipulated by laws and regulations, items required by the ordering party, and other items are set as limit values and are subjected to verification, and cases in which verification cannot be performed for items owing to insufficient information at the current stage. It is necessary to give appropriate consideration to these in each of the design, construction, and maintenance stages, after clarifying the items in the planning stage. Concepts regarding this consideration are described in "Basic Principles".

The effects of the environment on a structure are considered as environmental actions in the durability of the structure.

Chapter 3 Structural Planning

3.1 General

(1) In structural planning, it is necessary to define the type and form of a structure so that, after estimation of its behavior over its design service life and estimation of changes in its future environment, usage, and functions, with the environment of the construction site also taken into account, it will maintain its required performance and possess redundancy and robustness.

(2) In structural planning, it is necessary to set design service life, type of structure, materials used, and primary dimensions so as to meet the required performance of the structure over the service period.

(3) In structural planning, it is necessary to examine the construction work, maintenance, environmental performance, economic performance, and other aspects of the structure.

(4) In order to draft a structural plan, it is necessary to conduct required surveys in accordance with conditions at the planned construction site, the scale of the structure, and other factors.

Commentary: These Standard Specifications constitute a system for setting structural form, design service life, materials to be used, and primary dimensions in the planning stage after the setting of required performance to guarantee the use and functions of the structure and to verify these to confirm that the structure's performance satisfies the required performance. Therefore, the structural planning stipulated in this section is defined as the stage in which structural form, design service life, materials to be used, and primary dimensions are determined after the required performance of the structure has been determined.

When it is not possible to achieve the required performance of the structure using conventional methods, it is necessary to use special concretes and new technologies.

Compared to the cast-in-place method, the precast method shortens the construction period, reduces on-site labor and personnel requirements, and reduces quality control and inspection requirements. It offers advantages including enhancement of quality through the ability to produce structural members in a stable environment, mitigation of environmental impact, as well as mitigation of the impact of the construction site on the surrounding environment, and improvement of safety at construction sites.

In examining structures that are made with precast concrete or that otherwise differ from conventional structures, it is important to carry out such examinations from the structural planning stage.

Regarding (1): Structural planning, along with planning, design, construction work, and maintenance, is one of the most important actions in determining the significance of a civil engineering structure as an item of social capital and the contribution it makes to uses and functions demanded by the community. Verification of performance is performed under set conditions to demonstrate the presence or absence of performance under those conditions. It is accordingly necessary to recognize that, in reality, events may occur that exceed the set conditions. For that reason, the current design system requires that events that exceed the set conditions be addressed in the structural planning stage. Structural planning must bear in mind that structures are to be imbued with redundancy and robustness so that they and the systems that encompass them do not suddenly reach catastrophic states even in the case of events that exceed the settings of performance verification. In particular, it is necessary to fully examine the effects of earthquakes and associated phenomena such as tsunamis, rising seawater levels caused by climate change, and other accidental actions and changes in the natural environment caused by natural phenomena. Specifically, addressing events that exceed set conditions requires analysis of situations that can arise when actions occur that exceed settings, and the enactment of measures to minimize degree of impact when said situations do arise.

Regarding (2) and (3): In determining form, materials, and primary dimensions of a structure, the construction method, maintenance method, environmental performance, economic performance, and other aspects must be comprehensively examined so that the required performance (i.e., durability, safety, usability, and recoverability) stipulated in these Standard Specifications is satisfied. In particular, it can be said without exaggeration that not only the costs required for construction but also the costs of future maintenance are roughly determined in structural planning. Therefore, it is necessary to conduct examination with full consideration of future maintenance as well.

Regarding (4): A variety of surveys are required to plan, design, construct, and maintain a structure. Those required in the structural planning stage are conducted in accordance with conditions at the planned construction site, the scale of the structure, and other factors. Caution is required as the form or other aspects of the structure may be unsuited to local conditions, or significant changes to the plan may occur, if such surveys are inadequate.

3.2 Examination related to performance

In structural planning, the design service life, type of structure, materials to be used, and primary dimensions of a structure must be comprehensively examined, taking into account the structure's durability, safety, usability, and recoverability, as well as its construction, maintenance, environmental performance, and economic performance.

Commentary: In structural planning, it is necessary to examine structural and material properties and set the type of structure so as to satisfy the specified required performance. Within the aspects of required performance, environmental performance is one for which the impact of the structure on not only the natural environment but also on people's lives must be considered. As environmental performance, like economic performance, must be made to conform to societal demands, it is not always appropriate to treat it in the same manner as an aspect of performance determined by physical characteristics. For that reason, environmental performance, like economic performance, should be taken into account as an engineering value standard, *etc.*

With regard to seismic action, performance should be set and examination should be conducted with safety and usability of the structure during and after an earthquake, and repairability of the structure after an earthquake, taken comprehensively into account, as appropriate for the expected scale of earthquakes. In these Standard

Specifications, this performance is defined as seismic performance.

(i) Regarding durability: Based on Section 8, it is verified whether the deterioration of a structure is within a range that does not cause defects with respect to required performance during the design service life, and whether the structure possesses sufficient resistance to permeation of substances. Passing this verification is necessary in maintaining required safety, usability, and resiliency over the design service life. For that reason, in the structural planning stage, it is necessary to determine the material properties of the concrete and steel materials, while at the same time determining the cross-sectional form, arrangement of reinforcement, and other details of the structure. For concrete, the materials to be used (aggregate, cement, and admixtures) and the water–cement ratio must be examined and set.

In Section 8, verification related to the durability of the structure is performed, using the covering, stress intensity of the steel materials, concrete properties related to resistance to deterioration and permeation by substances, *etc.* When a structure is to be built in an environment with severe corrosion of steel materials or severe chemical erosion, a more detailed examination of durability must be performed if it is assumed in the structural planning stage that it will be difficult to set appropriate covering or ensure performance related to chemical erosion through the resistance of the concrete alone, and that countermeasures will entail considerable cost.

(ii) Regarding safety: Examination of safety can be carried out in the structural planning stage with reference to past cases when there is an adequate record of construction for the form and scale of the structure. In this case, if the scale of existing structures, the topographical and geological conditions of their construction sites, and other factors differ significantly from those of the structure in question, ensuring the safety of the structure in the verification stage will necessitate changes in its form, shape, and dimensions that were set in the structural planning stage, possibly entailing major changes to plans. Therefore, the preconditions for existing structures must be adequately confirmed before they can be applied.

If there is little or no record of construction for the form and scale of the structure in question, then performing a rough examination of safety is advisable to avoid changes to the type of structure and primary dimensions in the verification stage.

To further improve the safety of a structure, it is advisable to give it a structure such that the structure as a whole will not collapse even if some structural members reach their limit state for cross-sectional failure. Such properties are called redundancy and robustness. As an example of these, a statically indeterminate structure is a type of structure with redundancy and robustness higher than those of a statically determinate structure.

For concrete structures built in a tsunami inundation area, a large force will be applied when a tsunami strikes, carrying the risk of damage to or movement of the structure, or outflow of items from within it. If the tsunami action is significantly large, then resistance may not be possible even with changes to the structural member's dimensions or arrangement of steel materials. This should be addressed by appropriately setting the type of structure, the primary dimensions, and other aspects in the structural planning stage.

In Section 11, mechanical limit states for concrete structures are set and verified, primarily addressing the repairability of concrete structures. These limit states are set on the assumption of physical and non-physical aspects of response following a disaster. Therefore, in structural planning, both physical and non-physical aspects of the restoration of civil engineering structures must be taken into account and examined.

As the repairability of a structure is affected by its type, it is necessary to envision the environment and other factors of restoration actions at the construction site, and to examine a type of structure that makes repair and restoration of usability and function as easy as possible when the structure has been damaged.

For a given type of structure, it is likely that the time and construction expenditures needed for repair in the event of damage will differ greatly with not only the magnitude of the damage but also with the parts that were damaged. Therefore, it is advisable to identify locations of predicted damage and make these amenable to inspection and repair work.

3.3 Examination related to construction work

Constraints related to construction work must be taken into consideration in structural planning.

Commentary: For a structure to exhibit the use and functions required of it and to maintain required performance, it must be constructed so as to satisfy the conditions shown in design drawings. For that reason, structural planning must be carried out with adequate consideration of constraints related to construction work.

In general, when constraints are placed on the construction period and on on-site conditions, structures made using precast structural members have advantages over structures for which concrete is made on-site, as the on-site work will be limited by erection and joining. Moreover, standardizing the forms of structural members in the structural planning stage allows reduction in the cost of molds by increasing the number of times molds are used, and reduction of on-site labor through simplification, automation, and mechanization in production and construction work.

However, it is advisable to examine the weight and the lengths of structural members with constraints on transport and erection taken into account, and to examine the structure to ensure the integrity of the structure at the joints. In structures made with precast structural members, it is necessary to ensure performance not only upon completion but also during construction, and to verify performance with consideration of the effects that the stress state of structural members during construction will have on the performance of the structure after completion.

In some cases, the type of structure is determined by the construction method. As an example, a PC box girder form employing cantilevered erection is considered for bridges built over gorges in mountainous areas when timbering is topographically difficult, while a PC plate girder form employing movable timbering construction is considered for bridges built over even ground.

In this way, when determining the form based on topographical conditions at the construction site or other constraints, or when adopting such a form, the use of special concrete or specific work methods for molds and reinforcing bars may become attached as conditions. Therefore, structural planning must be proposed with adequate consideration of constraints on construction work.

3.4 Examination related to maintenance

In structural planning, the importance of the structure, its design service life, the service conditions, environmental conditions, the difficulty of maintenance, and other matters must be taken into consideration to facilitate maintenance during service.

Commentary: As shown in "Maintenance," in addition to the initial inspections carried out in the stage when maintenance is begun, numerous inspections must be carried out for structures currently in service, including daily and periodic inspections carried out regularly and provisional inspections carried out following accidental action due to disasters or the like. These may entail considerable cost and labor, depending on the inspection methods. It is also expected that repair, reinforcement, renewal, and other measures will be implemented according to the degree of deterioration in aspects of performance in an in-service structure. The costs required for these measures in such cases can be significant, depending on the degree of degradation in performance. Therefore, in structural planning, it is advisable to examine the type of structure and the materials to be used so that maintenance work during service can be performed efficiently with the costs of countermeasures reduced to the extent possible.

Making a clear maintenance plan for the structure in the structural planning stage allows such information to be properly incorporated into the maintenance plans formulated in the maintenance stage, contributing to the success of rational maintenance. Therefore, it is advisable to examine maintenance planning in the structural planning stage.

3.5 Examination related to environmental performance

In structural planning, the impacts of concrete structures on the environment, including natural and social environments, and the impact on people's lives must be considered.

Commentary: In structural planning, it is necessary to take into consideration the impact of concrete structures on the environment, including natural and social environments.

Concrete structures can have impacts on natural, social, and other environments in every stage, including during the manufacturing of component materials, erection of structures, service, and maintenance. In each stage, it is necessary to take note of and examine the elements that have effects on environments.

Impacts on the natural environment may be divided into the global and local environments. Impacts that must be examined include those caused by resource and energy consumption and by emissions of greenhouse gases, atmospheric pollutants, water pollutants, soil contaminants, and waste. Some of these are regulated by law, in which cases the regulated values are set as limit values and are verified to ensure compatibility with the environment.

It is also necessary to consider the impacts of structures on natural landscapes. A civil engineering structure is a form of social capital that can exist for a long period after construction; its very presence has the ability to affect the landscape. As the landscape is considerably dependent in part on the type of structure, a proper examination is necessary in the structural planning stage so that the surrounding natural landscape, including the structure, is not obstructed. Toward that end, the impacts caused by construction of a structure on the nearby natural landscape must be assessed in advance through models and composite photographs, and the structure must be planned with the aim of harmonizing with the surrounding natural environment after completion.

There is also a need to consider problems that could occur in the local environment around the construction site and in the work environment of construction workers, owing to noise and vibration during construction and other stages of a concrete structure.

Even in the maintenance stage of a concrete structure, formulating an appropriate maintenance plan and implementing maintenance in accordance with the purpose, function, and importance of the structure can of course extend its life and reduce its environmental load. In selecting materials and work methods for repairs and reinforcements to be performed in the maintenance stage, it is also important to consider environmental performance.

With regard to environmental conservation, it is expected to comply with relevant laws in order to contribute to the welfare of humankind and to the health and culture of both current and future generations. Within a framework of comprehensively evaluating environmental load, however, it is necessary to consider all factors that have impacts on the environment. Therefore, it is important to not only examine these legal structures but also consider the impacts on the global environment, local environment, work environment, and landscape at every stage during the design service life of the structure.

3.6 Examination related to economic performance

In structural planning, economic performance must be examined from the standpoint of the lifecycle cost of the structure.

Commentary: In structural planning, selecting a type of structure, structural member dimensions, materials, and other aspects that excel in economic performance is a matter of great importance. As economic performance is determined nearly entirely in the structural planning stage, a proper examination is needed.

A structure requires regular maintenance. If it is no longer able to meet required performance, then repairs, reinforcement, or renewal become necessary, thus incurring necessary management costs from the initial construction stage onward. Depending on constraints on construction work, renewal of a structure can often entail costs exceeding the initial cost of construction. For this reason, in the structural planning stage, it is important to perform not only an evaluation based on initial construction costs but also an evaluation that takes into consideration predictions of maintenance and renewal costs from the start of the plan into the future.

Chapter 4 Principles of Performance Verification

4.1 General

(1) As a general rule, in the performance verification of structures, limit states appropriate to the required performance are set for all structures and structural component members during construction and during the design service life. It is confirmed that structures and structural members having forms, dimensions, reinforcement arrangements, and other structural details assumed in the design do not reach the limit states.

(2) Limit states are generally set for durability, safety, usability, and recoverability.

(3) In performance verification of the structure, the effects of change over time due to environmental action on the performance of the structure may be ignored if Sections 8 and 12 are satisfied.

(4) The verification of limit states in the structure is performed in principle by setting appropriate verification metrics and comparing their limit values with response values.

Commentary: Regarding (1): These Standard Specifications adopt the principle of first setting multiple aspects of required performance to be conferred on the structure, and specifying the equivalent limit state that corresponds to each. When a structure or a part of it reaches the state known as the limit state, usability may deteriorate sharply or failure may occur in some cases. In this state, the structure no longer fulfills its uses and functions, various inconveniences arise, and required performance is no longer satisfied. In such cases, examination of limit states can replace performance verification of the structure. When setting limit states, metrics related to the state of the structure and structural members and the state of materials are selected, and limit values are assigned in accordance with required performance. The limit values are to be set taking into account the reliability of models and the analytical methods used to calculate response values. This concept should also be applied to ancillary equipment as necessary. If the structural system is composed of multiple structural elements and is not simple, then processes that could lead to the structural system lacking the required performance are to be identified and performance is to be set for each element, after which limit states may be set for structural members.

These Standard Specifications stipulate performance verification methods for reinforced concrete structures and prestressed concrete structures. Even when structures are configured using structures different from these, it is necessary to set limit states in accordance with required performance and to perform performance verification appropriate to the structure, following the key points of these specifications.

Regarding (2): When comprehensively considering safety during earthquakes and usability and recoverability afterward, corresponding appropriate limit states are to be set.

For safety, a limit state for the failure or collapse of the structure should generally be set. Other limit states for safety must be set separately in accordance with the use and functions of the structure. These include required performance related to the safety of use of the structure, such as safety of driving and walking, falls of ancillary equipment, peeling of concrete covering, and other public accidents affecting third parties and originating in the structure.

・Load capacity of the structure

The performance of the structure as a whole with respect to failure is closely related to the state of each of its constituent structural members. Regarding this safety, if the structure is composed of multiple structural members, then it is sufficient to verify that the structure as a whole will not fail even if some structural members undergo failure. To be on the safe side in performing verification of the failure of the structure, Section 9 of these Standard Specifications describes a specific verification method for use in cases in which this is defined as any structural member reaching failure. With regard to the load-bearing capacity of the structure, the following limit states should generally be set.

Cross-sectional failure: A limit state that expresses the ability of a structure to maintain its load-bearing capacity against all actions that occur during its design service life.

Fatigue failure: A limit state that expresses the ability of a structure to maintain its load-bearing capacity against all recurring variable actions that occur during its design service life.

・Stability of the structure

A limit state that expresses the ability to maintain a state by which a structure does not become unstable as a result of displacement, deformation, deformation of mechanisms or the foundation structure, etc. with respect to all actions that occur during the design service life.

With regard to usability, the following limit states should generally be set according to comfort in use and to other uses and functions of the structure, as performance that allows normal use of the structure. The limit values for these limit states are to be set in accordance with the purpose and functions of the structure.

・External appearance: A limit state for preventing cracking in concrete, surface staining, etc. from creating anxiety or discomfort and from hindering use of the structure.

・Noise/vibration: A limit state for preventing noise and vibration generated by a structure from interfering with its use and adversely affecting the surrounding environment.

・Ease of driving/walking: A limit state by which vehicles and pedestrians can comfortably drive and walk.

・Watertightness: A limit state for preventing impairment of use and function due to water leakage, water permeation, or moisture permeation in a concrete structure that requires watertightness.

・Damage: A limit state for preventing a structure from falling into a state of unsuitability for use due to the occurrence of damage caused by variable actions, environmental actions, etc.

With regard to recoverability, limit states should be set based on physical characteristics concerning the repairability of the structure.

The relationships between design actions to be considered, limit states, and verification metrics with respect to required performance specified in these Standard Specifications are shown in Commentary Table 4.1.1.

Limit states must be set in accordance with the uncertainty of actions and inconveniences such as the deterioration of performance that occur when a structure's response values exceed the limit states.

Commentary Table 4.1.1 Examples of required performances, limit states, verification metrics, and design actions.

Required performance	Limit state	Verification metrics	Design actions to consider
Safety	Cross-sectional failure	Power	All actions (maximum values)
	Fatigue failure	Stress intensity / Power	Recurring actions
	Displacement deformation / Mechanism	Deformation / Deformation due to foundation structure	All actions (maximum values) / Accidental actions
Usability	External appearance	Crack width, stress intensity	Actions of relatively frequently occurring magnitude
	Noise / Vibration	Noise / Vibration level	Actions of relatively frequently occurring magnitude
	Comfortability of driving, *etc.*	Displacement / Deformation	Actions of relatively frequently occurring magnitude
	Watertightness	Amount of water penetration of structure Crack width	Actions of relatively frequently occurring magnitude
	Damage (maintenance of functions)	Power / Deformation, *etc.*	Variable actions, *etc.*
Restorability	Repairability	Power / Deformation, *etc.*	Accidental actions (effects of earthquakes, *etc.*)

Regarding (3): In principle, verification of the required performance of a structure must consider the changes in performance that occur over the design service life. In these Standard Specifications, however, if the verification of durability in Section 8 and the verification of initial cracking in Section 12 are satisfied, then the safety, usability, and restorability indicated in these Standard Specifications can be verified without considering the deterioration of constituent materials during the design service life. Here, it is necessary to anticipate a degree of safety by assuming specifications of structural members, the preciseness of the arrangement of reinforcement, changes in the mechanical properties of materials used, and so on.

Regarding (4): When rationally conducting verification of the performance of a structure, the general principle is to compare limit values and response values using verification metrics that directly express the limit states as closely as possible.

4.2 Assumptions in verification

Performance verification of structures under these Standard Specifications assumes structural particulars that form preconditions for the verification method set forth in this volume and other structural particulars, construction methods, and concrete workability according to "Construction" and the implementation of maintenance set out in "Maintenance."

Commentary: The standard performance verification method indicated in these Standard Specifications is grounded in mechanical theories of structures and materials, etc. Most of these, however, make assumptions about the integration of concrete and reinforcing bars, localized stress states, etc. Under conditions for which these assumptions do not hold, the accuracy and scope of applicability of the verification method decline. For this reason, Section 13 stipulates that preconditions for the verification method are to be ensured. Should the scope of applicability expand in the future, the number of items that must be specified for the particulars and structural particulars that form preconditions for the verification method will decrease. Moreover, in these Standard Specifications, the safety margin is taken into consideration as a premise for the workability of concrete and construction methods indicated in "Construction." To ensure that settings for actual construction methods, filling performance, pumping performance, hardening properties, and other workability aspects of concrete, along with strength during construction, will equal or exceed the concrete properties set out in these Standard Specifications, it is presupposed in "Construction" that a construction plan will be formulated and that selection of materials will be carried out. With regard to maintenance, too, it is presupposed that the maintenance set out in "Maintenance" will be implemented so as to maintain the required performance set in "Design".

4.3 Verification methods

(1) Performance verification follows the basic principle of using mathematical models based on mechanical mechanisms of materials and structures or on based on experiments, *etc.* In cases of abundant past records and experience, quantitatively verified yield strength equations and empirical rules should be used.

(2) Verification of limit states is to be performed based on the methods specified in Sections 8, 9, 11, 12, and 19 with response values calculated from the methods specified in Section 7 using characteristic values of materials and actions and the safety coefficient specified in Section 4.5.

(3) Verification is generally performed according to:

$$\gamma_i \cdot S_d / R_d \leq 1.0, \qquad (4.3.1)$$

where: S_d: design response value

R_d: design limit value; and

γi: structural coefficient, per Section 4.5.

(4) Verification through experiments is to be performed based on the results of loading experiments using specimens that model the actual structure. In this case, however, an appropriate safety coefficient must be set with consideration of differences between the conditions of the experiments and the conditions of the actual structure, and verification of the actual structure must be carried out with analytical models and analytical methods both applied.

Commentary: Regarding (1): A variety of verification methods exist. These Standard Specifications follow the principle of using empirical results from experiments or mathematical models based on the mechanical mechanisms of materials and structures. Methods based on mechanics have general versatility and do not particularly limit the targets of verification. In these Standard Specifications, bending theory based on plane conservation, numerical analysis methods for shear yield strength verification, and so on fall into this category. Conversely, for verification subjects that were designed based on experience, it was decided that methods that have been comprehensively validated through experience may be used. The yield strength calculation equations and empirical rules specified in the standards, etc. of these Standard Specifications correspond to this.

Regarding (3): It is generally necessary to carry out verification of limit states using Equation (4.3.1) in the state at the end of the design service life, taking into consideration the effects of change over time in performance. Equation (4.3.1) expresses cases in which the performance limit value Rd is set as the lower limit value. In cases in which Sd represents the upper limit value, the direction of the inequality and the method of considering the safety coefficient differ from those in Equation (4.3.1).

Regarding (4): Verification through experiments is, in principle, to be carried out at the scale of the actual structure. In the case of a civil engineering structure, however, it is difficult to reproduce actual actions at the scale of the actual structure through experiments. In general, loading experiments, etc. are to be carried out using specimens that model the actual structure. In this case, the dimensions, cross-sectional specifications, details of arrangement of reinforcement, boundary conditions, loading conditions, etc. of the modeled specimen will generally differ from those of the actual structure. For that reason, when conducting verification through experiments, it is important to consider experimental results as being indicative of specific conditions, and to use experimental results in verifications with various differences from the actual structure's conditions taken into account. Therefore, even when verification is carried out primarily through experiments, a method should be used by which experiments are complemented through the application of analytical methods and models for which the estimation accuracy has been demonstrated with respect to actual behavior, and by which verification of the actual structure is performed.

4.4 Calculation of response values and limit values

(1) The function for calculating response values follows the principle of calculating average values of response values when actions, material properties, rigidity, and so on are set as actual values.

(2) The function for calculating limit values for the performance of a structure or structural members follows the principle of calculating the average values of limit values when material properties, rigidity, and so on are set as actual values.

Commentary: When proposing calculation equations for new limit values based on new knowledge, it is advisable to also take into consideration the precision of the equation and variability in the analytical values at that time, and to also propose a corresponding structural member coefficient γ_b.

4.5 Safety coefficients

(1) Safety coefficients are set for the material coefficient γ_m, action coefficient γ_f, structural analysis coefficient γ_a, structural member coefficient γ_b, and structure coefficient γ_i.

(2) The material coefficient γ_m is determined with consideration of factors including changes in material strength from the characteristic value in the undesired direction, differences in material properties between the specimen and the structure, the effects of material properties on limit states, and changes in material properties over time.

(3) The action coefficient γ_f is determined with consideration of factors including changes from the characteristic values of actions in the undesired direction, uncertainty in the calculation method for actions, changes in action during the design service life, and the effects of actions' properties on limit states.

(4) The structural analysis coefficient γ_a is determined with consideration of factors including the uncertainty of structural analysis when calculating response values.

The structural analysis coefficient γ_a can generally be set to 1.0.

(5) The structural member coefficient γ_b is determined with consideration of factors including uncertainty in calculation of structural member yield strength, the influence of unevenness in structural member dimensions, and the importance of structural members (*i.e.*, the impact on the structure as a whole when a given structural member reaches a certain limit state).

The structural member coefficient γ_b is determined for each limit value calculation.

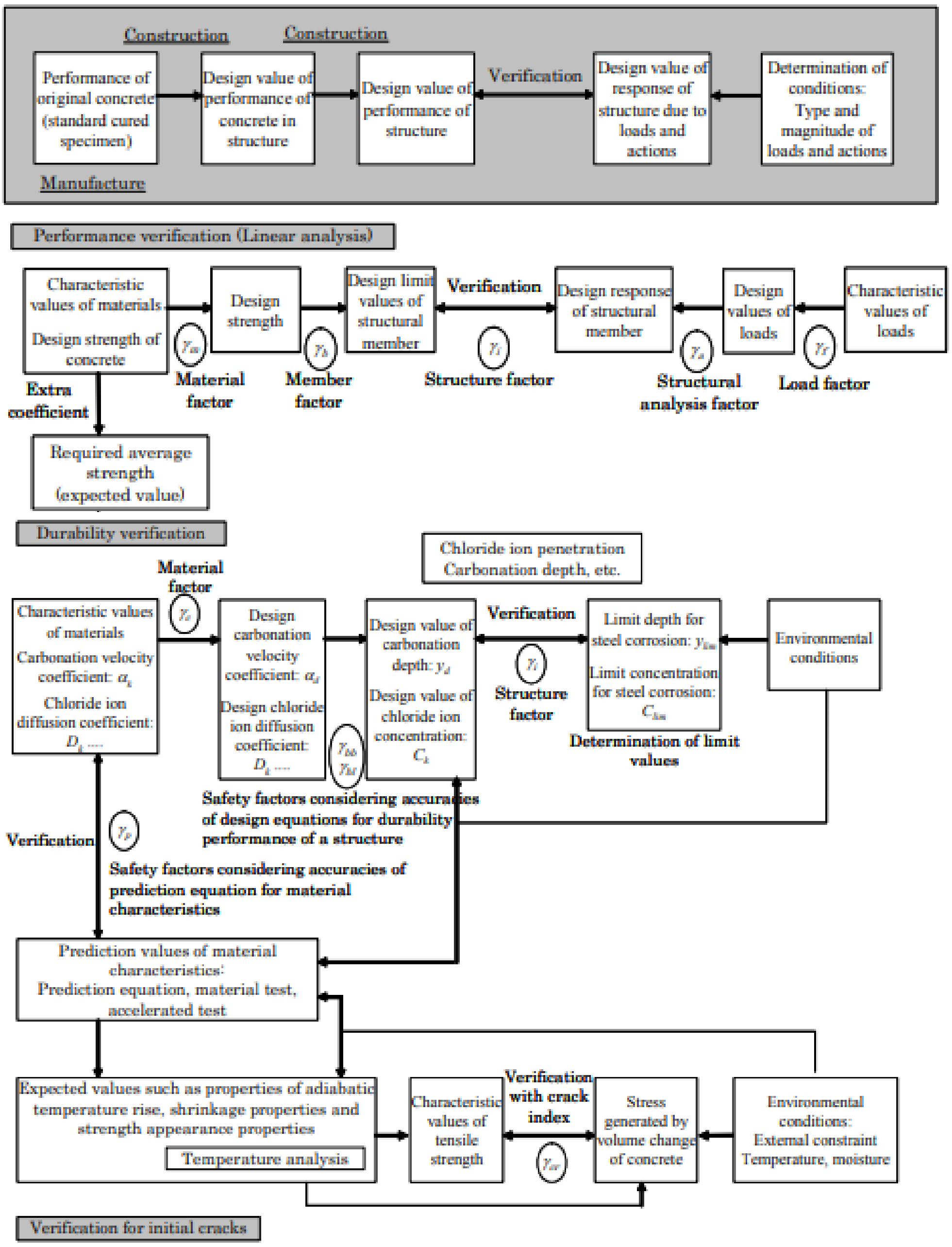
(6) The structure coefficient γ_i is determined with consideration of factors including the importance of the structure and the societal impact when the limit state is reached.

The structure coefficient γ_i can generally be set to 1.0–1.2.

(7) In performance verification using nonlinear analytical methods, this must be set appropriately with consideration of the purpose of the above safety coefficient, in accordance with the verification metrics used in the analytical methods.

(8) Safety coefficients used in verification of the effects of earthquakes must be set appropriately with consideration of the purpose of the above safety factors, in accordance with the verification methods.

Commentary: Regarding (1): In safety verification based on the limit state for cross-sectional failure of a structural member, the two safety coefficients γ_f and γ_a were set in the process of deriving design response values from characteristic values of actions, the safety coefficients γ_m and γ_b were set in the process of deriving design limit values from material strength, and the safety coefficient γ_i was set in the stage of comparing design response values and design limit values. Aside from their numerical values, these safety coefficients can be conceptually applied to other limit states. **Commentary Figure 4.5.1** shows the safety coefficient and the steps in performance verification indicated in "Design."



Commentary Figure 4.5.1 Safety coefficients for performance verification.

Regarding (3): The action coefficient γ_f varies with the type of action, as well as the type of limit state and the effect of the action on the response values occurring in the cross section under examination.

Regarding (4): As the function for calculating a response value follows the principle of calculating an average value according to Section 4.4, variation with γ_a in this function must be considered.

Regarding (5): The importance of a structural member is determined from its role in the structure, in the way that, for example, a primary structural member is more important than a secondary structural member. When there is a need to intentionally create a difference in degree of safety between bending failure and shear failure, or to cause failure in a specific member, these can be considered using the structural member coefficient γ_b.

As the function for calculating a limit value follows the principle of calculating an average value according to

Section 4.4, variation in this function must be considered using γ_b.

Regarding (6): The structure coefficient γ_i for the importance of the structure includes the societal impact when the target structure reaches the limit state, the importance of disaster prevention, the cost of reconstruction or repair, and other economic factors.

The content considered in ensuring safety with respect to the limit state of cross-sectional failure, and its handling, are summarized in Commentary Table 4.5.1.

Safety coefficients are determined in accordance with targeted limit states, and do not necessarily have the same values. In addition, although safety coefficients are separately assigned with conceivable uncertainties, they may be treated together.

The standard safety coefficient values used when applying an inspection system according to these Standard Specifications and "Construction" are shown in Commentary Table 4.5.2.

Commentary Table 4.5.1 Content considered according to safety coefficients (in verification of cross-sectional failure using linear analysis).

	Content considered	Items treated
Cross-sectional yield strength	1. Unevenness in material strength (1) Structural members for which determination is possible from material experimental data (2) Structural members for which determination is not possible from material experimental data (due to insufficient or skewed material experimental data, level of quality control, differences between the specimen and the structure in material strength, changes over time, *etc.*) 2. Degree of impact on limit state 3. Uncertainty in calculation of structural member cross-sectional yield strength, unevenness in structural member dimensions, importance of structural members, and failure properties	Characteristic value f_k Material coefficient γ_m Structural member coefficient γ_b
Sectional force	1. Unevenness in actions (1) Structural members for which determination is possible from statistical data on actions (2) Structural members for which determination is not possible from statistical data on actions (due to insufficient or skewed statistical data on actions, changes in actions during design service life, uncertainty in calculation method for actions, *etc.*) 2. Degree of impact on limit state 3. Uncertainty in structural analysis when calculating cross-sectional force, *etc.*	Characteristic value F_k Action coefficient γ_f Structural analysis coefficient γ_a
Importance of structure, socio-economic impact when limit state is reached, *etc.*		Structure coefficient γ_i

Commentary Table 4.5.2 Standard values for safety coefficients (when using linear analysis).

Safety coefficient / Required performance (limit state)	Material coefficient γ_m: Concrete γ_c	Material coefficient γ_m: Steel material γ_s	Structural member coefficient γ_b	Structural analysis coefficient γ_a	Action coefficient γ_f	Structure coefficient γ_i
Safety (cross-sectional failure)	1.3	1.0 or 1.05	1.1–1.3	1.0	1.0–1.2	1.0–1.2
Safety (fatigue failure)	1.3	1.05	1.0–1.3	1.0	1.0	1.0–1.1
Usability	1.0	1.0	1.0	1.0	1.0	1.0

Regarding (7): The safety coefficient when using linear analysis is indicated in **Commentary Figure 4.5.2** using verification of the limit state of cross-sectional failure as an example. In contrast, when using nonlinear analysis in examination of the limit state for cross-sectional failure, *etc.* of a structure, performance verification may be carried out using a metric other than cross-sectional force.

In other words, when deriving the design value of metric *S*, assuming the setting of material characteristic values with consideration of unevenness in strength and other material characteristics, the material coefficient should be set with consideration of the uncertainty factors for which both the structural analysis coefficient and structural member coefficient are considered in Commentary Table 4.5.1. Uncertainty factors include the accuracy and reliability of analysis, including boundary conditions, and unevenness in structural members. Limit value R_d should be set on the safe side, while taking into account the uncertainty factors considered through the structural member coefficient in Commentary Table 4.5.1. The treatment of safety coefficients when using nonlinear finite-element analysis based on these points is described in "Design: Standards", Volume 9.

Capacity		**Cross-sectional force**	
Material Strength Characteristic Values	$f_k(=\rho_m f_n)$	Characteristic value of action	$F_k(=\rho_f F_n)$
γ_m		γ_f	
Design Strength of Materials	$f_d=f_k/\gamma_m$	Design Actions	$F_d=\gamma_f F_k$
Sectional capacity	$R(f_d)$	Cross-sectional force	$S(F_d)$
γ_b		γ_a	
Design section strength	$R_d=R(f_d)/\gamma_b$	Design sectional force	$S_d=\Sigma\,\gamma_a S(F_d)$

Verification $\gamma_i S_d/R_d \leqq 1.0$

Commentary Figure 4.5.2 Safety coefficient when using linear analysis.

Regarding (8): Several structural analysis methods and verification methods exist that can be applied to verification of earthquake impacts, according to the safety, restorability, and other limit states to be verified. Several verification methods exist as well. For that reason, the safety coefficient is set considering the purpose of the safety coefficient used in the linear analysis indicated in this section, according to the limit state to be verified and the verification method.

4.6 Correction coefficients

(1) Correction coefficients are set to the material correction coefficient ρ_m and the action correction coefficient ρ_f.

(2) The material correction coefficient ρ_m is determined taking into account the difference between the characteristic value and standard value of material strength.

(3) The action correction coefficient ρ_f is determined for each limit state, with consideration of the difference between the characteristic value and the standard value or the nominal value of the action.

Commentary: When standard values or nominal values of a system that differs from characteristic values have been determined for material strength and action, these characteristic values are derived by converting the standard values or nominal values using the correction coefficient. The action correction coefficient ρ_f is derived for each limit state. The correction coefficient is specified in these Standard Specifications as a transitional measure until standard values and nominal values are specified in accordance with characteristic value definitions.

4.7 Design calculation sheet

(1) The design calculation sheet must indicate the set limit states of the structure or structural member when verifying the durability, safety, usability, seismic resistance, *etc.* of a structure, and must indicate the examined calculation process.

(2) The design calculation sheet adopts the principle of obtaining two significant figures at the final stage.

Commentary: <u>Regarding (2):</u> "Final stage" here means the value of $\gamma_i \cdot S_d / R_d \leq 1.0$. In order for this value to yield two significant digits, three significant digits are generally required for the response value and cross-sectional yield strength or for stress intensity and strength.

4.8 Design drawings

(1) A design drawing clearly indicates details of a structure and reinforcing steel materials, as well as the basic aspects for design calculation indicated below, construction and maintenance conditions, and other information. Note that 16-21 should be noted as reference values.

1 Design service life, environmental conditions

2 Combination of design actions and characteristic values of actions

3 Safety coefficient

4 Required performance and results of verification

- Design response value
- Design limit value

5 Characteristic values of materials used (concrete, steel)

6 Types of and standards for steel

7 Steel material covering and construction work errors for all structural members

8 The type of reinforcement joint, the position of the joint, or the range for which the joint can be set

9 Tensioning sequence, elongation, and tensioning force of the tension end of PC steel

10 Necessary matters in construction work and maintenance

11 Name and place of use of the structure

12 Signature of responsible engineer

13 Design date

14 Scale, dimensions, and units

15 Names of applied standards

16 Type of cement (type of admixture as necessary)

17 Maximum dimensions of coarse aggregate

18 Unit cement content (unit admixture content and admixture mixing rate as necessary)

19 Minimum slump or slump flow in concrete pouring

20 Water–cement ratio (water–binder ratio as necessary)

23 Air content

(2) Efforts must be made to preserve design drawings together with construction records during the period of the uses, function, and service of the structure.

Commentary: Design drawings can be considered the sole means of communicating the intent of the designer to the construction manager and the maintenance manager. For concrete, these Standards Specifications adopt the principle of setting the composition of materials, *etc.* so as to satisfy the various characteristic values described in the design drawings during construction. For that reason, all characteristic values set during design are to be included in the design drawings and followed in construction. At the same time, taking into account cases involving the use of a proven concrete mixture or the simplicity of mixing design during construction, it was decided to note types of cement, maximum dimensions of coarse aggregate, unit cement content, minimum slump or slump flow in pouring, water–cement ratio, and so on in design drawings as reference values (see Appendix 2 of "Design: Standards"). As information on admixtures may be used in the verification of resistance to deterioration and substance permeation, these are also shown as necessary. It is also important to clearly show the timing for removal of concrete molds and falsework, the compressive strength of concrete that can be prestressed if post-tensioned prestressed concrete is used, the thermal cracking index, shrinkage in structural members, creep, and other conditions assumed in construction. The construction period, construction method, procedures, and other matters related to construction work assumed in the design should also be included. With regard to securing the useful life of the concrete structure, the characteristics of the concrete used for structural members and covering, the cracking assumed in the design, and other matters are vital in acceptance inspections and maintenance. In order to communicate the results of the examination to managers in charge of construction and maintenance, it is necessary to clearly indicate the steel material covering, the design crack width, and the assumed construction error in reinforcing bars for all structural members in the design drawing. In locations where reinforcing bars, sheaths, anchor bolts, *etc.* are complicated and where the reinforcing bars of bearing parts and structural member joints are densely arranged, detailed two- or three-dimensional drawings should be prepared that take into account the thicknesses and processed forms of all arranged reinforcing bars and the forms and dimensions of equipment, and that confirm reinforcing bars, *etc.* do not interfere with each other and that the filling performance of concrete is ensured. These drawings should be included in the design drawings and preserved. In addition, calculation sheets, cross-sectional force diagrams, logs of meetings, data, informational materials, and other information that forms the design documents based on which structure is determined and that will aid in maintenance should be preserved. As well as the items indicated in this text, the following items may also be described as necessary:

・ Bill of materials
・ Detailed drawings of covering
・ Detailed drawings of locations where reinforcing bars, sheaths, anchor bolts, *etc.* are complicated
・ Pouring sequence
・ Position of construction joints
・ Position of crack-inducing joints
・ Falsework
・ Load-bearing capacity of the foundation ground and foundation piles
・ Geological map and *N* value

Chapter 5 Materials

5.1 Basics of materials

(1) In concrete structures, it is necessary to use concrete, steel reinforcing bars, and other materials that have properties necessary to ensure the required performance of the structure.

(2) Concrete must possess mechanical properties that satisfy the required performance and resistance to deterioration and substance permeation during the design service life of the structure. It must also have appropriate workability during construction.

(3) Steel must possess mechanical properties that satisfy the required performance of the structure when used together with concrete.

Commentary: Regarding (1): It was stipulated that performance verification for concrete structures should include examination of the properties of the concrete, steel, and other materials used to ensure the required performance of the structure.

The properties of materials that satisfy the required performance of structures are innumerable. Therefore, in design it is necessary to determine the properties of the concrete and steel materials to be used and the combination of these, taking environmental performance, economic performance, and other factors into consideration.

Regarding (2): Unlike steel materials, concrete is manufactured and worked at the construction site. Its material properties are greatly affected by workability at the site, which means that workability must be considered to ensure quality of the structure. Therefore, in the design, workability must be set in accordance with the arrangement of reinforcing bars and other details of the structure. However, even if efforts are made to enhance mechanical properties, resistance to deterioration and substance permeation, workability, and other properties, it will not necessarily be possible to satisfy all properties through the arrangement of reinforcing bars in the structure, the setting of conditions that determine the properties of concrete, and so on, and conflicts between individual properties may occur. Therefore, it is necessary to comprehensively consider and determine the properties of the concrete to ensure its quality, taking into account factors including the arrangement of steel materials (see "Design: Standards," Appendix 4).

Regarding (3): Steel materials are generally protected from corrosion by being covered with concrete. Therefore, it is necessary to determine the material properties of the concrete with the goal of ensuring the resistance of the steel materials to corrosion.

5.2 Design values of materials

(1) Material properties of concrete and steel materials include the compressive strength, tensile strength and other strength properties; Young's modulus and other deformation properties; thermal properties; resistance to deterioration and substance permeation; and watertightness, as required for performance verification. With regard to strength properties and deformation properties, the effect of loading rate must be considered as necessary.

(2) Assuming unevenness in experimental values, the material strength characteristic value f_k is to be set to a value such that most experimental values can be guaranteed to not fall below f_k.

(3) The design strength f_d of a material is to be set to a value derived by dividing the characteristic value of material strength f_k by the material coefficient γ_m.

(4) When the standard value f_n of material strength is set separately from its characteristic value, the characteristic value f_k of material strength is to be the value derived by multiplying the standard value f_n by the material correction coefficient ρ_m.

Commentary: Regarding (1): The concrete used in structural members of a structure must be of the appropriate type and must have appropriate properties, taking into consideration factors including purpose, function, environmental conditions, design service life, and construction conditions.

As necessary in performance verification, the properties of concrete are represented by not only compressive strength but also by quantities that express various material properties. Material properties can be broadly divided into physical properties, chemical properties, and mechanical properties such as strength properties and deformation properties.

Strength properties are expressed by various quantities of fatigue strength and static strength, including compressive strength, tensile strength, and bonding strength. Deformation properties are expressed by properties such as the non-time-dependent Young's modulus and Poisson's ratio, or the time-dependent creep coefficient and shrinkage strain. There are also mechanical properties such as stress–strain relationship that are expressed by the relationship between two mechanical factors. Fracture energy may be used to express crack resistance or toughness.

Physical properties also include density, watertightness, airtightness, and thermal properties such as thermal expansion coefficient and specific heat. At present, however, the quantitative treatment of density and thermal properties is generalized.

Chemical properties include resistance to sulfate alteration and acid erosion.

Concrete must possess resistance to meteorological action, intrusion and erosion by chemical substances, and various other effects, as well as resistance to the deterioration that these cause over time. Reinforced concrete further requires the resistance of steel materials over time. With respect to corrosion of steel materials in particular, verification of the resistance to water permeation and carbonation of concrete, intrusion by chloride ions, and permeation by other substances is to be performed.

Concrete properties can be greatly affected by not only the materials used and mixing conditions but also the construction conditions and environmental conditions under which the concrete is used. As these conditions are diverse, values for concrete made with primarily Portland cement and natural aggregate or artificial lightweight aggregate, worked at normal atmospheric temperature under normal environmental conditions, are presented as the general numerical values of the various properties normally used in the design stage. These numerical values take a single standard value and often have a small range of variation with respect to changes in conditions, but some exhibit a large range of variation. Therefore, if reliable numerical values for the material properties of concrete under varied conditions of materials used, mixture, construction, environment, etc. can be obtained, then it is advisable to use those fact-based values instead of the values shown here.

For steel materials, too, as with concrete, quantities that represent tensile strength and other material properties are used as necessary in performance verification.

Actions occurring in civil engineering structures include static actions such as the dead load acting throughout the service life of the structure; dynamic actions such as vehicle load, waves, and earthquakes; and shock actions such as explosions and collisions from flying objects. The speed of such actions is normally expressed by stress rate or strain rate. It has been clearly shown that failure modes in concrete structures are greatly affected by loading rate. Therefore, the mechanical properties of materials that compose concrete structures, i.e., concrete and steel materials, should be calculated according to stress rate and strain rate.

Material property values shown in this section may be used to verify limit states for static actions and normal dynamic actions. When there is a need to consider the effects of strain rate, then, as when considering shock, values obtained through reliable experiments must be used. The effects of loading rate on compressive strength, tensile strength, Young's modulus, strain at maximum stress, and other material properties should be examined as necessary.

When there is a need to consider the effects of strain rate on the yield stress, tensile strength, and breaking strain of steel materials, these must be derived through reliable experiments. The effect of strain rate on breakdown stress in reinforcing bars should be examined as necessary.

Regarding (2): The characteristic value of material strength should generally be derived using (Commentary 5.2.1):

$$f_k = f_m - k\sigma = f_m(1 - k\delta) \qquad \text{(Commentary 5.2.1)}$$

where

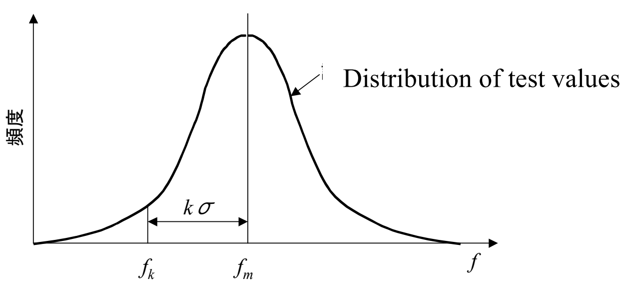
f_m: Average value of test values;

σ: Standard deviation of test values;

δ: Coefficient of variation of test values; and

k: Coefficient.

The coefficient is determined by the probability of obtaining a test value smaller than the characteristic value and by the distribution of the test value. If the probability of the value falling below the characteristic value is 5% and the distribution is a normal distribution, then the coefficient is 1.645 (see **Commentary Figure 5.2.1**).



Commentary Figure 5.2.1 Characteristic value for material strength.

5.3 Concrete

5.3.1 Strength

(1) In principle, the characteristic values of concrete strength are to be determined based on test strength at 28 days of material age. However, it may instead be determined based on test strength at an appropriate material age in accordance with the uses and functions of the structure, the period over which the primary load acts, the construction plan, *etc.*

(2) When ready-mixed concrete conforming to JIS A 5308 is used, the nominal strength specified by the purchaser may generally be used as the characteristic value f'_{ck} of compressive strength.

(3) The characteristic values for bonding strength and bearing strength of concrete are to be determined based on test strength obtained through appropriate testing.

(4) The bending crack strength of concrete is to be determined appropriately with consideration of the effects of drying, heat of hydration, and dimensions.

(5) The material coefficient of concrete γ_c is to be set appropriately in accordance with the performance to be verified.

5.3.2 Fatigue strength

The characteristic value of the fatigue strength of concrete is to be determined based on fatigue strength according to tests conducted with consideration of the type of concrete, the exposure conditions of the structure, *etc.*

Commentary: The standard treatment of fatigue strength of concrete is shown in "Design: Standards" Volume 3, Section 3.

5.3.3 Stress–strain curve

The stress–strain curve of concrete is to be assumed in accordance with the objectives of the limit state verification.

Commentary: In the case of ordinary concrete, too, the stress–strain curve differs considerably with the type of concrete, material age, acting stress state, loading rate, loading route, *etc.* However, differences in the stress–strain curve do not have a great effect in some cases, such as in the cross-sectional ultimate yield strength of rod structural members.

5.3.4 Breaking energy

In principle, the breaking energy of concrete is to be derived through testing.

Commentary: Safety verification of reinforced concrete can generally be performed by modeling concrete that undergoes tension as a completely brittle material.

5.3.5 Young's modulus

In principle, the Young's modulus of concrete is to be derived through testing.

Commentary: The Young's modulus of concrete is known to vary greatly with the type and quality of the aggregate and with the place of origin. The Young's modulus of concrete has a smaller effect than other characteristic values on the safety of a structure, but in cases in which the Young's modulus has a large effect on the performance of the structure, values measured using the actual materials to be used should be used as necessary, with conditions properly taken into account.

5.3.6 Poisson's ratio

The Poisson's ratio for concrete should generally be set to 0.2, within the elastic region. However, if it is subject to tension and if cracking is allowed, then it is to be set to 0.

5.3.7 Thermal performance

In principle, the thermal properties of concrete are to be determined based on experiments or past data.

5.3.8 Shrinkage

(1) In principle, the characteristic value of shrinkage in concrete is to be determined with consideration of the effects of the aggregate used, the type of cement, the mixing of the concrete, *etc.* For testing, the value used is that for a 100 × 100 × 400 mm rectangular prism specimen cured in water for 7 days, followed by a drying period of 6 months (182 days) under environmental conditions of a temperature of 20±2 °C and a relative humidity of 60±5%.

(2) In principle, the shrinkage of concrete in a structure is to be calculated with the temperature and relative humidity of the structure's environment, the cross-sectional shape and dimensions of structural members, the material age at the start of drying, and other factors affecting the characteristic value of shrinkage in concrete taken into consideration.

Commentary: Regarding (1): Shrinkage under unified curing conditions, environmental conditions, and form and dimensions were set as characteristic values of shrinkage in the concrete. In performance verification for structures, when calculating response values for structures that are affected by shrinkage of concrete, the characteristic values for shrinkage of concrete must be set in the design stage in the same manner as characteristic values such as for strength, and those values must be noted in the design drawings.

5.3.9 Creep

(1) As the creep strain of concrete is proportional to the elastic strain caused by acting stress, it is generally to be derived using:

$$\varepsilon'_{cc} = \varphi \cdot \sigma'_{cp}/E_{ct}, \tag{5.3.1}$$

where ε'_{cc}: Compressive creep strain of concrete;

φ: Creep coefficient;

σ'_{cp}: Acting degree of compressive stress; and

E_{ct}: Young's modulus of material age during loading.

(2) In principle, the creep coefficient of concrete is to be determined with consideration of factors including the effects of humidity around the structure, the cross-sectional shape and dimensions of structural members, the mixing of the concrete, and the material age of the concrete when stress acts on it.

Commentary: Regarding (1): If the stress intensity of concrete is 40% or less of its compressive strength, then creep strain is nearly linearly proportional to acting stress. Therefore, if acting stress is variable, then the principle of superposition should be applied. When concrete stress intensity is greater than this level, it is not appropriate to consider creep strain to be proportional to elastic strain caused by acting stress. For concrete in which cracking is absent, creep properties under tensile stress may be assumed to be the same as those under compressive stress.

Regarding (2): The design value of the creep coefficient of concrete must be determined with reference to test results, measurements from past tests or actual structures, *etc.*

5.3.10 Effects of high temperature

The compressive strength, Young's modulus, and tensile strength of concrete subjected to high temperature from fire or other causes are to be appropriately determined based on strength tests, *etc.*, based on values after heating and cooling.

Commentary: If the heated temperature is estimated to be in excess of 300 °C, then, in principle, the compressive strength of the concrete must be appropriately determined by collecting cores from parts of the structure subjected to fire damage and from parts not subjected to fire damage, or through other testing.

As Young's modulus is also dependent on factors including heated temperature, compressive strength at room temperature, and the type of aggregate, it must be determined appropriately with these taken into consideration.

The characteristic value of concrete strength that has been affected by high temperature due to fire or other causes is generally greatly dependent on the heated temperature. However, as the value at high temperature differs from the value during heating and cooling, it must be appropriately determined through testing or other means as necessary. In particular, when evaluation at high temperature is necessary, compressive strength, *etc.* must be confirmed through heated testing.

5.3.11 Effects of low temperature

(1) The characteristic value of concrete strength at low temperatures is to be determined based on tested strength, taking into consideration temperature and water content.

(2) The characteristic value of compressive strength at low temperatures may be derived by adding compressive strength at room temperature to the amount of increase determined by temperature and water content.

(3) The tensile strength and Young's modulus of concrete at low temperature may be derived from compressive strength at low temperature.

Commentary: The compressive strength of concrete at low temperature is derived by adding compressive strength at room temperature to the compressive strength increase due to the low temperature and water content.

Tensile strength at low temperature has a linear relationship with the 3/4th to 1st root of compressive strength at low temperature. Within the range of 0° to -100 °C, it is not dependent on temperature conditions and may be obtained from Equation 5.3.1 using compressive strength at low temperature. The value thus obtained is close to the lower limit in past data.

In cases in which the temperature could return to room temperature during the design service life, it is safer to not take the increase in strength due to low temperature into consideration. However, in cases in which temperature stress caused by a temperature decline or temperature gradient is a problem, as temperature stress is strongly dependent on structural member rigidity and tensile strength, it is important to derive the structural member rigidity and tensile strength appropriately, with temperature and water content taken into consideration.

If the Young's modulus used in the calculation of temperature stress is small, then temperature stress will be underestimated. Therefore, the Young's modulus should be derived with consideration of temperature and water content. At low temperatures, the Young's modulus of concrete has a linear relationship with compressive strength. The stress–strain curve of concrete at low temperature may be assumed to have a shape appropriate to the purpose of the verification, taking temperature and water content into consideration. The Poisson's ratio for concrete should generally be set to 0.2 within the elastic region, as at normal temperature. However, when cracking under tension is allowed, it is to be set to 0. As the thermal properties of concrete are generally temperature-dependent, they should be taken into account.

5.3.12 Water permeation velocity coefficient

In principle, the characteristic value of the water permeation velocity coefficient of concrete should be determined based on experiments or past data.

Commentary: The water permeation velocity coefficient is a coefficient for the supply of water that causes steel material corrosion in concrete. It stipulates the amount of water supply that reaches the position of steel materials through the covering when water exerts action from the concrete surface. In a general environment in which the supply of chloride ions is low, water permeation is an important factor that controls the corrosion of steel materials. Therefore, the water permeation velocity coefficient must be set appropriately.

When determining the water permeation velocity coefficient of concrete through experiments, a concrete test specimen should be made using materials, mixing, and curing methods that simulate the actual construction work to the extent possible. This should be subjected to action by water, after which the change in permeation depth over time can be measured and the water permeation rate coefficient calculated. The water permeation velocity coefficient of concrete is strongly affected by the material age and dry state of the concrete.

5.3.13 Carbonation velocity coefficient

In principle, the characteristic values of the carbonation velocity coefficient of concrete should be determined based on experiments or past data.

Commentary: The carbonation velocity coefficient is a proportionality constant wherein the depth at which carbonation progresses is proportional to the square root of the exposure period. This carbonation velocity coefficient is used to verify corrosion of steel material associated with carbonation. To prevent corrosion of steel material due to carbonization during the design service life, the carbonation velocity coefficient must be set appropriately, taking into consideration the environment to which the structure is exposed.

When deriving the carbonation velocity coefficient of concrete through experiments, the effects of the effective water–binder ratio of concrete and the type of binder must be taken into consideration. The carbonation velocity coefficient corresponds to the gradient of the linear regression equation for the relationship between the water–binder ratio and the value obtained by dividing the carbonation depth by the square root of the material age (in years). The above method can be used as a reference when newly deriving the carbonation velocity coefficient through experiments, *etc.*, but as the carbonation velocity coefficient is greatly affected by initial curing and environmental conditions, these must be appropriately determined in experiments.

5.3.14 Chloride ion diffusion coefficient

In principle, the characteristic value of the chloride ion diffusion coefficient of concrete should be determined based on experiments or past data.

Commentary: The diffusion coefficient is a proportional coefficient that appears in Fick's laws of diffusion and indicates the speed of diffusion. In "Design: Standards" Volume 2, Section 2, the diffusion coefficient used in Equation (2.1.16) to calculate the chloride ion concentration according to position in steel materials is derived based on the assumption that salinity intrusion into concrete is represented by a one-dimensional diffusion equation.

The chloride ion diffusion coefficient of concrete is affected by the materials used, mixing, and other factors. Methods for deriving the diffusion coefficient include the method of using the relational expression obtained by organizing the water–cement ratio and apparent diffusion coefficient from past data, the method of derivation through indoor experiments, and the method of derivation from exposed specimens and cores collected from existing structures that are considered to be in natural environments similar to the structure being designed and to be subjected to similar actions.

The relational equations for calculating the chloride ion diffusion coefficient of concrete as a function of the water–cement ratio are shown in "Design: Standards" Volume 2, Section 2, 2.1.4.2. These equations were derived from the results of past experiments. However, when using standard materials or construction methods, the characteristic value of the diffusion coefficient should be derived with reference to Equations (Commentary 2.1.4) to (Commentary 2.1.7).

When deriving the chloride ion diffusion coefficient of concrete through indoor experiments, *etc.*, the characteristic values of the diffusion coefficient must be obtained with full understanding of the properties and of the significance of the material properties obtained through the experiments. When converting the apparent diffusion coefficient from the effective diffusion coefficient that was obtained through the electrophoresis method, Equations (Commentary 2.1.9) to (Commentary 2.1.12) may be used as a reference if standard materials and construction methods are used.

When deriving the apparent diffusion coefficient of chloride ions in concrete through immersion and when deriving the total chloride ion distribution in concrete for an actual structure, the transport phenomenon that occurs during actual service, driven by factors including the concentration gradient of chloride ions and the advection of liquid water, is captured. Meanwhile, the diffusion coefficient is obtained from the distributions, including that of the fixation of chloride ions to the hardened cement. Therefore, the diffusion coefficient obtained here can be used as the apparent diffusion coefficient in direct verification. However, caution is needed as the obtained apparent diffusion coefficient is known to change with the material age, *etc.* for which the diffusion coefficient is measured. Although these Standard Specifications make Fick's laws of diffusion the basis for verification, the actual phenomenon of chloride ion intrusion into concrete is not controlled solely by the diffusion phenomenon driven by the concentration gradient assumed by Fick's laws of diffusion. Under conditions such as repeated wetting and drying, the mechanism of transport through the movement of interior liquid water dominates, and the hydration reaction continues as the material ages, with the pore structure of concrete becoming more minute. For these and other reasons, the measured apparent diffusion coefficient is known to decrease over time.

5.3.15 Relative dynamic modulus of elasticity in freeze–thaw testing

In general, the characteristic value of the relative dynamic modulus of elasticity of concrete is to be determined based on the relative dynamic modulus of elasticity.

Commentary: In the verification of freeze–thaw resistance, if the actual conditions of the freeze–thaw action are more severe than the conditions set in testing or if the design service life is set to a particularly great length, then the relative dynamic modulus of elasticity should be measured under severe settings for the freeze–thaw repetition cycle, the freeze–thaw temperatures, and the time required per cycle, in accordance with actual conditions. It has been reported that freeze–thaw action in the presence of an antifreeze agent that contains sodium chloride weakens the structure of concrete and causes greater decline in tensile strength and relative dynamic modulus of elasticity. The characteristic value of the relative dynamic modulus of elasticity must be appropriately determined under such conditions.

5.4 Steel materials

5.4.1 Strength

(1) The characteristic value f_{yk} of the tensile yield strength and the characteristic value f_{uk} of the tensile strength of steel material are to be determined from their respective test strengths.

(2) For cases conforming to JIS standards, the characteristic values f_{yk} and f_{uk} should be used as the JIS standard lower limits. The cross-sectional area of the steel material used in verification of the limit state should generally be used as the nominal cross-sectional area.

(3) The characteristic value f'_{yk} of the compressive yield strength of steel material should be set equal to the characteristic value f_{yk} of the tensile yield strength of the steel material.

(4) The characteristic value f_{vyk} of the shear yield strength of steel materials should generally be derived from:

$$f_{vyk} = f_{yk}/\sqrt{3} \qquad (5.4.1)$$

(5) For verification of the cross-sectional failure limit state, *etc.*, the material coefficient γ_s of the steel material should generally be set to the following values:

Reinforced and PC steel: 1.0

Other steel materials: 1.05

For verification with respect to fatigue failure, 1.05 should generally be used. For verification of usability, 1.0 should generally be used.

Commentary: Regarding (2): Although the cross-sectional area used to calculate strength differs with the type of steel material, it was decided that the nominal cross-sectional area may be used for verification of the limit state. The strength of reinforcing bars is calculated using the nominal cross-sectional area, and the strength of PC steel rods is calculated using the larger of actual cross-sectional area and nominal cross-sectional area, to be on the safe side. The strength of PC steel wire or PC steel stranded wire can be indicated as the strength per wire or stranded wire. However, the strength of structural steel material is calculated using the original cross-sectional area. If cross-sectional area based on nominal thickness is used, then actual strength may overestimated because of dimensional tolerance. However, this does not have a large effect on performance verification.

Regarding (4): It was decided that the von Mises yield criterion should be applied to the shear yield strength of steel material.

Regarding (5): When there is a possibility of impairing the strength of the base metal, as with threading or cutout, the effects must be taken into consideration.

5.4.2 Fatigue strength

The characteristic value of the fatigue strength of steel material is to be determined based on fatigue strength through testing conducted with consideration of factors including the type, form, and dimensions of the steel material, the jointing method, the magnitude and frequency of action of acting stress, and environmental conditions.

Commentary: The standard treatment of fatigue strength of steel material is shown in "Design: Standards" Volume 3, Section 3.

5.4.3 Stress–strain curve

The stress–strain curve of a steel material is to be assumed to have a shape appropriate to the purpose of the verification.

Commentary: The stress–strain curves of steel materials differ with the type of steel material, chemical composition, manufacturing method, and other factors. Therefore, a stress–strain curve matched to the purpose of verification must be used. However, depending on the content of the verification, differences in the stress–strain curve may not have significant effects.

5.4.4 Young's modulus

In principle, the Young's modulus of a steel material is to be determined from the results of tensile testing to obtain the stress–strain curve.

Commentary: The Young's modulus for a steel material varies with factors including the measurement method, but is generally within the range of 190 to 210 kN/mm^2. Different Young's moduli have been used for reinforcing bars, structural steel materials, and PC steel materials. In general, however, differences in the Young's moduli of steel materials have a relatively small effect on the results of calculation of a structural member's cross-sectional stress intensity, deformation of structural members, *etc.* Therefore, a value of 200 kN/mm^2 may be used as the Young's modulus for any steel material. When it is necessary to precisely derive deformation of structural members, such as when calculating elongation of PC steel materials by controlling the tensioning of PC steel materials, *etc.*, the Young's modulus obtained from test results should be used.

5.4.5 Poisson's ratio

The Poisson's ratio for steel materials should generally be set to 0.3.

Commentary: The Poisson's ratio for steel materials varies with factors including the measurement method, but generally does not have a significant effect on the verification of limit states. Therefore, it was decided that conventionally used values may be used.

5.4.6 Thermal expansion coefficient

The thermal expansion coefficient of steel materials may generally be set to that of concrete.

Commentary: The thermal expansion coefficient (or linear expansion coefficient) of steel materials in steel structures and in steel composite girders is generally 12×10^{-6}/°C. However, it was decided that for steel material in concrete, the value can be set to that of the thermal expansion coefficient of concrete.

5.4.7 Relaxation ratio of PC steel material

The relaxation ratio of PC steel material may be set to three times the 1000-hour test value derived through relaxation testing.

Commentary: The relaxation ratio of PC steel material is the value expressed as a percentage of the PC steel material's tensile stress intensity at which the decline in tensile stress intensity first occurred under constant strain condition. When performing relaxation testing at a temperature outside the normal temperature range, it is necessary to examine factors including the initial load, the loading method of the initial load, the heating or cooling method, the temperature measurement method, and the temperature variability range.

5.4.8 Effects of high temperature

(1) The characteristic values of the tensile yield strength and the tensile strength of steel materials subjected to the effects of high temperature due to fire, *etc.* are to be determined based on their respective test strengths.

Commentary: As the mechanical properties of steel materials are dependent on temperature history, the tensile yield strength and the tensile strength of steel materials subjected to the effects of high temperature due to fire, *etc.* must be appropriately determined in accordance with the heated temperature, and must be confirmed through tensile testing or other methods when damage is obvious.

5.4.9 Effects of low temperature

The characteristic values of the tensile yield strength and the tensile strength of steel materials at low temperatures are to be determined based on their respective test strengths.

Commentary: At low temperatures, it is necessary to use steel materials for which required characteristics have been confirmed. Although tensile yield strength and tensile strength increase with decline in temperature for both reinforcing bars and PC steel materials, for characteristic values of tensile yield strength and tensile strength, values for room temperature should be used, with effects of temperature ignored.

If the temperature could return to room temperature during the design service life, then the stress–strain curve for room temperature is to be used.

Although the Young's modulus increases at low temperatures, the rate of increase is small, and the Young's modulus at room temperature should be used. For the Poisson's ratio and thermal expansion coefficient at low temperatures, the values at room temperature should be used.

5.5 Other materials

If equipment, *etc.* composing a structure contains materials not shown here, then materials for which properties have been confirmed through testing, *etc.* are to be used.

Commentary: A structure is composed not only of reinforced concrete structures and prestressed concrete structures, but also of other structural elements.

The materials used in these structural elements, too, must possess material properties that satisfy the required performance of the structure.

Chapter 6 Actions

6.1 General

(1) In performance verification of structural objects, the actions envisioned during construction and during the design service life must be taken into account under appropriate combinations in accordance with the limit states for required performance. "Actions" include all actions that increase or decrease stress and deformation in a structural object or structural members or that create change in material properties.

(2) Design actions are to be determined by multiplying the characteristic values of actions by the action coefficient.

(3) Design actions are generally to be combined as shown in Table 6.1.1.

Table 6.1.1 Combinations of design actions.

Required performance	Limit state	Combinations that should be taken into consideration
Durability	All limit states	Permanent actions + variable actions
Safety	Cross-sectional failure, *etc*.	Permanent actions + primary variable actions + secondary variable actions
		Permanent actions + accidental actions + secondary variable actions
	Fatigue	Permanent actions + variable actions
Usability	All limit states	Permanent actions + variable actions
Restorability	All limit states	Permanent actions + accidental actions + secondary variable actions

Commentary: Regarding (1): In these Standard Specifications, in principle, verification of seismic actions calculates the response value by directly considering effects, without following steps for modeling of weight, force, and so on through dynamic analytical methods. Verification involving durability also includes methods of verifying the response of the structural object without modeling, such as by calculating the corrosion limit of reinforcing bars from environmental conditions at the structural object's location.

Permanent actions are actions that persist and for which variations are extremely rare or negligibly small relative to average values.

Variable actions are actions that occur continuously or frequently and for which variations relative to average values cannot be ignored.

Accidental actions are actions that occur very infrequently during the design service life but have a large effect when they occur.

However, when verifying safety with respect to a major tsunami striking after a massive earthquake, the effects of the tsunami, along with the effects of the earthquake, must both be taken into consideration as accidental actions.

Regarding (2): Combined actions are collectively referred to as "design actions" in some cases, but in these Standard Specifications, design actions are determined for individual actions.

Regarding (3): Performing verification involving safety and resilience for combinations that set a given group of variable actions as primary and other variable actions as secondary, or combinations of accidental and secondary variable actions, is practical and therefore has been specified.

In the examination of safety, the design cross-sectional force can generally be expressed using Equation (Commentary 6.1.1).

In verification involving usability and with respect to durability, the combinations of actions that should be examined for cracking, deformation, and other individual performance items are to be set. Therefore, it is not particularly necessary to distinguish between primary variable actions and secondary variable actions.

$S_d = \sum \gamma_{ap} S_p(\gamma_{fp} \cdot F_p) + \sum \gamma_{ar} S_r(\gamma_{fr} \cdot F_r) + \sum \gamma_{aa} S_a(\gamma_{fa} \cdot F_a)$, (Commentary 6.1.1)

where S_d : Design cross-sectional force;

S_p, S_r, S_a: Functions for deriving cross-sectional force due to permanent actions, primary variable actions, and secondary variable actions, respectively;

F_p, F_r, F_a: Characteristic values for permanent actions, primary variable actions, and secondary variable actions, respectively;

γf_p, γf_r, γf_a: Action coefficients for permanent actions, primary variable actions, and secondary variable actions, respectively; and

γ_{ap}, γ_{ar}, γ_{aa}: Structural analysis coefficients for permanent actions, primary variable actions, and secondary variable actions, respectively.

6.2 Characteristic values of actions

(1) The characteristic values of actions must be determined for the respective limit states of the required performance that should be examined.

(2) The characteristic values of the permanent actions, primary variable actions, and accidental actions used in verification involving safety are to be the expected maximum values that will occur during construction of the structural object and during its design service life. However, when a smaller value would be disadvantageous, the expected minimum value is to be used instead. The characteristic values of the secondary variable actions are to be determined in accordance with combinations of primary variable actions and accidental actions. The characteristic values of actions used in verification of fatigue are to be set with consideration of variation in actions during the design service life of the structural object.

(3) The characteristic values of actions used in verification involving usability are to be of magnitudes that occur relatively frequently during the construction of the structural object and during its design service life, and are to be set in accordance with the combination of limit states and actions with respect to the required performance that should be examined.

(4) The characteristic values of actions used in verification related to restorability are to be in accordance with the limit states of the set performance, with the expected maximum value that will occur during the design service life of the structural object as the upper limit value. Effects of earthquakes are to be in accordance with Section 6.4.10.

(5) The characteristic values of actions used for verification of durability are to be set to magnitudes that occur relatively frequently during the construction and design service life of the structural object.

(6) If the standard value or nominal value of an action is determined separately from the characteristic value, then the characteristic value of the action is to be the value obtained by multiplying the standard value or nominal value by the action correction coefficient ρ_f.

Commentary: Regarding (2): The maximum values or minimum values of actions during a reproduction period that extends beyond the design service life are used as the characteristic values of the permanent actions, the primary variable actions, and the accidental actions used in verification involving safety. However, in these Standard Specifications, it was decided to use the expected maximum value or expected minimum value as the characteristic value.

Secondary variable actions should also be considered in combination with primary variable actions and accidental actions. Therefore, their characteristic values should generally be set to smaller values than when the same variable actions are set to primary variable actions.

Regarding (3): Actions of "relatively frequently occurring magnitude" used in verification involving usability are those actions by which the limit states of cracking, deformation, *etc.* are not reached under actions occurring with that frequency.

Regarding (6): For live loads, *etc.*, it was decided that, when the standard value of an action is stipulated by laws, regulations, and so on, the characteristic value of the action may be obtained by multiplying the standard value by the action correction coefficient.

6.3 Action coefficients

The action coefficient, which is multiplied by the characteristic value of the action as a design action, can generally be determined from Table 6.3.1.

Table 6.3.1 Action coefficients.

Required performance	Limit state	Type of action	Action coefficient
Durability	All limit states	All actions	1.0
Safety	Cross-sectional failure, *etc.*	Permanent action	1.0–1.2*
		Primary variable effect	1.1–1.2
		Secondary variable effect	1.0
		Accidental action	1.0
	Fatigue	All actions	1.0
Usability	All limit states	All actions	1.0
Restorability	All limit states	All actions	1.0

* When the smaller permanent action would be disadvantageous, the action coefficient for the permanent action should be set to 0.9–1.0.

Regarding (1): Among permanent actions, variation in dead load is due to interaction between variation in the unit weight of the material and variation in the cross-sectional dimensions of the structural object, and some degree of unevenness should be taken into consideration using the action coefficient. As the coefficient of action for a fixed dead load is considered to exhibit little variation, it can be set to 1.0–1.1 in verification involving safety. Here, the dead weight of the structural object means the weight of the structural members that have an effect on the yield strength of the structural object, as calculated using the unit weight shown in Commentary Table 6.4.1.

The handling of prestress force in examination of safety is described in "Design: Standard" Volume 8. The corresponding action coefficient can generally be set to

1.0.

Accidental actions are actions for which the probability of occurrence during the design service life is extremely low. It was decided to set the action coefficient to 1.0 on the premise that the characteristic value will not be set on the unsafe side.

6.4 Types of actions

6.4.1 General

"Actions" refers to all workings that result in increased stress or deformation or in change in material properties over time in a structural object or structural members. In verification of performance, the actions indicated below should generally be taken into account:

- Dead load
- Live load
- Soil pressure
- Water pressure
- Fluid force
- Wave force
- Prestress force
- Wind load
- Snow load
- Effects of concrete shrinkage and creep
- Effects of temperature and of repeated heating and cooling
- Effects of solar radiation
- Effects of earthquakes
- Supply of humidity and moisture
- Concentrations of various substances
- Construction load
- Effects of fire
- Other

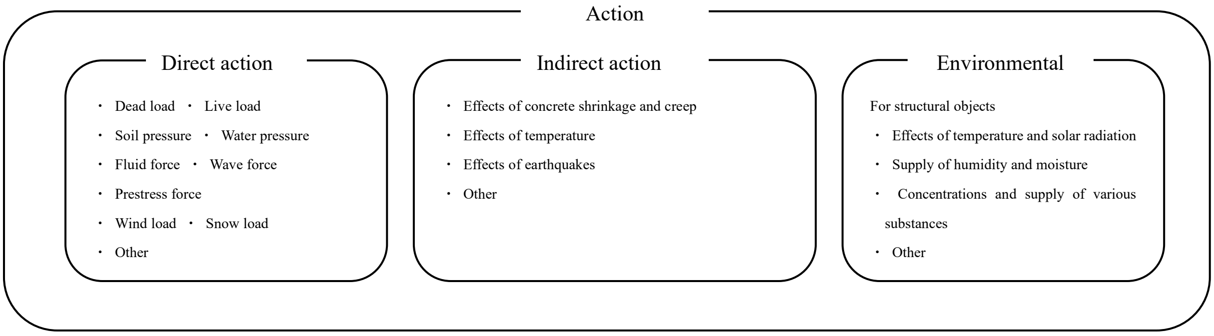
Commentary: In these Standard Specifications, actions were defined as all workings that result in increased stress or deformation or in changes in material properties over time in a structural object or structural members. In general, actions can be classified as follows:

Direct action: A force that acts directly on a structural object or structural member;

Indirect action: An action that causes a force to be generated against a structural object or structural member, such as forced displacement or a change in the volume of materials; and

Environmental action: An action that causes changes in properties and materials of a structural object, such as temperature, moisture, and substances.

The actions treated in this section can be classified as shown in **Commentary Figure 6.4.1**.



Commentary Figure 6.4.1 Relationships between actions.

6.4.2 Dead load

In principle, the nominal value of the dead load is to be calculated based on the dimensions in design documents.

Dead loads are to be divided into fixed dead loads and added dead loads. The action correction coefficient when deriving the characteristic value from the nominal value should generally be 1.0 for a fixed dead load. For an added dead load, it is to be determined with its variation taken into consideration.

Commentary: "Dead load" refers to the action caused by the weight of the materials that constitute or accompany the structural object.

The characteristic value of dead load is to be based on actual weight, with unevenness determined through investigation. As unevenness in weight is not very large in an actual structural object, it was decided to calculate the nominal value based on design dimensions. For the unit weight used in calculation, the values shown in Commentary Table 6.4.1 may be used as a reference. When the actual weight is known, however, that value should be used instead.

As the unit weight of lightweight aggregate concrete differs with the type of aggregate used and its mixing ratio, it should be set according to the type of aggregate and its mixing ratio.

Commentary Table 6.4.1 Unit weights of materials.

Material	Unit weight (kN/m^3)	Material	Unit weight (kN/m^3)
Steel/ Cast steel/ Forged steel	77	Concrete	22.5–23.0
Cast iron	71	Cement mortar	21.0
Aluminum	27.5	Wood	8
Reinforced concrete	24.0–24.5	Bituminous material	11
Prestressed concrete	24.5	Asphalt concrete pavement	22.5

6.4.3 Live load

The characteristic value of live load must be determined with its variable properties assumed.

In examination of fatigue failure, all variations during the design service life are to be taken into consideration, and actions of appropriate magnitude and the number of equivalent repetitions corresponding to these are to be determined.

Commentary: Dynamic response due to live load should be replaced with static action by appropriately evaluating the rate of increase with respect to the static response. If there is a standard value for live load, as with a railway bridge or traffic bridge, then this is to be multiplied by the action correction coefficient ρ_f appropriate to the verification item and set to the characteristic value.

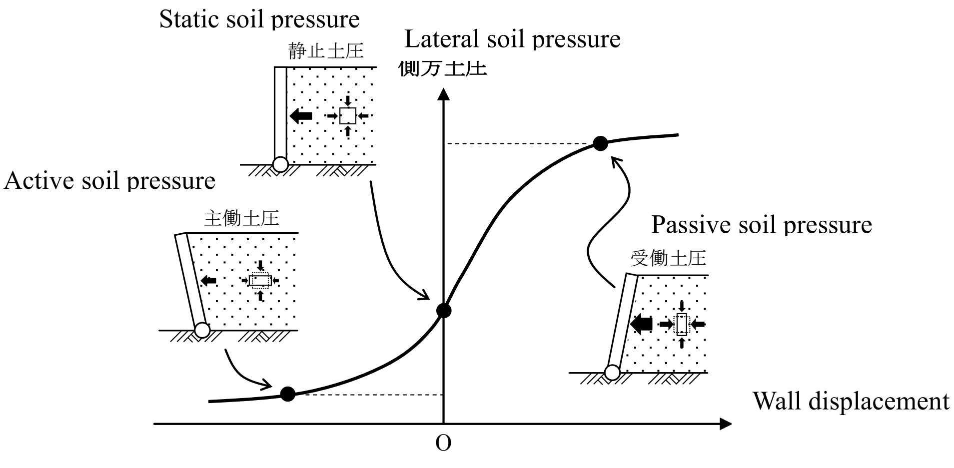
6.4.4 Soil pressure

(1) The soil pressure acting on the structural object is to be determined with consideration of the soil cover thickness, soil mass, soil looseness, construction method, *etc.*

(2) Soil pressure includes vertical soil pressure and lateral soil pressure, with the latter further including static soil pressure, active soil pressure, and passive soil pressure. These soil pressures are to be determined for each limit state in accordance with the type and rigidity of the structural object and the type of soil. In doing so, changes in the state of the structural object and the ground over time should be taken into consideration.

(3) If the nominal value of the soil pressure has been determined, then the characteristic value of soil pressure is to be set to the value obtained by multiplying the nominal value by the action correction coefficient ρ_f.

Commentary: The soil pressure acting on the structural object includes vertical soil pressure and lateral soil pressure, with the latter further divided into static soil pressure, active soil pressure, and passive soil pressure. In general, vertical soil pressure and static soil pressure can be regarded as direct actions determined by the state of the soil quality around the structure, while active soil pressure and passive soil pressure, as indicated in **Commentary Figure 6.4.2**, are interactions that accompany deformation of the structural object and are by their nature indirect actions. Regarding the upper load on the ground surface, the soil pressure must be determined with consideration of the relationship between the load distribution range and the soil cover thickness.



Commentary Figure 6.4.2 Lateral soil pressure as interaction with the structural object.

It is advisable to set the soil pressure assumed at the time of design with consideration of changes in the state of the ground and the structural object over time. As deriving variation in soil pressure over time through calculation is difficult at present, it should be determined based on experiments and measurement data. When that is not possible, it may be determined based on fundamental theories such as Rankine's soil pressure and Coulomb's theory of earth pressure based on limit theory.

6.4.5 Water pressure, fluid force, and wave force

Hydrostatic pressure, hydrodynamic pressure during earthquakes, fluid force, and wave force are to be determined in accordance with the type of structural object, environmental conditions, dimensions of structural members, *etc.*

Commentary: The characteristic value p_w of hydrostatic pressure of a structural object in contact with water or a structural object below groundwater level should generally be derived using:

$p_w = w_0 \cdot h$[kN/m^2], (Commentary 6.4.1)

where h: Depth (meters) below the water surface, determined for each limit state, and

w_0: Unit weight of water (kN/m^3).

However, for structural objects below the level of groundwater, this should be reduced if investigation of pore water pressure, *etc.* reveals that water pressure distribution is not as shown in Equation (Commentary 6.4.1). For a structural object in contact with liquid other than water, pressure derived using the unit weight of that liquid instead of the unit weight of water in Equation (Commentary 6.4.1) acts on the structural object.

The effects of hydrodynamic pressure during an earthquake must be determined with consideration of the coupling of the ground, the structural object, and water.

In general, the characteristic values of hydrodynamic pressure during an earthquake are to be determined with consideration of flow characteristics near the boundary of the structural object with water, the rigidity and form of the structural object, the water depth, and other factors. The characteristic value of fluid force due to flow is to be determined with consideration of the dimensions and cross-sectional shape of the structural object, flow conditions, and other factors.

The characteristic value of wave force is to be determined with consideration of the form and location of the structural object and the properties of waves. This is greatly affected by geography in the case of a tsunami, so the distance from the coast, altitude of the site, and the surrounding terrain should be fully taken into account, along with the waveform, flow velocity, and inundation depth of the tsunami.

6.4.6 Prestress force

Prestress force is to be calculated with consideration of the effects of changes over time on the tensile force of the tendons.

Commentary: Prestress force should be derived using "Design: Standards" Volume 8, Section 3.

6.4.7 Wind load

(1) Wind load is to be determined in accordance with the type of structural object, environmental conditions, dimensions of structural members, *etc*.

(2) Dynamic effects due to wind must be considered for structures and structural members with high flexibility.

Commentary: Regarding (1): The characteristic value W of wind load can generally be derived through:

$$W = \frac{1}{2}\rho \cdot v_z{}^2 \cdot C \cdot A \ \text{[N]} \qquad \text{(Commentary 6.4.3)}$$

where ρ: Air density (kg/m^3);

v_z: Design wind velocity (m/s), determined for each limit state;

C: Drag force coefficient, varying with cross-sectional shape of structural members; and

A: Projected cross-sectional area of structural members (m^2).

The design wind velocity is to be determined for each limit state. In general, it is to be derived from the 10-minute average velocity at an altitude of 10 m above the ground or sea surface, taking into account wind velocity observation records, design service life, wind velocity reproduction period, and other factors and correcting for elevation, horizontal and vertical length of the structural object, effects of ground roughness, effects of shielding/convergence by surrounding features, and effects of temporal and spatial variation in wind velocity.

For drag coefficient, the values shown in Commentary Table 6.4.2 should be used, according to the cross-sectional shape of structural members.

Commentary Table 6.4.2 Drag coefficients.

cross-sectional profile	drag coefficient	cross-sectional profile	drag coefficient
wind direction → circular cross-section	1.2 (0.7)*1	Wind direction → 2, r, d, 1 Rectangle ( ) when r>d/12	1.5 (1.1)
→ Flat or similar shape	2.2	→ Rectangle ( ) when r>d/12	2.1 (1.5)
→	1.8	→ 1 Rectangle 0.7	2.7
→ Square (Diagonal direction)	1.5	→ 1 Rectangle ( ) when r>d/29 0.5	2.3 (2.1)

Note 1: Indicates a value above the critical Reynolds number.

Regarding (2): For elongated structural members such as suspension materials and structural objects prone to sagging, as in suspension bridges and cable-stayed bridges, the influence of vibration caused by wind must be considered.

6.4.8 Snow load

The characteristic value of snow load is to be determined in accordance with regional conditions and the condition of the structural object.

Commentary: In regions where snow load must be considered, the value must be set in accordance with actual conditions. In general, this should be derived through:

$SN = w_s \cdot z \cdot I$ [N/m^2], (Commentary 6.4.3)

where SN : Characteristic value of snow load;

w_s: Design unit weight of snow (N/m^3);

z: Design ground snow depth (m);

I: Coefficient due to gradient:

$I = 1 + (30 - \theta)/30$

however, I=1.0 when $\theta \leq 30°$ and I=0 when $\theta \geq 60°$; and

θ: Gradient of the surface of the structure where snow falls (°).

The unit weight of snow varies with snow quality and the state of snow cover. As snow depth varies significantly by area and by whether snow removal is performed, it is advisable to determine snow load in accordance with actual conditions in the region and the maintenance status of the structural object, based on snow observation data and taking into consideration the design service life of the structural object.

6.4.9 Effects of concrete shrinkage and creep

(1) The effects of concrete shrinkage and creep must be determined with consideration of factors including materials, environmental conditions, and dimensions of structural members. If changes occur in the structural system during and after construction, then those effects must be considered.

(2) In the design of statically indeterminate structural objects such as rigid frames and arches, it is generally assumed that shrinkage and creep effects are uniform over the cross sections of the structural object.

Commentary: The effects of shrinkage and creep should generally be considered in the verification of usability and of fatigue failure. In a statically indeterminate structural object, a statically indeterminate force is generated by shrinkage of the concrete and by deformation of structural members owing to the effects of creep; accordingly, these effects must be taken into consideration. In a statically determinate structural object, too, if free deformation is restrained or if significant differences exist in contraction deformation in a given cross section, then this effect must be taken into consideration.

6.4.10 Effects of earthquakes

(1) With regard to the effects of earthquakes, all actions arising from seismic motion must be taken into account.

(2) In the verification of seismic action in the structural object, seismic motion in accordance with required performance is to be set.

(3) For seismic motion, the properties of seismic motion required in a verification are to be extracted with consideration of factors including the degree of seismic activity around the construction site, seismic source properties, and the properties of propagation of seismic motion from the seismic source to the construction site.

Commentary: Regarding (1): The following are generally taken into consideration as effects of earthquakes:

① Inertial force caused by the mass of the load and negative load of the structure;

② Dynamic interaction between the structural object and the ground;

③ Hydrodynamic pressure during an earthquake; and

④ Liquefaction of the ground and resulting lateral flow.

The mass that gives rise to inertial force owing to vibration during an earthquake includes the mass of the structural object as well as the mass loaded onto the structure. Cases such as inertial force due to mass loaded onto the structural object acting at the same time as an earthquake should be considered. In general, it is sufficient to take into consideration mass that creates permanent actions and secondary variable actions.

Actions caused by the dynamic interaction between the structural object and the ground arise from the relative differences in the dynamic responses of the two.

In a tank or other structural object containing liquid, it is necessary to consider the hydrodynamic pressure caused by negative load and the hydrodynamic pressure associated with liquid oscillation generated during an earthquake.

Taking measures to prevent ground liquefaction is the basis of seismic design. When such measures are technically difficult or costs are significant, it is necessary to perform design with full consideration of the effects of ground liquefaction on the performance of the structural object. If the ground surface is sloped or unsymmetrical soil pressure acts constantly, then liquefaction may cause ground flow to occur, and the effects of this must be considered.

Regarding (2): "Design: Standards" Volume 5 states that two levels of seismic intensity, Level 1 and Level 2, should be used. It was decided that seismic action made to correspond to a level requiring that the structural object is not damaged, and to a level that considers damage occurring to the structural object and extends to the damage process, should not be physically determined independent of performance. Moreover, it was decided that seismic motion for verification should be set to correspond to the performance required for the structural object and its level.

Regarding (3): In principle, seismic motion used in verification should be set with consideration of factors including the degree of seismic activity around the construction site, seismic source properties, and the properties of propagation of seismic motion from the seismic source to the construction site. In general, it should be set based on a method that considers multiple observed waveforms and the crustal fracturing process at the seismic source area. If seismic motion at the construction site cannot be set appropriately, then a simulated seismic motion waveform that incorporates a vibration component with a large effect on the structural object may be used.

The ground surface or the engineering base surface is to be set as the reference position, and characteristic values such as maximum acceleration, maximum velocity, and maximum displacement, or appropriate values from the response spectrum, the Fourier spectrum, time-history waveform, and so on that match the purpose of verification should be selected as seismic properties.

6.4.11 Environmental actions

(1) The effects of temperature change are to be determined with consideration of the temperature around the structural object and changes in it over time.

(2) The effects of moisture are to be determined with consideration of the relative humidity around the structural object, the supply of moisture, and the changes in these over time.

(3) The effects of substances that affect the structural object are to be determined with consideration of their concentration around the structural object, the status of their supply, and changes in them over time.

Commentary: Regarding (1): In addition to introducing thermal stress into the structural object during service, temperature changes initially affect the occurrence of thermal cracking caused by heat of hydration of cement in concrete. In the long term, temperature affects the rate and degree of progress of various deterioration phenomena.

In the design of statically indeterminate structural objects such as rigid frames and arches, it is generally assumed that temperature rises and falls are uniform over the cross sections of the structural object. However, in the case of a structural object for which it is not possible to ignore effects due to temperature differences between structural members or in parts of structural members, these effects are to be taken into consideration. The characteristic values of temperature rise and fall are to be determined from the difference between the maximum and minimum annual average temperatures and monthly average temperatures.

Temperature differences arise between structural members affected by solar radiation and those in the shade. In the case of a structural object for which it is not possible to ignore the statically indeterminate force generated by this temperature difference, the characteristic value of the temperature difference is to be determined with consideration of the site conditions of the structural object, weather conditions, *etc.* In the case of low-temperature and high-temperature tanks, a significant temperature difference occurs between the inside and outside of the structural members, and the effects of this temperature difference must be considered.

The atmospheric temperature at the time that construction of the structural object is completed may not match the annual average temperature. When construction is completed in the summer or winter, the temperature change initially differs from that shown in this section, but thermal stress can be considered to vary above and below that of the annual average temperature due to concrete creep. Therefore, in the design of a statically indeterminate structural object, it is generally not necessary to consider the time of construction work. However, when calculating sinking allowance, amount of expansion and contraction, *etc.*, the temperature at the time of construction must be taken as a benchmark.

In areas where temperatures below freezing occur repeatedly, there is concern about deterioration of concrete due to freezing damage. The main causes of the occurrence and progression of frost damage are the freeze–thaw action and the water-content state of the concrete. Therefore, in examining frost damage, the minimum temperature, the number of freeze–thaw cycles, and the meteorological actions that supply water to concrete must be considered as environmental actions.

Regarding (2): The progression of drying in concrete is a cause of drying shrinkage. The water-content state of concrete also affects carbonation, freezing damage, intrusion of chloride ions, and corrosion of steel materials in concrete. The water-content state of concrete is affected not only by the surrounding relative humidity but also by rainfall, solar radiation, and moisture from the ground. These may also differ by component of the structural object, which must be kept in mind when precisely deriving the water-content state of the concrete in the structural object.

The corrosion of steel material in concrete under an environment not subjected to the effects of chloride ions is affected by the water-content state of the concrete surrounding the steel material, which is associated with supply of moisture. Therefore, it is necessary to accurately assess the status of the supply of water into the structural object. Moreover, carbonation of concrete, which lowers the resistance of steel material to corrosion, is greatly affected by the degree of drying of the concrete. Carbonization of concrete progresses through the intrusion of atmospheric carbon dioxide. In this case, carbonization progresses more quickly in concrete exposed to surfaces that receive a relatively large amount of solar radiation and are prone to drying, than in an environment that is subject to rainfall or that does not facilitate drying. As civil engineering structural objects are often constructed outdoors, when examining carbonation, the degree of drying of concrete, which affects intrusion by carbon dioxide, must be considered as an effect of environmental actions.

Regarding (3): The intrusion of chloride ions into concrete is greatly affected by the supply of chloride ions to the concrete surface, owing to airborne salt, seawater spray, tides, antifreeze agents, and so on.

In the examination of salt damage, a method is used in which the effect of environmental action is considered from the concentration of chloride ions on the concrete surface. The amount of salt that impacts a structure and affects chloride ion concentration on the surface of concrete is affected not only by distance and height from the coast but also by regional characteristics. Even in the same region, it can differ greatly with how rainwater strikes the surface. Therefore, it is appropriate to set the chloride ion concentration on concrete surfaces in accordance with weather conditions in the target region and the conditions surrounding the structural object.

When examination of chemical erosion is necessary, erosion strength must be determined as an environmental action related to chemical erosion, with consideration of the types of chemical substance that erode the concrete, concentrations of the chemical substances, temperature and humidity conditions, and other factors.

6.4.12 Load during construction work

During construction work, it is necessary to determine the load with respect to dead load of the structure, construction equipment, wind, earthquakes, *etc.*, taking into consideration the construction method, the structural system during construction, and the duration of the construction period.

Commentary: During construction, the effects of wind load and earthquakes may be set at values lower than the characteristic values of actions derived according to Sections 6.4.7 and 6.4.10, in accordance with the duration of the construction period and the importance of the structural object.

6.4.13 Effects of fire

With regard to the effects of fire, it is necessary to set characteristic values of actions, taking into consideration the relationship between damage to the structural object due to high temperature and the required performance of the structural object after the fire.

Commentary: The effects of fire vary with the environment of the structural object, but characteristic values should be set with consideration of factors including the relationship between the combustion time and the temperature of the structural materials, in accordance with the environment of the structural object.

6.4.14 Other actions

When it is necessary to consider actions other than the above, these will be determined in accordance with actual conditions.

Commentary: Other actions include the effects of collision load, ground motion, and movement and rotation of supporting points.

When performing evaluations, collision loads can be classified into static force actions, such as in an automobile or other collision, and into a special failure mode, such as the crash of a flying object. In the former case, the collision load can be replaced by a static action using a method in which mechanical equivalence is considered, after which verification involving safety is performed. In the latter case, intrusion, backside peeling (the scattering of concrete from the backside of the collision surface), and penetration are considered as special states due to the crash of a flying object, and in verification the velocity, weight, and size of the flying object must be determined.

If ground motion due to consolidation settlement of the foundation after completion of the structural object, or the effects of movement or rotation of the fulcrum of a statically indeterminate structural object are assumed, then those effects must be appropriately considered.

Chapter 7 Calculation of Response Values

7.1 General

In the calculation of response values, a structure is modeled in line with its form, threshold conditions, the state of actions, and the limit states to be considered, after which analysis is performed using an analytical model with demonstrated reliability and accuracy. Cross-sectional force, deflection, stress, strain, crack width, carbonation depth, chloride ion concentration, and other values must be calculated in line with verification indicators.

Commentary: This chapter presents matters related to methods of calculating the response values that are used in verification of limit states for the required aspects of performance set in Chapter 4. It addresses modeling, selection of methods of structural analysis, and techniques for the calculation of design response values. To ensure the reliability of verification results, response values must be calculated in accordance with the principles of this chapter. When calculating response values using advanced nonlinear analysis or other analytical methods, particular attention should be paid to the assurance of reliability. In the verification of durability and initial cracking, too, design response values for the structure must be derived in accordance with the principles of this chapter.

7.2 Modeling

7.2.1 General

(1) The scope of analysis and analytical dimensions are set and the actions of a structure are modeled in line with the structure's response properties due to actions.

(2) In modeling, the scope of analysis, encompassing the structure, the ground, boundary elements, etc., is set in line with the range over which response occurs.

(3) When the scope of analysis is set to include the ground, modeling must be able to appropriately consider its effects.

Commentary: In modeling a structure, the range over which responses are generated by actions must be collectively set as the scope targeted by analysis. However, when the effects of these are small or when the effects can be taken into account by the boundary conditions of the area of analysis, modeling may be performed with the scope of analysis separated into structural elements.

7.2.2 Modeling of actions

(1) Actions are modeled appropriately according to the properties of the actions and their effects on the limit states to be verified.

(2) Actions that result in aggregates of forces or in deformation and internal constraint in structures may be modeled as loads. Actions due to the environment may be modeled as the infiltration and transport of substances that act as deterioration factors in materials.

(3) Loads may be modeled as having effects equivalent to actual loads or may be modeled more conservatively by means such as simplifying the state of distribution or replacing dynamic actions with static actions.

(4) In the calculation of response values, loads should be considered in the manner that is most unfavorable with respect to verification results.

Commentary: Regarding (3): When calculating response values of a structure by replacing dynamic effects of impacts from vehicles, wind, earthquakes, etc. with static loads, the shape of the load distribution and the structure must be modeled so that the static response values are equivalent to dynamic response values or are more conservative.

7.2.3 Modeling of structures

(1) A structure should be modeled in line with its form, etc.

(2) Depending on its form, a structure may be analyzed by hypothesizing a simplified structural model consisting of slabs, beams, columns, Rahmen frames, arches, shells, and combinations of these.

(3) Structural members may be modeled as rod members or as plane members.

(4) Wire rods or plane members may be modeled using finite elements or wire rods.

Commentary: Regarding (1): When modeling a structure, columns and beams may be modeled as rod members, and walls, floors, and other members with a broad surface may be modeled as slabs, shells, or other plane members, with the structure modeled as an aggregate of these. The progress of carbonation, permeation of water, and infiltration of chloride ions in the verification of durability may generally be handled through one-dimensional analysis that progresses from the surface of the structure to the interior.

Regarding (2): Structural analysis of these and related verification methods should be according to "Design: Standards" Volume 1.

Regarding (3) and (4): Modeling of structures has typically been performed using wire rods. With advances in numerical analysis technologies, however, modeling by the finite element method has also been adopted. In the finite element method, the form of a structure can be modeled with considerable precision through the use of 2D or 3D elements, but this may entail problems including a dramatic increase in required computational capacity. It should also be noted that the precision of derived response values differs by analytical technique.

7.3 Structural analysis

(1) Methods that enable the calculation of verification indicators must be used in structural analysis, in line with the modeling of actions and structures. However, if verification indicators cannot be directly obtained from structural analysis, structural analysis methods that allow conversion to verification indicators by an appropriate method may be used.

(2) The nonlinear effects of the structural members that compose a structure should be considered, in line with responses. However, when the effects of non-linearity in structural members do not affect verification indicators such as cross-sectional force, or when these clearly yield a conservative and rational evaluation, structural members may be treated as linear.

Commentary: Regarding (1): An analytical theory can be broadly classified as a (material) linear theory or nonlinear theory, depending on whether the assumed stress-strain relationship of the material is linear or not. It can also be broadly classified as a first-order theory or a second-order theory, depending on whether secondary effects of deformation (geometric nonlinearity) are ignored. Linear analysis is performed according to elastic first-order theory; cases involving other combinations are nonlinear analyses. Analytical theories that are applicable to the limit state of mechanisms include plastic analysis that assumes rigid plastic bodies.

Verification does not necessary require the use of the same structural model or analytical theory for each limit state. Structural models and analytical theories suited to particular limit states can be used. The structural analysis coefficient γ_a must be selected as appropriate for the analytical theory used to calculate response values.

Regarding (2): Other than linear analysis, analytical methods include rigorous nonlinear analysis that takes into account material and geometric nonlinearity; equivalent linear analysis that takes into account approximate effects of nonlinearity by using structural member stiffness lowered below elastic stiffness to perform linear analysis; and plastic analysis methods used for beams and slabs.

In cases such as calculating the design response values used in verification of damage or collapse of a structure or verification of earthquake effects, when performing analysis with consideration of hysteresis characteristics and nonlinear properties that include the plastic region of structural members, it is necessary to use a nonlinear hysteresis model as the material model to derive a nonlinear model or a hysteresis model for structural members, or to use a hysteresis model or nonlinear model based on past research findings.

Using discretization techniques such as finite element analysis based on nonlinear theory to verify safety with respect to fracture also enables verifying the safety of structures and structural members using verification indicators other than cross-sectional force and cross-sectional load bearing capacity. In such cases, verification may be performed by setting indicators for the failure mode and the limit state and comparing the derived design response values and design threshold values. The safety coefficient must be set through empirical validation of accuracy and scope of application. The use of nonlinear finite element methods should be according to "Design: Standard," Volume 10.

7.4 Calculation of design response values

7.4.1 General

Design response values should be calculated by using an appropriate method to convert response values obtained in **7.3** into verification indicators.

Commentary: Methods for calculating design response values include that of calculating cross-sectional force using a wire rod model and that of using the finite element method to calculate stress, strain, etc. as response values. This section presents a calculation method for converting design response values from cross-sectional force to verification indicators, such as material stress intensity.

7.4.2 Calculation of cross-sectional force

(1) When the axial force and flexural moment acting on a structural member are derived using the finite element method, calculation may be performed by using stress distribution inside the cross section to integrate in the direction of cross-sectional depth. Shear force may be calculated from the equilibrium condition of flexural moment distribution.

(2) When using a structural member model based on wire rods, the cross-sectional force obtained through structural analysis may be used to calculate the cross-sectional force of structural members.

Commentary: Regarding (1): Cross-sectional force is often used as a verification indicator for structural members. In the finite element method, however, methods have been proposed for using strain and other localized information to calculate localized properties such as cracks and reinforcement yield in structural members, as well as properties including axial yield strength, flexural yield strength, and shear yield strength. Therefore, taking advantage of the features of the finite element method, indicators that use localized information substituted for states that correspond to the cross-sectional force required for verification may be used as response values.

Regarding (2): When using an analytical method that confers nonlinearity using the relationship between the flexural moment and the angle of rotation in the wire rod model, the cross-sectional force obtained through structural analysis may be used without change as the cross-sectional force of structural members. When a fiber model is used, it is acceptable to use the cross-sectional force that was substituted for element stress in the same manner as cases in which the finite element method is used.

7.4.3 Calculation of stress intensity

(1) When using finite element analysis, the stress intensity of concrete and reinforcing bars may be calculated based on the element strain.

(2) When using a wire rod model, the stress intensity of concrete and reinforcing bars may be calculated based on the compatibility conditions for displacement within the cross section where the design cross-sectional force acts.

In general, when adhesion between reinforcing bars and concrete can be assumed, the stress intensity due to axial force and flexural moment may be calculated based on the following assumptions:

① Fiber strain is proportional to the distance from the neutral axis of the structural member cross section.

② In concrete and steel materials, stress and Young's modulus are set based on the stress-strain relationship.

③ The tensile stress of concrete is generally ignored.

Commentary: When an analytical method such as a fiber model that makes direct use of the stress-strain relationship of materials is applied as the structural analytical method, stress intensity may be calculated in the same manner as in finite element analysis, or the response values of materials may be calculated according to **7.4.3**, using the cross-sectional force obtained from **7.4.2**.

When calculating thermal stress associated with the hydration reaction of cement in concrete at a young material age, the effects of creep and changes over time in Young's modulus and other physical properties of concrete associated with the progress of hydration must be appropriately considered.

7.4.4 Calculation of strain

When using finite element analysis, element strain may be used to calculate strain that represents material states of damage such as cracking, peeling of cover concrete, compression failure in concrete, yielding of reinforcement bars, and buckling.

Commentary: Cracking, peeling of cover concrete, compression failure, yielding of reinforcing bars, buckling, and other states may be used as limit states for repairability.

7.4.5 Calculation of crack width

(1) In principle, crack width in reinforced concrete structural members and prestressed concrete structural members should be calculated using Equation (7.4.1), with consideration of (2) and (3).

$$w = l(\varepsilon_s - \varepsilon_c) \tag{7.4.1}$$

where, w : crack width

l : crack spacing

ε_s : average strain in reinforcing bars between cracks

ε_c : average strain in concrete surface between cracks

(2) When calculating flexural crack width, the following items must be considered:

(a) Flexural crack spacing should be calculated with consideration of the steel material and concrete adhesion properties and the reinforcing bar arrangement surrounding the tensile steel.
(b) The strain of steel material should be calculated based on the state in which the stress intensity of concrete at the position of the steel material is 0.
(c) For the surface strain of concrete between cracks, factors including the effects of concrete shrinkage and creep should be considered, taking into account the effects of the concrete mix conditions, water supply conditions spanning the concrete construction work stage to the service period, and the material age at which cracking occurred.
(3) Shear crack width may be calculated from Equation (7.4.1), using appropriate crack spacing.

Commentary: Regarding (1): Equation (7.4.1) expresses the compatibility conditions for crack width, elongation of steel material, and elongation or shrinkage of concrete between cracks.

(2) Regarding (a): Crack spacing in reinforced concrete and prestressed concrete is affected by many factors, with the type of steel material, cover, effective cross-sectional area of concrete, diameter of steel material, ratio of steel material, number of layers of steel material, surface shape of steel material, and properties of the concrete identified in past research as among the chief factors.

The crack spacing gradually decreases as stress acting on the steel material increases, and is smallest when a stable state is reached in which cracking no longer occurs. Therefore, care must be taken as using crack spacing for a stable state may underestimate crack width.

(2) Regarding (b): In principle, calculation of stress intensity in steel material should consider the effects of all axial steel material arranged in the cross section. Here, "effects of axial steel" refers to consideration of the effects of constraint by steel material due to concrete shrinkage and creep in the calculation of stress intensity in steel material. Stress intensity should be considered as a reaction force due to the steel material until the reaction force of the concrete at the location of the steel material reaches 0.

When calculating the stress intensity of the steel material, it is necessary to take into account the fact that stress intensity will be greater in cases in which the steel material was subjected in the past to actions greater than the actions considered in calculating stress intensity.

(2) Regarding (c): The surface strain of concrete between cracks should be calculated with consideration of the effects of shrinkage and creep in the concrete between cracks. Calculation should be performed with consideration of the test value of shrinkage strain and the material age at which cracking occurs, taking into account construction work processes such as concrete pouring and removal of formworks and falsework in the structure under consideration.

Regarding (3): Shear crack width can also be calculated from Equation (7.4.1) in the same manner as flexural crack width. However, because shear crack spacing is known to differ from flexural crack width spacing and equations for flexural crack width estimation based on experimental results cannot be applied, it must be derived separately. While shear crack width is mainly affected by the elongation of shear reinforcement steel, it also intersects, and is thus affected by, tensile reinforcement steel. It is necessary to consider the fact that, unlike flexural cracking, shear crack width does not perpendicularly intersect with reinforcing steel.

7.4.6 Calculation of displacement/deformation of structural members

(1) Short-term displacement/deformation should be calculated with consideration of the effects of damage such as cracking, cover concrete peeling, compression failure in concrete, yielding of reinforcing bars, and buckling.

(2) The amount of long-term displacement/deformation must be calculated based on the mechanical and thermodynamic properties of concrete, with consideration of the effects of shrinkage and creep associated with water transport and the propagation of hydration in structures, and taking into account the mix conditions of the concrete and environmental conditions such as temperature and humidity.

Commentary: Regarding (1): Points of inflection in the stiffness of structural members due to peeling of cover concrete, compression failure, yielding of reinforcing bars, buckling, etc. may be used as limit states for repairability.

Regarding (2): Calculating the displacement of a concrete structure based on thermodynamics and mechanics means calculating autogenous shrinkage, drying shrinkage, and creep in concrete as behaviors of a substance that possesses a microstructure.

For the case of a structure that possesses a standard structural form with a demonstrated track record, it was decided that the effects of concrete creep and shrinkage may be modeled for each part of a structural member with consideration of mechanical and thermodynamic properties of concrete, and that techniques for calculation based on the linear creep law, which assumes linearity in stress and creep deformation due to external forces, may be used.

7.4.7 Calculation of response values related to durability

(1) The progress of steel corrosion should be derived with consideration of carbonation, permeation by water, infiltration by chloride ions, effects of cracking, and environmental actions.

(2) The permeation of water into concrete should be derived with consideration of the properties of the concrete, the effects of cracking, and environmental actions such as wetting and drying.

(3) The progress of carbonation in concrete should be derived with consideration of the properties of the concrete, the effects of cracking, and environmental actions such as wetting and drying.

(4) The infiltration of chloride ions into concrete should be derived with consideration of the properties of the concrete, the effects of cracking, and environmental actions such as supply of salt.

(5) The progress of freezing in concrete should be derived with consideration of the properties of the concrete and environmental actions such as temperature and the supply of water.

(6) The progress of chemical erosion in concrete should be derived with consideration of the properties of the concrete and environmental actions such as the concentration of substances that cause erosion.

Commentary: Regarding (1): For the progress of steel corrosion in an environment in which the supply of chloride ions is low, focus must be placed on carbonation and permeation by water, with consideration given to the concrete properties, cracking, and environmental actions that affect these. Conversely, in the case of steel corrosion associated with infiltration by chloride ions, the passive film on the surface of the steel material will be destroyed and corrosion will progress if the chloride ion concentration in the concrete at the location of the steel material exceeds the threshold concentration for occurrence of steel corrosion. When placed in an environment of cyclical wetting and drying or other environment accompanied by oxygen supply, the rate of corrosion increases as the chloride ion concentration at the position of steel material increases, and, depending on conditions, corrosion may progress rapidly.

Regarding (2): The permeation of water into concrete is affected by environmental actions such as supply of seawater and rainfall as well as by the denseness of the concrete. The possibility that water may reach the position of steel material through cracks must also be considered.

Regarding (3): The progress of carbonation in concrete is generally expressed by carbonation depth. Factors that affect the progress of carbonation include water content, air permeability, and alkali content. The properties of concrete are often expressed using a carbonation rate coefficient that incorporates the effects of these factors. As cracking in concrete affects air permeability, the effects of this must also be appropriately considered. Because the water content of concrete affects the progress of carbonation, it is necessary to consider temperature, humidity, solar radiation, and other environmental actions that affect the water content of concrete.

Regarding (4): The infiltration of chloride ions into concrete proceeds under the effects of factors including transport and concentration associated with cyclical wetting and drying, the concentration diffusion of chloride ions in pore moisture, and the fixation of chloride ions by hardened cement. In general, calculation models by which the transport of chloride ions in concrete observed as a result of the preceding is approximated as a diffusive transport phenomenon are often used. In this case, properties of concrete regarding the transport of chloride ions are expressed using the diffusion coefficient. If cracking is present, the diffusion coefficient must be evaluated with consideration of the effect of the cracking. The environmental action with the greatest effect is the amount of chloride ions supplied to the concrete surface by airborne salt, seawater spray, tides, and deicing agents.

Regarding (5): The progress of freezing damage in concrete is expressed as damage to the surface, such as scaling and popout, and as damage to the concrete inside the structure. However, because predicting the progress of freezing damage under given conditions is difficult, and because the progress of freezing damage can be prevented if the freezing resistance of concrete above a certain level is secured through the air content and the water-to-cement ratio, it is often unnecessary to derive response values for freezing damage.

Regarding (6): The progress of chemical erosion in concrete is expressed by erosion depth. The causes of erosion include various substances and bacteria. Erosion depth is determined by the resistance of concrete to specific erosion actions and by environmental actions such as the concentration of substances on the surface that indicates the strength of erosion action.

Chapter 8 Verification of Durability

8.1 General

(1) Verification must be performed to confirm that a structure will retain required durability.

(2) Verification must be performed to ensure that the required performance of a structure will not be compromised due to steel corrosion caused by carbonation, permeation by water, or salt damage, or due to deterioration of concrete caused by freezing damage or chemical erosion. When these deterioration phenomena occur in combination or when they cannot be comprehensively addressed by the method of verification, the effects of the phenomena must be taken into consideration.

Commentary: Appropriate investigation, planning, and design for the assurance of durability are prerequisites for the verification of durability. In order to ensure the required performance of a structure over its service life, it is general practice to perform design in such a manner that no deterioration or change in state of materials will occur due to environmental actions during the design service life or so that any material deterioration that does occur will remain within a minor range that will not cause a decline in the performance of the structure. The alkali-silica reaction and salt damage caused by chloride ions present in concrete from the time of mixing should be given consideration based on investigations of the materials used, as shown in Chapter 2, 2.1 (2) and (4) in "Design: Standards," and appropriate remedial measures must be taken when these present a possibility of compromising durability. Decisions such as the choice to use a mineral admixture affect not only the verification of every item concerning durability but also the verification of cracking caused by hydration of cement, of cracking associated with drying shrinkage, and of usability (cracking, long-term displacement/deformation), and therefore require special care.

To confirm that a structure passes verification of safety, usability, and restorability as indicated in Chapters 9, 10, and 11 and that it will retain required safety, usability, and restorability over its design service life, the structure must pass verification of durability and must be guaranteed against defects caused by the deterioration of materials over the design service life.

Factors that affect the durability of concrete structures may act independently or may act in combination, which can often be addressed by evaluating the effects of the dominant factors independently. However, when structures are subjected to combined actions instead of independent actions, deterioration may proceed more rapidly. In such cases, modifications to structural details, the use of high-durability materials, or other remedial measures are necessary to ensure durability.

Standards should be used as reference for specific methods of verification.

Chapter 9 Verification of Safety

9.1 General

(1) Verification must be performed to confirm that a structure will retain required safety throughout its design service life.

(2) As a general rule, verification of safety with respect to the failure or collapse of a structure should be performed by confirming that the structure will not reach threshold states for failure under design actions.

(3) As a general rule, threshold states for failure should be defined as threshold states for load bearing capacity, stability, *etc*. in the structure, and physical quantities such as cross-sectional force, strain, displacement, and deformation should be set as indicators.

(4) In the verification of safety with respect to factors other than the failure or collapse of a structure, threshold states must be set in line with the usage and functions of the structure, and it must be confirmed that the threshold states will not be reached.

Commentary: Regarding (1): A structure must maintain required safety throughout its design service life. Threshold states that are consistent with safety must be set and must be subjected to examination, using means for which the accuracy and scope of application have been made clear. These Standard Specifications adopt methods of verifying the safety of structures without considering the deterioration of materials during the design service life, on the assumption that the verification of durability in Chapter 8, the verification of initial cracking in Chapter 12, and the construction work performance in the "Construction" volume are satisfied.

Regarding (2) and (3): Threshold states with respect to load bearing capacity are threshold states with respect to variable actions or other actions that occur during the design service life and that are reasonably resisted by the load bearing capacity of the structure. Conversely, threshold states with respect to stability are threshold states with respect to seismic actions, *etc*. that occur during the design service life. Restated, they are threshold states with respect to actions that are reasonably resisted by the load bearing capacity of the structure and by resistance to displacement and deformation, for the purpose of preventing the structure from failing to maintain its form, losing stability, and progressing to failure when displacement and deformation exceed certain values.

Regarding (4): Specifically, safety with respect to factors other than the failure or collapse of a structure involves the safety of vehicle operation and other effects on users, the effects of cover concrete spalling on third parties around the structure, *etc*.

Standards should be used as reference for specific methods of verification.

Chapter 10 Verification of Usability

10.1 General

(1) Verification must be performed to confirm that a structure will retain its required usability throughout its design service life.

(2) As a general rule, verification of usability should be performed by confirming that all constituent structural members and the structure will not reach threshold states for usability under design actions.

(3) As a general rule, threshold states for usability should be set in line with intended use, including aspects of usage and functions demanded of the structure such as watertightness, fire resistance, and comfort during use (external appearance, vibration, *etc*.), and should use physical quantities such as stress, cracking, displacement, and deformation as indicators.

Commentary: A structure or structural member must maintain sufficient comfort and other aspects of usage and functions consistent with intended use during its design service life. Threshold states consistent with these aspects of performance must be set and must be subject to examination, using means for which accuracy and scope of application have been made clear.

For verification indicators and threshold states for comfort during use and other aspects of usage and functionality, a variety of states may be considered, depending on the usage conditions of the structure. The threshold states and verification indicators shown in **Commentary Table 4.1.1** are generally used. These Standard Specifications address comfort during use through verification with respect to appearance, vibration, displacement, and deformation, and address other aspects of usage and functions through verification with respect to watertightness and damage due to fire. Performance items not presented in this chapter must be verified as necessary, using appropriate methods.

Threshold states for usability are often set with the compressive stress intensity of concrete and the tensile stress intensity of reinforcing bars with respect to flexural moment and axial force constrained to appropriate values. The stress intensity of materials in a structure does not correspond to a specified performance item included in aspects of usability of the structure; it has meaning as a premise for verification of usability. However, given that it has been considered in verifications of usability in past designs, it is included as one usability item in these Standard Specifications.

Standards should be used as reference for specific methods of verification.

Chapter 11 Verification of Restorability

11.1 General

(1) Verification must be performed to confirm that a structure will retain required restorability throughout its design service life.

(2) Verification of restorability should be performed by verifying the repairability of a structure, determined according to the environment of the structure, the state of management structures for restoration, *etc*.

(3) The accidental actions addressed in verification of restorability should be effects of earthquakes, effects of fires, and collisions.

(4) Verification of the repairability of a structure should be performed with consideration of the ease of recovering from a decline in functions assumed to be caused by accidental actions. In general, the verification should be performed based on the degree of damage to structural elements.

Commentary: <u>Regarding (1)</u>: These Standard Specifications presume that safety with respect to uncommon accidental actions, such as the effects of earthquakes, will be verified according to Chapter 9. This chapter further defines restorability for the purpose of quickly recovering structural functions that can be expected to decline due to accidental actions so as to enable continuous use. However, this chapter does not presume or address design that sufficiently inhibits damage from major variable actions and environmental actions other than accidental actions, or restoration from such actions.

<u>Regarding (2) and (3)</u>: The restorability of a structure is greatly affected by factors including the ease of securing and inspecting post-disaster restoration materials and delivery routes, restoration technologies, and the establishment of management structures for restoration. Therefore, it was decided that threshold states should be set for each structural system and structural element in accordance with the environment of a structure and the status of management structures for restoration, and that restorability should be secured for the structure through verification of its repairability.

Although the time required for restoration of a structure depends on the extent of damage to structural members, fast restoration is generally possible when restoration materials can be brought in easily, when damage can be induced in areas that facilitate the use of scaffolding and other construction work, and when the structure does not reach the point of collapse. In such cases, the most important consideration is that verification of the structure's stability according to Chapter 9 is satisfied. Therefore, it is vital to ensure sufficient toughness in structural members. At the same time, when access to the structure is difficult, as in the case of locations lacking an access route or sited by a river, the repairability of the structure should be set to enable the maintenance of functions through simple repairs. However, even if damage to structural members can be inhibited, it is important to make the toughness of the structural members sufficiently high.

It was decided that verification of restorability should be performed with respect to decline in functions caused by accidental actions. Accidental actions with a low probability of occurrence include earthquake effects, fire effects, collisions, and strong winds. These Standard Specifications address earthquakes effects, fire effects, and collisions.

Damage due to environmental actions includes steel corrosion, freezing damage, cracking due to chemical erosion, and concrete fragment spalling. These Standard Specifications presume design that is intended to prevent the occurrence of damage due to environmental actions during the design service life or to prevent the performance of a structure from falling below predetermined values if deterioration of materials does occur. Therefore, the decision was made that the repairability of structures is to be addressed through maintenance.

Regarding (4): Because the maintainability of functions when a structure has been subjected to damage varies with the scale and form of the structure, threshold states for repairability of the structure must be set based on the state of damage to structural elements, with consideration of the properties of the structure. In general, threshold values can be set for each of the following: (i) the state in which functions are solidly maintained and in which continuous use is possible without repairing cracks that may occur; (ii) the state in which functions can be restored through simple repairs and in which restoration of structural properties such as load bearing capacity and deformation performance is not required; (iii) the state in which restoration is required for the purpose of recovering structural properties such as load bearing capacity and deformation performance; and (iv) the state in which catastrophic damage occurs (even if not involving loss of life or property) and the structure as a whole cannot be repaired. For the repairability of a structure, threshold values can be set based on the degree of damage to each structural element according to (i) to (iv), and the repairability of the structure can be confirmed by damage not exceeding these.

Standards should be used as reference for specific methods of verification.

Chapter 12 Verification of Initial Cracking

12.1 General

(1) It is necessary to confirm that initial cracking caused by the shrinkage of concrete and by the heat of hydration of cement does not affect the required performance of the structural object.

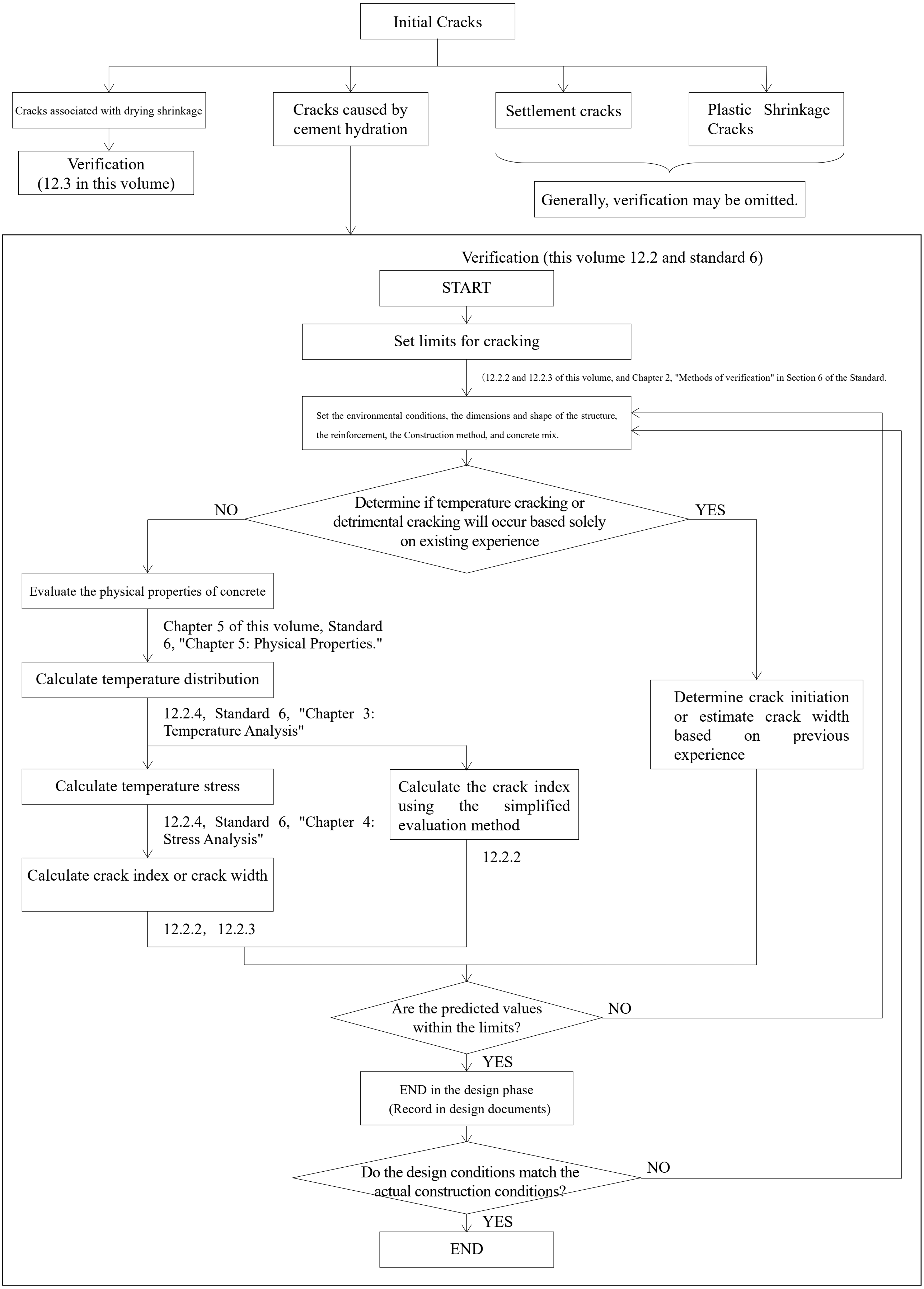
(2) Verification of subduction cracking and plastic shrinkage cracking may generally be omitted.

(3) When cracking caused by the hydration of cement is a problem, verification must be performed by means of evaluation based on past performance or evaluation based on thermal stress analysis.

(4) When crack-inducing masonry joints are provided for the purpose of controlling cracking, their structure and positions must be determined so as to avoid compromising the performance of the structural object.

Commentary: Regarding (1): Because verification of durability, safety, usability, and restorability assumes that initial cracking that affects the required performance of the structural object will not occur, it should be kept in mind that the verification of initial cracking in this chapter, too, is performed at the design stage. In some cases, however, it may be more reasonable to perform verification of initial cracking at the construction work stage or at both the design stage and the construction work stage. Even in such cases, the time at which to perform verification of initial cracking must be determined at the design stage.

Regarding (2): Cracking caused primarily by material segregation and rapid drying before hardening occurs and cracking caused by changes to concrete volume due to hydration and drying were noted as the main types of cracking that occur at the construction work stage. However, while subsidence cracking may occur on the upper surface of reinforcing bars or at non-uniform cross-sections due to subsidence of aggregates or material segregation, this can generally be prevented though tamping performed at the appropriate time. In addition, plastic shrinkage cracking may occur when the amount of water evaporation from the surface is greater than the rate of ascent of bleeding water, but this can generally be prevented by preventing rapid drying from the surface after concrete pouring.



Commentary Figure 12.1.1 Flow of verification for initial cracks (Note: Includes areas omitted in this document)

In other words, when construction work is performed following "Construction Work," the occurrence of problematic subsidence cracking and plastic shrinkage cracking can be prevented, and thus verification of these forms of cracking may be omitted. Verification may also be omitted for cracking caused by the hydration of cement in cases of extremely fine cracking that is judged to pose no problem even under careful consideration of safety, usability, durability, aesthetics, *etc*.

In a structural object for which cracking caused by the hydration of cement is a concern and for which the concrete should be treated as mass concrete, the dimensions of structural materials vary by structural form, materials used, and construction work conditions, which makes general decisions difficult. As a rough guideline, the thickness should be 80 to 100 cm or more for broad slabs and 50 cm or more for walls for which the lower end is constrained. When rich-mix concrete is used, as in the case of prestressed concrete structural objects, even thinner structural members must be treated in the same manner as mass concrete, depending on the constraint conditions.

Regarding (3): Verification of cracking caused by the hydration of cement can be broadly divided into two methods: evaluation based on past performance and evaluation using thermal stress analysis. To improve the accuracy of analytical evaluation when performing verification using the latter method, it is advisable to set physical values for the materials used in construction, along with physical values for the ground and bedrock at the site, as values to be used in design. When measured values are not used, the design values for materials may be set on the basis of reliable data.

Regarding (4): It is typically difficult to control thermal cracking in massive wall-like structural objects, *etc*. using remedial measures aimed at materials and the concrete mix. Moreover, when concrete requires watertightness, cracking renders achievement of the intended purpose impossible. One method that can be used in such cases is to provide parts with reduced cross-sectional area at regular intervals in the longitudinal direction of the structural object, then induce cracking in those parts to prevent cracking from occurring in other parts and to facilitate later treatment at the locations with cracking. To ensure that cracking occurs at the planned locations, the degree of reduction in cross-sectional area at the crack-inducing masonry joints must be about 50%. The interval of the crack-inducing masonry joints must be determined with consideration of factors that greatly affect the interval, including the dimensions of the structural object, the amount of reinforcing bars, the pouring temperature, and the pouring method. Due consideration must also be given to matters such as selection of the filling material to be used in the areas with masonry joints, methods for preventing corrosion of the reinforcing bars in the masonry joints, and methods of maintaining the specified cover. The provision of crack-inducing masonry joints enables the control of cracking in wall-like structural objects, *etc*. with relative ease. Because crack-inducing masonry joints can become points of structural weakness, however, their structure and positions must be determined appropriately with reference made to past performance, *etc*.

Standards should be used as reference for specific methods of verification.

Chapter 13 Premises Concerning Reinforced Concrete

13.1 General

This chapter presents basic concepts concerning structures as premises for designing reinforced concrete and prestressed concrete structures.

Commentary: This chapter presents requirements for structures that must be followed when designing reinforced concrete and prestressed concrete structures for which verification will be performed based on these Standard Specifications. The standard methods of performance verification presented in these Standard Specifications are grounded in mechanical theories of structures and materials. However, many of these methods make assumptions concerning the integrity of concrete and reinforcing bars, localized stress states, and other matters. When these assumptions do not hold, deviation from the scope of the verification methods may occur and accuracy may decrease. Therefore, these Standard Specifications specify that preconditions for the verification methods should be ensured. In the absence of special considerations, Volume 7 of "Design: Standards" should be followed in order to satisfy the preconditions.

Standards should be used as reference for specific methods of verification.

Standards

Part1 Structural analysis of members

Chapter 1 General

1.1 Scope of application

(1) This volume presents standard methods for verifying the performance of structural members and structural objects using the linear analysis methods specified in Chapter 7 of "Design: Main Volume". This volume may also serve as a reference when performing structural analysis of structural members and structures using non-linear analysis.

(2) When designing structural members and structures using this volume, matters related to arrangement of rebar should in principle follow "Design: Standards" Volume 7.

Commentary: Regarding (1): This volume presents simple methods for structural analysis based on linear analysis methods when verifying the performance of structural members and structures. It also describes verification methods matched to different types of structural members and structures, as well as structural details. These standards may also be used as a reference when performing structural analysis using non-linear analysis methods.

Regarding (2): The structural member and structures design methods specified in this volume depend on satisfying the content of "Design: Standards" Volume 7.

Chapter 2 General of Structural Analysis of Members

2.2 Concrete shrinkage and creep

(1) The design value of cross-sectional average shrinkage of concrete used in the calculation of response values for a structure should in principle be calculated with consideration of the effects that temperature and relative humidity in the structure's environment, cross-sectional shape and dimensions of structural members, material age at the start of drying, and other factors have on the characteristic value of shrinkage in the concrete used.

(2) The design value for the creep coefficient of concrete used in the calculation of response values for a structure should in principle be calculated with consideration of the effects of humidity around the structure, the cross-sectional form and dimensions of structural members, the mixing of the concrete, the material age of the concrete at the time that stress acts on it, and other factors.

(3) The design value for the shrinkage or creep coefficient of concrete used in the calculation of response values of a structure must appropriately consider distribution over the cross section of structural members, according to the purpose of verification.

Commentary: Regarding (1): The characteristic value of shrinkage is to be set according to shrinkage strain determined from empirical results and from test values for 100 × 100 × 400 mm concrete specimens of the same materials and mixture as the concrete to be used and cured in water for 7 days, followed by a drying period of 6 months under environmental conditions with a temperature of 20 °C and a relative humidity of 60%.

When not based on test values or empirical results, the characteristic value may be set by referring to the shrinkage properties of concrete calculated using the following equation:

$$\varepsilon'_{sh} = 2.4\left(W + \frac{45}{-20+30\cdot C/W}\cdot\alpha\cdot\Delta\omega\right), \qquad \text{(Commentary 2.2.1)}$$

where ε'_{sh}: estimated value of test value of concrete shrinkage ($\times 10^{-6}$);

W : unit water content of concrete (kg/m^3) ($W \approx 175$ kg/m^3);

C/W : cement-to-water ratio;

a : coefficient that expresses effects of aggregate quality (α = 4 to 6). For standard aggregate, α = 4 may be used;

$\Delta\varpi$: water content of aggregate:

$$\Delta\omega = \frac{\omega_S}{100+\omega_S}S + \frac{\omega_G}{100+\omega_G}G;$$

ϖ_S and ϖ_G : rate of water absorption in fine aggregate and coarse aggregate, respectively (%); and

S and G : unit fine aggregate content and unit coarse aggregate content, respectively (kg/m^3).

The estimated value according to Equation (Commentary 2.2.1) is the average value of test values for the shrinkage of concretes made with various aggregates in use in Japan. For each test value, it has been determined that unevenness is within approximately ±50%. For concrete using an aggregate for which the test value of shrinkage has not been confirmed, this must be taken into consideration when setting the characteristic value of shrinkage based on the estimated value according to the equation.

When calculating the response value for a structural member that has been approximated as a linear member (such as cross-sectional force due to shrinkage or effective prestressing in prestressed concrete) under the assumption that shrinkage strain is uniform over the cross section of the structural member, or when calculating the shrinkage strain of concrete between cracks in order to derive crack width in reinforced concrete, the change over time in the shrinkage strain of the concrete in the structural member may be derived using the following equation:

$$\varepsilon'_{ds}(t,t_0) = \frac{\frac{1-RH/100}{1-60/100}\cdot\varepsilon_{sh,inf}^{(t-t_0)'}}{\left(\frac{d}{100}\right)^2\cdot\beta+(t-t_0)}, \qquad \text{(Commentary 2.2.2)}$$

where $\varepsilon'_{ds}(t, t_0)$: dry-shrinkage strain in the structural member;

t, t_0 : material age of concrete and material age at the start of drying (days) ($t_0 \geq 3$ days);

RH : average relative humidity in the environment of the structure (%) ($45\% \leq \mathrm{RH} \leq 80\%$);

d : effective thickness of structural member (mm). In the case of a linear member for which the entire surface is in a dry state, the length of one side may be used. A general cross section may be calculated using the following equation (see **Commentary Figure 2.2.1**):

$d = \frac{4V}{S}$;

V/S : volume-to-surface-area ratio (mm), where the surface area of the part in contact with outside air is used for surface area;

$\varepsilon'_{sh,inf}$: final value of dry-shrinkage strain; and

β : coefficient expressing change over time in dry-shrinkage strain.

The terms $\varepsilon'_{sh,inf}$ and β can be derived by using the following equation to regress the curve of change over time $\varepsilon'_{sh}(t, 7)$ in shrinkage strain in a 100 × 100 × 400 mm concrete specimen under a temperature of 20 °C and relative humidity of 60% after curing in water for 7 days:

$$\varepsilon'_{sh}(t,7) = \frac{\varepsilon_{sh,inf}^{(t-7)'}}{\beta+(t-7)}. \qquad \text{(Commentary 2.2.3)}$$

When the shrinkage strain of the 100 × 100 × 400 mm concrete specimen is not based on the curve of change over time, $\varepsilon'_{sh,inf}$ and β may be obtained using:

$\varepsilon_{sh,inf}^{\left(1+\frac{\beta}{182}\right)'_{sh}\,'}$ and (Solution 2.2.4)

$$\beta = \frac{30}{\rho}\left(\frac{120}{-14+21C/W} - 0.70\right), \qquad \text{(Commentary 2.2.5)}$$

where ε'_{sh} : the JIS A 1129 test value (for a 100 × 100 × 400 mm concrete specimen cured in water for 7 days, followed by a drying period of 6 months under environmental conditions of a temperature of 20 °C and relative humidity of 60%), and when estimating shrinkage strain ε'_{sh} using Equation (Commentary 2.2.1), unevenness in the estimated value must be taken into consideration; and

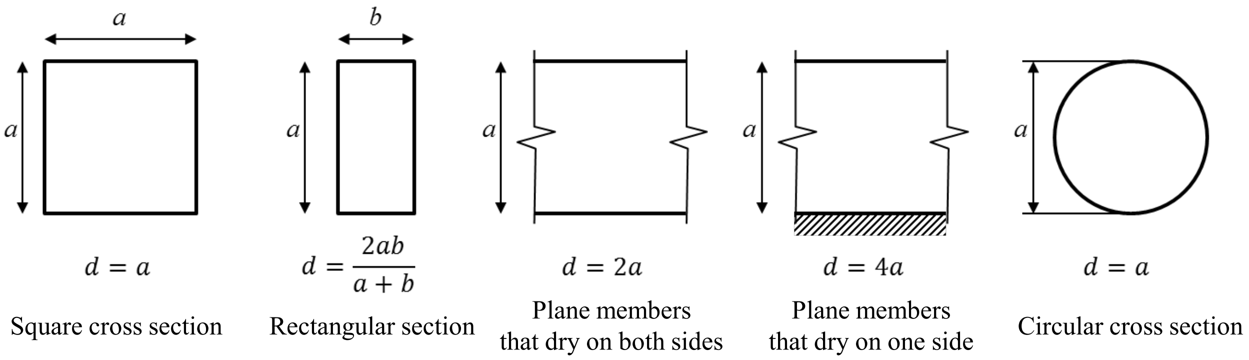
ρ : May be derived from the mass of a unit volume (g/cm^3) and the mixture of concrete.

Equation (Commentary 2.2.2) was created based on experimental results with the material age at the start of drying t_0 being 3 days or more. Therefore, the calculated shrinkage strain includes autogeneous shrinkage and drying shrinkage after the start of drying. However, when the material age at the start of drying is less than approximately 3 days and the effect of material age on drying shrinkage strain at the start of drying is great, or when the binding-material-to-water ratio is high and the effect of autogeneous shrinkage is great, Equation (Commentary 2.2.2) cannot be applied. In such cases, the change in shrinkage strain over time must be derived through testing or a reliable prediction method.

To accurately derive shrinkage strain in individual structural objects, not only meteorological data for temperature and humidity in the area around the structure but also the drying conditions of the structure should be considered as precisely as possible.

Indoors, concrete generally dries less readily and shrinkage strain is smaller than when outdoors. At the same time, solar radiation accelerates the drying of concrete but also causes expansion strain by raising the temperature of the concrete. To consider the effects of rainfall and solar radiation on concrete shrinkage strain in fine detail, the deformation of structural members should be evaluated based on analysis of thermal conduction and moisture transfer in concrete. To consider the effects of rainfall and solar radiation on shrinkage strain in structural members in a simpler manner using Equation (Commentary 2.2.2), the shrinkage strain should be calculated using the effects of rainfall and solar radiation, as well as of the apparent relative humidity.

Equation (Commentary 2.2.2) was created based on shrinkage strain in linear members with a square cross section. Obtaining effective structural member thickness d for structural members with varied cross-sectional forms should follow **Commentary Figure 2.2.1**



Commentary Figure 2.2.1 Effective structural member thickness d for structural members with varied cross-sectional forms

Equation (Commentary 2.2.2) was created based on measured shrinkage strain in specimens with thicknesses of 400 mm or less. However, the bulk of measurement data is for shrinkage strain that occurred over a short period of several years. The predictive accuracy of shrinkage strain may decline when structural member dimensions are large or after a long period of time has elapsed. When calculating the response values of structures, predictive accuracy must be appropriately considered in line with circumstances. However, when calculating long-term deflection in structural members, the fact that shrinkage and creep are not spatially uniform must be taken into consideration. Specifically, calculation should be performed following Section 3.3.3.2.

The final value $\varepsilon'_{sh,inf}$ for dry-shrinkage strain, derived from the shrinkage strain in 100 × 100 × 400 mm specimens after 7 days of curing in water in an environment with a temperature of 20 °C and a relative humidity of 60%, does not incorporate the effects of temperature surrounding the structural members, the effects of temperature history on the concrete during the curing period, or the effects of changes in concrete quality due to bleeding and other factors that occur during construction work. Therefore, shrinkage strain in concrete in the structure should be calculated with consideration of these effects.

Regarding (2): The design value of the creep coefficient of concrete must be determined with reference to test results, past tests, measurements of the actual structure, *etc*.

However, creep strain per unit of stress in the concrete used when calculating cross-sectional force introduced by the shrinkage of concrete, the effective prestressing of prestressed concrete, and the statically indeterminate force when structural systems change during and after construction of the structure can be estimated using:

$$\varepsilon'_{cc}(t,\ t')/\sigma'_{cp} = \frac{4W(1-RH/100)+350}{12+f'_c(t')} \cdot log_e(t - t' + 1) \quad ,$$

(Commentary 2.2.6)

where $\varepsilon'_{cc}(t,t')/\sigma'_{cp}$: creep strain per unit of stress at material age t (days) in concrete for which initial loading was performed at material age t' (days) ($\times 10^{-6}$/(N/mm^2));
W : unit water content of concrete (kg/m^3) (W ≈ 175 kg/m^3);
RH : relative humidity (%) (45% ≈ RH);
t' and t : effective material age (days) of concrete at the time of loading and while under load, using the value corrected by (t' ≈ 7 days):

For t' and t, $= \sum_{i=1}^{n} \Delta t_i \cdot exp\left[13.65 - \frac{4000}{273+T(\Delta t_i)/T_o}\right]$;

(Commentary 2.2.7)

Δt_i: number of days in period during which temperature is T (°C);
T_o : 1 °C; and
f'_c’(t): compressive strength (N/mm^2) of concrete at effective material age t' ’days) at the time of loading, for which Equation (Commentary 2.2.8) may be used if there is no measured value: $f'_c(t) = \frac{1.11t'}{4.5+0.95t'}\left(-20 + 30\ {}^{C}/_{W}\right)$
(Commentary 2.2.8)

The creep coefficient may be calculated from Equation (5.3.1) in Section 5.3.9 in "D“sign: Main Volume":” $\varphi(t,\ t') = \frac{4W(1-RH/100)+350}{12+f'_c(t')} \cdot log_e(t - t' + 1) \cdot E_{ct}$, (Commentary 2.2.9)

where $\varphi(t,t')$: creep coefficient at material age t (days) of the concrete for which initial loading was performed at material age t' ’days) and
E_{ct} : Young's modulus of concrete at effective material age t' (days) at the time of loading.

Equation (Commentary 2.2.6) was created based on the results of indoor tests on specimens with thicknesses of 400 mm or less, over a testing period of 3 years or less. The predictive accuracy of creep strain per unit of stress in concrete may decline when a long period of time has elapsed or when the structural member dimensions are large. When calculating the response value of a structure, this must be taken into consideration in line with circumstances.

Regarding (3): Drying shrinkage, the primary component of shrinkage in concrete, is caused by the dissipation of water from inside the concrete. The degree of drying shrinkage is affected by temporal and spatial changes in the temperature; the relative humidity and other environmental conditions around the structure; the effects of rainfall and solar radiation; the types of binders, aggregates, and other materials used; the mixing of the concrete; and the cross-sectional forms and dimensions of structural members. Therefore, the degree of drying also differs between linear members and plane members, and, even within a structure, shrinkage is not spatially uniform. To rationally consider the effects of environmental conditions, materials used, structural member forms and dimensions, and other factors when calculating shrinkage in concrete in a structure, it is advisable to derive the shrinkage of concrete for each structural member based on analysis of moisture transfer in the concrete. Moreover, concrete structures are unable to contract freely, owing to restraint by steel materials, restraint between structural members, and other external restraints, as well as internal restraint caused by the distribution of shrinkage within structural members. Therefore, to calculate the response values for deformation and stress that occur in structures and structural members, stress analysis must be performed with consideration of shrinkage in the concrete and accompanying restraints.

Part2 Durability design and durability verification

Chapter 1 General Provisions

This volume presents the basics of durability design and standard methods for dealing with steel corrosion in concrete and concrete deterioration for verification related to durability that satisfies Chapter 8 of "Design: Main Volume."

Commentary: This volume presents the basics of durability design and standard methods for the verification of steel corrosion in concrete due to water permeation into concrete and carbonation of concrete, steel corrosion in concrete due to chloride ion penetration into concrete as well as concrete deterioration due to freezing damage and chemical erosion as presented in Chapter 8 of "Design: Main Volume.

Chapter 2 Basics of Durability Design

2.1 General

(1) Every structure must be designed for durability, taking into account the importance and maintenance classification of the structure and economic efficiency from a life-cycle cost perspective.

(2) In design, the environmental conditions in which the structure is placed, the construction conditions, and the materials used must be fully understood.

(3) In design, structural details and properties of materials that satisfy durability of the structure must be set.

(4) Materials appropriate to environmental actions must be selected, and design must be performed so as to enable reliable construction work that will ensure durability.

(5) In design, a plan for the maintenance envisioned at the design stage should be formulated and should be carried forward into maintenance work.

Commentary: Regarding (1): In structures that are of importance, are difficult to maintain, or will be used in severe environments, design that confers considerable durability leads to reduced maintenance and other overall costs, even if the initial cost at the time of construction is somewhat higher. When verification of all factors related to durability is difficult, such as in an environment of severe cold in which large amounts of anti-freezing agents are dispersed or in a case of inability to completely eliminate the possibility of deterioration caused by alkali-silica reaction (ASR) or other quality of the materials used, it is important to ensure redundancy that will prevent a decline in structural performance under the actions of deterioration factors by making preparations and implementing multiple protective measures for durability.

Regarding (2): In design, the environmental actions that a structure will be subjected to during its service life must be fully surveyed and understood. Items to be surveyed include the amount of airborne salt from the sea, amount of anti-freezing agents spread on road surfaces, air temperature, precipitation, and number of freeze-thaw cycles. Structural deterioration including steel corrosion in concrete, freezing damage, and ASR is governed by the presence or absence of water supply to structures. It is important to consider the presence or absence of water exposure and other localized effects on structures, in the same manner as weather actions and other macroscopic environmental conditions. Devising structural details that prevent water from acting on structures to the greatest extent possible is also effective in ensuring durability.

In ensuring durability, it is important to ensure the quality of structures through reliable construction work based on appropriate design. Therefore, constraints on construction work such as construction work period, construction work methods, curing, concrete types, manufacturing methods, and supply conditions must be fully investigated in design. Types of cement, aggregates, *etc*. planned for use should also be examined to understand their applicability under the intended environment of the structure and the potential for durability issues caused by the materials.

Regarding (3): Methods of ensuring durability with respect to deterioration factors caused by environmental actions include ensuring that the concrete used in structures is dense and durable; enhancing resistance in terms of material properties through the use of rebars with high anti-corrosion performance and reinforcing materials with high corrosion resistance; and controlling the intrusion of deterioration factors through refinement of structural details such as an increase in cover concrete or application of surface protection, drainage slopes, and drainage devices; and by adopting several of these methods. Methods for ensuring durability must be selected from among these in durability design, with consideration of the function of the structure, the degrees of environmental actions, design service life, and aspects of economic efficiency such as life-cycle cost.

In the case of road bridge deck slabs or other structural members that deteriorate through the compounding of repeated load actions caused by running vehicles and environmental actions such as rainwater, freezing and thawing, and chloride ions, methods that enhance the resistance of materials with respect to a singular environmental action alone may be insufficient for dealing with other actions. In such cases, it is important to examine both the properties of the materials used and structural details to ensure durability with respect to degradation caused by the compounding of environmental actions and load actions.

This volume stipulates methods of performance verification in cases that involve ensuring the durability of structures primarily through the resistance of concrete as a material. Methods for controlling deterioration factors through refinements to structural details should be examined in accordance with the functions and properties of the structure, as in 2.2.

Regarding (4): Examples of matters for consideration in the assurance of durability are shown in **Commentary Table 2.1.1**. In the case of structures in which large amounts of chloride ions may be supplied by airborne salt or anti-freezing agents or in the case of a severe environment of chemical erosion, the use of mixed cements or admixtures should be considered. In addition, when factors affecting durability cannot be determined through verification or when there is a possibility that the concrete alone will be unable to resist the action of deterioration factors, multiple measures should be taken. As an example, the application of multiple measures such as the use of reinforcing materials treated for corrosion protection and stainless steel rebars with highly corrosion-resistant, concrete surface protection method to prevent penetration by chloride ions, and electrochemical corrosion control method should be considered.

Whether a structure will maintain the required durability over its design service life against degradation caused by the quality of the materials used, such as ASR is difficult to confirm through verification. As a general rule, materials for which quality has been ascertained should be selected, in accordance with "Construction Work." In some cases, however, the possibility of ASR deterioration cannot be completely excluded on the basis of maintenance records of similar structures or based on regional characteristics. Moreover, in an environment in which anti-freezing agents are spread or in which airborne salt acts, alkali is supplied to concrete from the outside. In addition to the use of control measures indicated in "Construction Work," minimizing the risk of deterioration under such circumstances requires consideration of measures such as control of the total amount of alkali in the concrete combined with the use of mixed cement that exhibits an alkali-aggregate reaction inhibiting effect, or reduction of the amount of alkali to considerably less than 3.0 kg/m^3, which is the upper limit for controlling the total amount of alkali in concrete. It is even more effective to supplement these countermeasures with means such as concrete surface protection to prevent intrusion by water and other deterioration factors.

It must also be noted that choosing to use mixed cements or admixtures affects the verification of not only the durability-related matters indicated in this volume but also cracking caused by hydration of cement. Similarly, when using concrete made with aggregates for which

shrinkage characteristic values have not been clarified through testing or past results, it is necessary to examine the cracking of concrete due to drying shrinkage, taking into account effects on usability (cracking, long-term displacement/deformation), *etc*.

At the same time, in ensuring durability, it is important to ensure the quality of the structure through reliable construction work as well as selection of appropriate materials. Under constraints on construction work, appropriate construction work plans are to be considered in accordance with "Construction Work." In some cases, such as when the adoption of special concrete has been proposed, it may be necessary to review the materials used.

Regarding (5): In these Standard Specifications, the maintenance management category of the structure is to be assumed at the structural planning stage, based on which durability design for the structure is to be performed. Therefore, information that is important in performing maintenance of the structure must be reflected in the maintenance plan and carried forward into maintenance work.

Examples of this important information include the properties of the materials selected as measures against salt damage and freezing damage, concrete cover thickness, anti-corrosion measures for rebars, the application of embedded formwork, and other matters that must be considered in the maintenance plan. Maintenance scenarios of structures envisioned at the design stage should also be included in the maintenance plan and carried over to the maintenance work. When maintenance scenarios differ from those envisioned during design, appropriate measures must be taken at the maintenance stage.

Commentary Table 2.1.1 Examples of considerations for ensuring durability

Materials	Considerations for durability
Concrete	・Assuming a significant supply of chloride ions, examine the upper limit of the amount of chloride ions present in the concrete from the time of mixing. ・Application of mixed cements and admixtures in severe chloride attack and chemical erosion environments. Combined use of concrete surface protection works where necessary. ・Combined use of alkali total volume control and use of mixed cement/admixture. Use of aggregates that are determined to be non-toxic in alkali-silica reactivity tests. Consideration of limits on total alkali limits where alkali is supplied from the external environment. ・Examination of cracking of concrete due to drying shrinkage when using concrete with aggregates for which the characteristic values of shrinkage have not been clarified by tests or actual results.
Reinforcing bars	・In severe salt and chemical corrosive environments, corrosion protection measures; Use of reinforcements, plastic sheaths, etc.; In combination with electrochemical corrosion control method.

2.2 Durability design for structures

The properties of materials used, structural details, and structural systems must be specified in design so as to satisfy durability, taking into account that the structure will be affected by a combination of load actions and environmental actions.

Commentary: The deck slabs of road bridges are subjected to both repeated load actions caused by running vehicles and environmental actions such as rainwater, freezing and thawing, and chloride ions. This causes clear and significant reduction in durability compared with cases in which such actions occur singly. In ensuring durability, methods of enhancing the resistance of concrete materials with respect to a singular environmental action alone may be insufficient for dealing with other actions. Therefore, varied methods should be used in combination to ensure durability with respect to environmental actions and load actions.

Durability must also be ensured for devices that compose a structure, in the same manner as structural members. Bearings in particular are important devices that transmit the load of the upper structure to the lower structure, and it is important that their durability is ensured in the same manner as structural members. Therefore, the use of materials that are highly resistant to environmental actions or measures such as corrosion prevention must be considered for bearings. Rainwater permeates from girder joining work and expansion devices, which tends to create a poor environment for bearing sections. Therefore, regardless of whether joining work and expansion devices exhibit drainage performance, it is advisable to not only ensure the durability of the bearing body but also to enhance resistance to rainwater and other deterioration factors by means such as installing a pedestal directly under the bearing or providing a drainage slope on the girder seating surface to secure drainage.

The service life of expansion devices is generally shorter than that of the structure itself. When designing expansion devices , drainage plans for the bridge surface, the service life and degree of water stoppage performance of the expansion devices , measures for the durability of bearing sections, and other factors should be comprehensively considered in terms of the structural system overall, taking into consideration that the innate function of expansion devices is to ensure the operability of vehicles.

Chapter 3 Verification Related to Durability

3.1 Verification of steel corrosion in concrete

3.1.1 General

Corrosion of the steel in concrete must not impair the required performance of the structure during its design service life. In general, after confirming (i) below, (ii) and (iii) should be verified with consideration of the degree of impact on performance when the limit state is exceeded. Consideration of steel corrosion may generally be omitted for temporary structures.

(i) The width of cracks on the concrete surface is below the limit value of crack width for steel corrosion.

(ii) The corrosion depth of steel due to carbonation and water permeation during the design service life is below the limit value.

(iii) In an environment of salt damage, the concentration of chloride ions in concrete at the locations of rebars will not reach the critical chloride concentration during the design service life.

Commentary: This section presents a method for the consideration of steel corrosion due to carbonation and water permeation, and a method for the consideration of steel corrosion caused by the penetration of chloride ions into concrete under an environment of salt damage. Both of these assume one-dimensional material transfer from the concrete surface to the rebars. Such verification methods rest upon the premise that localized steel corrosion does not occur at the locations of cracking. For this purpose, crack widths must be small. Accordingly, given confirmation that crack width according to (i) is kept below the crack width limit value for corrosion of rebars, and taking into account the degree of effect on performance when the limit state for steel corrosion has been exceeded, (ii) verification of steel corrosion depth due to carbonation and water permeation or (iii) verification of chloride ion concentration in concrete at the locations of rebars under an environment of salt damage is to be performed. When it is difficult to calculate the depth of steel corrosion due to water permeation, verification of carbonation may be substituted for verification of steel corrosion due to water permeation.

Structures used in environments that present no risk of water permeation into concrete or chloride ion penetration do not require verification of (ii) and (iii). Even in such cases, however, it is advisable to constrain crack width to below the limit value to prevent crack width from becoming excessively large.

In an environment in which the supply of chloride ions is low, the amounts of water and oxygen supplied are the controlling factors in the progress of corrosion, and steel corrosion progresses more gradually than in an environment of salt damage. In an environment in which the supply of chloride ions is low, verification that steel corrosion depth will not reach its limit value during the design service life is required, with the progress of carbonation and water permeation into the locations of rebar taken into consideration.

When rebar corrodes because of penetration of chloride ions, the passive film on the surface of the rebar will be destroyed and corrosion will progress if the chloride ion concentration in the concrete at the location of the rebar exceeds the critical chloride concentration. The rate of corrosion increases as the chloride ion concentration at the locations of rebar increases, and steel corrosion generally progresses rapidly. Therefore, in an environment of salt damage in which chloride ions are supplied, it is advisable that the occurrence of steel corrosion during the design service life is not tolerated. Verification that chloride ions at the locations of rebar will not reach the critical chloride concentration during the design service life is required.

3.1.2 Verification of crack width

(1) Regarding cracking in concrete, it should be confirmed that the width of cracks on the concrete surface is below the crack width limit value for steel corrosion.

(2) In reinforced concrete, the limit value of crack width with respect to steel corrosion may be set to 0.005*c* (where *c* is concrete cover). However, 0.5 mm is set as the upper limit.

(3) In reinforced concrete structural members, verification of crack width may be omitted if steel stress caused by permanent actions satisfies the limiting values of steel stress intensity shown in **Table 3.1.1**.

Table 3.1.1 Limiting values for rebar stresses due to permanent action in members for which crack width considerations may be omitted σ_{sl1} (N/mm^2)

Constantly dry environment (e.g. underside of girders unaffected by rainwater)	Dry-wet cycling environment (e.g. on top of girders, in humid environments close to the coast or river surface).	Constantly humid environment (e.g. components in the soil)
140	120	140

Commentary: Regarding (1): Examination by appropriate methods must be performed to ensure that cracks do not impair the required performance of the structure. The limit value for crack width with respect to steel corrosion can be considered to normally vary with factors such as concrete cover, the environmental conditions of the structure, the design service life, and the quality of the concrete, and may be set with reference to testing or to past results in similar structures.

Calculation of the design response value for crack width may be according to the method shown in Volume 4 of "Design: Standards."

Regarding (2): When cracks of excessive width are present in concrete cover, localized corrosion of rebar may occur. The steel corrosion limit value for crack width can generally be set to 0.005*c* (where *c* is concrete cover). For the crack width at which steel corrosion occurs, the limit value for crack width was defined as a function of concrete cover, out of recognition of the effects of concrete cover and with consideration of consistency with past results.

The limit value for crack width with respect to steel corrosion in PRC structures is according to Volume 8 of "Design: Standards."

Regarding (3): The values in **Table 3.1.1** set the limit values of rebar stress intensity caused by permanent actions in structural members in which consideration of flexural crack width may be omitted, in accordance with the environment of the structure. Taking into account the effects of water on steel corrosion, the values set rebar stress intensity that results in smaller crack width in a constantly humid environment or a dry-wet cycling environment in which water and oxygen are assumed to be supplied, than in a constantly dry environment.

The upper limit for rebar stress intensity is higher in a constantly humid environment. This is because the stress intensity of rebar was set so that crack width is the same when the details of the rebar arrangement are identical. Shrinkage and creep are small in a constantly humid environment. Therefore, under identical rebar stress intensity, crack width is smaller than in a cyclic dry-wet environment.

When the effect of variable actions may be greater than those of permanent actions, crack width must be considered.

For shear and torsional cracking, verification of crack width can be omitted by limiting the stress intensity of shear reinforcing bars and torsional reinforcing bars to the values shown in **Table 3.1.1**.

3.1.3 Verification of steel corrosion due to carbonation and water permeation

3.1.3.1 General

(1) With regard to steel corrosion due to carbonation and water permeation, verification of steel corrosion is to be performed in principle to confirm that the steel corrosion depth will not reach the limit value for steel corrosion depth during the design service life.

(2) When using carbonation depth to perform verification of steel corrosion due to carbonation and water permeation, confirmation that carbonation depth will not reach the steel corrosion occurrence limit concentration depth during the design service life can be treated as verification of steel corrosion.

Commentary: Regarding (1): Civil engineering structures are erected within the natural world, and therefore are subject to varied effects from water, including direct effects of rainwater, effects of water leaks that occur when rainwater is transmitted across the surface of structures, and condensation that occurs according to site conditions. Corrosion of rebar caused by such water effects is a major issue in the maintenance of structures.

Regarding (2): Depending on the environment of a concrete structure, calculation of steel corrosion depth due to carbonation and water permeation may be difficult in some cases. In such cases, a method of verification for confirming that carbonation depth will not reach the steel corrosion occurrence limit depth during the design service life may be applied. This is based on the idea that if the pH of the pore solution in concrete does not decrease because of atmospheric carbon dioxide or other effects, steel corrosion will not progress readily.

3.1.3.2 Verification of steel corrosion depth

Verification of steel corrosion depth is in principle performed by confirming that the ratio of the steel corrosion depth design value sd (mm) to the steel corrosion depth limit value slim (mm), multiplied by the structure coefficient γi, is no greater than 1.0.

$\gamma_i \cdot s_d / s_{\lim} \leq 1.0$ (3.1.1)

where, γ_i: structure coefficient. This may generally be set to between 1.0 and 1.1.

slim: limit value of steel corrosion depth (mm). This is to be set appropriately with consideration of the importance of the structure, the maintenance category, the uncertainty and reliability of verification, etc.

s_d: design value of steel corrosion depth (mm). This may generally be derived using Equation (3.1.2).

$s_d = \gamma_w \cdot s_{dy} \cdot t$ (3.1.2)

where, γ_w: safety coefficient with inconsistency in the steel corrosion depth design value sd taken into account

sdy: design value of steel corrosion depth per year (mm/year).

t: service life (years) with respect to steel corrosion due to carbonation and water permeation. In general, 100 years is set as the upper limit of service life.

Commentary: The steel corrosion depth limit value $s_{\lim}$ must be set appropriately with consideration of the importance of the structure and the maintenance category. In general, the steel corrosion depth limit value should be set to prevent cracking, spalling, or other deformation in concrete caused by steel corrosion at the initial stage.

For general structures, the steel corrosion depth limit value may be calculated from Equation (Commentary 3.1.1).

$s_{\lim} = 3.81 \times 10^{-4} \cdot c$ (mm) (Commentary 3.1.1)

When $c > 35$ mm, $s_{\lim}$ is set to 1.33×10^{-2} (mm).

c: concrete cover (mm)

The limit value for steel corrosion depth given in Equation (Commentary 3.1.1) was set to 0.3 times the steel corrosion depth at the time structural member surface cracking occurred, as derived through elastoplastic finite element method (FEM) analysis assuming general concrete and with sufficient leeway given the certainty of verification. When accurate and reliable limit values can be derived through means such as experiments that simulate the environment during service or the target structure, those values should be used.

The design value of steel corrosion depth per year according to Equation (3.1.2) must be determined with consideration of the progress of carbonation and the amount of moisture and oxygen supplied to the locations of rebars in concrete. An example of handling rainfall, a common environmental action, is shown below.

In the case of effects of rainfall, the design value of steel corrosion depth per year may be determined with consideration of the geographical conditions (number and duration of rainfall events, *etc.*), the duration of the corrosion environment of the rebar in the targeted parts, the progress of carbonation, the permeation rate of water in concrete, and the effects of steel concrete cover. When the number and duration of rainfall events and other geographical conditions of the structure are not considered, in general, the design value s_{dy} (mm/year) for steel corrosion depth per year may be derived from Equation (Commentary 3.1.2).

$s_{dy} = 1.9 \cdot 10^{-4} \cdot \exp(-0.068 \cdot (c - \Delta c_e)^2 / q_d^{\,2})$ (Commentary 3.1.2)

c_e: construction error in concrete cover (mm). This may generally be set to 15 mm for columns and bridge piers, 10 mm for beams, and 5 mm for slabs.
q_d: design value of water permeation velocity coefficient of concrete (mm/$\sqrt{hr}$)
$qd = \gamma c \cdot qk$
γ_c: material coefficient of concrete. This generally may be set to 1.3.
q_k: characteristic value of water permeation velocity coefficient of concrete (mm/$\sqrt{hr}$)

The design value for the steel corrosion depth per year was set as a constant, on the assumption that the passage of years causes little difference at the stage before concrete cracking occurs. When it has been determined at the design stage that steel corrosion depth per year changes considerably with the number of years in service life, an appropriate value should be set for each year.

The design value of steel corrosion depth per year given by Equation (Commentary 3.1.2) was set on the assumption that carbonation will progress and steel corrosion will gradually progress under the repeated supply of water and oxygen to the locations of the rebar. The presence or absence of a supply of water and oxygen to the locations of the rebar was determined from the relationship between concrete cover and the water permeation depth in concrete, with said permeation depth derived from the water permeation velocity coefficient of the concrete and the hours of action of water.

Verification of this assumed cases in which the main cause of water exposure in concrete is precipitation. Therefore, Equation (Commentary 3.1.2) was determined from annual rainfall volume, duration per rainfall event, *etc*. calculated on the conservative side from rainfall records in all areas of Japan. When using Equation (Commentary3.1.2), $c \geq c_e$ must be specified.

Depending on the environment of the concrete structure, the frequency of water action and other conditions may differ from those assumed in Equation (Commentary 3.1.2). When the progress of carbonation is significant and the supply of water to the locations of the rebar is large, the corrosion velocity may be greater than that given by Equation (Commentary 3.1.2). In such cases, it is necessary to conduct experiments and measurements that reproduce actual environmental conditions, to fully understand the state of deterioration in the actual structure under the same conditions, and to use prediction equations that incorporate the findings.

3.1.3.3 Setting the water permeation velocity coefficient of concrete

The characteristic value q_k of the water permeation velocity coefficient of concrete may be set using the predicted value q_p, which is estimated from the water-to-binder ratio of the concrete and the type of binder based on experiments or past data.

Commentary: When the binder used in concrete is ordinary Portland cement, Type-B blast furnace cement, or Type-B fly ash cement, the predicted value q_p (mm/$\sqrt{hr}$) for the water permeation velocity coefficient of concrete may be predicted from the water-to-binder ratio of the concrete using Equation (Commentary 3.1.3) instead of through experiments.

$$q_p = 31.25 \cdot (W/B)^2 \quad (0.40 \leq W/B \leq 0.60) \qquad \text{(Commentary 3.1.3)}$$

W/B is the water-binder ratio. Equation (Commentary 3.1.3) was obtained from experiments on concrete with water-binder ratios of 0.4, 0.5, and 0.6. It was set based on the outcomes of previous experiments, on the assumption that curing equivalent to the standard curing stipulated in the "Construction Work" volume of the Standard Specifications for Concrete Structures was performed. The water permeation velocity coefficient of concrete is thought to vary with the types, amounts, and quality of the cement, admixtures, aggregates, and other materials, as well as with the conditions of casting, compaction, and curing. Predictions should be made with the effects of these taken into consideration as necessary.

The predicted value q_p for the water permeation velocity coefficient in concrete is derived using JSCE-G 582-2018 (Test method for water penetration rate coefficient of concrete subjected to water in short term). JSCE-G 582-2018 assumes moisture permeation caused by rainfall, transitory water actions, or other short-term water exposure. Therefore, after the specified curing, test specimens are left indoors for 91 days, then immersed in water. After 5 to 48 hours, the specimens are split and the water permeation velocity coefficient is derived from the permeation depth of water confirmed from the split surface. Although JSCE-G 582-2018 deals with vertical upward permeation, permeation behavior has been confirmed to be unaffected by the direction of water absorption. Drying and other effects may cause the water permeation velocity coefficient to increase over time from the value obtained in experiments. Therefore, it is advisable to set the water permeation velocity coefficient on the conservative side rather than use the value obtained in experiments.

3.1.3.4 Verification of steel corrosion due to carbonation

(1) When verifying that the carbonation depth of concrete will not reach the steel corrosion occurrence limit depth during the design service life to make the determination that steel corrosion depth will not reach the limit value, the following methods may be used as verification of steel corrosion.

(2) Verification of steel corrosion due to carbonation is in principle performed by confirming that the ratio of the carbonation depth design value yd to the steel corrosion occurrence limit depth ylim, multiplied by the structure coefficient γi, is no greater than 1.0.

$$\gamma_i \cdot y_d / y_{\lim} \leq 1.0 \tag{3.1.3}$$

where, γ_i: structure coefficient. This may generally be set to between 1.0 and 1.1.

$y_{\lim}$: steel corrosion occurrence limit depth This may generally be derived using Equation (3.1.4).

$$y_{\lim} = c_d - c_k \tag{3.1.4}$$

where, c_d: design value (mm) of concrete cover used for verification related to durability. This is to be derived using Equation (3.1.5), taking construction error into account.

$$c_d = c - \Delta c_e \tag{3.1.5}$$

c: concrete cover (mm)

Δc_e: construction error in concrete cover (mm). This may generally be set to 15 mm for columns and bridge piers, 10 mm for beams, and 5 mm for slabs.

c_k: remaining non-carbonated concrete cover thickness (mm). This may generally be set to 10 mm under normal environments. In an environment in which the effects of chloride ions are not negligible, it should be set to between 10 and 25 mm.

y_d: design value of carbonation depth (mm). This may generally be derived using Equation (3.1.6).

$$y_d = \gamma_{cb} \cdot \alpha_d \sqrt{t} \tag{3.1.6}$$

where, α_d: design value of carbonation velocity coefficient (mm/$\sqrt{year}$)

$= \alpha_k \cdot \beta_e \cdot \gamma_c$

α_k: characteristic value of carbonation velocity coefficient (mm/$\sqrt{year}$)

β_e: coefficient that expresses degree of environmental action. This may generally be set to 1.6.

γ_c: material coefficient of concrete. This may generally be set to 1.0. However, it should be set to 1.3 for upper surface parts.

γ_{cb}: safety coefficient with inconsistency in the carbonation depth design value y_d taken into account. This may generally be set to 1.15. However, it should be set to 1.1 when high-fluidity concrete is used.

t: service life (years) with respect to carbonation. For carbonation depth calculated from the Equation (3.1.6), a service life of 100 years is generally set as the upper limit.

Commentary: Regarding (2): When using carbonation depth of concrete to perform verification of steel corrosion due to carbonation and water permeation, whether the carbonation depth is less than the steel corrosion occurrence limit depth is to be confirmed. However, depending on the required performance and the degree of importance of the concrete structure, verification may be performed with the occurrence of concrete cracking originating in steel corrosion caused by carbonation set as the limit state.

Many studies and surveys of structures have revealed that corrosion of steel begins before the carbonation depth, as defined using coloration in phenolphthalein spray testing, reaches the location of the steel. The time at which corrosion begins is often controlled by the remaining non-carbonated concrete cover thickness, *i.e.*, the difference between the concrete cover and the carbonation depth. When the remaining non-carbonated concrete cover thickness is at least 10 mm, any corrosion that occurs often remains minor.

The concrete cover used in Equation (3.1.4) is a value that takes construction errors into account. Given that steel corrosion due to carbonation can be caused by insufficient concrete cover and that precise concrete cover can be difficult to ensure in construction work, a value with construction error subtracted, as in Equation (3.1.5), was set for the concrete cover used in verification. According to the findings of studies comparing concrete cover in design with measured concrete cover, the shortage compared to concrete cover in design is at most about 10% of the concrete cover. The value for construction error in concrete cover may be set by referring to Chapter 4 of this volume.

In an environment in which the effects of chloride ions are not negligible, this was set to between 10 and 25 mm. This value is set with consideration of the fact that chloride ions fixed in the cement hydrate dissociate through the progress of carbonation and become concentrated in non-carbonated areas, which can hasten the onset of corrosion. However, the upper limit of 25 mm is the value when chloride ions are already present in the concrete, which corresponds to an environment of severe salt damage. When no data is available for setting the remaining non-carbonated concrete cover thickness at the start of corrosion, it should be set to 25 mm as a safety measure. When this has been sufficiently confirmed from the findings of surveys and experiments involving structures under similar conditions, the value may be set to less than 25 mm, with reference to said findings.

Carbonation depth was set in proportion to the square root of service life because this is the most general approximation based on past data.

In general, 1.15 can be used as a safety coefficient that takes inconsistency of the design value of carbonation depth in Equation (3.1.6) into account. High-fluidity concrete exhibits a high resistance to material separation and high reliability in ensuring the uniformity of concrete in structures, a prerequisite for the application of predictive methods. When high-fluidity concrete is used, 1.1 can be used as a safety coefficient that takes inconsistency of the design value of carbonation depth into account.

The material coefficient of concrete can generally be set to 1.0. However, it should be set to 1.3 when there is a possibility of fluctuations in the quality of the concrete in upper surface parts during casting.

3.1.3.5 Setting the carbonation velocity coefficient of concrete

The characteristic value of the carbonation velocity coefficient of concrete, α_k, may be set using the predicted value α_p estimated from the effective water-to-binder ratio of concrete and the type of binder based on experiments or past data.

Commentary: The predicted value α_p of the carbonation velocity coefficient of concrete may be predicted from the water-to-binder ratio of concrete and the type of binder, using Equation (Commentary 3.1.4).

$$\alpha_p = a + b \cdot W/B \qquad \text{(Commentary 3.1.4)}$$

where, a, b: coefficients determined from past results, according to type of cement (binder)

W/B: effective water-to-binder ratio

Equation (Commentary 3.1.5) is a regression equation derived from 17 types of experimental data using ordinary Portland cement or moderate-heat Portland cement.

$$\alpha_p = -3.57 + 9.0 \cdot W/B \quad (\mathrm{mm}/\sqrt{year}) \qquad \text{(Commentary 3.1.5)}$$

where, W/B: effective water-to-binder ratio = $W/(C_p + k \cdot A_d)$

W: mass of water per unit volume

B: mass of effective binder per unit volume

C_p : mass of Portland cement per unit volume

A_d: mass of admixture per unit volume

k: constant set according to type of admixture

For fly ash, $k = 0$

For blast furnace slag fine powder, $k = 0.7$

Equation (Commentary 3.1.5) was derived from the linear regression equation of the relationship between the water-to-binder ratio and the value obtained by dividing the carbonation depth by the square root of the material age (years), based on multiple data items. In the experiments, a cylindrical specimen with dimensions of φ150 mm × 300 mm and cured in water for 14 days was exposed to outdoor conditions and was then cut after reaching the specified material age. Carbonized portions were determined through spraying with phenolphthalein, and the carbonation depth was calculated from the results.

3.1.4 Verification of steel corrosion in an environment of salt damage

3.1.4.1 Verification of steel corrosion due to penetration by chloride ions

(2) Verification of steel corrosion due to penetration of chloride ions is in principle performed by confirming that the ratio of the design value C_d of chloride ion concentration at the location of rebar to the critical chloride ion concentration C_{lim}, multiplied by the structure coefficient γ_i, is no greater than 1.0.

$$\gamma_i \frac{C_d}{C_{lim}} \leq 1.0 \tag{3.1.7}$$

where, γ_i: may generally be set between 1.0 and 1.1.

C_{lim}: critical chloride ion concentration (kg/m^3). This may be determined through reference to measurements and test results from similar structures. Otherwise, it may be determined using Equations (3.1.8) to (3.1.11). However, the range of W/C is to be between 0.30 and 0.55. When the structure is subjected to freezing and thawing action, a value even smaller than these should be set.

(When ordinary Portland cement is used)

$$C_{\mathrm{lim}} = -3.0(W/C) + 3.4 \tag{3.1.8}$$

(When the equivalent of Type-B blast furnace cement or the equivalent of Type-B fly ash cement is used)

$$C_{\mathrm{lim}} = -2.6(W/C) + 3.1 \tag{3.1.9}$$

(When low-heat Portland cement or high-early-strength Portland cement is used)

$$C_{\mathrm{lim}} = -2.2(W/C) + 2.6 \tag{3.1.10}$$

(When silica fume is used)

$$C_{\mathrm{lim}} = 1.20 \tag{3.1.11}$$

C_d: design value of chloride ion concentration at the location of rebar. This may generally be derived using Equation (3.1.12).

$$C_d = \gamma_{cl} \cdot C_0 \left(1 - erf\left(\frac{0.1 \cdot c_d}{2\sqrt{D_d \cdot t}}\right)\right) + C_i \tag{3.1.12}$$

where, C_0: chloride ion concentration on concrete surface (kg/m^3). In general, the value given in 3.1.4.3 may be used.

c_d: design value (mm) of concrete cover used for verification related to durability. This is to be determined using Equation (3.1.13), taking construction error into account beforehand.

$$c_d = c - \Delta c_e \tag{3.1.13}$$

c: concrete cover (mm)

Δc_e: construction error (mm). In general, this may be set to 15 mm for columns and bridge piers, 10 mm for beams, and 5 mm for slabs.

t: service life (years) with respect to penetration by chloride ions. In general, for the chloride ion concentration at the location of rebars calculated from Equation (3.1.12), 100 years is to be used as the upper limit of design lifetime.

γ_{cl}: safety coefficient that considers the inconsistency in the design value C_d of the chloride ion concentration at the location of rebars. This generally may be set to 1.3. However, it may be set to 1.1 when high-fluidity concrete is used.

D_d: design diffusion coefficient with respect to chloride ions (cm^2/year). This may generally be calculated using Equation (3.1.14).

$$D_d = \gamma_c \cdot D_k + \lambda \cdot \left(\frac{w}{l}\right) \cdot D_0 \tag{3.1.14}$$

where, γ_c: material coefficient of concrete. This may generally be set to 1.0. However, it should be set to 1.3 for upper surface parts.

D_k: characteristic value of the diffusion coefficient for chloride ions in concrete (cm^2/year)

λ: coefficient that expresses the effects of the presence of cracking on the diffusion coefficient. In general, this may be set to 1.5.

D_0: constant that expresses the effects of cracking on the migration of chloride ions in concrete (cm^2/year). In general, this may be set to 400 cm^2/year.

w/l: ratio of crack width to crack spacing. This may generally be derived using Equation (3.1.15).

$$\frac{w}{l} = \left(\frac{\sigma_{se}}{E_s}\left(or\,\frac{\sigma_{pe}}{E_p}\right) + \varepsilon'_{csd}\right) \tag{3.1.15}$$

Here, the definitions of σ_{se}, σ_{pe}, and ε'_{csd} use the values used to calculate the design response value of crack width, following Volume 4 of "Design: Standards."

$erf(s)$ is expressed as $erf(s) = \frac{2}{\sqrt{\pi}}\int_0^s e^{-\eta^2} d\eta$.

C_i: initial chloride ion concentration (kg/m^3). In general, this may be set to 0.30 kg/m^3.

(2) When passing verification according to (1) is difficult, reinforcing materials that have high corrosion resistance or that have been treated with rustproofing, concrete surface coating that inhibits steel corrosion, electrochemical measures that prevent the onset of corrosion, *etc*., should be used in principle. In this case, the efficacy of these must be evaluated through appropriate methods after maintenance plans have been considered.

(3) When the structure is affected by chloride ions from the external environment, its concrete cover should be at least 3/2 times the maximum dimension of the coarse aggregate.

(4) When the structure is not affected by chloride ions from the external environment, if the total amount of chloride ions contained in the concrete at the time of mixing is 0.30 kg/m^3 or less, it can be assumed that the required performance of the structure will not be lost because of chloride ions. However, in cases such as the use of PC steel, which is prone to stress corrosion, the value should be made even smaller.

(5) For concrete structures that are expected to be subjected to anti-freezing agents, chloride ions should be inhibited from penetrating the concrete by considering the occurrence of salt damage and by appropriately installing reliable waterproofing and drainage works.

Commentary: Regarding (1): In the verification of structure performance with respect to chloride ion penetration, the safest measure is to set the non-occurrence of corrosion in steel during the design service life as a condition. It should be verified that chloride ion concentration at the location of rebars is not higher than the steel corrosion occurrence limit concentration. However, if verification is possible using appropriate methods, then, depending on the required performance and the degree of importance of the concrete structure, verification may be performed with the occurrence of concrete cracking originating in steel corrosion caused by penetration of chloride ions set as the limit state. Here, chloride ion concentration is not the actual chloride ion concentration in the liquid phase in concrete; rather, it expresses the mass of total chlorine per unit volume of concrete, with kg/m^3 as the unit.

The steel corrosion occurrence limit concentration of chloride ions at the locations of rebar differs by type of cement and the water-cement ratio. Therefore, in verification of individual structures, it is advisable to determine the steel corrosion occurrence limit concentration with reference to measurements and experimental results under conditions in which materials and concrete mix are similar to those of the target structure. In other cases, it was decided that the steel corrosion occurrence limit concentration may be derived using Equations (3.1.8) to (3.1.11). The range of application of these equations was set to a water-cement ratio of between 0.30 and 0.55, with consideration of required strength and of the concrete mix in terms of durability with respect to salt damage.

Chloride ion concentration in concrete can be expressed as follows: (1) chloride ion concentration $[Cl^-]$ (mol/l) in the pore solution (hereinafter noted as ion concentration), (2) ratio of the total chlorine mass to cement mass (mass % of cement) (hereinafter noted as cement weight), and (3) mass of total chlorine per unit volume of concrete (kg/m^3) (hereinafter noted as total concrete weight). Total chlorine in (2) and (3) is sometimes referred to as soluble chlorine. These Standard Specifications adopt (3) total concrete weight notation.

In the penetration of chloride ions into concrete, advection caused by water migration and diffusion that is dependent on the concentration gradient of chloride ions occur. These are accompanied by fixation, adsorption, *etc*. to cement hydration products and cement components. Here, with respect to chloride ion penetration, the chloride ion concentration at the locations of rebar at the time the design service life is reached may be estimated from the equation, with consideration of the solution to the Fick diffusion equation shown in Equation (3.1.12) and to initial chloride ion concentration. This equation is based on the concept of considering penetration by chloride ions as a phenomenon that is uniform in the axial direction of the rebar. The concrete cover used in Equation (3.1.12) is a value that takes construction errors into account. According to the findings of studies comparing concrete cover in design with measured concrete cover, the shortfall compared to concrete cover in design is at most about 10% of the concrete cover. The value for the construction error in concrete cover may generally be set to the value indicated in **Commentary Table 4.1.1**.

When flexural cracking is present in the cover concrete, the chloride ion concentration at the location of rebar can be considered to vary with distance from the cracking. However, when the crack width is small, the non-uniformity of the chloride ion penetration caused by cracking is generally small. In dense concrete, cracking may result in non-uniform distribution of chloride ions. However, the density of areas other than where cracking occurs is thought to function effectively in protecting the rebar. From the above, by considering the averaged effects of cracking, chloride ion penetration can be viewed as a uniform phenomenon in the axial direction of the rebar, and the assumption can be made that engineering issues will not arise even if the occurrence limit of steel corrosion occurrence is determined. Therefore, for cases in which the flexural cracking width is kept below the crack width limit value indicated in 3.1.2, chloride ion concentration at the location of rebar should be estimated using Equation (3.1.12), by calculating the diffusion coefficient with consideration of the effects of cracking and the quality of the concrete according to Equation (3.1.14). Equation (3.1.14) attempts to evaluate the average diffusion coefficient for chloride ions in cover concrete, with respect to both concrete quality and the effects of cracking. The ratio of crack width to crack spacing ratio, *w/l*, is a physical quantity introduced to express the averaged effects of cracks. Even when cracking is not present in the cover concrete, if the integrity of the coarse aggregate and the mortar is lost due to bleeding, shrinkage, *etc*., resistance to chloride ion penetration at the coarse aggregate interface will lessen. Therefore, assurance of quality through appropriate selection of materials and reliable construction work is a prerequisite for verification.

With respect to thermal cracking, shrinkage cracking, and other initial cracking at the construction work stage, too, verification related to steel corrosion due to penetration of chloride ions is in principle appropriately performed by deriving crack width and spacing. When it is difficult to derive the initial crack spacing and when crack width is less than the crack width limit value in 3.1.2, Equation (Commentary 3.1.6) may be used.

$$D_d = D_k \cdot \gamma_c \cdot \beta_{cl}$$

(Commentary 3.1.6)

where, β_{cl}: coefficient that considers the effects of initial cracking. This may be set to 1.5.

The safety coefficient γ_{cl} that considers the inconsistency in the design value of the chloride ion concentration at the location of rebar in Equation (3.1.12) may generally be set to 1.3. This was introduced to provide a margin of safety, taking into account the accuracy of prediction methods and the fact that penetrating chloride ions exert a localized effect on steel corrosion. High-fluidity concrete exhibits a high resistance to material separation and high reliability in ensuring the uniformity of concrete in structures, a prerequisite for the application of predictive methods. Therefore, when high-fluidity concrete is used, the safety coefficient may be set to 1.1, taking into account inconsistency in the design values of chloride ion concentration at the locations of rebar. It is also expected that, in cases such as when curing is performed by a method more advanced than general curing methods, the quality of the concrete surface layer will be improved and the resistance of the concrete cover concrete to penetration by chloride ions will be enhanced. In such cases, taking into account the inconsistency of the design value of chloride ion concentration at the location of rebar, the safety coefficient should be determined appropriately through experiments, *etc*.

The material coefficient γ_c of concrete may generally be set to 1.0 but should be set to 1.3 for the upper surface parts of the structure. This is because bleeding may cause the quality of concrete on the upper surface to be inferior to that of other parts. When there is no difference in quality between the concrete of the structure and that of standard cured specimens, the value may be set to 1.0 for all parts.

Regarding (2): In particularly severe environments or where corrosion is unacceptable, it may be difficult to pass verification even if concrete with a low diffusion coefficient and the concrete cover is increased. In such cases, measures such as the use of reinforcing materials treated for corrosion protection (e.g., epoxy resin-coated rebar) and highly corrosion-resistant stainless steel rebars, concrete surface protection treatment to prevent intrusion by chloride ions, and electrochemical anti-corrosion methods to prevent the occurrence of corrosion may be economical. When using epoxy resin-coated rebars or stainless steel rebars, methods of verification of chloride ion penetration-based steel corrosion should follow those indicated in the respective guidelines for these rebars.

Regarding (3): The migration of chloride ions at the interface between coarse aggregate and mortar in concrete is known to be fast. This tendency is even more pronounced when bleeding or shrinkage is significant. Therefore, when this boundary surface is continuous from the concrete surface to the rebar, chloride ions are readily supplied to the rebar, and corrosion of the rebar will be accelerated if the concrete cover is light. To prevent this, the decision was made to set concrete cover to at least 3/2 times the maximum size of the coarse aggregate.

Regarding (4): If chloride ions in the concrete do not exceed the steel corrosion occurrence limit concentration on the steel surface during the service period, corrosion of the rebar is not likely to reduce the performance of the structure. Therefore, even in concrete into which chlorides may have been introduced during mixing, problems will not occur if the chloride content does not exceed the steel corrosion occurrence limit concentration during the service period. In some cases, however, chloride ions may not be distributed uniformly in the concrete because of bleeding during concrete casting, or chloride ions may migrate for various reasons after the concrete has hardened, with the result that ions become concentrated in certain parts. Taking these factors into consideration, it is not always appropriate to apply the steel corrosion occurrence limit concentration as the limit value for chloride ion content during mixing.

Therefore, the chloride ion content in concrete during mixing was set to a value conventionally used as the practical value capable of limiting deterioration in structures caused by steel corrosion to an acceptable level, with the total amount to be no more than 0.30 kg/m^3. However, when chloride ions are not expected to penetrate into concrete from the outside following the start of service life, the acceptable value for chloride ion content may be increased to 0.60 kg/m^3 if the concrete is made dense enough to prevent free migration of chloride ions, with material separation prevented by making the water-cement ratio, unit water content, *etc*. as small as possible and by casting the concrete with care.

Regarding (5): In some cases, the method in (1) can be used in verification of the effects of anti-freezing agents. At present, however, it is difficult to set chloride ion concentration in a concrete surface and to set the parts where salt damage caused by anti-freezing agents is likely to be concentrated. This results in difficulty in ensuring the reliability and accuracy of predictions. Therefore, when salt damage caused by anti-freezing agents is expected, appropriate waterproofing and/or drainage work should be installed. Means of preventing moisture containing chloride ions from reaching the concrete should in principle be used, such as the adoption of a structural form without expansion devices. The use of anti-freezing agents that do not contain chloride ions should also be considered. In this case, factors including the chemical stability of the concrete and the effects of the new anti-freezing agent on the surrounding environment must be taken into consideration. When the subject of design is a particularly important structure or when there is a need to prepare for unexpected changes in the environment or for age-related deterioration of waterproofing and wastewater treatment devices under long-term service, it is advisable to enact varied measures in combination. As an example, the use of dense concrete with salt-blocking properties, epoxy resin-coated rebars or stainless steel rebars with high corrosion-resistant, concrete surface protection to prevent penetration by chloride ions, or electrochemical methods in combination can reduce the risk of deterioration caused by anti-freezing agent-based salt damage. When considering the scope of construction work under these construction methods, it is effective to identify structural members and parts in which significant salt damage caused by anti-freezing agents occurs, by using maintenance records from similar existing structures.

3.1.4.2 Setting the chloride ion diffusion coefficient of concrete

The characteristic value D_k for the chloride ion diffusion coefficient of concrete is to be derived using one of the following methods.

(i) Relational expression for the water-cement ratio and apparent diffusion coefficient

(ii) Laboratory or natural exposure experiments using migration test or immersion method

(iii) Surveys of actual structures

Commentary: Regarding (i): Several regression equations for use in predicting the apparent diffusion coefficient from the materials used and the composition of concrete have been derived based on past experimental results. When no results from laboratory experiments or natural exposure experiments are available, the following prediction equation based on past data may be used. For high-early-strength Portland cement, equations such as the below should be obtained through experiments.

(a) When using ordinary Portland cement

$$log_{10}D_k = 3.0(W/C) - 1.8$$

($0.30 \leq W/C \leq 0.55$) (Commentary 3.1.7)

(b) When using low-heat Portland cement

$$log_{10}D_k = 3.5(W/C) - 1.8$$

($0.30 \leq W/C \leq 0.55$) (Commentary 3.1.8)

(c) When using Type-B blast furnace cement equivalent and silica fume

$$log_{10}D_k = 3.2(W/C) - 2.4$$

($0.30 \leq W/C \leq 0.55$) (Commentary 3.1.9)

(d) When using Type-B fly ash cement equivalent

$$log_{10}D_k = 3.0(W/C) - 1.9$$

($0.30 \leq W/C \leq 0.55$) (Commentary 3.1.10)

Regarding (ii): When deriving the diffusion coefficient through laboratory experiments, the Test Method for Effective Diffusion Coefficient of Chloride Ion in Concrete by Migration (JSCE-G 571-2003) (draft) and the Test Method for Apparent Diffusion Coefficient of Chloride Ion in Concrete by Submergence in Salt Water (JSCE-G 572-2003) (draft) should be followed. The migration method measures the effective diffusion coefficient that expresses the ease of migration of chloride ions in the pore solution of concrete, not the apparent diffusion coefficient indicated in these Standard Specifications. Therefore, the value thus obtained cannot be used as-is for verification under these Standard Specifications. When obtaining the apparent diffusion coefficient from the effective diffusion coefficient, the following equation may be used when converting without conducting testing. For high-early-strength Portland cement, the relationship between $k_1 \cdot k_2$ and water-cement ratio should be obtained experimentally.·

$$D_{ae} = k_1 \cdot k_2 \cdot D_e \quad \text{(Commentary 3.1.11)}$$

where, D_{ae}: apparent diffusion coefficient (cm^2/year) converted from the effective diffusion coefficient by migration testing

D_e: effective diffusion coefficient (cm^2/year) by migration testing

k_1: coefficient related to the balance of chloride ion concentrations on the concrete side at concrete surface and the solution at the cathode side respectively.

k_2: coefficient related to the immobilization of chloride ions in cement hydrate

(a) When using ordinary Portland cement

$k_1 \cdot k_2 = 0.21\exp\{1.8\ (W/C)\}$ $(0.30 \leq W/C \leq 0.55)$ (Commentary 3.1.12)

(b) When using low-heat Portland cement

$k_1 \cdot k_2 = 0.15\exp\{3.1\ (W/C)\}$ $(0.30 \leq W/C \leq 0.55)$ (Commentary 3.1.13)

(c) When using Type-B blast furnace cement equivalent

$k_1 \cdot k_2 = 0.14\exp\{1.6\ (W/C)\}$ $(0.30 \leq W/C \leq 0.55)$ (Commentary 3.1.14)

(d) When using Type-B fly ash cement equivalent

$k_1 \cdot k_2 = 0.37\exp\{1.1\ (W/C)\}$ $(0.30 \leq W/C \leq 0.55)$ (Commentary 3.1.15)

Regarding (iii): The apparent diffusion coefficient can be calculated by measuring the chloride ion concentration distribution of samples collected from the structure. The apparent diffusion coefficient of concrete in actual structures derived in this manner can be directly incorporated as a design value into the verification of salt damage of a new structure. However, the apparent diffusion coefficient may differ by exposure period and the material age being measured. Therefore, care must be taken when determining the design value based on data obtained from testing that is short-term, particularly in comparison to the period of service life.

3.1.4.3 Concrete surface chloride ion concentration

The concrete surface chloride ion concentration used in verification of salt damage should be set according to the airborne salt content of the area in question.

Commentary: When performing verification of salt damage by the method indicated in 3.1.4.1, the effect of external salt on the amount of chloride ion penetration is expressed by the concrete surface chloride ion concentration C_0.

When past results from similar structures or measurement data are not used, the concrete surface chloride ion concentration may be derived from **Commentary Table 3.1.1** according to the geographical zone where the structure is sited and the distance from the coast.

Commentary Table3.1.1 Chloride ion concentration C_0 at concrete surface (kg/m^3)

		Splash zone	Distance from coast (km)				
			Near shoreline	0.1	0.25	0.5	1.0
Region with high airborne chloride concentration	Hokkaido, Tohoku, Hokuriku, Okinawa	13.0	9.0	4.5	3.0	2.0	1.5
Region with low airborne chloride concentration	Kanto, Tokai, Kinki, Chugoku, Shikoku, Kyusyu		4.5	2.5	2.0	1.5	1.0

The penetration of chloride ions into concrete is affected by the amount of airborne salt that adheres to and remains on the concrete surface. In bridges running parallel to the coastline, the amount of chloride ions on the concrete surface is greater on the sea-facing side than on the land-facing side. Salt adhering to the concrete surface may also be washed away by rain, wind, snow, *etc*. The values indicated in **Commentary Table 3.1.1** do not reflect such localized conditions; rather, they deal with parts in the structure that are at risk of salt damage from large amounts of airborne salt. Cases in which surface chloride ion concentration is less at more elevated positions should be given appropriate consideration.

When reliable data on airborne salt content is available for the area where a structure in question was constructed, the following equation (Commentary 3.1.16) may be used to derive the concrete surface chloride ion concentration C_0.

$$C_0 = -0.016 \times C_{ab}^2 + C_{ab} + 1.7 \qquad (C_{ab} \leq 30.0)$$

(Commentary 3.1.16)

where, C_0: concrete surface chloride ion concentration (kg/m^3)

C_{ab}: airborne salt content (mdd, mg/dm^2/day)

Measured airborne salt is known to fluctuate greatly with the season, with past data indicating variation of the order of tens to hundreds of times even at a single location. When measuring airborne salt, measurements must be continued monthly to eliminate fluctuations caused by transitory changes in wind speed or direction and wave conditions. To further eliminate the effects of typhoons, seasonal variations between summer and winter, and similar factors, it is advisable to continue measurements in one-year units and to use the resulting average value for the airborne salt content C_{ab}. When measurements are possible for only a period of less than one year, the season with the highest airborne salt content during the year should be chosen. Screens or obstacles present in front of or behind the locations of airborne salt content measurement may greatly affect measured values. In such cases, steps must be taken to obtain reliable data.

3.2 Verification of deterioration of concrete

3.2.1 Verification of freezing damage

(1) In principle, verification of freezing damage should be performed separately for internal damage and for surface damage (scaling).

(2) In cases in which concrete inside a structure has undergone deterioration, verification of internal damage should in principle be performed by confirming that the value of the ratio of the minimum limit value $E_{\min}$ of the relative dynamic modulus of elasticity in freeze-thaw testing to its design value E_d, multiplied by the structure coefficient γ_i, is no more than 1.0. For general structures, however, this verification may be omitted if the characteristic value of the relative dynamic modulus of elasticity in freeze-thaw testing is at least 90%.

$$\gamma_i \frac{E_{min}}{E_d} \leq 1.0 \qquad (3.2.1)$$

where, γ_i: may generally be set between 1.0 and 1.1.

E_d: design value of the relative dynamic modulus of elasticity in freeze-thaw testing

$E_d = E_k / \gamma_c$

E_k: characteristic value of the relative dynamic modulus of elasticity in freeze-thaw testing. In the case of ordinary concrete for which general concrete materials are selected and for which the air content is 4-7%, the value indicated in **Table 3.2.1** may be used as the characteristic value of the relative dynamic modulus of elasticity in freeze-thaw testing of the concrete.

γ_c: material coefficient of concrete. This may generally be set to 1.0. However, it should be set to 1.3 for upper surface parts.

$E_{\min}$: the minimum limit value of relative dynamic modulus of elasticity in freeze-thaw testing that satisfies performance related to freezing damage. This may generally be according to **Table 3.2.2**.

Table 3.2.1 Relative dynamic modulus of elasticity of concrete in freeze-thaw test and the water-cement ratio to be satisfied (%).

	Water-cement ratio (%)			
	65	60	55	45 以下
Relative dynamic modulus of elasticity of concrete in freeze-thaw tests (%)	60	70	85	90

The values of the relative dynamic modulus of elasticity in freeze-thaw test between the water-cement ratios given in the table may be obtained by linear interpolation.

Table 3.2.2 Minimum limits for the relative dynamic modulus of elasticity in freeze-thaw tests to satisfy the performance of concrete structures with respect to frost damage, $E_{\min}$ （%）

Weather conditions / Cross-section / Exposed state of the structure	In cases of often repeated freeze-thaw cycles		In cases where temperatures below freezing are a rare occurrence	
	Thin cross section[2)]	General cross sections	Thin cross section[2)]	General cross sections
(1) Where continuously or often saturated with water [1)]	85	70	85	60
(2) In case of normal exposure and not classified under (1)	70	60	70	60

1) Water channels, water tanks, bridge abutments, bridge piers, retaining walls, tunnel linings, etc. that are close to the water surface and saturated with water, as well as girders, slabs, etc. that are away from the water surface but saturated with water due to snow melt, running water, water spray, etc., in addition to these structures.

2) Parts of the cross-section with a thickness of less than about 20 cm, etc.

(3) In cases in which concrete on the surface of structure has undergone deterioration, verification of surface damage (scaling) should in principle be performed by confirming that the value of the ratio of design value d_d of the amount of scaling of the concrete to its limit value $d_{\lim}$, multiplied by the structure coefficient γ_i, is no more than 1.0.

$$\gamma_i \frac{d_d}{d_{lim}} \leq 1.0 \tag{3.2.2}$$

where, γ_i: may generally be set between 1.0 and 1.1.

$d_{\lim}$: limit value of the scaling amount of concrete (g/m^2)

d_d: design value of the scaling amount of concrete (g/m^2)

(4) For general structures, verification of freezing damage may be omitted if the characteristic value of the relative dynamic modulus of elasticity in freeze-thaw testing is at least 90%, or if the water-cement ratio is no more than 45% and the air content is at least 6% when effects of chloride caused by anti-freezing agents, seawater, *etc*. are present.

Commentary: Regarding (1): Because freezing damage to the interior concrete of a structure differs from surface freezing damage such as scaling and popouts in terms of effects on the performance of the structure, verification should address both interior damage and surface damage. When there is no risk of freezing of the concrete, verification of the performance of the structure with respect to freezing damage may be omitted.

Regarding (2) and (3): The resistance of concrete to freezing damage is affected by a number of factors including the properties of the concrete, the minimum temperature of the usage environment, the number of freeze-thaw cycles, and the degree of water saturation. While accurately evaluating these is not easy, the concrete used in general civil engineering structures is AE concrete with *W/C* of no more than 55% and possesses sufficient freezing resistance in normal environments. For structures constructed under environmental conditions identical to those constructed in the past, verification of resistance to freezing damage may be omitted if it is clear from past results that resistance to freezing damage is sufficient.

Among the freezing deterioration in concrete caused by freeze-thaw action, for internal damage to the structure, verification should confirm that the design value of the relative dynamic modulus of elasticity in freeze-thaw testing is not less than the minimum limit value of the relative dynamic modulus of elasticity in freeze-thaw testing. In the case of ordinary concrete for which general concrete materials are selected and for which the air content is 4-7%, the value indicated in **Table 3.2.1** may be used as the characteristic value of the relative dynamic modulus of elasticity in freeze-thaw testing of the concrete.

Conversely, for surface damage such as scaling, the amount of scaling (i.e., the amount of concrete mass loss due to scaling due to freeze-thaw action) should be used as a metric, and verification should confirm that the scaling amount does not reach the limit value set on the basis of deformation of specimen surfaces in single-face freeze-thaw testing. Using methods including RILEM CDF and ASTM C672, single-face freeze-thaw testing of concrete is able to derive the scaling amount of concrete caused by freeze-thaw action.

When there is a requirement for high durability that maintains the initial soundness of a structure during its period of service life without impairment of its aesthetics, there must be no reduction of mass caused by surface damage (scaling) or reduction in the relative dynamic modulus of elasticity of specimens in freeze-thaw testing. In some cases, it may be advisable not to allow scaling to occur, or to set the value of the relative dynamic modulus of elasticity of concrete in freeze-thaw tests appropriately by setting it to 95% or more.

3.2.2 Verification of chemical erosion

Verification related to chemical erosion is in principle performed by confirming that the ratio of the design value of the chemical erosion depth y_{ced} to the concrete cover c_d, multiplied by the structure coefficient γ_i, is no greater than 1.0. However, if the concrete satisfies the required chemical erosion resistance, it may be assumed that the required performance of the structure will not be lost due to chemical erosion, and this verification may be omitted.

$$\gamma_i \frac{y_{ced}}{c_d} \leq 1.0 \qquad (3.2.3)$$

where, γ_i: may generally be set between 1.0 and 1.1.

y_{ced}: design value of depth of chemical erosion.

$$y_{ced} = \gamma_c \, y_{ce}$$

y_{ce}: characteristic value of depth of chemical erosion.

γ_c: material coefficient of concrete. This may generally be set to 1.0. However, it should be set to 1.3 for upper surface parts.

c_d: design value (mm) of concrete cover used for verification related to durability. It is derived using

Equation (3.2.4).

$$c_d = c - \Delta c_e \tag{3.2.4}$$

c: concrete cover (mm)

Δc_e: construction error (mm).

Commentary: When the design value for concrete cover has already been set based on other design conditions, verification can be considered to have been passed by setting the design value of the depth of chemical erosion to a value equal to or lower than the design value of concrete cover used in verification related to durability. When it is necessary to inhibit the chemical erosion of concrete to a degree that does not affect the required performance of the structure, it should be set to no more than the water-cement ratio indicated in **Commentary Table 3.2.1**, in accordance with the degradation environment.

Commentary Table 3.2.1 Maximum water cement ratio to ensure resistance to chemical erosion.

Degraded environment	Maximum water-cement ratio (%)
In contact with soil or water containing more than 0.2% sulphate as SO_4	50
In case of using an anti-freezing agent	45

Note: For those confirmed by actual results, research results, etc., the values in the table may be plus 5 to 10.

Conversely, when the chemical erosion depth for the design service life is set first, verification can be satisfied by setting the design value of concrete cover to a greater value than the corrosion depth. In this case, however, it must be considered in comparison with design values for concrete cover set according to other design conditions, and the highest of the values must be as the design value for concrete cover.

Because the types and concentrations of erosive substances that cause chemical erosion are varied, it is difficult to evaluate the resistance of concrete using uniform testing methods. Therefore, when conducting testing according to the type and the strength of external forces of environmental deterioration, the limit value of resistance to chemical erosion must be determined for each test.

The characteristic value of chemical erosion depth in Equation (3.2.3) should be determined from the results of accelerated testing and real-world exposure testing matched to actual environments. In cases such as undergoing action by sulfates, verification may be performed by using an appropriately set characteristic value for sulfur penetration depth to confirm that the specified depth is not reached during the design service life. Around acidic rivers, hot springs, or other environments in which erosive action is extremely harsh and deterioration of concrete cannot be completely inhibited, the most reliable method of verifying performance in concrete specimens is to expose them to actual environments. In evaluation at that time, the erosion rate of the concrete is derived from the exposure period, and verification is performed by confirming that concrete deterioration will not reach the limit depth of the structure during its design service life. In an environment such as sewers in which erosion action is extremely harsh and microorganisms such as sulfur-oxidizing bacteria are involved in concrete erosion, deterioration occurs through a mechanism by which hydrogen sulfide gas generated from sewage sludge is oxidized by sulfur-oxidizing bacteria, which results in sulfuric acid that erodes concrete. The erosion rate of concrete varies with the types and growth conditions of the sulfur-oxidizing bacteria. Therefore, it is reasonable to perform verification by confirming that deterioration will not progress to the rebar locations during the design service life by conducting exposure testing, *etc*. with consideration of the state of deterioration of individual facilities, deriving the rate of deterioration progress from the exposure period.

Chapter 4 Concrete cover of Structures in General Environments

4.1 Scope of application

This chapter presents standard values for concrete cover that satisfy Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume with respect to ordinary concrete structures constructed under general environments.

Commentary: In the case of structures constructed in general environments in which salt damage is not significant, it is possible in practice to easily satisfy Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume if concrete exhibiting excellent durability at or below a specified water-cement ratio is used, if concrete cover equal to or greater than a certain level is secured, and if crack width is restricted to a certain range. Therefore, design work for such structures can be simplified by pre-selecting the concrete cover and water-cement ratio within a range that will pass verification of steel corrosion due to carbonation and water permeation. For beams, columns, slabs, and other parts, this chapter indicates the minimum values of concrete cover and the maximum values of water-cement ratio that will satisfy concrete structures constructed in general environments. Here, a general environment refers to an environment that is free of risk of salt damage, freezing damage, and chemical erosion. If concrete cover and a water-cement ratio within the ranges shown here are selected and if the crack width limit values indicated in 8.2.2 in Chapter 8 of "Design: Main Volume" and 3.1.2 in Chapter 3 of this volume are satisfied, then the verification of durability in Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume can be omitted.

4.2 Maximum values for water-cement ratio and standard values for minimum concrete cover

(1) When ordinary concrete structures constructed under general environments satisfy the water-cement ratio and concrete cover for concrete indicated in **Table 4.2.1** and when crack width satisfies the crack width limit values indicated in 3.1.2 in Chapter 3, the durability verification in Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume may be considered to have passed. Additions to or subtractions from the value for concrete cover are to follow **Table 4.2.2**, in accordance with the environment for water exposure on the concrete structure.

Table 4.2.1 Minimum concrete cover and maximum water-cement ratio of structures satisfying durability*.

	Maximum W/C (%)	Minimum concrete cover c (mm)	Construction error □c_e(mm)
Column	50	45	15
Beam	50	40	10
Slab	50	35	5
Bridge pier	55	55	15

* Assumes a design service life of 100 years.

Table 4.2.2 Additions to or subtractions from the value for concrete cover in accordance with the environment for water exposure

Classification	Division		Examples of applicable parts of the structure	Additions to or subtractions from the value for concrete cover
0	Constantly dry environment	Parts of the structure that are not subject to water exposure and are constantly dry.	・Parts of the structure that are not subject to water exposure and are not prone to condensation. However, this excludes parts that are subject to water in the event of the loss of function of nearby waterproofing systems.	−5mm
I	Dry-wet cycling environment	Parts of the structure that are often exposed to water but begin to dry out as soon as the water supply ceases.	・General concrete structures, such as bridges and viaducts, where Parts of structures that are not classified as classifications 0, II or III.	±0mm
II		Parts of the structure that are often subjected to water and where the action of water continues for a long time and where there is a lot of repetition of dry and wet cycles.	・Parts of the structure that remain wet for a long period of time after rainfall due to leakage from the superstructure, etc. ・Parts of bridge piers, etc. in rivers that are in the environment described in the left column.	＋5mm
III	Constantly humid environment	Parts of the structure that are always wet, with little change in the dry and wet environment, and where the supply of oxygen to the steel is limited.	・Parts of the structure that are permanently underwater ・Parts of the structure that are in the soil below the permanent groundwater table.	−5mm

(2) When the verification of the provisions of Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume has been omitted by adopting the water-cement ratio and concrete cover indicated in **Table 4.2.1**, the values of these must be recorded in design drawings.

Commentary: Regarding (1): For concrete structures in a general environment in which salt damage is not significant, the rusting of steel can be prevented by setting the concrete cover so as to satisfy sufficient durability with respect to deterioration factors caused by the actions of water and oxygen. This indicates the combination of standard concrete quality (water-cement ratio) and concrete cover considered to be acceptable even when verification of durability is omitted under normal environmental conditions. **Table 4.2.1** indicates respective structural member-specific standard values for the maximum values of water-cement ratio for concrete made with ordinary Portland cement, minimum values of concrete cover, and construction errors. When this table is satisfied and when crack width satisfies the crack width limit values indicated in 3.1.2 in Chapter 3 of this volume, verification of steel corrosion due to penetration by water, as indicated in 3.1.3 in Chapter 3 of this volume, can be considered to have been passed. The values indicated in **Table 4.2.1** can also be used for Type-B blast furnace cement and Type-B fly ash cement. **Table 4.2.1** does not assume cases in which inspection and repair following completion of the structure is difficult even under general environments, cases of harsh construction work conditions, cases in which precast structural members are used, *etc*. In such cases, the satisfaction of required durability must be verified based on Chapter 8 of "Design: Main Volume" or Chapter 3 of this volume.

Steel corrosion in concrete is strongly affected by water and oxygen. Therefore, concrete cover in **Table 4.2.2** may be reduced at parts that constantly remain dry and in which the supply of water to rebars is limited, or at parts that constantly remain wet and in which the supply of oxygen to rebars is limited, as steel corrosion does not occur readily in these parts. Conversely, because steel corrosion occurs readily in an environment in which water acts over a long period to repeatedly reach the locations of rebars and in which dry-wet cycling occurs, additional concrete cover must be provided. The addition or reduction of concrete cover due to contact with water, as indicated in **Table 4.2.2**, is based on values calculated using the method indicated in 3.1.3.2 with consideration of the effects of environments of localized action by water.

Regarding (2): When the verification of durability in Chapter 8 of "Design: Main Volume" and Chapter 3 of this volume has been omitted by adopting the concrete cover and water-cement ratio indicated in **Table 4.2.1**, the concrete cover and water-cement ratio must be recorded in design drawings in place of characteristic values related to verification of durability. Descriptions of the assumed construction errors must also be provided.

Part3 Safety verification

Chapter 1 General Provisions

(1) Structural objects must be checked to maintain required safety over their design lifetime. In general, if this volume and "Design: Standards" Volume 7 are satisfied, then checking can be considered to be satisfied.

(2) In general, checking of load-carrying capacity is to be performed by confirming that the limit states of failure of member cross section and fatigue failure are not reached.

(3) In principle, these standards are to be used in the checking of limit states of failure of member cross section with force of member cross section used as a metric.

(4) In principle, these standards should be followed in the checking of limit states of fatigue failure using force of member cross section or stress intensity as metrics.

(5) In general, checking of stability with respect to the effects of earthquakes should be performed, following "Design: Standards" Volume 5.

(6) In checking of the functional safety of structural objects, limit states must be set according to the functions of the structural object and checking must confirm that the limit states are not reached.

Commentary: Regarding (1): This volume presents standard techniques for use in verifying that a structural object will maintain the required safety throughout its design lifetime. The load-carrying capacity calculation methods and other matters provided here are premised on satisfying the provisions of "Design: Standards" Volume 7. This volume describes the series of steps in design actions and their combinations, structural analysis, and the calculation of design response values and design critical values in checking.

Regarding (2), (3), and (4): This volume stipulates checking methods premised on the use of force of member cross section and stress intensity calculated from force of member cross section as design response values, primarily through a structural analytical method that models structural objects as wire rods. It presents standards for the use of force of member cross section as a metric for failure of member cross section and the use of stress intensity or force of member cross section as a metric for fatigue failure. When modeling using wire rods is difficult or impractical, the structural object may be modeled as finite elements according to "Design: Standards" Volume 9 and checking may be performed using strain, *etc*. as metrics.

Regarding (5): Checking of the stability of a structural object with respect to the effects of earthquakes was set according to "Design: Standards" Volume 5. When performing checking of stability with respect to actions other than the effects of earthquakes, "Design: Standards" Volume 5 should be used as reference.

Regarding (6): The functional safety of a structural object is its performance as determined from the functions of the structural object, and checking is performed with limit states set separately according to the functions of the structural object.

Safety with respect to the spalling of cover concrete or other public accidents involving users of a structural object or third parties may be regarded as satisfied at the design stage by satisfying "Design: Standards" Volume 2. When it is considered that spalling of cover concrete would have significant impact on human life or property around the structural object, actions should be taken such as implementing anti-spalling preventive measures at the design stage and stipulating such actions in maintenance plans as items to be inspected at the maintenance stage.

Chapter 2 Checking of failure of member cross section

2.1 General

(1) In principle, checking of safety is to be conducted by confirming that no structural members reach the failure of member cross section limit state under the design actions.

(2) In principle, checking of the limit state of failure of member cross section using force of member cross section should follow this chapter.

(3) Checking of the limit state of failure of member cross section is to be performed by confirming that the ratio of the design force of member cross section S_d to the design capacity of member cross section R_d, multiplied by the structure factor γ_i, is no greater than 1.0.

$$\gamma_i S_d / R_d \leq 1.0 \qquad (2.1.1)$$

where, S_d: design force of member cross section

R_d: design capacity of member cross section

γ_i: may generally be set between 1.0 and 1.2.

Commentary: Regarding (1): This chapter presents standard methods for verifying the safety of a structural object without consideration of the deterioration of materials during its design lifetime, premised on the satisfaction of placing performance in accordance with "Design: Standards" Volume 2, "Design: Standards" Volume 6, and "Construction Work" in the Standard Specifications.

All specifications used in checking, including material strength specifications, follow Chapter 5 of "Design: Main Volume."

Regarding (2) and (3): In the checking of safety with respect to failure of member cross section under combined actions such as bending moment and axial force, comparison of design force of member cross section and design capacity of member cross section under the combined actions in question should be performed.

2.2 Design actions and combinations of design actions

(1) The characteristic values of permanent actions, primary variable actions and accidental actions used in checking of failure of member cross section are to be the maximum values expected to occur under construction and during the design lifetime of the structural object. When a small value would be disadvantageous, the minimum value expected to occur is to be used. The characteristic values of secondary variable actions are to be determined in accordance with combinations of primary variable actions and accidental actions.

(2) When the standard value or nominal value of an action is determined separately from its characteristic value, the characteristic value of the action is to be set to the value obtained by multiplying the standard value or nominal value by the action correction factor ρ_f .

(3) Design actions are to be determined by multiplying the characteristic values of actions by an action coefficient. Action coefficients may be determined from **Table 2.2.1**.

Table 2.2.1 Action coefficient

Required performance	Limit state	Kind of action	Action coefficient
Safety	Failure of member cross section	Permanent action	1.0～1.2*
		Primary variable action	1.1～1.2
		Secondary variable action	1.0

* When a smaller permanent action other than dead load would be disadvantageous, the action coefficient for the permanent action should be set between 0.9 to 1.0.

(4) Design actions are generally combined as shown in **Table 2.2.2**.

Table 2.2.2 Combinations for design actions

Required performance	Limit state	Action combinations to be considered
Safety	Failure of member cross section	Summation of permanent actions, primary variable actions, and accidental actions

Commentary: Regarding (1):

Secondary variable actions are actions that should additionally be taken into consideration in combination with primary variable actions and accidental actions. Their characteristic values may generally be set to smaller values than when the same variable action is used as the primary variable action.

Regarding (3): The action coefficient for fixed dead load can be considered to exhibit little inconsistency, and therefore may be set between 1.0 to 1.1 with respect to checking of safety. For the self-weight of the structural object, the action coefficient may be set to 1.0, given that variation in self-weight on the high side generally results in an increase in capacity. The self-weight of a structural object refers to the weight of structural members that affects the capacity of the structural object, calculated using the unit weight indicated in 6.4.2 in "Design: Main Volume."

The action coefficient for additional dead loads, including exterior materials, handrails, pavement, and ballast load, can be considered to exhibit high inconsistency, with its magnitude likely to change in the future. With respect to considerations of safety, it may be set between 1.1 and 1.2.

As described in "Design: Standards" Volume 8, the action coefficient for prestress force in considerations of safety may generally be set to 1.0.

Regarding (4): It is practical to perform checking involving safety and seismic performance for combinations of actions with a given set of variable actions set as primary variable actions and other variable actions set as secondary variable actions. When combining secondary variable actions, the probability of the maximum expected value occurring at the same time as the primary action can generally be considered low. Therefore, it is advisable to reduce or otherwise appropriately set the characteristic values.

Design force of member cross section S_d in considerations of safety can generally be expressed using Equation (Commentary 2.2.1).

$$S_d = \sum \gamma_{ap} S_p(\gamma_{fp} \cdot F_p) + \sum \gamma_{ar} S_r(\gamma_{fr} \cdot F_r) + \sum \gamma_{aa} S_a(\gamma_{fa} \cdot F_a) \quad \text{(Commentary 2.2.1)}$$

where

S_d :design force of member cross section;

S_p, S_r, S_a : functions for deriving force of member cross section caused by permanent actions, primary variable actions, and secondary variable actions, respectively;

F_p, F_r, F_a : characteristic values of permanent actions, primary variable actions, and secondary variable actions, respectively;

$\gamma_{fp}, \gamma_{fr}, \gamma_{fa}$: action coefficients for permanent actions, primary variable actions, and secondary variable actions, respectively; and

$\gamma_{ap}, \gamma_{ar}, \gamma_{aa}$: structural analysis factor for permanent actions, primary variable actions, and secondary variable actions, respectively.

2.3 Calculation of design force of member cross section

2.3.1 General

(1) In the checking of failure of member cross section, design force of member cross section may generally be used as the design response value.

(2) The design force of member cross section S_d is the total of force of member cross sections S (where S is a function of F_d) calculated through structural analysis using the combined design action F_d, multiplied by the structural analysis factor γ_a.

$$S_d = \Sigma \gamma_a S\ (F_d) \quad (2.3.1)$$

where, S_d: design force of member cross section

S: force of member cross section

F_d: design action

γ_a: may generally be set to 0.1.

2.3.2 Structural analysis for checking of failure of member cross section

(1) Structural analysis for the checking of failure of member cross section should, in principle, consider the effects of nonlinearity of structural members. When the nonlinearity of structural members is negligible in its effects on force of member cross section or other design response values, the design response values may be calculated with structural members treated as linear. In this case, the structural analysis factor γ_a is set to 1.0.

(2) The effects of the non-linearity of materials are generally taken into consideration in the non-linearity of structural members. Effects of geometrical non-linearity are to be considered as necessary.

(3) When calculating force of member cross section through linear analysis, the effects of nonlinearity may be considered through a simple method premised on compliance with structural specifications, *etc*.

Commentary: Regarding (1): When calculating deformation in the limit state of failure of member cross section, consideration of nonlinearity is essential. The use of nonlinear analysis for the calculation of force of member cross section is rational.

However, when the ultimate strain of concrete in the modeled stress-strain curve for concrete shown in **Figure 2.4.1** is used as the design critical value of the limit state of failure of member cross section in a structural member that possesses a flexural failure morphology, it is advisable to apply an equivalent linear analysis method, *etc*., that approximates the effects of nonlinearity, using stiffness equivalent to the stress level in question. In that case, the design response value for the limit state of failure of member cross section may be calculated with the value of the structural analysis factor γ_a set to 1.0.

When the design critical value of the limit state of failure of member cross section is set to a value greater than the ultimate strain shown above, it is necessary to set appropriate checking metrics such as concrete compression strain, curvature, and structural member angle by applying a nonlinear analysis method using incremental analysis or other means, and to perform checking of the design critical values. In this case, the nonlinearity of structural members must comply with "Design: Standards" Volume 5.

When force of member cross section is used as a checking metric in a statically determinate structure such as a girder or a cantilever beam, the change in force of member cross section due to the stiffness of the structural member is small. Therefore, for convenience, the stiffness of the structural member may generally be considered elastic.

Regarding (2): Depending on the morphology of the structural object, the effects of geometric nonlinearity may not be negligible when the amount of deformation in structural members increases. Therefore, the effects of geometric nonlinearity should also be considered as required.

Regarding (3): Methods that consider the nonlinearity of structural members and structural objects in a simple manner by using a linear analysis method to stipulate structural details may also be used. The following is one simple method, in which the structural analysis factor γ_a may be set to 1.0.

(i) When linear analysis is used, redistribution of moments may occur and therefore ductile rotational capacity must be ensured. In general, the reinforcing bar ratio in all cross sections should be set to no more than 75% of the balanced reinforcing bar ratio.

(ii) Redistribution of bending moments at the support points or nodes of continuous beams, continuous slabs, Rahmen frames, and so on may be performed based on values from linear analysis. The bending moments that are redistributed should be within 15% of the linear analysis value and should be no less than 70% of the value of bending moments in all cross sections prior to redistribution. In this case, the reinforcing bar ratio in all cross sections must be set to no more than 50% of the balanced reinforcing bar ratio.

(iii) Force of member cross section due to the effects of normal temperature changes, shrinkage, creep, *etc*. can be ignored. In such cases, the reinforcing bar ratio in all cross sections must be set to no more than 50% of the balanced reinforcing bar ratio. When change occurs in the structural system between the time of under construction and the time of completion of construction, the change in force of member cross section due to the effects of creep are to be taken into consideration.

At the stage at which checking of failure of member cross section should be performed, structural members generally present major deformation, with the proportion of deformation caused by the effects of shrinkage, creep, and temperature changes attributable to weather conditions remarkably small. Response values resulting from these forced deformations can be ignored when calculating design response values in checking of failure of member cross section using linear analysis. In this case, the reinforcing bar ratio in all cross sections should be set to no more than 50% of the balanced reinforcing bar ratio. Compliance with this stipulation is not necessary when performing checking using nonlinear analysis.

Depending on the construction method, when the structural system differs between the time of under construction and the time of completion of construction, the change in force of member cross section due to creep may reach a magnitude that cannot be ignored and thus must be considered. In some cases, it is not possible to ignore the effects that temperature changes, *etc*. not caused by weather conditions have on design response values involved in checking of failure. In such cases, design response values may be calculated by considering the reduction in stiffness of structural members due to cracking, *etc*., based on empirical results or established theory.

2.3.3 Calculation of force of member cross section

2.3.3.1 Calculation of force of member cross section using wire rod models

When using a structural member model that uses wire rods, the force of member cross section obtained through structural analysis may be used to calculate the force of member cross section of structural members.

Commentary: When using an analytical method that confers nonlinearity using the relationship between the bending moment and the angle of rotation in the wire rod model, the force of member cross section obtained through structural analysis may be used without change as the force of member cross section of structural members. When a fiber model is used, it is acceptable to use the force of member cross section that was substituted for element stress, in the same manner as cases in which the finite element method is used.

2.3.3.2 Calculation of force of member cross section using the finite element method

When using the finite element method to derive the axial force and bending moment that act on structural members, calculation may be performed using the stress distribution inside the cross section and integrating in the direction of section depth. Shear force may be calculated from the equilibrium condition for bending moment distribution.

Commentary: Taking advantage of the features of the finite element method, metrics using localized information substituted for the state corresponding to the force of member cross section required for checking may be set as response values.

2.4 Calculation of design critical values

2.4.1 General

(1) In the checking of failure of member cross section, design capacity of member cross section may be used as the design critical value.

(2) Design capacity of member cross section is to be calculated for axial force, bending moment, shear force, and torsion.

(3) The design capacity of member cross section R_d is the value obtained by calculating capacity R (where R is a function of f_d) of the structural member cross section using design strength f_d, divided by the member factor γ_b.

$$R_d = R\,(f_d)\,/\,\gamma_b \tag{2.4.1}$$

where, R_d: design capacity of member cross section

R: capacity of structural member cross section

f_d: design strength

γ_b: member factor; the value indicated for each node may be used.

Commentary: In the case of an empirical equation, capacity calculation values are calculated as average values. In the equation for calculation of the shear capacity of structural members that do not have reinforcing bars, the effect of dimensions is large. Therefore, 1.3 was used in principle as the member factor, even when the accuracy of the empirical equation is high. If the empirical equation has been confirmed to be applicable to large structural members, this value may be reduced to about 1.15. The equation for calculation of the torsional capacity for structural members that do not have reinforcing bars exhibits considerable inconsistency and is susceptible to the effects of cracking caused by factors other than load. Because dimensional effects are also likely, the member factor was in principle set to 1.3, in the same manner as shear capacity.

For calculation equations for capacity of reinforcing bars yield type caused by truss-like shear mechanisms in structural members that have reinforcing bars, in this volume the coefficient was set in principle to 1.1, taking into account the effects of inconsistency in the dimensions of structural members.

2.4.2 Checking of bending moment and axial force

2.4.2.1 Design capacity of member cross section

(1) For structural members subjected to axial compressive force, the upper limit N'_{oud} of axial compressive capacity when hoop reinforcing bars is used is calculated from Equation (2.4.2). When spiral rebars are used, calculation is performed using the greater of Equation (2.4.2) or (2.4.3).

$$N'_{oud} = \left(k_1 f'_{cd} A_c + f'_{yd} A_{st}\right) / \gamma_b \tag{2.4.2}$$

$$N'_{oud} = \left(k_1 f'_{cd} A_e + f'_{yd} A_{st} + 2.5 f_{pyd} A_{spe}\right) / \gamma_b \tag{2.4.3}$$

where, A_c : cross-sectional area of concrete

A_e : cross-sectional area of concrete surrounded by spiral rebars

A_{st} : total cross-sectional area of longitudinal rebars

A_{spe} : converted cross-sectional area of spiral rebars ($= \pi d_{sp} A_{sp} / s$, where, d_{sp} : diameter of cross section surrounded by spiral rebars)

A_{sp} : cross-sectional area of spiral rebars

s : pitch of spiral rebars

f'_{cd} : design compressive strength of concrete

f'_{yd} : design compressive yield strength of longitudinal rebars

f_{pyd} : design tensile yield strength of spiral rebars

k_1 : reduction factor of strength ($= 1 - 0.003 f'_{ck} \leq 0.85$, where, f'_{ck} : characteristic value of concrete strength (N/mm^2))

γ_b : may generally be set to 1.3.

(2) Calculation of design capacity of member cross section of structural members subjected to bending moment and to bending moment and axial force in accordance with the direction of action of force of member cross section, for structural member cross section or structural member unit width, is to be performed based on assumptions ① to ④ below. When doing so, the member factor γ_b may generally be set to 1.1.

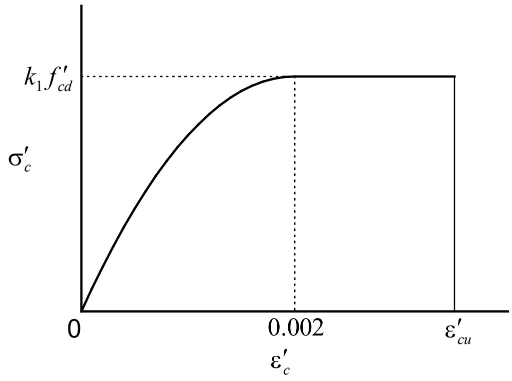
① Fiber strain is proportional to the distance from the neutral axis of the cross section.

② The tensile stress in the concrete is ignored.

③ The stress-strain curve of concrete is in principle in accordance with **Figure 2.4.1**.

④ The stress-strain curve of reinforcing bars is in principle in accordance with **Figure 2.4.2**.

(3) When calculating capacity by means such as verifying the limit state of failure of member cross section in structural members that are subjected to bending moment and to bending moment and axial force, the stress-strain curve of concrete may generally use the stress-strain curve presented in **Figure 2.4.1**. In the case of lightweight aggregate concrete, the stress-strain curve in **Figure 2.4.1** may be used.



$k_1 = 1 - 0.003 f'_{ck} \quad \leq 0.85$

$$\varepsilon'_{cu} = \frac{155 - f'_{ck}}{30000} \qquad 0.0025 \leq \varepsilon'_{cu} \leq 0.0035$$

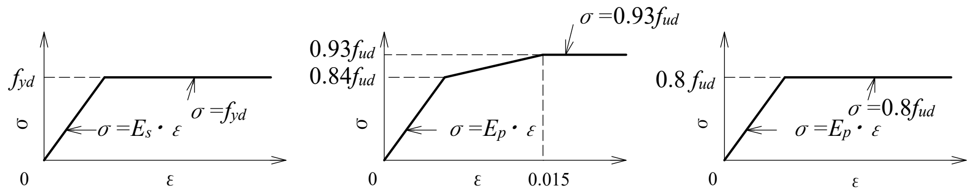
where the unit of f'_{ck} is N/mm²

The stress-strain formula of curve section

$$\sigma'_c = k_1 f'_{cd} \times \frac{\varepsilon'_c}{0.002} \times \left(2 - \frac{\varepsilon'_c}{0.002}\right)$$

Figure 2.4.1 Stress-strain curve of concrete

(4) When calculating capacity by means such as checking of the limit state of failure of member cross section, the stress-strain curve of reinforcing bars may generally use the stress-strain curve presented in **Figure 2.4.2**.



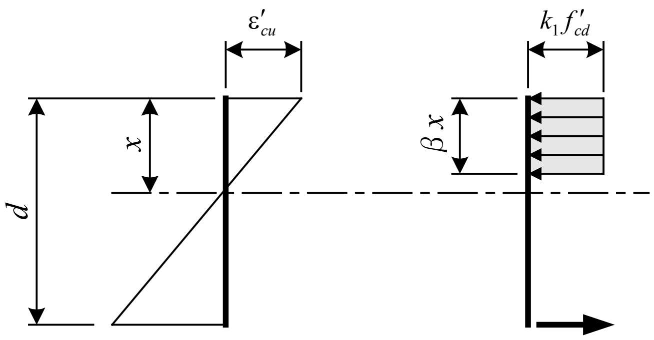
(a) The stress-strain curve of steel bars and structural materials

(b) The stress-strain curve of PC wire, PC strand and PC tendon standard No. 1

(c) The stress-strain curve of PC tendon standard No. 2

Figure 2.4.2 Modeled stress-strain curve of reinforcing bars

(5) Except in cases in which the strain of cross sections of structural members is entirely compressive, the distribution of compressive stress intensity of concrete may be assumed to be the distribution of rectangular compressive stress intensity (equivalent stress block) shown in **Figure 2.4.3**.



$k_1 = 1 - 0.003 f'_{ck} \qquad \leq 0.85$

$$\varepsilon'_{cu} = \frac{155 - f'_{ck}}{30000} \qquad \leq 0.0035$$

$f'_{ck} \leq 80$ N/mm²

$\beta = 0.52 + 80\,\varepsilon'_{cu}$

Figure 2.4.3 Equivalent stress block

(6) The design capacity of member cross section of a structural member that is subjected to simultaneous biaxial bending moment and axial force may be calculated based on the assumptions shown in (2).

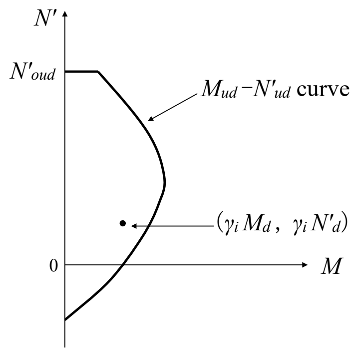
(7) When the effects of axial force are small, capacity of member cross section may be calculated as for a flexural structural member. Cases in which the effects of axial force are small may be considered cases in which

$e/h \geq 10$. h is the section depth; eccentricity e is the ratio of design bending moment M_d to design axial compressive force N'_d.

Commentary: Regarding (1): For high-strength concrete, k_1, which is dependent on compressive strength, was introduced.

An upper limit was set for design axial compressive capacity, and the member factor was set to 1.3.

Regarding (2): The relationship between design axial capacity and bending capacity when axial force and bending moment act simultaneously on the structural member cross section can be derived as a curve like that shown in **Commentary Figure 2.4.1**. In the checking of safety with respect to axial force and bending moment, the basic concept is that the point ($\gamma_i M_d$, $\gamma_i N'_d$) should be inside the (M_{ud}, N'_{ud}) curve, *i.e.*, should lie on the origin side, as indicated in the figure below.



Commentary Figure 2.4.1 The relationship between axial and bending capacity

Bending moment and capacity in structural analysis must be derived for the same axis.

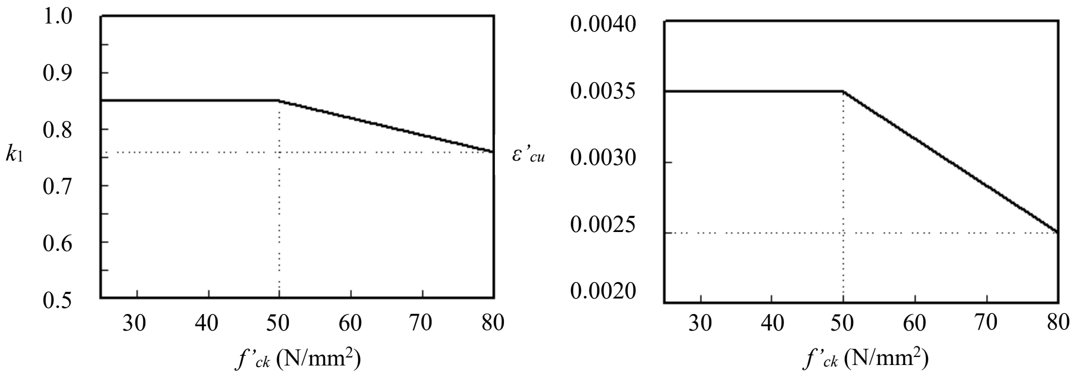
Assumption ① stipulated in the main text relates to strain distribution in structural member cross sections, and assumptions ② and ③ relate to stress distribution in concrete.

Assumption ④ relates to the stress-strain curve of reinforcing bars. The stress-strain curve may be determined based on appropriate empirical results when available.

Regarding (3): In some cases, differences in stress-strain curves do not have a significant effect, as in the ultimate capacity of member cross section of rod members. In such cases, a suitable shape may be assumed for the stress-strain curve, such as the commonly used shape shown in **Figure 2.4.1** or a rectangle. k_1 was made dependent on f'_{ck}, taking into account that the difference between cylinder strength and the strength back-calculated from the capacity of the structural member becomes large when strength is high. Given that the failure would also be brittle, ultimate strain was also reduced (**Commentary Figure 2.4.2**). **Figure 2.4.1** is a model for calculating ultimate strength when subjected to bending and axial force. When performing detailed examination extending to deformation that leads to the ultimate state, it is necessary to assume a stress-strain curve that follows reality, including in the descending region. In the descending region of the stress-strain curve, the compressive strain of concrete is not uniform within the specimen but rather is localized. Therefore, the element dimensions that define the stress-strain curve, the length of the specimen, the size of the localized region, and similar matters should be specified.

For lightweight aggregate concrete, it was decided that the stress-strain curve in **Figure 2.4.1** may be used. However, because it differs somewhat from ordinary concrete in reality, an appropriate curve must be assumed in the examination of deformation, toughness, *etc*.

For concrete surrounded by hoop reinforcing bars, spiral rebars, *etc*., results that have been appropriately obtained through experiments, *etc*. may be used.



Commentary Figure 2.4.2 The relationship between k_1, ε'_{cu} and f'_{ck}

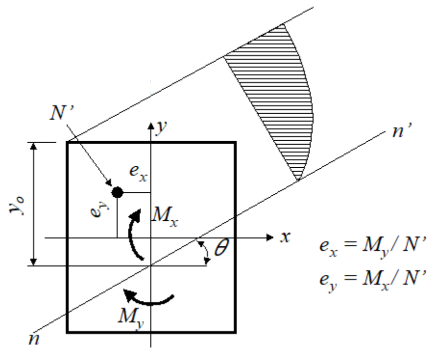
Regarding (4): The stress-strain curve of reinforcing bars varies by type of steel, chemical composition, manufacturing method, and other factors. Therefore, an appropriate form matched to the purpose of the checking must be assumed. The stress-strain curve indicated in **Figure 2.4.2** may generally be used in checking of capacity, stress intensity of structural member cross section, *etc*.

Regarding (5): The equivalent stress block indicated in **Figure 2.4.3** was set on the basis of the stress-strain curve shown in **Figure 2.4.1**. For simplicity, the stress of the stress block was set to be identical to the maximum stress in the stress-strain curve, and the height of the stress block was set so that resultant force and bending moment nearly match.

Regarding (6): The capacity of member cross section of a structural member that is subjected to simultaneous biaxial bending moment and axial force can be calculated in the same manner as that of a structural member that is subjected to uniaxial bending moment and axial force. When doing so, all rebars arranged around the cross section should be considered. As a point of caution in analysis, the positions of the arranged rebars must be considered accurately. In addition, in an ordinary rectangular cross-section column, it is necessary to take into account the fact that the inclination of the neutral axis of the cross section generally does not align with the main axis direction of the cross section except in the case of uniaxial bending moment.

Commentary Figure 2.4.3 is an example of a rectangular cross section. The capacity of member cross section when axial compressive force is applied at a position eccentric to the centroid can be derived by performing repeated calculations with the neutral axis position y_0 and the angle of inclination θ of the neutral axis as unknowns, until the calculated horizontal and vertical eccentricities e_x and e_y converge to a predetermined value.

The cross section may be divided into minute elements, stress may be assumed to be constant within the elements, and stress may be derived from strain at the centroid positions of the elements.



Commentary Figure 2.4.3 Rectangular cross section subjected to biaxial bending

Regarding (7): It was decided that the effects of axial force may be ignored when $e/h \geq 10$ in order to simplify the calculation and to perform checking on the conservative side. However, when the axial force is tension, the axial force must not be ignored.

2.4.3 Checking of shear force

2.4.3.1 General

(1) In checking of safety with respect to shear force, factors including the boundary conditions of structural members, the loading conditions, the direction of action of shear force, and the types of rod members, plane members, *etc*. must be considered.

(2) The method used for calculating the shear capacity of rod members must be in line with the load-carrying mechanism and must take into consideration the boundary conditions of the structural members and the loading conditions. In the case of a reinforced concrete structure, this is generally performed as follows.

(a) Rod members modeled in simple supports and cantilever supports are to be checked for safety with respect to design shear capacity V_{yd} and V_{wcd} derived using 2.4.3.2. When the shear span ratio is small, safety is to be confirmed for the design shear compression failure capacity V_{dd} instead of these design shear capacities.

(b) For rod members modeled in supports anchored at both ends, shear capacity is in principle to be calculated using a shear capacity calculation method or nonlinear finite element analysis with consideration of the effects that support conditions, loading conditions, *etc*. have on load-carrying mechanisms. When no special examination is undertaken, design shear capacity V_{yd} in (a) may be used.

(3) When a plane member is subjected to out-of-plane shear force, checking of the out-of-plane shear force is to be performed in the same manner as for rod members. At the same time, when a concentrated load acts partially, checking of punching shear failure with respect to the concentrated load V_d is to be performed following 2.4.3.3.

(4) When a plane member is subjected to in-plane shear force, the shear capacity should in principle be calculated using nonlinear finite element analysis with consideration of the boundary conditions of the structural member and the loading conditions. When conforming to the model presented in 2.4.3.4, checking of the in-plane force may be performed following 2.4.3.4.

(5) When it is necessary to transmit shear force V_d at a surface where cracking is likely to occur, at a joint surface, *etc*., checking of direct shear transfer on the shear plane is to be performed, following 2.4.3.5.

(6) For parts in which the flow of force changes significantly within a concrete structural object, parts in which cross section changes suddenly, corners, openings, anchorage zones of reinforcing bars, and other parts for which theories of beams and plates are difficult to apply (*i.e.*, discontinuous regions), demonstration experiments of capacity or nonlinear finite element analysis should be performed.

Commentary: Regarding (1): The behavior and the mechanism of failure of structural members under shear force actions differ by type of structural member (rod member, plane member, *etc*.). For plane members, they also differ according to whether the shear force acts in the out-of-plane or the in-plane direction. Therefore, checking of safety must be performed using a method that takes these into consideration.

When using concrete in which autogenous shrinkage is significant and the modulus of elasticity suddenly becomes large, or when using concrete for which drying shrinkage is large in thin structural members, reduction of shear capacity by means such as reduction of the tensile strength of the concrete must be considered.

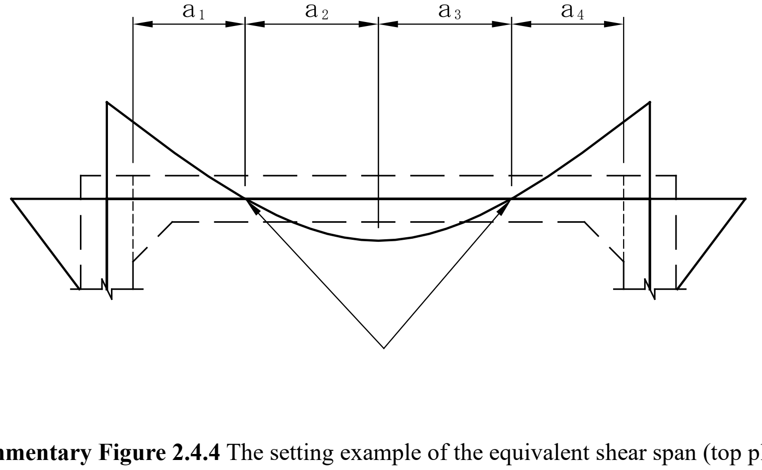
Regarding (2): The load-carrying mechanism of rod members differs with the boundary conditions of the structural members. Therefore, the calculation method must be matched to the load-carrying mechanism.

Regarding (2) (a):

For rod members that use shear reinforcing bars, safety with respect to yielding of the shear reinforcing bars is to be confirmed using Equation (2.4.4), and safety with respect to diagonal compression failure of web concrete is also to be confirmed using Equation (2.4.8). When the shear span ratio (shear span/effective depth) is small, the shear capacity will be larger than the shear capacity calculated using Equation (2.4.4). Therefore, checking of design shear compression failure capacity should be performed using Equation (2.4.9).

Regarding (2) (b): In the calculation of specific load-carrying capacity, numerical analysis such as nonlinear finite element analysis or methods based on experimental studies may be used. When no special examination is undertaken, then even if design shear capacity V_{yd} shown in the previous item (a) is used for convenience, shear capacity will be on the conservative side. Therefore, design shear capacity V_{yd} indicated in (a) may be used.

In structural members on which multiple distributed loads act simultaneously and for which shear span cannot be clearly determined, as in the top plate, bottom slab, and side walls of a box culvert, the equivalent shear span for the cross section to be checked may be set based on the bending moment distribution for each combination of actions. Specifically, as shown in **Commentary Figure 2.4.4**, for the spans a_1 and a_4 that are directly supported by the side walls, shear capacity is calculated with the shear span from the end of the structural member to the inflection point of bending moment set as shear span *a*, and with the span assumed as rod members or deep beams in line with the shear span ratio *a*/*d* (*d*: effective depth of the structural member). In the indirectly supported spans a_2 and a_3, because a truss-like load-carrying mechanism dominates, shear capacity is calculated with the span assumed as rod members regardless of the shear span ratio. The shear span when no bending moment inflection point is present may be used as the structural member length. The shear span of the center column may be set to half of the length of the structural member.



Commentary Figure 2.4.4 The setting example of the equivalent shear span (top plate)

Regarding (3): Checking of the shear force of one-way slabs or other plane members that exhibit beam-like behavior may be performed in the same manner as rod members.

Regarding (4): A variety of methods may be used in checking of the safety of plane members that are subjected to in-plane shear force. Here, as a checking method for plane members that employ a highly practical arrangement of rebars in two orthogonal directions, a method is shown for confirming safety by following 2.4.3.4 (1) to derive the forces T_{xd} and T_{yd} that act on the rebars in each direction from the in-plane forces and to derive the diagonal compression force C'_d that acts on concrete, and then comparing these with the capacities T_{xyd} and T_{yyd} derived from 2.4.3.4 (2) and with C'_{ud}, respectively.

Regarding (5): In the case of a thin T-shaped beam with a large flange width, a large shear force acts on the base portion of the flange and shear failure may occur in the flange. It is advisable to perform checking of this as a direct transfer of shear force.

Regarding (6): For beams, columns, slabs, and other structural members for which results from past design and construction work are abundant and for which structural member cross sections and reinforcing bars arrangement methods that satisfy the required performance are widely understood empirically, rational safety design is possible through the application of this section to design. However, for the load-carrying capacity of structural members that include corners, openings, and parts with sudden changes in cross section, methods exist for verifying safety by deriving the load-carrying mechanism through experiments or nonlinear finite element analysis for the assumed structural member form and arrangement of materials. A strut-and-tie model may be applied. Methods of calculation using a strut-and-tie model are to follow "Design: Standards" Volume 10. The validity of a strut-and-tie model must be demonstrated before it is used.

2.4.3.2 Design shear capacity of rod members

(1) The design shear capacity V_{yd} may be derived using Equation (2.4.4).

However, when bent rebar and stirrups are used together as shear reinforcing bars, no less than 50% of the shear force to be borne by the shear reinforcing bars should be borne by the stirrups.

$$V_{yd} = V_{cd} + V_{sd} \tag{2.4.4}$$

However, $p_w \cdot f_{yd} / f'_{cd}$ should be set to ≤ 0.1.

where, V_{cd} :design shear capacity of rod members that do not use shear reinforcing bars, following Equation (2.4.5).

$$V_{cd} = \beta_d \cdot \beta_p \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b \tag{2.4.5}$$

$$f_{vcd} = 0.20\sqrt[3]{f'_{cd}} \quad (\text{N/mm}^2)\ f_{vcd} \le 0.72\ (\text{N/mm}^2) \tag{2.4.6}$$

$\beta_d = \sqrt[4]{1000/d}$ (d: mm) Set to 1.5 when $\beta_d > 1.5$.

$\beta_p = \sqrt[3]{100 p_v}$ Set to 1.5 when $\beta_p > 1.5$.

b_w: width of web (mm)

d: effective depth (mm)

$p_v = A_s / (b_w \cdot d)$

A_s: cross-sectional area of reinforcing bars at tension side (mm^2)

f'_{cd}: design compression strength of concrete (N/mm^2)

γ_b: may generally be set to 1.3

V_{sd}: : design shear capacity borne by the shear reinforcing barss, following Equation (2.4.7)

$V_{sd} = [A_w f_{wyd} (\sin \alpha_s + \cos \alpha_s) / s_s] \, z/\gamma_b$ (2.4.7)

A_w: total cross-sectional area of shear reinforcing bars in interval s_s (mm^2)

f_{wyd}: design yield strength of the shear reinforcing bars, with the lower of $25 f'_{cd}$ (N/mm^2) or 800 N/mm^2 as its upper limit

α_s: angle formed between shear reinforcing bars and structural member axis

s_s: spacing in arrangement of shear reinforcing bars (mm)

z: distance from the acting position of the resultant force of compressive stress to the centroid of the tensile reinforcing bars; may generally be set to d/1.15

$p_w = A_w / (b_w \cdot s_s)$

γ_b: may generally be set to 1.1.

(2) For directly supported rod members, checking of V_{yd} is not necessary for the interval from the bearing frontal face to half of the total height h. However, in this interval, shear reinforcing bars is to be arranged in an amount equal to or more than that required for the cross section at a distance of only h/2 from the bearing frontal face. For structural members with non-uniform cross section, the value at the bearing frontal face may be used as the structural member height. For haunches, parts at an angle more gradual than 1:3 are to be considered valid.

When performing checking of plane members subjected to out-of-plane shear force, treating the plan members as rod members in accordance with 2.4.3.1 (3), the design shear capacity near the structural member support must be calculated using an evidence-based method.

(3) Design diagonal compressive failure capacity V_{wcd} with respect to shear in web concrete may be calculated using Equation (2.4.8).

$V_{wcd} = f_{wcd} \cdot b_w \cdot d/\gamma_b$ (2.4.8)

where, $f_{wcd} = 1.25\sqrt{f'_{cd}}$ (N/mm^2) $f_{wcd} \leq 9.8$ (N/mm^2)

γ_b: may generally be set to 1.3.

(4) Measuring the web width of structural members

Except in the case of a circular cross section, when the width of the web changes in the height direction of the structural member, the minimum width within the range of effective depth d is to be used as b_w. In the case of multiple webs, the total width is to be used as b_w. In the case of a solid circular cross section or a hollow circular cross section, the total width of the web of a square box of equal area (equivalent area box shape) or one side of a square of equal area (equivalent area square) is to be used as b_w. The axial tensile reinforcing bars cross-sectional area A_s may be used as the cross-sectional area of the reinforcing bars at tension side 1/4 (90°) portion, and the effective depth d may be used as the distance to the centroid of the reinforcing bars, with the distance from the compression edge of the equal area square or equal area box shape as A_s.

However, such a method of determining the cross-sectional area of the axial tension reinforcing bars must not be used for calculation of bending capacity.

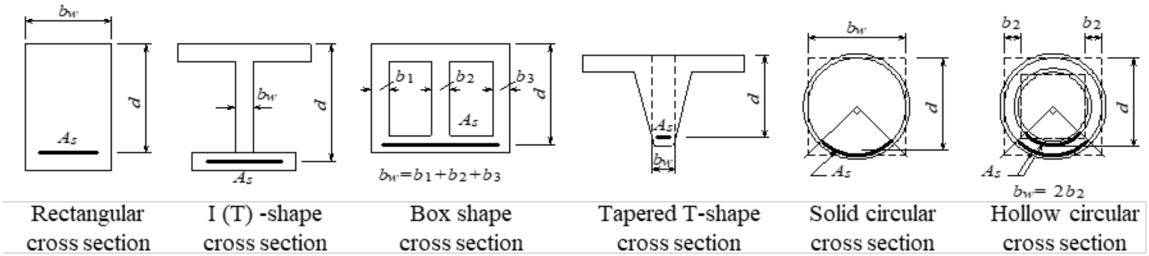


Figure 2.4.4 Method for determining b_w and d for various cross-sectional shapes

(5) Design shear compressive failure capacity may be calculated using Equation (2.4.9).

$$V_{dd} = \beta_d \cdot \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b \tag{2.4.9}$$

where, V_{dd}: design shear compressive failure capacity (N)

$$f_{dd} = 0.19\sqrt{f'_{cd}} \quad \text{(N/mm}^2\text{)} \tag{2.4.10}$$

$\beta_d = \sqrt[4]{1000/d}$ Set to 1.5 when β_d >1.5.

$\beta_p = \dfrac{1+\sqrt{100p_v}}{2}$ Set to 1.5 when β_p >1.5.

$\beta_a = \dfrac{5}{1+(a/d)^2}$

b_w: width of web (mm)

d: loading point in the case of simple beams; effective depth (mm) at the support frontal face in the case of cantilever beams

a: distance from the support frontal surface to the loading point (mm)

$p_v = A_s / (b_w \cdot d)$

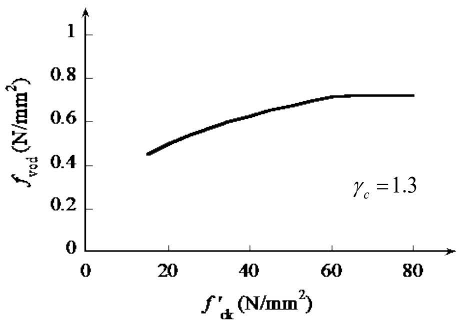
A_s: cross-sectional area of reinforcing bars at tension side (mm^2)

f'_{cd}: design compression strength of concrete (N/mm^2)

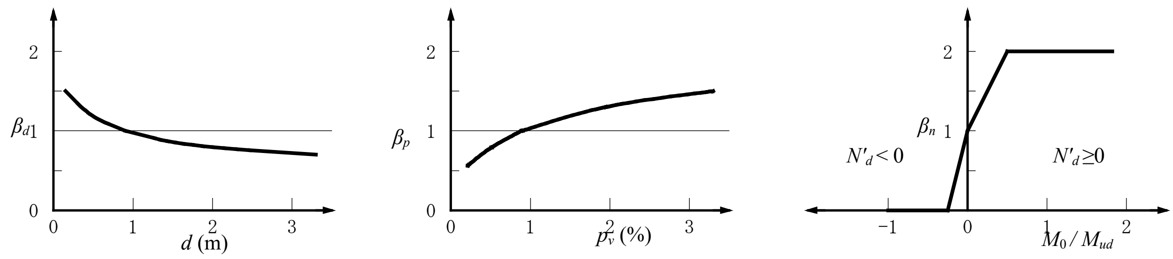
γ_b: may generally be set to 1.3.

Commentary: Regarding (1): Design shear capacity V_{yd} is expressed as the sum of the share to be borne by the concrete V_{cd} and the share to be borne by shear reinforcing bars V_{sd}, as indicated in Equation (2.4.4). When the amount of shear reinforcing bars is relatively small, actual shear strength may greatly exceed V_{yd}. However, because actual shear strength has not yet been accurately evaluated, Equation (2.4.4) was provided as a calculation that is practical and on the conservative side.

Equation (2.4.5) was derived with consideration of the effects of concrete strength, structural member height, and reinforcing bar ratio on V_{cd}. **Commentary Figure 2.4.5** shows the relationship between the design specified strength of concrete and shear strength in this equation. The value f'_{cd} used in Equation (2.4.6) is the design compression strength of concrete and is obtained by dividing the design specified strength of concrete f'_{ck} by the material factor γ_c. The effects of the effective depth of the structural member and the longitudinal rebar ratio are shown in **Commentary Figure 2.4.6 (a)** and **(b)**, respectively. Equation (2.4.5) ignores the effect of the shear span ratio a/d to provide a safer calculated value than that of the equation of Futaba *et al*. Even when the material factor for concrete with a design specified strength in excess of 60 N/mm^2 is changed to 1.3, Equation (2.4.5) may be used with the upper limit of f_{vcd} set to 0.72 N/mm^2 and the member factor γ_b set to 1.3.



Commentary Figure 2.4.5 The relationship between f_{vcd} and design specified strength of concrete f'_{ck}



(a) (b) (c)

Commentary Figure 2.4.6 The influence of various factors affecting shear capacity

The design shear capacity of RC structural members that are subjected to axial force is in principle to be calculated using nonlinear finite element analysis. However, in the case of bridge piers or other structural members in which axial compressive stress intensity is small with respect to the compression strength of concrete, calculation of design shear capacity without any special considerations may use the following equation.

$$V_{cd} = \beta_d \cdot \beta_p \cdot \beta_n \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b \qquad \text{(Commentary 2.4.1)}$$

where

$$f_{vcd} = 0.20\sqrt[3]{f'_{cd}} \ \ (\text{N/mm}^2);\ f_{vcd} \leq 0.72\ (\text{N/mm}^2); \qquad \text{(Commentary 2.4.2)}$$

$\beta_d = \sqrt[4]{1000/d}$ (d: mm), Set to 1.5 when $\beta_d > 1.5$;

$\beta_p = \sqrt[3]{100 p_v}$, Set to 1.5 when $\beta_p > 1.5$;

$\beta_n = 1 + 2M_0 / M_{ud}$ (when $N'_d \geq 0$),

Set to 2 when $\beta_n > 2$;

$= 1 + 4M_0 / M_{ud}$ (when $N'_d < 0$),

Set to 0 when $\beta_n > 0$;

N'_d: design axial compressive force;

M_{ud}: pure bending capacity without consideration of axial force;

M_0: bending moment required to cancel out the stress generated by the axial force at the tensile edge with respect to design bending moment M_d;

b_w: width of web (mm);

d: effective depth (mm); and

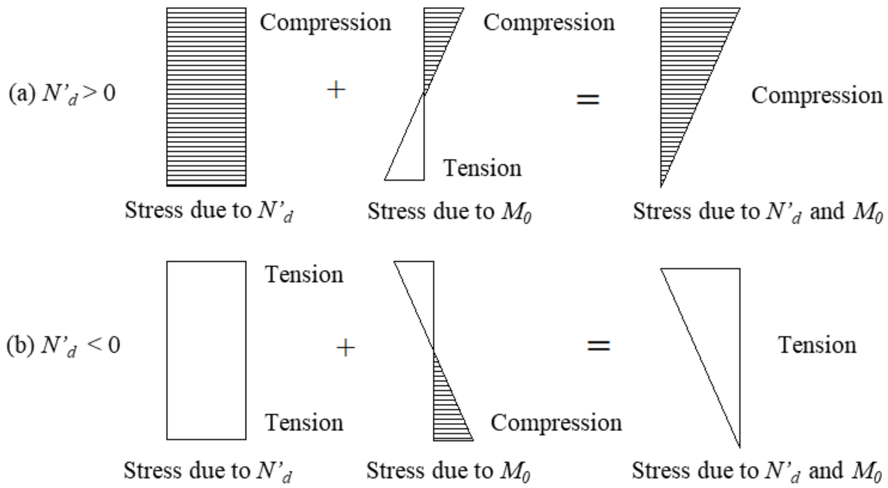
$p_v = A_s / (b_w \cdot d)$.

The failure of structural members subjected to axial compressive force is to be calculated from $\beta_n = 1 + 2M_0 / M_{ud}$ on the basis of compatibility with experimental results. Comparison was made with empirical values from past experiments, and

$\beta_n = 1 + 4M_0 / M_{ud}$ was set (see **Commentary Figure 2.4.6 (c)**). When $M_d > M_{ud} / 2$, $M_{ud} = 2M_d$ can used as a safe approximation. M_{ud} is flexural strength and M_d is design bending moment. M_0 and M_{ud} are calculated with 1.0 as the member factor.

The concept behind M_0 is shown in **Commentary Figure 2.4.7**. This calculation of M_0 may be performed for concrete cross sections with all cross sections considered valid for simplicity.

Equation (2.4.5) is for general ordinary concrete. For lightweight aggregate concrete, 70% of the value may be used.



Commentary Figure 2.4.7 Concept of M_0 (when $M_d > 0$)

The use of exceedingly high-strength rebars as shear reinforcing bars should be avoided, and the design yield strength f_{wyd} of the shear reinforcing bars must be limited.

Because little experimental data is available for cases in which design yield strength exceeds 800 N/mm^2, the upper limit was set to 800 N/mm^2.

At least 1/2 of the shear force to be borne by the shear reinforcing bars must be borne by the stirrups.

When Equation (2.4.7) is applied, it should be confirmed that effective depth is about four times that of the covering of the compression edge. Efficacy should be demonstrated using nonlinear finite element analysis, and shear capacity should be calculated

This volume sets design shear capacity to the sum of the shear capacity borne by the shear reinforcing bars and the shear capacity borne by elements other than the shear reinforcing bars, as shown in Equation (2.4.4). Using existing calculation equations as criteria, arrangement of excessive shear reinforcing bars is to be limited by setting an upper limit for the shear capacity borne by the shear reinforcing bars. Shear compression failure of the concrete at the compression edge may occur when the ratio ($p_w \cdot f_{wyd} / f'_{cd}$) of the product ($p_w \cdot f_{wyd}$) of the shear reinforcing bars ratio p_w (=A_w /($b_w \cdot s_s$)) and shear reinforcing bars yield strength f_{wyd} to the design compression strength of concrete (f'_{cd}) exceeds approximately 0.1. This should be used as a guideline in setting an upper limit on the amount of shear reinforcing bars. When shear reinforcing bars are arranged in excess of this guideline, or when using concrete for which f'_{cd} is 50 N/mm^2 or more, efficacy should be demonstrated using nonlinear finite element analysis and shear capacity should be calculated.

Checking when biaxial shear force acts on a rectangular cross section may be performed by confirming that Equation (Commentary 2.4.3) is satisfied.

$$(\gamma_i V_{dx}/V_{yx})^2 + (\gamma_i V_{dy}/V_{yy})^2 \leq 1.0 \qquad \text{(Commentary 2.4.3)}$$

where

V_{yx}: design uniaxial shear capacity on the x-axis;

V_{yy}: design uniaxial shear capacity on the y-axis;

V_{dx}: design shear force on the x-axis when biaxial shear force acts; and

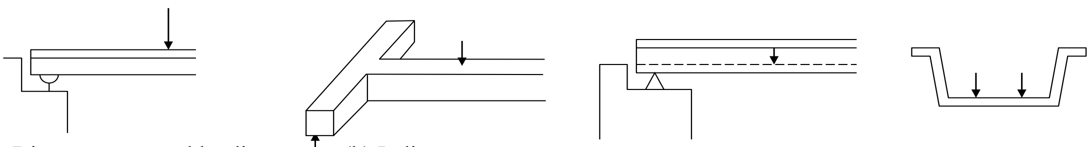
V_{dy}: design shear force on the y-axis when biaxial shear force acts.

In the case of a circular cross section, hoop rebars and spiral rebars may be considered shear reinforcing bars.

Regarding (2): Taking simplicity of design into account, checking of shear capacity (V_{yd}) based on truss action may be omitted for the interval from the bearing frontal face to half of the total height.

In special cases such as a large concentrated load anchored near a support point, checking of design shear compressive failure capacity V_{dd} must be performed (see Equation (2.4.9)). Because shear capacity increases at the corners and near the supports of rod members subjected to distributed loads, checking should be performed using a more appropriate method.

A directly supported structural member is a structural member in which compressive stress occurs in the web due to the load and support point reaction force, as in **Commentary Figure 2.4.8 (a)**. In cases such as **(b)** in which the support point reaction force is transmitted as shear force through cross beams, *etc*. to main girders, or in cases such as **(c)** in which the loaded state is such that compressive stress does not occur in the web, this item must not be applied.



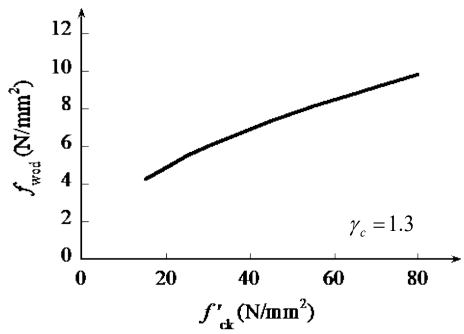
(a) Direct support and loading (b) Indirect support (c) Indirect loading

Commentary Figure 2.4.8 Direct support and indirect support

Shear reinforcing bars are typically not used in plane members such as retaining walls and box culverts. In such cases, it may not be rational to apply design methods as-is near the supports of rod members on the assumption that shear reinforcing bars will be arranged. For plane members or the corners of structural members that are subjected to such distributed loads, checking may be performed using more accurate evidence-based methods.

Regarding (3): This paragraph is provided for the purpose of avoiding cases in which compression failure in the web concrete leads to failure of structural members. Equation (2.4.8) was set on the conservative side, given the lack of experimental data. **Commentary Figure 2.4.9** indicates the relationship between f_{wcd} and design specified strength f'_{ck}.

This equation was derived for ordinary concrete. However, its validity has also been demonstrated for concrete with a degree of high strength, and the value when f'_{ck} is 80 N/mm^2 was set as the upper limit. When f'_{ck} is greater than 80 N/mm^2, validity must be confirmed through testing.



Commentary Figure 2.4.9 The relationship between f_{vcd} and specified design strength of concrete f'_{ck}

Regarding (5): When the shear span ratio *a/d* is small, a directly supported rod member exhibits properties similar to those of a tied arch with tensile rebars tied, even after the occurrence of diagonal cracking. Because the tensile rebars become the ties of a tied arch in such a case, they must not be anchored along the span, and the entire amount of rebar that is required with respect to maximum bending moment must be continuously and sufficiently anchored past the support point.

For cases in which shear reinforcing bars is not arranged, Equation (Commentary 2.4.4) obtained from past research results was simplified to be on the conservative side with respect to empirical results so that the shear span ratio *a/d* is approximately 2.5 and approaches the normal shear capacity equation for rod members, and the shear compressive failure capacity equation was determined. Cantilever beams and simple beams are assumed in the formation of arch ribs. Therefore, when the form of the arch ribs is assumed to be different, separate detailed checking should be performed as necessary.

$$V_c = \frac{0.24 \cdot f_c'^{2/3} \cdot \left(1 + \sqrt{100 p_v}\right) \cdot (1 + 3.33 r/d)}{1 + (a/d)^2} b_w \cdot d$$

(Commentary 2.4.4)

where

r : structural member axial length of bearing plate.

It has been confirmed empirically that beams for which *a/d* is small exhibit a shear reinforcing bars effect when horizontally oriented rebars are arranged in the web. In this case, design shear compressive failure capacity V_{dd} can be derived by substituting tensile reinforcing bars ratio p_v (derived from Equation (Commentary 2.4.5)) into Equation (Commentary 2.4.6).

$$p_v = p_{v1} + p_{v2} \cdot d_2 / d_1 \qquad \text{(Commentary 2.4.5)}$$

where

p_v: tensile reinforcing bars ratio;

p_{v1}: tensile reinforcing bars ratio of tensile reinforcing bars;

p_{v2}: tensile reinforcing bars ratio of horizontally oriented rebars arranged in the web of the beam;

d_2: distance from compression edge of tensile rebar; and

d_1: distance from the compression edge of horizontally oriented rebar arranged in the web of the beam.

When taking the effects of shear reinforcing bars into account, Equation (Commentary 2.4.6) may be used to perform calculation. In this case, shear reinforcing bars must be arranged so that the shear reinforcing bar ratio is 0.2% or higher.

$$V_{dd} = (\beta_d + \beta_w)\beta_p \cdot \beta_a \cdot \alpha \cdot f_{dd} \cdot b_w \cdot d / \gamma_b$$

(Commentary 2.4.6)

where

V_{dd}: design shear compressive failure capacity (N);

α: coefficient that considers the effects of the structural member axial length (r) of the bearing plate, determined as follows. In general, r/d may be set to 0.1;

α=(1+3.33 r/d) / (1+3.33・0.05);

$f_{dd} = 0.19\sqrt{f'_{cd}}$ (N/mm^2);

$\beta_d = \sqrt[4]{1000/d}$ Set to 1.5 when $\beta_d > 1.5$;

$\beta_w = 4.2\sqrt[3]{100p_w} \cdot (a/d - 0.75)/\sqrt{f'_{cd}}$

Set to 0 when $\beta_w < 0$;

$\beta_p = \frac{1+\sqrt{100p_v}}{2}$ Set to 1.5 when $\beta_p > 1.5$;

$\beta_a = \dfrac{5}{1+(a_v/d)^2}$;

b_w: width of web (mm);

d: loading point in the case of simple beams; effective depth (mm) at the support frontal face in the case of cantilever beams;

a_v: distance from the support frontal surface to the loading point (mm);

$p_v = A_s/(b_w \cdot d)$;

A_s: cross-sectional area of tensile reinforcing bars (mm^2);

p_w : shear reinforcing bar ratio

$p_w = A_w/(b_w \cdot s_s)$ Set p_w to 0 when $p_w <$ 0.002;

A_w: total cross-sectional area (mm^2) of shear reinforcing bars orthogonal to the structural member axis in the interval s_s ;

s_s: spacing in arrangement of shear reinforcing bars orthogonal to the structural member axis (mm)

f'_{cd} : design compression strength of concrete (N/mm^2); and

γ_b: member factor; generally, set to 1.2.

When not following the above, effects should be demonstrated through non-linear finite element analysis, empirical studies, *etc*. for which applicability has been demonstrated, and shear capacity should be calculated. When not following empirical studies, loading and other experiments should be conducted using specimens that represent the actual structural objects. Capacity in actual structures should be comprehensively examined by also performing nonlinear analysis and by examining the sensitivity of influencing factors and the differences between experimental conditions and actual conditions of the structure.

Equation (2.4.9) can also be used when concrete with a compressive strength of 60 to 80 N/mm^2 is used.

The scope of application of Equation (2.4.9) was determined on the conservative side, using the results of past studies as reference. When subjected to distributed loads or when subjected to multiple loads within the span, the distance from the center of gravity to the support frontal surface may be used as a. In the case of structural members that are not directly supported or when the required structural specifications are not satisfied, a risk exists that the type of load-carrying mechanism indicated here will not be formed; therefore, (5) must not be applied.

Equation (2.4.9) applies to ordinary concrete. The value must be reduced for lightweight aggregate concrete, and generally may be set to 70% of the value in Equation (2.4.9).

2.4.3.3 Design punching shear capacity of plane members

(1) When the loading surface is at a distance from the free edge or opening of the structural member and the eccentricity of the load is small, the design punching shear capacity V_{pcd} may be derived using Equation (2.4.11).

$$V_{pcd} = \beta_d \cdot \beta_p \cdot \beta_r \cdot f_{pcd} \cdot u_p \cdot d / \gamma_b \tag{2.4.11}$$

where,

$$f_{pcd} = 0.20\sqrt{f'_{cd}} \quad (\text{N/mm}^2) \qquad f_{pcd} \le 1.2 \text{ N/mm}^2 \tag{2.4.12}$$

$\beta_d = \sqrt[4]{1000/d}$ (d : mm) Set to 1.5 when $\beta_d > 1.5$.

$\beta_p = \sqrt[3]{100 p_v}$ Set to 1.5 when $\beta_p > 1.5$.

$\beta_r = 1 + 1/(1 + 0.25u/d)$

f'_{cd} : design compression strength of concrete, with N/mm^2 as the unit

u: peripheral length of load surface

u_p: Peripheral length of the cross section to be checked, calculated at a position d/2 distant from the load surface

d and p: effective depth and reinforcing bar ratio, set to average values for rebars in two directions.

γ_b: may generally be set to 1.3.

(2) When the load surface is close to the free edge or opening of the structural member, reduction in punching shear capacity must be considered.

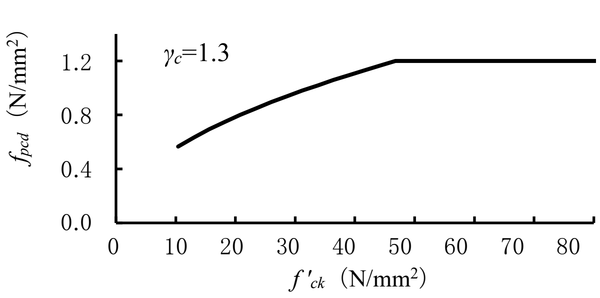
(3) When the load is eccentric to the load surface, the effects of flexing and torsion must be considered.

Commentary: Regarding (1): When localized loads act, as in footings or connecting parts between columns and slabs, safety with respect to punching shear failure must be checked.

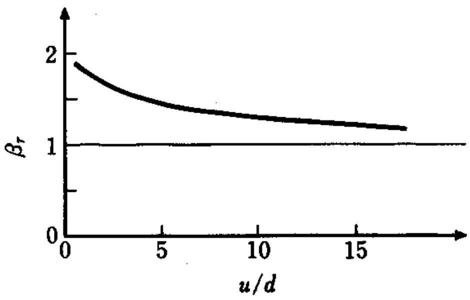
As, deriving the punching shear capacity of slabs is theoretically difficult, it was assumed here that the punching shear capacity of slabs is expressed in essentially the same format as the shear capacity calculation equation for beams. Equation (2.4.11) provides the coefficient based on past punching shear experiment results for slabs and footings.

Commentary Figure 2.4.10 shows the relationship between the value obtained by Equation (2.4.12) and the design specified strength of concrete. β_r in Equation (2.4.11) is a coefficient for considering the effects of the size of the loaded area and takes a value between 1 and 2 as indicated in **Commentary Figure 2.4.11**.

In the case of high-strength concrete, this should in principle be confirmed through testing. When confirmation by testing is not performed, the value when f'_{cd} is 50 N/mm^2 should be set as the upper limit.

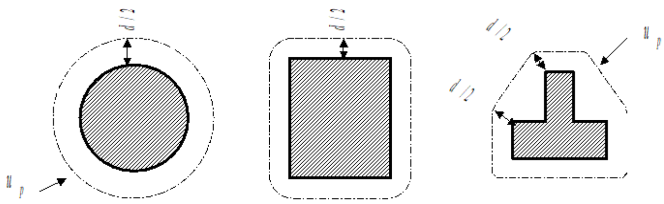


Commentary Figure 2.4.10 The relationship between f_{pcd} and specified design strength of concrete f'_{ck}



Commentary Figure 2.4.11 The influence of the size of the loaded area for the capacity of punching shear failure

Equation (2.4.11) was derived with the cross section to be checked for calculation of capacity set to a position d/2 distant from the loaded zone periphery, as indicated in **Commentary Figure 2.4.12**.



Commentary Figure 2.4.12 Cross sections to be checked

When subjected to pull-out force as in the case of steel tower foundation footings, the failure state is similar to that of punching shear failure. Performing calculation of design capacity in the same manner as punching shear may follow the method below.

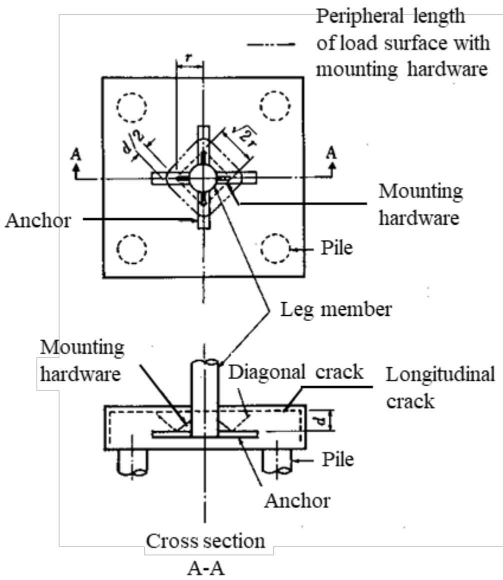
(i) When performing checking of pull-out shear in the same manner as punching shear, the peripheral length u of the load surface is to be calculated using Equation (Commentary 2.4.7) (see **Commentary Figure 2.4.13**).

$$u = 4 \cdot \sqrt{2} \cdot r \text{ : when mounting hardware is present}$$
$$= 2 \cdot \pi \cdot r \text{ : when mounting hardware is not present} \qquad \text{(Commentary 2.4.7)}$$

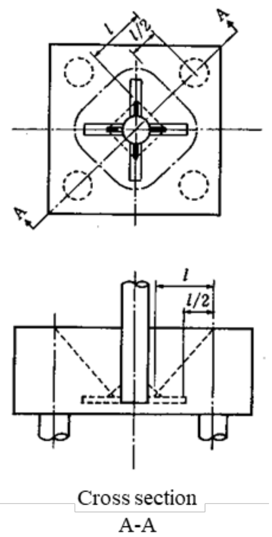
where

r: distance from the center of the leg member to the end of the mounting hardware if present, or the distance from the center of the leg material to the connecting part between the leg material and the anchor if there is no mounting hardware. The effective depth d is the distance from the upper surface of the anchor to the longitudinal rebars.

(ii) When the distance l from the end of the load surface to the center of the pile is no more than the effective depth d, the cross section to be checked is generally to be set to a position half-way between the end of the load surface and the center of the pile (see **Commentary Figure 2.4.14**).



Commentary Figure 2.4.13 Cross sections to be checked for pull-out shear



Commentary Figure 2.4.14 Cross sections to be checked when the pile pitch is small

(iii) When the effects of shear reinforcing bars have been recognized through experiments, *etc*., said effects may be evaluated to calculate pull-out shear capacity.

When a circular plate that is supported simply around the periphery, such as the bottom plate of a tank, is subjected to a uniformly distributed load or a linear load (see **Commentary Figure 2.4.15**), it is appropriate to perform the calculation following Equation (Commentary 2.4.8).

In checking of the cross section of a slab, when checking of shear force is performed based on the slab's assumed resistance behavior and considering the slab a rod member, shear strength will increase relative to that of a rod member of small width. Based on results of demonstrations in past experiments, *etc*., in a simply supported circular slab on which uniformly distributed load acts, when the ratio of the loaded diameter to the effective depth of the slab is 10 or less, the constraining effect caused by circumferential rebars may be converted to axial compressive force and V_{cdp} may be calculated using Equation (Commentary 2.4.8), which was modified from Equation (2.4.5).

$$V_{cdp} = \beta_d \cdot \beta_p \cdot \beta_\theta \cdot \beta_{a/d} \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b$$

(Commentary 2.4.8)

where

$\beta_\theta = 1 + 2M_0 / M_{ud}$;

$M_0 = P_n(d - h/3)$;

$M_u = p_r f_y d^2 \left(1 - \dfrac{p_r f_y}{1.7 f'_c}\right)$;

$P_n = \dfrac{f_y A_{s\theta}}{r} n$;

d: effective depth;

h: structural member height;

p_r: radial reinforcing bar ratio;

$A_{s\theta}$: circumferential rebar cross-sectional area (per rebar);

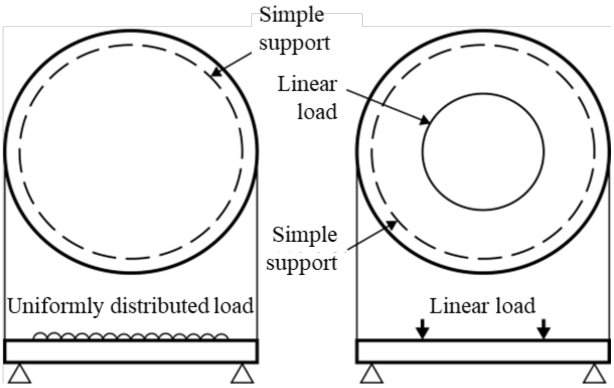
r: radius of position of cross section to be checked;

n: number of circumferential reinforcing bars that are arranged outside the cross section to be checked and that are subjected to tensile force;

$\beta_{a/d} = 0.75 + \dfrac{1.4}{a/d}$;

a/d : shear span ratio; and

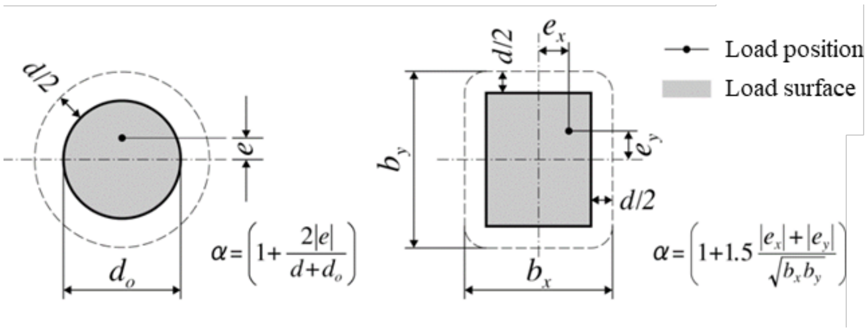
$\beta_\theta \cdot \beta_{a/d} \leq 1.5.$



Commentary Figure 2.4.15 Circular plates subjected to a uniformly distributed load or a linear load

Regarding (2): When a unidirectional slab is close to the free edge, punching shear capacity is small. In this case, it should be considered a rod member that has a valid width, and design shear capacity of the rod member should be calculated according to 2.4.3.2.

Regarding (3): When bending moment and torsional moment act simultaneously to add to vertical force as in corner columns supporting a slab, or when load acts eccentrically with respect to the load surface, Equation (2.4.11) cannot be used as is. One method for addressing this is to reduce punching shear capacity to 1/α times its value when only vertical force acts (see **Commentary Figure 2.4.16**).



Commentary Figure 2.4.16 Method to determine the capacity reduction factor *1/a* when load acts eccentrically

2.4.3.4 Design capacity of plane members subjected to in-plane force

(1) When a plane member in which rebars are arranged in two orthogonal directions is subjected to in-plane force, the tensile forces T_{xd} and T_{yd} in the directions of the rebars and the diagonal compressive force C'_d that acts on the concrete may be derived from Equations (2.4.13), (2.4.14), and (2.4.15) as design in-plane forces.

$$T_{xd} = N_1 \cdot \cos^2\alpha + N_2 \cdot \sin^2\alpha + (N_1 - N_2)\ \sin\alpha \cdot \cos\alpha \quad (2.4.13) \tag{2.4.13}$$

$$T_{yd} = N_1 \cdot \sin^2\alpha + N_2 \cdot \cos^2\alpha + (N_1 - N_2)\ \sin\alpha \cdot \cos\alpha \tag{2.4.14}$$

$$C'_d = 2\ (N_1 - N_2)\ \sin\alpha \cdot \cos\alpha \tag{2.4.15}$$

where, T_{xd}, T_{yd}: design tensile force per unit of structural member width acting on x-direction rebars and y-direction rebars

α: angle formed between principal in-plane force N_1 and x-direction rebars; $\alpha \leq 45°$

C'_d : design diagonal compressive force per unit of width acting on concrete

N_1, N_2: principal in-plane force; $N_1 \geq N_2$; N_1 is tension.

(2) When performing checking of the in-plane force of plane members with respect to the design force of member cross section derived in (1), the design yield strength T_{xyd} and T_{yyd} of the rebars and the design compressive failure capacity C'_{ud} of concrete may be derived from Equations (2.4.16), (2.4.17), and (2.4.18).

(i) Design yield strength of rebars

$$T_{xyd} = p_x \cdot f_{yd} \cdot b \cdot t/\gamma_b \tag{2.4.16}$$

$$T_{yyd} = p_y \cdot f_{yd} \cdot b \cdot t/\gamma_b \tag{2.4.17}$$

where, p_x and p_y: reinforcing bar ratio in the x direction and y direction (A_s/bt)

b: width of structural member; generally, unit width

t: structural member thickness

γ_b: may generally be set to 1.1.

(ii) Design compressive failure capacity of concrete

$$C'_{ud} = \cdot\ \mathrm{b} \cdot \mathrm{t}\ /\ \gamma\mathrm{b}\ f'_{ucd} \tag{2.4.18}$$

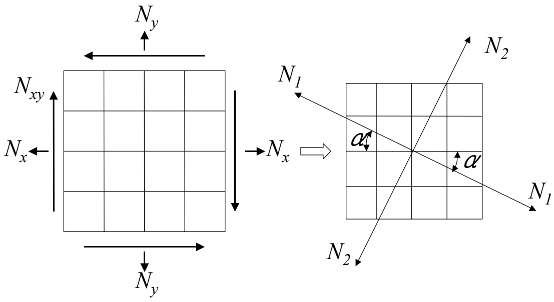
where,

$$f'_{ucd} = 2.8\sqrt{f'_{cd}}\ (\text{N/mm}^2)\ ;\ \text{however,}\quad f'_{ucd} \leq 17\ \text{N/mm}^2 \tag{2.4.19}$$

γ_b: may generally be set to 1.3.

Commentary: Regarding (1): When in-plane shear force and in-plane force in the direction of rebars act simultaneously in a reinforced concrete plate with an orthogonal rebar mesh, these can be expressed as principal in-plane forces N_1 and N_2 at angle α relative to the direction of rebars, as indicated in **Commentary Figure 2.4.17**.

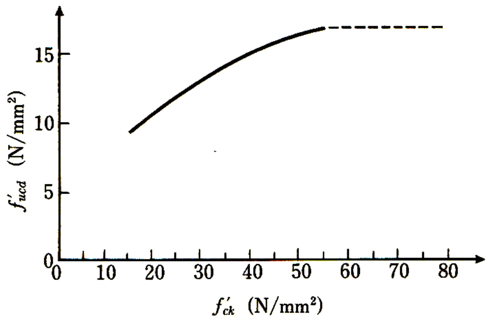
Equations (2.4.13) to (2.4.15) are derived such that compressive force acts in the direction diagonal to the orthogonal rebar mesh, cracking occurs at an angle of 45° from the direction of the rebars, the shear force occurring on the cracking surface is divided into a component parallel to the rebars and a component in a direction diagonal to the rebar mesh, and shear force is resisted by the axial tension component of the rebars and the compressive stress component of the concrete.



Commentary Figure 2.4.17 Biaxial tension of reinforced concrete plate with an orthogonal rebar mesh

Regarding (2): In cases in which in-plane force and in-plane shear force act in combination, these are resisted by the axial tensile force of the rebars and the compressive force of the concrete after cracking occurs. Assuming that in-plane force and in-plane shear force act equally in the direction of cross-sectional thickness, Equations (2.4.16) and (2.4.17) are presented as equations that express the yielding capacity of the rebars.

Equation (2.4.18) was set to provide a simple calculation of compression strength reduction effects on the conservative side. The relationship between f'_{ucd} in this Equation and the design specified strength of concrete is shown in **Commentary Figure 2.4.18**.



Commentary Figure 2.4.18 The relationship between f_{ucd} and specified design strength of concrete f'_{ck}

According to recent research findings, the reduction factor of compressive failure capacity with respect to in-plane force in reinforced concrete structural members in which cracking is significant can be derived from the expected maximum average tensile lateral strain ε_t (strain in the direction perpendicular to the direction of crack openings and the direction of compressive force) (see **Commentary Table 2.4.1**). Therefore, f'_{ucd} may be derived from Equation (Commentary 2.4.9).

$$f'_{ucd} = \lambda \cdot f'_{cd} \qquad \text{(Commentary 2.4.9)}$$

where,

λ: the reduction factor of compression strength given in **Commentary Table 2.4.1** as a function of lateral strain ε_t.

Commentary Table 2.4.1 Reduction factor λ as a function of lateral strain ε_t

Lateral strain ε_t^* ($\times 10^{-6}$)	Less than 2400	3600	More than 4800
Λ	0.8	0.7	0.6

* Average tensile strain in the direction perpendicular to the direction of crack openings

For conditions other than those indicated in **Commentary Table 2.4.1**, evaluation may be performed through interpolation.

The value for the reduction factor indicated here was obtained from experimental findings in which orthogonal compressive force and tensile force or pure shear were caused to act on specimens made with ordinary concrete with a compression strength of no more than 50 N/mm^2 and ordinary deformed bars, and with approximately 2% set as the upper limit for the reinforcing bar ratio in each direction of the rebars. The upper limit for tensile lateral strain is set to 5000×10^{-6}. When this is applied to cases in which loading is continued until tensile lateral strain occurs by which positive/negative cyclical action is even greater, or cases in which the repeating effects are large, the derived reduction factor should be reduced by 0.2.

2.4.3.5 Design shear transfer capacity

(1) When rebars are arranged in a shear plane, the design shear transfer capacity V_{cwd} when axial force acts on the shear plane may be derived using Equation (2.4.20). Cases in which rebars that do not conform to JIS standards are used must be considered separately.

$$V_{cwd}=\{(\tau_c+p\cdot\tau_s\cdot\sin^2\theta-\alpha\cdot p\cdot f_{yd}\cdot\sin\theta\cdot\cos\theta)\ A_c+V_k\}/\gamma_b \tag{2.4.20}$$

where, $\tau_c=\mu\cdot f'^{\,b}_{cd}\ (\alpha\cdot pf_{yd}-\sigma_{nd})^{1-b}$

$\tau_s=0.08\,f_{yd}/\alpha$

$\alpha=0.75\{1-10\,(p-1.7\sigma_{nd}/f_{yd})\}$

$0.08\sqrt{3}\leq\alpha\leq0.75$ (for reinforcing bars)

$f_{yd}\leq490\text{N/mm}^2$

σ_{nd}: average stress intensity that acts perpendicularly to the shear plane; in the case of compression, $\sigma_{nd}=-\sigma'_{nd}$ /2.

In either case, $(\alpha\cdot p\cdot f_{yd}-\sigma_{nd})$ must be positive.

σ'_{nd}: average compressive stress intensity that acts perpendicularly to the shear plane

p: reinforcing bars ratio at the shear plane; only rebars that have sufficient development length on both sides from the shear plane are to be considered

A_c: area of the shear plane

θ: angle formed between the shear plane and rebars

b: coefficient (0 to 1) representing the plane form, with the following values set as standards:

2/3 = cracked surface (ordinary-strength concrete)

1/2 = joints in the case of cracking in construction joint surfaces (treated) or in high-strength concrete, or in the case of adhesive used in the joints of precast structural members

μ: average friction coefficient for solid contact; may be set to 0.45

V_k: shear capacity attributable to shear key

$V_k=0.1A_k\cdot f'_{cd}$ $\quad A_k$: cross-sectional area of shear surface of the shear key

γ_b: may generally be set to 1.3.

(2) Design shear transfer capacity V_{cwd} when bending moment and axial force act on the shear plane may be obtained from Equation (2.4.21) by deriving the neutral axis when bending moment and axial force have acted, by dividing the

neutral axis into a tension side and a compression side, and, following steps (i) to (iii) and for the tension side and compression side, by using $V_{cwd,t}$ and $V_{cwd,c}$ that were derived using Equation (2.4.20). However, when the characteristic value of the yield strength of rebars exceeds 490 N/mm^2, the upper limit should be set to 490 N/mm^2.

$$V_{cwd}=\beta_M \cdot V_{cwd,t}+V_{cwd,c} \tag{2.4.21}$$

where, $V_{cwd,t}$: shear transfer capacity borne on the tension side of the shear plane

$V_{cwd,c}$: shear transfer capacity borne on the compression side of the shear plane

β_M: reduction factor with the effects of bending moment taken into account, with the following set as standards:

$\beta_M = 4\ (1 - M_d / M_y)$

$\beta_M\ \leq 1$: for joint surfaces and cracked surfaces

$= 0$: for the juncture surfaces of precast structural members

M_d: design bending moment that acts on the shear plane

M_y: action moment when the outermost rebars on the tension side yield

(i) Calculation of the axial force borne by rebars and concrete when bending moment and axial force act

P_{st}, P'_{sc} , and P'_c when design axial compressive force N'_d and design bending moment M_d act on the shear plane are derived based on the assumptions of (i) to (iv) in 2.4.2.1 (2). Equations (2.4.22) and (2.4.23) may be assumed for the stress-strain relationships of concrete and rebars, respectively.

Stress-strain relationship in concrete: $\sigma'_c = E_c\ \ \varepsilon'_c\ \ (\varepsilon'_c \geq 0)$ (2.4.22)

Stress-strain relationship in rebars: $\sigma = E_s \cdot \varepsilon$ (2.4.23)

where, N'_d : design axial compressive force that acts on the shear plane

P_{st}: sum of rebar axial tensile forces borne by rebars on the tension side

P'_{sc} : sum of rebar axial compressive forces borne by rebars on compression side

P'_c : axial compression force borne by concrete on the compression side

(ii) Calculation of yield moment M_y

Using the same method as (i), the acting moment M_y is to be calculated so that stress intensity generated in the outermost rebars on the tension side is f_{yd}, which is the design tensile yield strength of the outermost rebars.

(iii) Calculation of shear transfer capacity $V_{cwd,t}$ and $V_{cwd,c}$

a) $V_{cwd,t}$ is derived, following Equation (2.4.20) and using $\sigma_{nd}=P_{st}/A_{ct}$. Shear capacity V_k attributable to the shear key on the tension side is to be neglected.

b) $V_{cwd,c}$ is derived, following Equation (2.4.20) and using $\sigma_{nd}=-(1/2)(P'_{sc}+P'_c)/A_{cc}$.

where, A_{ct}: cross-sectional area of the shear plane on the tension side

A_{cc}: cross-sectional area of the shear plane on the compression side

(3) When the entire surface of the shear plane is compressive, the design shear transfer capacity V_{cwd} may be derived following Equation (2.4.20), using $\sigma'_{nd} = N'_d / A_c$ and ignoring the effects of bending moment.

Commentary: Regarding (1):

If the reinforcing bar ratio is 1% or less, full cross-sectional yielding in the axial direction can be assumed for the constraining force caused by rebars. The reduction factor α in Equation (2.4.20) expresses the increase in shear capacity and the decrease in axial capacity in the shear reinforcing bars; the shear transfer capacity equation was derived by adding the shear direction components of the concrete and rebars.

The coefficient b in Equation (2.4.20) represents the effect of roughness. Roughness declines in the cracking surface of high-strength concrete because of splitting of the coarse aggregate. Therefore, the coefficient b must be made small. In cases in which construction joint surface treatment is omitted, the coefficient b may be set to 2/5. The decision was made to use $\sigma_{nd} = -\sigma'_{nd}/2$ when σ_{nd} is compression to be on the conservative side. When σ_{nd} is tension, it is to be used as is.

Equation (2.4.20) is applied when rebars are arranged in a shear plane. When no rebars are arranged, $p = 0$ may be used. Rebars that have sufficient development length are those rebars that are anchored on both sides from the shear plane for a length of no less than 10 times the diameter of the rebars. When using rebars that are not specified in JIS standards, the upper limit of the characteristic value of yield strength f_{yd} is to be 490 N/mm^2.

Adhesive, mortar, *etc*. are used in the connecting masonry joints of precast structural members. The case indicated here is that of adhesives for which many experiments and examples of usage exist. Cases in which mortar, *etc*. are used must be considered separately. In the case of precast concrete, because shear keys are often used, shear capacity is given as the sum of frictional force caused by axial force and the capacity of the shear key.

<u>Regarding (2)</u>: When bending moment acts on the shear plane, the transmissible shear stress within the shear plane varies with crack width and other factors. Therefore, Equation (2.4.20), which assumes uniform shear stress, cannot be applied as is. Equation (2.4.21) is to be adopted as an equation for calculating shear transfer capacity when bending moment and axial force act.

The coefficient β_M, which considers the effects of bending moment, is introduced to evaluate shear transfer capacity on the tension side, in line with the degree of unevenness of the shear plane and acting bending moment. When adhesive is used on juncture surfaces of precast structural members, a certain degree of unevenness is thought to be formed. Therefore, when β_M can be set with consideration of this effect, the shear transfer capacity on the tension side may be considered. When using rebars not specified in JIS standards, the upper limit of the characteristic value of yield strength f_{yd} is to be 490 N/mm^2 in the calculation of yield moment M_y.

In the calculation of axial force borne by rebars and concrete when bending moment and axial force act, a linear relationship was used for the stress-strain relationships of concrete and rebars. The reason for this is that, in the evaluation of β_M, when a large bending moment acts by which the outermost rebars on the tension side yield, the shear transfer capacity on the tension side was ignored out of consideration of safety.

In the case of full cross-section tension, the neutral axis is outside the shear plane. Therefore, evaluation should be performed for only the shear transfer capacity on the tension side.

<u>Regarding (3)</u>: It was confirmed from the strict analytical results indicated in (2) that when the entire cross section is compressive, bending moment has relatively little effect on shear transfer capacity. Therefore, it was decided that Equation (2.4.20) can be applied when the entire cross section is compressive.

2.4.4.1 General

(1) Checking of safety with respect to torsion must follow this section. However, when performing checking of the limit state of displacement or deformation with consideration of the effects of change in stiffness caused by torsion, it is not necessary to follow this section. When using reinforcing bars for which the characteristic value of yield strength exceeds 490 N/mm^2, the upper limit is to be set to 490 N/mm^2.

(2) In the case of structural members on which torsional moment has a small effect and in the case of deformation compatibility torsional moment, checking of safety with respect to torsion in 2.4.4 may be omitted in entirety.

A structural member on which torsional moment has a small effect is defined as a structural member for which the ratio of design torsional moment M_{td} to design pure torsional capacity M_{tcd} according to 2.4.4.2 when torsional reinforcing bars are not present, multiplied by the structure factor γ_i, yields a value of less than 0.2 in all cross sections.

(3) When design torsional moment M_{td} and design torsional capacity M_{tud} (derived from 2.4.4.2, when torsional reinforcing bars are not present) satisfy Equation (2.4.24) in all cross sections, checking according to 2.4.4.3 may be omitted. In this case, minimum torsional reinforcing bars must be arranged following 2.3.3 in "Design: Standards" Volume 7.

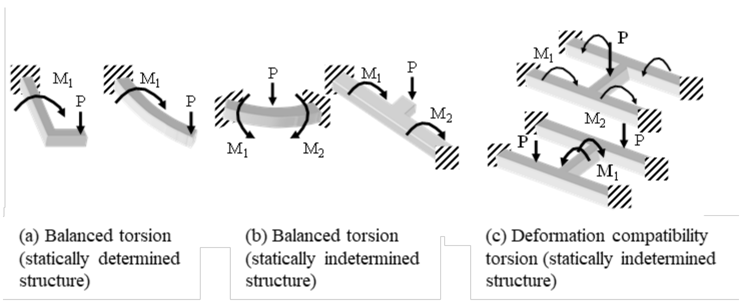
$$\gamma_i \, M_{td} / M_{tud} \leq 0.5 \qquad (2.4.24)$$

(4) When the design torsional moment M_{td} does not satisfy Equation (2.4.24), torsional reinforcing bars must be arranged following 2.4.4.3.

(5) When torsional moment and bending moment act simultaneously or torsional moment and shear force act simultaneously, the effects of each interaction must be taken into account in the checking of safety.

Commentary: Regarding (1): When using rebars that are not specified in JIS standards, checking is to be carried out with the upper limit of the characteristic value of yield strength f_{yd} set to 490 N/mm^2.

Regarding (2): When organizing torsional moments from a standpoint of structural design, the moments can be classified into balanced torsion and deformation compatibility torsion (see **Commentary Figure 2.4.19**).



Commentary Figure 2.4.19 Balanced torsion and deformation compatibility torsion

In the ultimate limit state, deformation compatibility torsion was considered negligible in the equilibrium calculation of force, and it was made clear that checking of safety with respect to failure of member cross section under action by torsional moment is to be performed only in the case of balanced torsional moment. In the case of deformation compatibility torsion, the torsional stiffness of structural member cross sections must be assumed to be 0.

In actual design, checking of torsion following 2.4.4 is complicated when only a slight balanced torsional moment acts. The range $\gamma_i M_{td} / M_{tcd} < 0.2$ is specified as a range that does not impair the safety of the structural object and defines cases in which torsional moment has a small effect. When this condition is satisfied, checking of safety with respect to torsional moment may be omitted.

Regarding (3): Even when verifying safety with respect to torsional moment, if the design torsional moment is no more than the design torsional capacity that can be resisted by concrete alone, arranging torsional reinforcing bars is generally not necessary. However, because cracking may be caused by factors including constraint of structural member ends, concentration of stress, drying shrinkage of concrete, and temperature differences, an appropriate minimum amount of rebars must be arranged to prevent sudden failure.

The minimum amount of torsion reinforcing bars was set to ensure 1/2 of the torsional capacity when torsional reinforcing bars are not present.

2.4.4.2 Design torsional capacity when torsional reinforcing bars are not present

(1) The design torsional capacity M_{tud} when only rod members not having torsional reinforcing bars are subjected to torsional moment may be derived using Equation (2.4.25).

$$M_{tud} = M_{tcd} \tag{2.4.25}$$

where, M_{tcd}: design pure torsional capacity

$$M_{tcd} = \beta_{nt} \cdot K_t \cdot f_{td} / \gamma_b \tag{2.4.26}$$

K_t: torsion coefficient indicated in **Table 2.4.1**

β_{nt}: coefficient for prestress force or other axial compressive force

$$\beta_{nt} = \sqrt{1 + \sigma'_{nd} / (1.5 f_{td})}$$

f_{td}: design tensile strength of concrete

σ'_{nd}: acting average compressive stress intensity caused by axial force; must not exceed $7 f_{td}$

γ_b: may generally be set to 1.3.

(2) The design torsional capacity M_{tud} when bending moment M_d and torsional moment M_{td} act simultaneously may be derived from Equation (2.4.27).

$$M_{tud} = M_{tcd} \cdot \left(0.2 + 0.8\sqrt{1 - \gamma_i \cdot M_d / M_{ud}}\right) \tag{2.4.27}$$

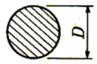
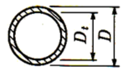
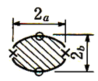
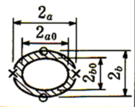
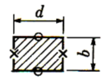
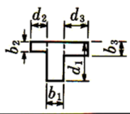
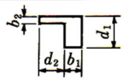
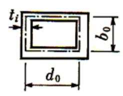
where, M_{ud}: design bending capacity derived from 2.4.2.1

(3) The design torsional capacity M_{tud} when shear force V_d and torsional moment M_{td} act simultaneously may be derived using Equation (2.4.28).

$$M_{tud} = M_{tcd} \cdot \left(1 - 0.8\gamma_i \cdot V_d / V_{yd}\right) \tag{2.4.28}$$

where, V_{yd}: design shear capacity derived from Equation (2.4.4)

Table 2.4.1 Coefficients related to torsion1

Cross section	K_t	Remarks
	$\frac{\pi D^3}{16}$	
	$\frac{\pi(D^4-D_i^4)}{16D}$	
	○ $\pi ab^2/2$ × $\pi a^2b/2$	
	○ $\pi ab^2(1-q^4)/2$ × $\pi a^2b(1-q^4)/2$	$q=a_0/2$ $=b_0/2$
	○ b^2d/η_1 × $b^2d/(\eta_1\eta_2)$	$\eta_1=3.1+\frac{1.8}{d/b}$ $\eta_2=0.7+\frac{0.3}{d/b}$
 	$\Sigma\frac{b_i^2d_i}{\eta_{1i}}$ b_i and d_i are the lengths of the short and long sides of the divided rectangular cross section, respectively	The division into rectangles should be such that the torsional stiffness is large
	$2A_m t_i$ In principle, K_t for a box-shaped cross section is obtained as a hollow cross section. However, if the ratio of the thickness of the member to the total width of the box-shaped section in the thickness direction exceeds 0.15, it is better to consider the section as a solid section and calculate K_t	A_m is the area enclosed by the wall thickness center t_i is the web thickness

Commentary: Regarding (1): Before cracking occurs, an elastic theory equation is to be applied to the concrete cross section with effects of rebars ignored and with the coefficients shown in **Table 2.4.1** used. Box-shaped cross section K_t is in principle derived as a hollow cross section. Specifications for the box-shaped cross section in **Table 2.4.1** were determined through calculation that uses the ratio of the structural member thickness to the total width of the box-shaped cross section in the direction of the thickness as a parameter.

The coefficient β_{nt} for prestress force or other axial compressive force in Equation (2.4.26) corresponds to β_n in Equation (Commentary 2.4.1) for calculating design shear capacity. This coefficient is based on the maximum principal tensile stress theory, which states that failure occurs when principal tensile stress caused by the combination of axial stress intensity and torsional shear stress intensity reaches the tensile strength of the concrete.

In the formula for β_{nt}, 1.5 f_{td} is an approximation of the average value for tensile strength of concrete. Axial compressive stress intensity σ'_{nd} is the average value in the cross section.

When not subjected to uniform axial compressive stress but instead to axial compressive stress that has a triangular distribution, as in prestress, experimental results reveal that the elastic theory value is considerably on the conservative side. Because of the effects of flexing, the ultimate axial stress distribution differs from that caused by axial force alone. Given these factors, average stress intensity may be used to calculate β_{nt}.

When σ'_{nd} exceeds 7 f_{td}, the structural member enters the region in which compression failure occurs not because of the maximum principal tensile stress but rather because of a combination of axial compressive stress and compressive stress caused by torsional moment. Because this represents an extremely dangerous case, a limit was set.

For structural members that are subjected to axial tensile force, when axial tensile force is small and does not cause cracking ($\sigma_{td} < f_{td}$), β_{nt} derived from Equation (Commentary 2.4.10) may be used. When cracking occurs only because of axial tensile force ($\sigma_{td} \geq f_{td}$), $\beta_{nt} = 0$ is to be used.

$$\beta_{nt} = \sqrt{1-\sigma_{td} / f_{td}} \quad (0 \leq \beta_{nt} \leq 1) \qquad \text{(Commentary 2.4.10)}$$

where

σ_{td}: acting average tensile stress intensity caused by axial tensile force.

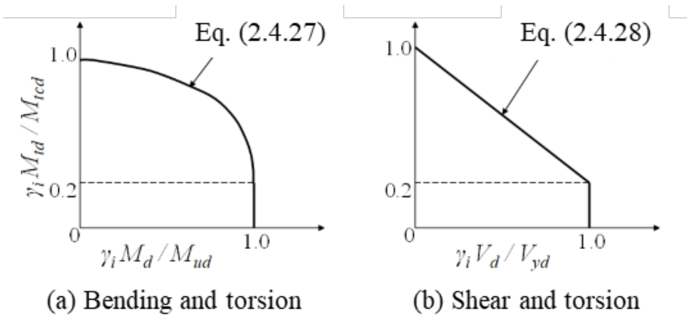
When performing design such that the entire structural member is subjected to axial tensile force, consideration of the combined actions of torsional moment and axial tensile force results in decreased torsional capacity of the structural member, and therefore requires a large amount of torsional reinforcing bars. In such cases, it is advisable to introduce prestressing or otherwise take measures to avoid design that results in structural members being subjected to the combined action of torsional moment and axial tensile force.

Regarding (2): Capacity in cases in which torsional moment and bending moment act in combination was based on the correlation derived from the maximum principal stress theory.

When torsional reinforcing bars are not present, the absence of cracking is a foundation of design within the range in which torsional moment dominates. However, for parts that become the tension side because of flexing, Equation (2.4.27), which assumes that cracking will occur, was indicated here.

Regarding (3): For capacity when torsional moment and shear force act in combination, the absence of cracking within the range in which torsional moment dominates when torsion reinforcing bars are not present was made a foundation of design. Within the range in which shear force dominates, Equation (2.4.28), which assumes that shear cracking will occur, was indicated.

For cases in which torsional moment and bending moment act simultaneously or torsional moment and shear force act simultaneously, the respective interaction relationships are as shown in **Commentary Figure 2.4.20 (a)** and **(b)** when design force of member cross section, design capacity of member cross section, and the structure factor are used. Therefore, checking of safety is to be performed by confirming that the value obtained through a given combination of force of member cross sections is inside the curve, *i.e.*, is on the origin side. The reason the correlation of these is shifted up to $\gamma_i M_{td} / M_{tcd} = 0.2$ is that checking of torsion can be completely omitted when $\gamma_i M_{td} / M_{tcd} < 0.2$.



Commentary Figure 2.4.20 Correlation diagrams

2.4.4.3 Design torsional capacity when torsional reinforcing bars are present

(1) Design diagonal compressive failure capacity M_{tcud} with respect to torsion in web concrete may be derived using Equation (2.4.29).

$$M_{tcud} = K_t \cdot f_{wcd} / \gamma_b \tag{2.4.29}$$

where, $f_{wcd} = 1.25\sqrt{f'_{cd}}$ (N/mm^2) $f_{wcd} \leq 9.8$ N/mm^2 (2.4.30)

K_t: torsion coefficient indicated in **Table 2.4.1**

γ_b: may generally be set to 1.3.

(2) Design torsional capacity M_{tyd} in rectangular, circular, and toric cross sections may be derived using Equation (2.4.31).

$$M_{tyd} = 2A_m \sqrt{q_w \cdot q_l} / \gamma_b \tag{2.4.31}$$

where, A_m: torsional effective cross-sectional area (rectangular cross section, $b_0\ d_0$; circular and toric cross section, $\pi d_0^2 / 4$)

b_0: length of short leg of transverse rebar

d_0: length of long leg of transverse rebars in the case of a rectangular cross section; diameter of the concrete cross section enclosed by transverse rebars in the case of circular and toric cross sections

$q_w = A_{tw} \cdot f_{wd} / s$

$q_l = \Sigma A_{tl} \cdot f_{ld} / u$

ΣA_{tl} : cross-sectional area of longitudinal rebars that effectively act as torsional reinforcing bars

A_{tw}: cross-sectional area of single transverse rebar that effectively acts as torsional reinforcing bar

f_{ld}, f_{wd}: design yield strength of transverse rebars and longitudinal rebars

s: axial spacing of transverse rebars that effectively act as torsional reinforcing bars

u: length of central line of transverse rebars (rectangular cross section, $2(b_0 + d_0)$; circular and toric cross section, πd_0)

γ_b: may generally be set to 1.3.

When $q_w \geq 1.25q_l$, $q_w = 1.25q_l$; when $q_l \geq 1.25q_w$, $q_l = 1.25q_w$.

(3) The design torsional capacity of T-, L- and I-shaped cross sections may be set to the sum of the values of M_{tydi}, derived by dividing the cross section into rectangles and, following (i) to (iv) below, using Equation (2.4.31) for each. Each value of M_{tydi} must not exceed the value of $\xi \cdot A_{mi}$.

where, A_{mi}: torsional effective cross-sectional area of the divided rectangles

ξ: value of M_{tydi} / A_{mi} in divided rectangles having maximum torsional effective cross-sectional area

(i) A_{mi} may be set to the area enclosed by transverse rebars as shown in **Figure 2.4.5**.

(ii) Longitudinal rebars for torsion must not be counted twice in each divided rectangle.

(iii) In a T-shaped cross section in which the flange is continuous, transverse rebars in the flange may be deemed valid even if not enclosing the longitudinal rebars. When the upper and lower amounts of rebar in the flange differ, the smaller amount of rebar is to be used as the limit.

(iv) The one-side effective width λ_t of the flange with respect to torsional moment may be derived using Equation (2.4.32).

λ_t=3 t_i (2.4.32)

Cantilever arm $\lambda_t \leq l_c$

Middle segment $\lambda_t \leq l_b/2$

where, t_i: average thickness of flange

l_c, l_b: overhang length of cantilever plate and clear distance of girders, respectively

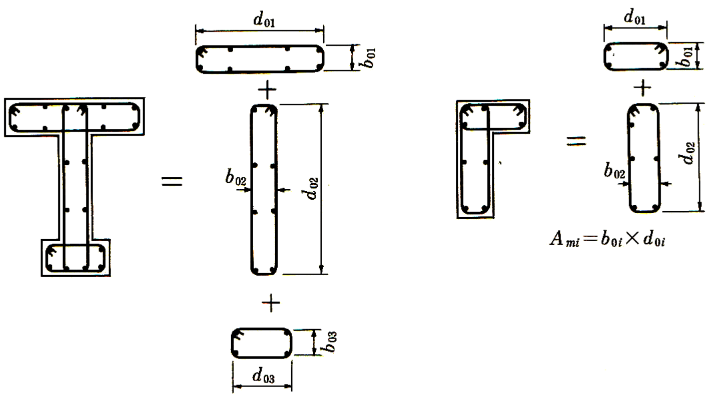


Figure 2.4.5 Calculation method of A_{mi} for T-shaped and L-shaped cross section

(4) Box-shaped cross sections must be designed as solid cross sections when the minimum value of the ratio of wall thickness to total width of the box-shaped cross section in the direction of thickness is 1/4 or higher.

However, if the minimum value of the ratio of wall thickness to total overall width of the box-shaped cross section in the direction of thickness is less than 1/4, (7) is to be followed.

(5) In the case of rectangular, circular, and torus cross sections, checking of safety when bending moment M_d and torsional moment M_{td} act simultaneously is to be performed by confirming that Equations (2.4.33) to (2.4.35) are satisfied.

When $M_{ud} \geq M'_{ud}$ and $\gamma_i \mid M_d \mid \leq M_{ud} - M'_{ud}$

$$\gamma_i M_{td} / M_{tu\ \min} \leq 1.0 \tag{2.4.33}$$

When $M_{ud} \geq M'_{ud}$ and $M_{ud} - M'_{ud} \leq \gamma_i \mid M_d \mid \leq M_{ud}$

$$\gamma_i \left[\left(\frac{1.3(M_{td} - 0.2M_{tcd})}{M_{tu\min} - 0.2M_{tcd}} \right)^2 + \frac{|M_d| - M_{ud} + M'_{ud}}{M'_{ud}} \right] \leq 1.0 \tag{2.4.34}$$

When $M_{ud} < M'_{ud}$ and $\gamma_i \mid M_d \mid \leq M_{ud}$

$$\gamma_i \left\{ \left(\frac{1.15(M_{td} - 0.2M_{tcd})}{M_{tu\min} - 0.2M_{tcd}} \right)^2 + \frac{|M_d|}{M_{ud}} \right\} \leq 1.0 \tag{2.4.35}$$

where, $M_{tu\ \min}$: the smaller of M_{tucd} and M_{tyd}

M_d: design bending moment

M_{ud}: absolute value of design bending capacity when the primary rebars arranged on the tension side under the action of M_d are considered tensile rebars

M'_{ud}: absolute value of design bending capacity when the primary rebars arranged on the compression side under the action of M_d are considered tensile rebars

(6) In the case of rectangular, circular, and torus cross sections, checking of safety when shear force V_d and torsional moment M_{td} act simultaneously may be performed by confirming that Equation (2.4.36) is satisfied.

$$\gamma_i\,[M_{td}/M_{tu\ \min} + (1-0.2M_{tcd}/M_{tu\ \min})(V_d/V_{yd})\,] \leq 1.0 \tag{2.4.36}$$

where, $M_{tu\,\min}$: the smaller of M_{tcud} and M_{tyd}

V_{yd}: design shear capacity derived from Equation (2.4.4)

(7) In a box-shaped cross section, when the minimum value of the ratio of wall thickness to total width of the box-shaped cross section in the direction of thickness is less than 1/4, the design torsional strength M_{tyd} may be derived from Equation (2.4.37).

$$M_{tyd} = 2A_m\,(V_{odi})_{\min} \tag{2.4.37}$$

where, $(V_{odi})_{\min}$: minimum value of in-plane shear capacity per unit length of each wall

In this case, the anchoring method of rebars and connecting parts of each wall must follow 2.5 in "Design: Standards" Volume 7.

When bending moment or shear force acts simultaneously with torsional moment, checking of safety may be performed in the same manner as a rectangular cross section.

Commentary: Regarding (1): It was decided to avoid cases in which torsional reinforcing bars are arranged in such amounts that the bars will not yield in either the longitudinal or transverse directions. Failure capacity in this case is provided in Equation (2.4.29). This equation is an approximation proposed in place of restricting the amount of torsional reinforcing bars to less than the amount of balanced rebars.

Regarding (2): As a method for analytical modeling of load-carrying mechanisms in this case, a design method based on three-dimensional truss theory was specified for its ability to easily calculate torsional capacity caused by the yielding of torsional reinforcing bars.

Equation (2.4.31) gives the design torsional capacity induced by the balance of forces, assuming that reinforcing bars yield in both directions when composed of rebars in the longitudinal direction of the structural member and transverse rebars perpendicular to those. When the amount of rebars in one direction dominates, 1.25 times the lesser amount is to be deemed the rebar amount for torsional reinforcing bars. Equation (2.4.31) can also be considered an equation for calculating torsional capacity by replacing the solid cross section with a simulated thin box-shaped cross section and considering the ultimate shear flow as $q = \sqrt{q_w \cdot q_t}$. In this case, the area enclosed by the central line of the thin partition wall (the central line of the shear flow) is the torsional effective cross-sectional area. Here, this is assumed to be the area enclosed by the central lines of the transverse reinforcing bars. Because torsional capacity derived using Equation (2.4.31) may be slightly large, the decision was made to use 1.3 as γ_b. The difference in effective cross-sectional area according to the respective methods is large for small cross sections and small for large cross sections. Therefore, when the accuracy of Equation (2.4.31) can be sufficiently confirmed, γ_b may be reduced to approximately 1.15.

Regarding (3): Based on the same treatment of cases in which torsional reinforcing bars are not present, design torsional capacity is assumed to be the sum of the torsional capacities of the divided rectangles. However, the method for deriving A_{mi}, used in Equation (2.4.31), was determined with consideration of the actual form of arrangement of the transverse rebars.

Regarding (4): This stipulation was set as a simple method for handling box-shaped cross sections.

Regarding (5): The correlation among capacity of member cross sections of a structural member that is subjected to simultaneous bending moment and torsional moment is affected by the arrangement of longitudinal rebars.

A number of correlation equations for capacity have been proposed. The following two equations, which are able to express the effects of rebar arrangement in a simple form, are presented here with modification.

$$\frac{F_{ty}}{F_{by}}\left(\frac{M_{td}}{M_{tyd}}\right)^2+\frac{M_d}{M_{ud}}=1 \qquad \text{(Commentary 2.4.11)}$$

$$\left(\frac{M_{td}}{M_{tyd}}\right)^2-\frac{F_{by}}{F_{ty}}\left(\frac{M_d}{M_{ud}}\right)=1 \qquad \text{(Commentary 2.4.12)}$$

where

F_{ty}, F_{by}: tensile force at the time when the upper and lower longitudinal rebars yield, respectively.

Equations (2.4.33), (2.4.34) and (2.4.35) were derived by using M_{ud} and M'_{ud} in place of F_{by} and F_{ty}, respectively, by ignoring the increase in torsional capacity caused by the effects of flexing from a safety perspective, and by considering consistency with the torsion negligible threshold ($\gamma_i M_{td} < 0.2M_{tcd}$). When the structure factor γ_i is set to 1.15 and when M_d / M_{ud} is small and the effects of torsion dominate, this yields greater safety than conventional equations. When M_d / M_{ud} is large and the effects of flexing become dominant, the degree of safety with respect to torsion decreases more than under conventional equations. When γ_i is smaller than 1.15, the degree of safety gradually increases even within the region in which the effects of flexing dominate.

These equations are relational expressions for cases in which rebars yield, but they can also be applied to cases in which concrete undergoes compression failure before the rebar yields.

Regarding (6): Equation (2.4.36) expresses the correlation between torsion and shear as a linear relationship and takes into account consistency with the torsion negligible threshold.

Regarding (7): The thinner the walls of the box-shaped cross section, the more that torsional deformation and capacity will differ from those of a solid cross section. Therefore, the in-plane shear capacity of each wall is derived, and from this, torsional capacity of the box-shaped cross section is derived. When in-plane shear capacity is derived using 2.4.3.4, the in-plane shear capacity V_{od} is set to the smaller of T_{xyd} or T_{yyd}.

When calculating T_{xyd} and T_{yyd}, the value γ_b should be set to 1.3. If the accuracy of Equation (2.4.37) can be sufficiently confirmed by means such as experiments involving torsional loading on a large box-shaped cross section, γ_b may be reduced to approximately 1.15.

In cases in which shear force and torsional moment act simultaneously on a box-shaped cross section, and in cases in which bending moment and torsional moment act simultaneously, checking of safety may be performed in the same manner as a rectangular cross section.

Chapter 3 Checking of Fatigue Failure

3.1 General

(1) In principle, checking of safety is to be conducted by confirming that no structural members reach the fatigue failure threshold under the design actions.

(2) In principle, checking of the limit states of fatigue failure using stress intensity or force of member cross section should follow this volume.

(3) Checking of the limit states of fatigue failure are to follow (i) to (iii) below.

(i) Checking of safety with respect to fatigue is in principle performed by confirming that the ratio of the design variable stress intensity σ_{rd} to the value obtained by dividing design fatigue strength f_{rd} by member factor γ_b, multiplied by the structure factor γ_i, is no greater than 1.0.

$$\gamma_i \sigma_{rd}/(f_{rd}/\gamma_b) \leq 1.0 \quad (3.1.2)11$$

Here, design fatigue strength f_{rd} is the value obtained by dividing the characteristic value f_{rk} of fatigue strength of the material by the material factor γ_m.

(ii) Checking of safety with respect to fatigue may be performed by confirming that the ratio of the design variable force of member cross section S_{rd} to the design fatigue capacity R_{rd}, multiplied by the structure factor γ_i, is no greater than 1.0.

$$\gamma_i S_{rd}/R_{rd} \leq 1.0 \quad (3.1.3)12$$

Here, the design variable force of member cross section S_{rd} is the value of variable force of member cross section S_r (F_{rd}) derived using the design variable action F_{rd}, multiplied by the structural analysis factor γ_a. The design fatigue capacity R_{rd} is the fatigue capacity R_r (f_{rd}) derived using the design fatigue strength f_{rd} of the material, divided by the member factor γ_b.

(iii) The member factor γ_b used in the checking of safety with respect to fatigue may generally be set between 1.0 and 1.3.

(4) Checking of safety in beams is generally to be performed with respect to flexing and shear.

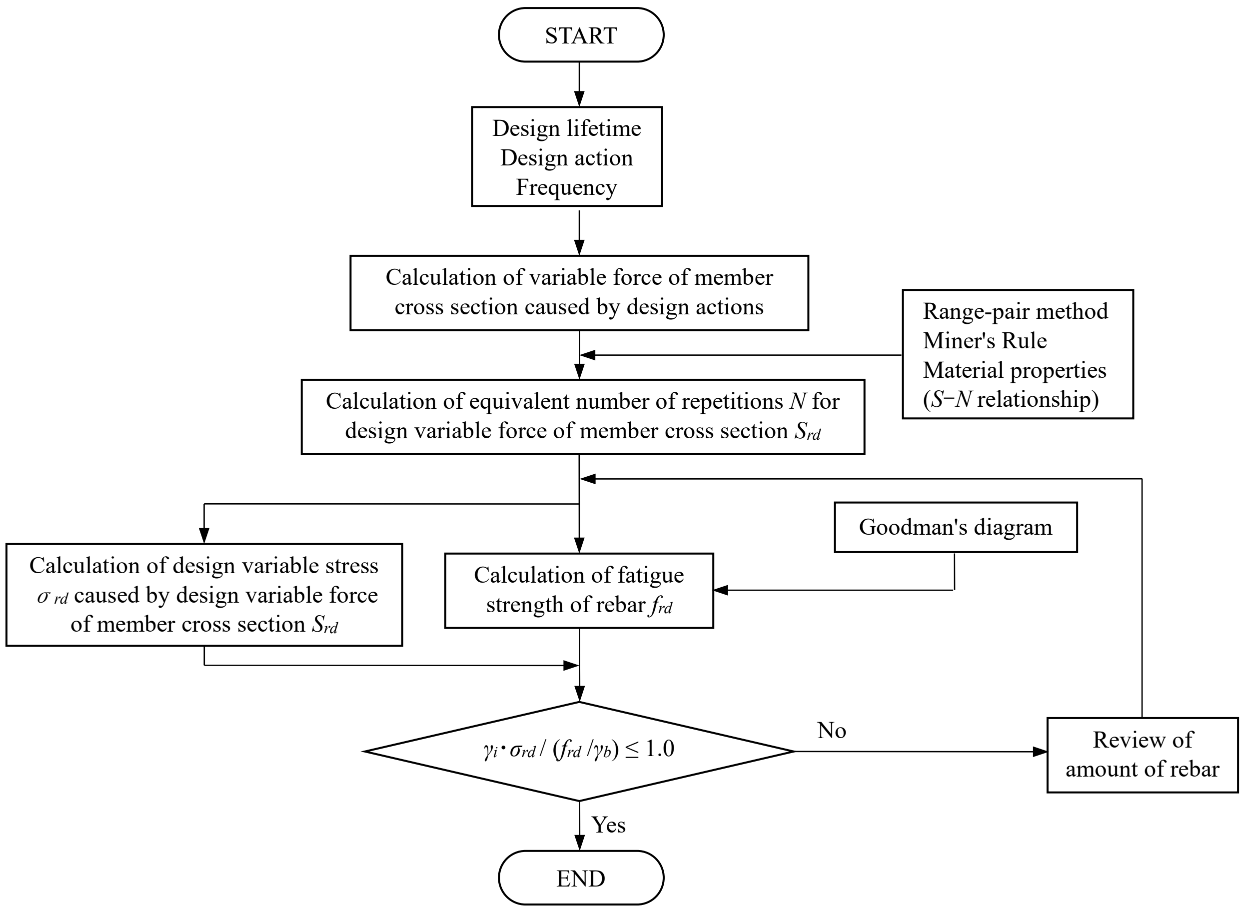
(5) Checking of safety in slabs is generally to be performed with respect to flexing and punching shear.

(6) In general, checking of performance may be omitted for columns. However, when the effects of bending moment or axial tensile force are particularly large, checking should be performed in the same manner as beams.

Commentary: Regarding (1): This chapter presents standard methods for verifying the safety of structural objects without consideration of the deterioration of materials during the design lifetime, premised on the satisfaction of placing performance in accordance with "Design: Standards" Volume 2, "Design: Standards" Volume 6, and "Construction Work" in the Standard Specifications. All specifications used in checking in this chapter, including material strength specifications, follow Chapter 5 of "Design: Main Volume" and this chapter.

Regarding (3): The fatigue capacity of a structural member cross section is determined by the fatigue strength of the reinforcing bars or concrete that composes the structural member. Checking of safety with respect to fatigue, based on the fatigue strength of the material, is also stipulated. Of the performance requirements with respect to fatigue failure, this chapter presents standards for safety checking related to fatigue failure of materials. In general, safety with respect to fatigue loads and environmental conditions during design lifetime should be checked. When checking of usability with respect to fatigue is necessary, a method that has been separately demonstrated through experiments or other means should be used.

Irregular variable force of member cross sections caused by variable actions may be broken down into sets using the range-pair method or other method, then replaced by N repetitions of design variable cross-section force S_{rd} (generally, the maximum variable force of member cross section) through the application of Miner's Rule, and used in the checking of fatigue failure. The procedures are shown in **Commentary Figure 3.1.1**, using fatigue failure in reinforcing bars as an example.



Commentary Figure 3.1.1 Procedure for checking the fatigue failure of rebars caused by variable actions

Regarding (4) and (5): Checking of the limit states of failure of member cross section with respect to fatigue should generally be performed for fatigue failure in shear reinforcing bars and primary rebars that are subjected to repeated tensile stress. In special cases, however, checking must also be performed for fatigue in concrete.

Such special cases involve lightweight aggregate concrete and concrete in a wet state, as shown in 3.4.2. Because shear capacity decreases in structural members in water, even structural members that have shear reinforcing bars must undergo the checking of fatigue capacity that is applied to structural members that do not have shear reinforcing bars, as well as checking of stress intensity of the shear reinforcing bars.

3.2 Design actions and combinations of design actions

(1) For actions used in checking of fatigue, characteristic values and frequencies are determined with consideration of the variability of the actions during the design lifetime of the structural object.

(2) A design action is determined by multiplying the characteristic value of the action by an action coefficient. Action coefficients can be determined from **Table 3.2.1**.

Table 3.2.1 The action coefficient

Required performance	Limit state	Kind of actions	Action coefficient
Safety	Fatigue failure	All actions	1.0

(3) Design actions are generally combined as shown in **Table 3.2.2**.

Table 3.2.2 Combination of design action

Required performance	Limit state	Action combinations to be considered
Safety	Fatigue failure	Summation of permanent actions and variable actions

Commentary: The variable action used in checking of fatigue failure must be set with consideration of the effects of all variable actions that will occur under normal use during the design lifetime. Not only the magnitude but also the frequency of the actions must be set. However, because considering the effects of all variable actions is complicated, in general it is acceptable to model several design variable actions and their frequencies.

3.3 Calculation of design response values

3.3.1 General

(1) In the checking of fatigue failure, design variable stress intensity or design force of member cross section may generally be used as the design response value.

(2) The design variable force of member cross section S_{rd} is the values of force of member cross section S_r (a function of Frd) calculated using the design variable action F_{rd}, multiplied by the structural analysis factor γ_a and totaled.

$$S_{rd} = \Sigma \gamma_a S_r \ (F_{rd}) \qquad (3.3.1)$$

where, S_{rd}: design variable force of member cross section

S_r: variable force of member cross section

F_{rd}: design action

γ_a: may generally be set to 1.0.

(3) Design variable stress intensity σ_{rd} is calculated using the design variable force of member cross section S_{rd}.

3.3.2 Structural analysis for checking of fatigue failure

(1) Structural analysis for the checking of fatigue failure should, in principle, consider the effects of nonlinearity of structural members, in accordance with structural properties. When the nonlinearity of structural members is negligible in its effects on design response values, the design response values may be calculated with structural members treated as linear. In this case, the structural analysis factor γ_a is set to 1.0.

(2) The effects of decline in stiffness caused by cracking are generally taken into consideration in the non-linearity of structural members. In principle, the effects of normal temperature changes, shrinkage, creep, and other factors are to be considered in the calculation of response values.

Commentary: Regarding (1): The design response values used in the checking of fatigue failure must be calculated with consideration of factors including changes in the stiffness of structural members caused by actions during design lifetime and other actions.

Various analytical methods exist that take the effects of decline in the stiffness of structural members into account. However, an equivalent linear analytical method that uses stiffness equivalent to a certain stress level should be applied. In this case, the structural analysis factor γ_a may be set to 1.0. Stiffness when the variable action is repeated becomes higher than that at the time of initial loading. Accordingly, when the effects of stiffness cannot be ignored, the stiffness effects should be taken into consideration.

Regarding (2): In general, checking of fatigue failure is based on stress conditions during normal use. Therefore, force of member cross sections caused by normal temperature changes, shrinkage, creep, *etc.* cannot be ignored.

3.3.3 Calculation of force of member cross section

3.3.3.1 Calculation of force of member cross section using wire rod models

When using a structural member model that uses wire rods, the force of member cross section obtained through structural analysis may be used to calculate the force of member cross section of structural members.

Commentary: When using an analytical method that confers nonlinearity using the relationship between the bending moment and the angle of rotation in the wire rod model, the force of member cross section obtained through structural analysis may be used without modification as the force of member cross section of structural members. When a fiber model is used, it is acceptable to use the force of member cross section that was substituted for element stress in the same manner as cases in which the finite element method is used.

3.3.3.2 Calculation of force of member cross section using the finite element method

When the axial force and bending moment acting on a structural member are obtained using the finite element method, the stress distribution inside the cross section may be used and integrated in the direction of section depth. Shear force may be calculated from the equilibrium condition of bending moment distribution.

Commentary: Taking advantage of the features of the finite element method, metrics that use localized information substituted for the state corresponding to the force of member cross section that is required for checking may be used as response valu

3.3.4 Calculation of stress intensity

The design stress intensity of materials is calculated using (1) to (3) below.

(1) The design stress intensity of materials caused by bending moment or by bending moment and axial force is calculated based on the assumptions presented in i) to iv) below. The flexural compressive stress intensity of concrete may be set as the stress intensity of a rectangular stress distribution in which the position of resultant force occurs at the same position as the position of resultant force of stress in a triangular distribution that was calculated based on this section.

i) Fiber strain is proportional to the distance from the neutral axis of the structural member cross section.

ii) Concrete and reinforcing bars are generally treated as elastic bodies.

iii) The tensile stress of concrete is generally ignored.

iv) The Young's modulus of elasticity of concrete and reinforcing bars is based on 5.2.5 and 5.3.4 in "Design: Main Volume."

(2) The design stress intensity of the shear reinforcing bars caused by shear force is calculated using Equations (3.3.2) and (3.3.3).

$$\sigma_{wrd} = \frac{(V_{pd} + V_{rd} - k_r \cdot V_{cd})s}{Aw \cdot z \cdot (\sin\theta + \cos\theta)} \cdot \frac{V_{rd}}{V_{pd} + V_{rd} + V_{cd}} \tag{3.2.2}$$

$$\sigma_{wpd} = \frac{(V_{pd}+V_{rd}-k_r \cdot V_{cd})s}{A_w \cdot z \cdot (sin\theta + cos\theta)} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}} \tag{3.2.3}$$

where, σ_{wrd}: design variable stress intensity of shear reinforcing bars

σ_{wpd}: design stress intensity of shear reinforcing bars caused by permanent action

V_{pd}: design shear force caused by permanent action

V_{rd}: design shear force caused by variable action

V_{cd}: design shear force of rod members that do not use shear reinforcing bars, following 2.4.3.2.

In this case, the member factor γ_b is generally set to 1.3 in checking of fatigue failure.

k_r: coefficient for considering the effects of the frequency of variable action; may generally be set to 0.5. In structural members for which the repetition of variable actions does not matter, it is set to 1.0.

s: spacing in the arrangement of shear reinforcing bars

A_w: total cross-sectional area of shear reinforcing bars in the interval s

θ: angle formed between shear reinforcing bars and structural member axis

z: distance from the loaded position of the resultant force of compressive stress to the centroid of the tensile reinforcing bars; generally, set to $d/1.15$

d: effective depth

(3) The design stress intensity of torsional reinforcing bars caused by torsion is calculated using Equation (3.3.4).

$$\sigma_{wpd} = \frac{M_{tpd} - 0.7M_{t1}}{M_{t2} - 0.7M_{t1}} \cdot f_{wyd} \tag{3.3.4}$$

where, σ_{wpd}: design stress intensity of transverse torsional reinforcing bars caused by permanent action

M_{tpd}: design torsional moment caused by permanent action

$$M_{t1} = M_{tcd} \cdot (1 - 0.8V_{pd}/V_{yd})$$

$$M_{t2} = 0.2M_{tcd} \cdot V_{pd}/V_{yd} + M_{tyd} \cdot (1 - V_{pd}/V_{yd})$$

M_{tcd}: design pure torsional capacity when no torsional reinforcing bars are present, following 2.4.4.2

In this case, the member factor γ_b and material factor γ_c of concrete are generally set to 1.0.

M_{tyd}: design torsional capacity determined by yielding in torsional reinforcing bars, following 2.4.4.3

In this case, the member factor γ_b is generally set to 1.0.

V_{pd}: design shear force caused by permanent action

V_{yd}: design shear capacity of rod members, following 2.4.3.2

In this case, the member factor γ_b and material factor γ_c of concrete are generally set to 1.0.

f_{wyd}: design tensile yield strength of transverse torsion reinforcing bars

Commentary: Regarding (1): Assumptions that have been conventionally used in the calculation of stress intensity of the concrete and reinforcing bars in reinforced concrete structures under normal usage conditions are shown. When the finite element method or an analytical method such as a fiber model that makes direct use of the stress-strain relationship of materials is applied as the structural analysis method, the force of member cross section obtained in 3.3.3 may be used to calculate the response values of materials in accordance with this section.

Based on previous studies, if the position where the resulting force of the compressive stress of concrete does not change when subjected to repeated action by which a stress gradient occurs, fatigue life should be calculated by using stress intensity with stress distribution (rectangular stress distribution) considered, such as when the resultant force acting position is set to the centroid, to be the magnitude of the repeating stress.

Regarding (2): When vertical stirrups and bent rebar are used together for shear reinforcing bars, the stress intensity of each may be calculated using Equations (Commentary 3.3.1) through (Commentary 3.3.4).

Vertical stirrup:

$$\sigma_{wrd} = \frac{V_{pd} + V_{rd} - k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s} + \frac{A_b \cdot z \cdot (\cos\theta_b + \sin\theta_b)^3}{s_b}} \cdot \frac{V_{rd}}{V_{pd} + V_{rd} + V_{cd}} \tag{Commentary 3.3.1}$$

$$\sigma_{wpd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)^3}{s_b}} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}}$$

(Commentary 3.3.2)

Bent rebar:

$$\sigma_{brd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s \cdot (\cos\theta_b+\sin\theta_b)^2}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)}{s_b}} \cdot \frac{V_{rd}}{V_{pd}+V_{rd}+V_{cd}}$$

(Commentary 3.3.3)

$$\sigma_{bpd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s \cdot (\cos\theta_b+\sin\theta_b)^2}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)}{s_b}} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}}$$

(Commentary 3.3.4)

where

σ_{wrd}: design variable stress intensity of vertical stirrup;

σ_{wpd}: design stress intensity of vertical stirrup caused by permanent action;

σ_{brd}: design variable stress intensity of bent rebars;

σ_{bpd}: design stress intensity of bent rebars caused by permanent action;

s: spacing in arrangement of vertical stirrups;

s_b: spacing in arrangement of bent rebars;

A_w: total cross-sectional area of vertical stirrups in the interval s;

A_b: total cross-sectional area of bent rebars in the interval s_b; and

θ_b: angle formed between bent rebars and structural member axis.

Regarding (3): Equation (3.3.4) was derived on the assumption that the stress intensity of transverse torsional reinforcing bars increases with the occurrence of cracking and becomes f_{wyd} during failure. The coefficient 0.7 for M_{t1} in Equation (3.3.4) represents 70% of cracking moment M_{tc}.

Equation (3.3.4) incorporates the effect of shear force on the stress intensity of lateral torsional reinforcing bars, with consideration of the correlation between capacity caused by shear and torsional moment during cracking and during failure.

In cases in which bending moment is applied at the locations where torsional cracking occurs on the upper and lower surfaces of beams, or in which torsional moment is applied at the locations where flexural cracking occurs on the upper and lower surfaces of beams, the effects are smaller than in the case of a combination of torsional moment and shear. Consideration of cases in which bending moment and torsional moment act simultaneously is not necessary.

Consideration of cracking with respect to deformation compatibility torsion is not necessary. However, if U-shaped stirrups are used in beam structural members, excessive cracking may occur on the upper surface of the beams. Therefore, when the section depth for a rectangular cross section is not greater than three times the structural member width, closed stirrups should be used.

3.3.5 Calculation of the equivalent number of repetitions of design variable force of member cross section

Irregular variable force of member cross sections may be broken down into sets of independent variable force of member cross sections, and, applying Miner's Rule, may be replaced by the action of the equivalent number of repetitions N for the design variable force of member cross section S_{rd}.

Commentary: In general, the variable force of member cross section that acts on structural member cross sections through variable action presents a complicated shape. Therefore, it must be broken down into sets of independent variable force of member cross sections (S_{r1}, S_{r2}, …, S_{rm}) and the number of repetitions of these (n_1, n_2,

…, n_m). Many specific methods for doing so have been proposed, including the range-pair method. A method must be selected in line with the properties of the variable action.

Next, Miner's Rule is applied to the sets of independent variable force of member cross sections and the equivalent number of repetitions N for the design variable force of member cross section S_{rd} is obtained.

$$\sum_{i=1}^{m} \frac{n_i}{N_i} = 1.0 \qquad \text{(Commentary 3.3.5)}$$

When the capacity of the structural member cross section is determined from the fatigue strength of the reinforcing bars and when the slope of the S-N line is given by Equation (3.4.4), *etc*., then, applying Miner's Rule, the equivalent number of repetitions N for the design variable force of member cross section S_{rd} can be derived for bending moment (M_{rd}, M_{ri}) using Equation (Commentary 3.3.6) and can be derived for shear force (V_{rd}, V_{ri}) using Equation (Commentary 3.3.7).

$$N = \sum_{i=1}^{m} n_i (M_{ri}/M_{rd})^{1/k} \qquad \text{(Commentary 3.3.6)}$$

$$N = \sum_{i=1}^{m} n_i \left[\frac{V_{ri}}{V_{rd}} \cdot \frac{V_{ri}+V_{pd}-k_2V_{cd}}{V_{rd}+V_{pd}-k_2V_{cd}}\right]^{1/k} \qquad \text{(Commentary 3.3.7)}$$

where

k: constant expressing the slope of the S-N line for reinforcing bars, following 3.4.3 (2), *etc*.;

V_{pd}: design shear force caused by permanent action;

V_{cd}: design shear capacity of rod members that do not use shear reinforcing bars; and

k_2: coefficient for considering the effects of the frequency of variable action; may generally be set to 0.5.

When the capacity of the structural member cross section is determined by the fatigue strength of the concrete and when design fatigue strength is given by Equation (3.4.3), then, applying Miner's Rule, the equivalent number of repetitions N for the design variable force of member cross section S_{rd} can be derived using Equation (Commentary 3.3.8).

$$N = \sum_{t=1}^{m} n_i \cdot 10^{\frac{K(S_{ri}-S_{rd})}{k_{1f}S_d(1-\sigma_p/f_d)}} \qquad \text{(Commentary 3.3.8)}$$

where

S_d: force of member cross section when stress intensity reaches f_d;

σ_p: stress intensity caused by permanent action; and

k_{1f}, f_d and K : follow 3.4.2 (3).

3.4 Calculation of design critical values

3.4.1 General

(1) In the checking of fatigue failure, design fatigue strength or design fatigue capacity may be used as the design critical value.

(2) Design fatigue strength f_{rd} is the value obtained by dividing the characteristic value of fatigue strength f_{rk} by the material factor γ_m.

$$f_{rd} = f_{rk} / \gamma_m \qquad (3.4.1)$$

where, f_{rd}: design fatigue strength

f_{rk}: characteristic value of fatigue strength

γ_m: member factor; the value indicated for each node may be used

(3) Design fatigue capacity R_{rd} is the value obtained by dividing the characteristic value of fatigue capacity R_{rk} by the material factor γ_b.

$$R_{rd} = R_{rk} / \gamma_b \qquad (3.4.2)$$

where, R_{rd}: design fatigue capacity

R_{rk}: characteristic value of fatigue capacity

γ_b: member factor; the value indicated for each node may be used.

3.4.2 Fatigue strength of concrete

(1) The characteristic value of the fatigue strength of concrete is determined based on fatigue strength according to testing conducted with consideration of the type of concrete, the exposure conditions of the structural object, *etc*.

(2) The material factor of concrete γ_c is generally set to 1.3 for the fatigue limit state.

(3) The design fatigue strength f_{rd} of compression, flexural compression, tension, and flexural tension of concrete may generally be derived using Equation (3.4.3) as a function of fatigue life *N* and stress intensity σ_p caused by permanent action.

$$f_{rd} = k_{1f} f_d \left(1 - \sigma_p / f_d\right) \left(1 - \frac{logN}{K}\right) \tag{3.4.3}$$

$$N \leq 2 \times 10^6$$

where, f_d: respective design strength of concrete; may be derived with material factor γ_c set to 1.3.

For f_d, each design strength for f'_{ck}= 50 N/mm^{2} is set as the upper limit.

(i) In the case of ordinary concrete that is continuously or frequently saturated with water and in the case of light aggregate concrete, *K* is set to 10.

In other general cases, *K* is set to 17.

(ii) k_{1f} may generally be set as follows.

In the case of compression and flexural compression, k_{1f}= 0.85.

In the case of tension and flexural tension, k_{1f}= 1.0.

(iii) σ_p is the stress intensity of concrete caused by permanent action. It is generally set to 0 when subjected to alternating loads.

Commentary: Regarding (2): When fatigue failure does not result in the ultimate state of the structural object, the value of γ_c may be set to less than 1.3.

Regarding (3): With reference to past experimental data on the compressive fatigue strength of ordinary concrete, estimating the relationships among the ratio of minimum stress to static strength $S_{\min}$, the ratio of maximum stress to static strength $S_{\max}$, the ratio of stress amplitude to static strength $S_r (S_r = Smin_{max})$, and fatigue life *N*, design fatigue strength can be closely expressed by Equation (Commentary 3.4.1) when *N* is 2×10^{6} or fewer iterations.

$$logN = 17 \frac{1-S_{max}}{1-S_{min}} = 17\left(1 - \frac{S_r}{1-S_{min}}\right) \quad \text{(Commentary 3.4.1)}$$

Taking characteristic value f'_{ck} as static strength, an equation with f_d from Equation (3.4.3) as f'_{ck} can be obtained as an equation that incorporates most of the experimental data on the conservative side, provided that k_{1f} is 1.0 and K is 17. However, taking strength reduction caused by permanent action and other factors into consideration, k_{1f} was set to 0.85 for the purpose of safety. Because sufficient data is not available for cases in which f'_{ck} exceeds 50 N/mm^{2}, the applicable range of

Equation (3.4.3) was set to 50 N/mm^2 or less. When it exceeds 50 N/mm^2, the design fatigue strength for 50 N/mm^2 may be used.

The design fatigue strength is divided according to the exposure conditions of the structural object. Because lightweight aggregate absorbs large amounts of water, a large amount of water can be assumed to be contained in the concrete. Therefore, the compression fatigue properties of lightweight aggregate concrete were considered equivalent to those of ordinary concrete that is in a constant wet state.

For other special concretes, design fatigue strength is in principle determined through testing to ensure a safety level equivalent to that of ordinary concrete.

Although there have been no reports concerning fatigue strength under alternating loads, compressive fatigue strength is calculated as complete pulsating repeated stress with tensile stress ignored, and tensile fatigue strength is calculated as pulsating repeated stress with compressive stress ignored.

Because sufficient information is not available for cases in which fatigue life N exceeds 2×10^6 iterations, design fatigue strength is in principle determined through testing. Because Equation (3.4.3) is considered to provide values on the conservative side when N exceeds 2×10^6 iterations, it may be used as is in checking of the limit state of fatigue failure.

3.4.3 Fatigue strength of reinforcing bars

(1) The characteristic value of the fatigue strength of reinforcing bars is determined from fatigue strength based on testing conducted with consideration of factors including the type, shape, and dimensions of the reinforcing bars, the jointing method, the magnitude and frequency of acting stress, and environmental conditions.

(2) For reinforcing bars conforming to JIS standards, the design fatigue strength f_{srd} of the reinforcing bars may generally be derived using Equation (3.4.4) as a function of fatigue life N and the stress intensity σ_{sp} of the reinforcing bars caused by permanent action.

$$(\mathrm{N/mm^2})\ f_{srd} = 190\frac{10^a}{N^k}\left(1-\frac{\sigma_{sp}}{f_{ud}}\right)/\gamma_s \qquad (3.4.4)$$

$$N \le 2\times10^6$$

where, f_{ud}: design tensile strength of rebar

γ_s: material factor for rebar; may generally be set to 1.05.

(i) a and k are in principle determined through testing.

(ii) When fatigue life is no more than 2×10^6 iterations, a and k may generally be used as the values in Equation (3.4.5).

where, φ: diameter of rebar (mm)

k_{0f}: coefficient related to the form of the ribs of the rebars; may generally be set to 1.0.

(3) The design fatigue strength of gas pressure welds may generally be set to 70% of that of the base metal. The fatigue strength of rebars assembled through welding, rebars with bends, and intersecting parts with shear cracking may be set to 50% that of the base metal.

Commentary: Regarding (1): Because the fatigue strength of reinforcing bars is affected by numerous factors including material quality, form, and dimensions, it is in principle determined through testing.

In the case of the PRC structures presented in "Design: Standards" Volume 8, because stress fluctuations in PC steel are relatively large, checking of fatigue failure is necessary.

The fatigue strength of PC steel is in principle determined through fatigue testing using the actual PC steel and anchoring devices. However, when test data or reliable documentation is not available, it can generally be derived using Equations (Commentary 3.4.2) and (Commentary 3.4.3).
In the case of PC steel, the fatigue strength of the anchorage zone tends to be lower than that of the base metal. Therefore, especially when using unbonded PC steel or the external cable method, checking of fatigue failure of the anchorage zone is necessary.

PC wire and strand (N/mm^2)

$$f_{prd} = 280\frac{10^{a_r}}{N^k}\left(1-\frac{\sigma_{pp}}{f_{pud}}\right)/\gamma_s \qquad \text{(Commentary 3.4.2)}$$

PC steel rod (N/mm^2)

$$f_{prd} = 270\frac{10^{a_r}}{N^k}\left(1-\frac{\sigma_{pp}}{f_{pud}}\right)/\gamma_s \qquad \text{(Commentary 3.4.3)}$$

where

f_{prd}: design fatigue strength of PC steel;
N: fatigue life;
σ_{pp}: stress intensity of PC steel caused by permanent action;
f_{pud}: design tensile strength of PC steel;
$a_r$and$k$: may generally be set to the values indicated in **Commentary Table 3.4.1**; and
γ_s: material factor for PC steel; may generally be set to 1.05.

Commentary Table 3.4.1 α_r and k

	PC wire and strand	PC steel rod
a_r	1.14	0.96
k	0.19	0.16

Structural steel is generally assembled into structural members by welding or by using high-strength bolt connections. Therefore, fatigue strength is affected by residual stress and stress concentration in welds. Many fatigue tests have been conducted on steel plates; fatigue strength may be determined based on the data from these tests.

Regarding (2): The fatigue strength of reinforcing bars is affected by factors including the rebar diameter, the form of the ribs, and the rebar strength. Equation (3.4.4) was obtained by organizing data on fatigue strength in rebar conforming to JIS standards. The data applies to 2×10^6 or fewer iterations. k_{0f} in Equation (3.4.5) is generally 1.00 when no arc is present at the base of ribs and the angle between ribs and the rebar axis is 60° or greater. The value may be set to 1.05 when no arc is present at the base of ribs and the angle between ribs and the rebar axis is less than 60°, or 1.10 when an arc is present at the base of ribs. Because sufficient data is not available for cases in which fatigue life N exceeds 2×10^6 iterations, design fatigue strength is in principle determined through testing. Because Equation (3.4.5) is considered to provide values for α and k on the conservative side when N exceeds 2×10^6 iterations, values from Equation (3.4.5) may be used as is in checking of limit states.

When the aforementioned α and k are used for the design fatigue strength of high-strength rebars that are not specified in JIS standards, values on the hazardous side will be provided. Therefore, in the case of rebars for which the characteristic value of tensile yield strength is 685 N/mm^2, it is advisable to determine the values of α and k based on fatigue testing using the actual rebars or based on reliable data. If values based on these are not used, values obtained from Equation (Commentary 3.4.4) may be used.

$$a = k_{0f}(1.34 - 0.003\phi),\quad k = 0.22 \qquad \text{(Commentary 3.4.4)}$$

where

γ_s: may generally be set to 1.05. The value may be set to less than 1.05 when the fatigue failure of rebars does not result in the ultimate state of the structural object.

Regarding (3): In addition to gas pressure welding joints, rebar joints include mechanical joints and other types. Because mechanical joints involve various construction methods, it is advisable to determine fatigue strength by confirming the fatigue properties of specific construction methods through testing.

When rebars are welded or are bent as in shear reinforcing bars, fatigue strength should be set to 50% of the normal value unless confirmed experimentally.

3.4.4 Design fatigue capacity of structural members that do not have shear reinforcing bars

The design fatigue capacity of structural members that do not have shear reinforcing bars may be obtained as follows.

(1) The design shear capacity V_{rcd} of rod members that do not use shear reinforcing bars may generally be derived using Equation (3.4.6).

$$V_{rcd} = V_{cd}(1 - V_{pd}/V_{cd})\left(1 - \frac{logN}{11}\right) \qquad (3.4.6)$$

where, V_{cd}: following Equation (2.4.5) in Chapter 2

N: fatigue life

(2) The design punching shear fatigue capacity V_{rpd} of reinforced concrete slabs treated as plane members may generally be derived using Equation (3.4.7).

$$V_{rpd} = V_{pcd}(1 - V_{pd}/V_{pcd})\left(1 - \frac{logN}{14}\right) \qquad (3.4.7)$$

where, V_{pcd}: following Equation (2.4.11) in Chapter 2

Commentary: In general, structural members that do not have shear reinforcing bars include footings and retaining walls. Particularly in cases in which fatigue is a problem, checking is to be performed using Equation (3.4.6) or Equation (3.4.7). These equations should be applied within the range of approximately 2×10^6 or fewer iterations.

Even when a structural member is subjected to repeated action by water, the fatigue capacity may be calculated by applying Equation (3.4.6) or Equation (3.4.7). When doing so, the design compressive strength f'_{cd} of concrete required in calculation of V_{cd} or V_{pcd} must be based on values that have been tested in water.

As a calculation on the conservative side, the fatigue capacity of structural members that do not have shear reinforcing bars should be used as the design value.

In the case of a moving load that acts repeatedly, such as in highway bridge deck slabs, capacity must be estimated by appropriate methods such as experiments, empirically-based evaluation methods, and nonlinear finite element analysis. Equation (3.4.7) was derived from the results of experiments involving fixed load points and must not be applied to cases involving moving loads.

Part4 Usability Verification

Chapter 1 General

In principle, verification of usability is to be performed by confirming that all component structural members and structural objects do not reach their limit states for usability under design actions.

Commentary: A structural object and structural members must retain sufficient comfort and other functionalities to suit their usage purposes during the design service life. Limit states must be set to suit these aspects of performance and must be examined using methods with demonstrated accuracy and applicable range. This volume presents standards for methods of confirming that, in verification of durability in "Design: Standards" Volume 2 and of initial cracking in "Design: Standards" Volume 6, limit states are not reached. It also presents methods for verifying the usability of structural objects without considering deterioration of materials during the design service life, on the assumption that workability is satisfied according to the "Construction" volume.

Chapter 2 Verification of Appearance Caused by Cracking

2.1 General

(1) In principle, the width of cracks is to be used to verify that the appearance of a structural object is not impaired by cracking. When it is not possible to appropriately calculate a response value for crack width, verification may be performed using the stress intensity of the rebar.

(a) Verification using crack width is performed by multiplying the structural coefficient γ_i by the ratio of the design response value of the crack width occurring in the structural object to the design limit value of crack width (as determined by the appearance required for the structural object), and confirming that the value obtained is 1.0 or lower:

$$\gamma_i\, w_d / w_a \leq 1.0, \qquad (2.1.1)1$$

where w_a : design limit value of crack width;

w_d : design response value of crack width; and

γ_i : structural coefficient, set to 1.0.

(b) Verification using stress intensity in rebar is performed by multiplying the structural coefficient γ_i by the ratio of the rebar stress intensity corresponding to the design response value of the crack width occurring in the structural object to the rebar stress intensity corresponding to the design limit value of crack width (as determined by the appearance required for the structural object), and confirming that the value obtained is 1.0 or lower:

$$\gamma_i\, \sigma_d / \sigma_a \leq 1.0, \qquad (2.1.2)$$

where σ_a : rebar stress intensity corresponding to the design limit value of crack width;

σ_d : rebar stress intensity corresponding to the design response value of crack width; and

γ_i : structural coefficient, set to 1.0.

(2) In a reinforced concrete structure, if the edge tensile stress intensity of the concrete effective across the entire cross section is equal to or less than the flexural crack strength under conditions of combined permanent action and variable action, then examination of flexural crack width may be omitted by setting the stress intensity of tensile rebar due to permanent action to be equal to or less than the rebar stress intensity limit value σ_{sl1} in accordance with Table 3.1.1 in "Design: Standards" Volume 2.

(3) When the design shear force V_d of structural members subject to shear force is smaller than 70% of the design shear capacity V_{cd} of rod members that do not use the shear reinforcement steel (as derived using Equation (2.4.5) in "Design: Standards" Volume 3), then the verification of appearance using the width of shear cracks may be omitted. In this case, however, γ_b and γ_c are generally set to 1.0.

(4) When the design torsional moment M_{td} is less than 70% of the design torsional capacity M_{tud} when torsional reinforcing bars are not present (as derived using Section 2.4.4.2 in "Design: Standards" Volume 3), the effects of torsional cracks on appearance do not have to be taken into consideration. In this case, however, γ_b and γ_c are generally set to 1.0.

(5) In a PC structure, this can generally be replaced by verification of appearance based on cracking, by confirming that cracking is not occurring.

Commentary: Regarding (1), (2) and (3): Although research on shear crack width is making progress, a general-purpose calculation has not been established. Therefore, in addition to verification based on crack width, a method is also described for indirectly confirming that adverse effects due to cracking will not occur, by maintaining the stress of the rebar below the limit value.

Regarding (4): With regard to the usability limit state, examination of torsional cracks can be omitted only when the design torsional moment is less than 70% of the design torsional capacity, for reasons of safety. In the examination of the limit state of cross-sectional failure, examination of compatibility torsion is omitted based on the assumption that a decrease in torsional rigidity due to cracking will be taken into consideration. In examination of the limit state of usability, too, it is generally not necessary to examine crack width due to compatibility torsion.

When U-shaped stirrups are used in beam members, excessive cracks may occur on the upper surface of the beams. For this reason, if the height of a rectangular cross section is not greater than three times the member width, then closed stirrups should be used.

Even with respect to the equilibrium torsional moment, it is thought that if the design torsion moment is no more than the design torsion capacity resisted by the concrete, then torsion cracks will not occur. Therefore, with consideration of safety, examination is not necessary when the design torsional moment is less than 70% of design torsional capacity

2.2 Design action and combinations of design actions

(1) The characteristic values of actions used in verification of usability are set to magnitudes that occur relatively frequently during the construction of structural objects and during their design service life, and are set in line with the combinations of limit states and actions for the required performance to be examined.

(2) When the standard value or nominal value of an action is determined separately from its characteristic value, then this characteristic value is to be set to the value obtained by multiplying the standard value or nominal value by the action correction coefficient ρ_f.

(3) Design actions are to be determined by multiplying the characteristic values of actions by an action coefficient. Action coefficients may be determined from **Table 2.2.1**.

Table 2.2.1 Action coefficient

Required performance	Limit states	Type of action	Action coefficient
Usability	All limit states	All actions	1.0

(4) Design actions are generally to be combined as shown in **Table 2.2.2**.

Table 2.2.2 Combination of design actions

Required performance	Limit states	Combinations of actions
Usability	All limit states	permanent action and variable action

Commentary: Regarding (1): Actions of "relatively frequently occurring magnitude" used in verification related to usability refer to actions by which the limit states of cracking, deformation, *etc*. are not reached under actions occurring with that frequency. Therefore, these must be determined according to the properties of the respective structural objects, the type of action, and the limit state to be examined. Because variable actions exhibit significant variation during use, the magnitude of characteristic values of actions must be set in line with the purpose of verification.

Regarding (4): In verification related to usability, combinations of actions to be examined for cracking, deformation, and other individual performance items are set. Therefore, it is not particularly necessary to distinguish between primary variable actions and secondary variable actions.

2.3 Calculation of design response values

2.3.1 General

(1) In the verification of appearance based on crack width, the crack width may generally be calculated as the design response value.

(2) The design response value S_d of crack width is set to the crack width S (where S is a function of F_d) calculated through structural analysis using the combined design action F_d multiplied by the structural analysis coefficient γ_a:

$$S_d = \gamma_a S(F_d), \tag{2.3.1}$$

where S_d : design response value of crack width;

S : crack width:

F_d : design action; and

γ_a : structural analysis coefficient.

(3) When crack width cannot be properly evaluated, the rebar stress intensity may be calculated as the design response value.

(4) The design response value S_d of rebar stress intensity is set to the rebar stress intensity S (where S is a function of F_d) calculated through structural analysis using the combined design action F_d multiplied by the structural analysis coefficient γ_a:

$$S_d = \gamma_a S(F_d), \tag{2.3.2}$$

where S_d : design response value of rebar stress intensity;

S : rebar stress intensity;

F_d : design action; and

γ_a : structural analysis coefficient.

2.3.2 Structural analysis for verification of appearance

When verifying appearance using crack width, in principle, the effects of decline in rigidity due to cracks should be taken into consideration. However, when the effects of non-linearity of structural members can be ignored, a calculation of design response values may treat structural members as linear. In that case, the structural analysis coefficient γ_a is set to 1.0. In principle, the effects of normal temperature changes, shrinkage, creep, and other factors are to be taken into consideration in the calculation of response values.

Commentary: For structural members in which cracking is expected, calculating cross-sectional force due to temperature changes and shrinkage with consideration of decline in rigidity due to cracking is rational without requiring excessive reinforcement bars. When considering decline in rigidity of structural members due to cracking, the rigidity should be determined through experiments or on the basis of theoretical analysis with solid grounding.

This section focuses on cracks caused by mechanical action, particularly by flexural moment, axial force, shear force, or torsional moment.

Verification of cracks is to be performed under a state of normal use. When there are any changes in a structural system between the time of its erection and its completion, examination must be carried out after consideration of the effects of these changes in the calculation of cross-sectional force. In general, because the effects of normal temperature changes, shrinkage, creep, and so on cannot be ignored in stress states during use, consideration of these effects was set as a principle. When considering the effects of temperature changes, shrinkage, and creep, the values shown in "Design: Standards" Volume 1, Chapter 2 may be used.

2.3.3 Calculation of cross-sectional force

2.3.3.1 Calculation of cross-sectional force using wire models

When a structural member model using wire rods is used, the cross-sectional force obtained from structural analysis may be used to calculate the cross-sectional force of structural members.

Commentary: When a wire rod model is used, the cross-sectional force obtained through structural analysis may be used without change as the cross-sectional force of structural members. When a fiber model is used, the cross-sectional force may be used in place of element stress.

2.3.3.2 Calculation of cross-sectional force using the finite-element method

When the axial force and flexural moment acting on a structural member are obtained using the finite-element method, the stress distribution inside the cross section may be used and integrated in the direction of cross-sectional height. Shear force may be calculated from the equilibrium condition of the flexural moment distribution.

Commentary: An advantage of the finite-element method is the ability to directly obtain stress and strain at localized positions, as well as nodal force at element nodes. Conversely, while cross-sectional force generally cannot be obtained directly, it can be calculated by integrating the stress distribution at Gaussian points, or element node forces that integrate that stress, in the direction of cross-sectional height.

2.3.4 Calculation of design response value of flexural crack width

(1) The design response value of flexural crack width may be calculated using:

$$w = 1.1k_1k_2k_3\{4c + 0.7(c_s - \phi)\}\left[\frac{\sigma_{se}}{E_s}\left(or\,\frac{\sigma_{pe}}{E_p}\right)\right] + \varepsilon'_{csd} \quad (2.3.3)$$

where k_1 : coefficient that expresses the effect of the surface form of steel on crack width; this may generally be set to 1.0 for reinforcing steel and 1.3 for plain round steel and PC steel;

k_2 : coefficient that expresses the effect of concrete quality on crack width, according to:

$$k_2 = \frac{15}{f_c'+20} + 0.7; \quad (2.3.4)$$

f_c' : compressive strength of concrete (N/mm^2); the design compressive strength f_{cd}' may generally be used;

k_3 : coefficient that expresses the effect of multiple layers of tensile steel on crack width, according to:

$$k_3 = \frac{5(n+2)}{7n+8}; \quad (2.3.5)$$

n : number of layers of tensile steel;

c : concrete covering (mm);

c_s : center-to-center distance of steel (mm);

φ : steel diameter (mm);

ε_{csd}' : numerical value that takes into account the increase in crack width due to shrinkage of concrete, creep, *etc*.; the values shown in **Table 2.3.1** may be used as standard values;

Table 2.3.1 Numerical value that takes into account the increase in crack width due to shrinkage of concrete, creep, etc

Environmental conditions	Permanently dry environment (e.g. Bottom surface of girders unaffected by rainwater)	Repeated dry and wet environments (e.g. Top surface of girders, humid environments close to the water surface of coasts and rivers)	Permanently wet environment (e.g. Members of underground, etc.)
Members that crack under their own weight (assuming a material age of 30 days)	450×10^{-6}	250×10^{-6}	100×10^{-6}
Members that crack under permanent action (assuming a material age of 100 days)	350×10^{-6}	200×10^{-6}	100×10^{-6}
Members that crack during variable action (assuming a material age of 200 days)	300×10^{-6}	150×10^{-6}	100×10^{-6}

(2) The rebar and PC steel to be subjected to examination for flexural cracking should in principle be tensile steel located closest to the surface of the concrete, with stress intensity derived according to 2.3.5.

σ_{se} : amount of increase in rebar stress intensity, from the state in which stress intensity at the position of the steel is 0 (N/mm^2); and

σ_{pe} : amount of increase in PC steel stress intensity, from the state in which stress intensity at the position of the steel is 0 (N/mm^2).

Commentary: Regarding (1): Equation (2.3.3) was established with reference to the results of past research. When using a combination of rebars having differing diameters, and when using high-strength reinforcing steel with a characteristic value not less than 490 N/mm^2, crack spacing should be derived using some other appropriate method.

The spacing between flexural cracks is affected by the adhesive properties of the steel and the concrete. In Equation (2.3.3), k_1 is a constant that, among the effects of the adhesive properties of steel and concrete on crack width, takes into account the effect of the form of the steel surface. Equation (2.3.3) is based on cases involving the use of reinforcing steel. How crack width changes when ordinary round steel or PC steel is used remains unclear, but, for the time being, k_1 = 1.3 may be used. If the pretension method is used with PC steel and the adhesiveness of the steel can be considered to be good based on the surface form, then k_1 of 1.3 or less (though $k_1 \geq 1.0$) may be used.

In Equation (2.3.3), k_2 is a coefficient that expresses the effect of changes in the adhesive properties of steel and concrete, due to concrete quality, on crack spacing. Equation (2.3.4) uses compressive strength as a metric to represent the quality of hardened concrete, by reducing crack width and crack spacing to express the effect of concrete quality on crack width. Even in concrete of low strength, when bleeding is reduced and a uniform concrete covering is applied, this value may be lowered to 0.9.

In Equation (2.3.3), k_3 is a coefficient that expresses the effect of the steel material on the second and subsequent layers from the outermost edge on surface crack width in the case of multi-layer arrangement of reinforcement bars. Equation (2.3.5) is a simplified equation that expresses the effect of the number of rebar layers in the existing equation for calculating crack width.

For concrete surface strain between cracks, the values in Table 2.3.1 may be used when concrete is used for which shrinkage properties have been confirmed to fall within the normal range (generally concrete with a six-month shrinkage strain of no more than 1000 μm, according to the JIS A1129 test). In a dry-wet cycling environment, dry-shrinkage is small and therefore was set to approximately half that of a dry environment. Although dry-shrinkage does not proceed in a constantly humid environment, it was decided to consider 100□ due to the increase in crack width caused by creep.

When shrinkage-compensating concrete is used, ε'_{csd} may be reduced in consideration of the crack suppression effect of the shrinkage-compensating concrete and the effect of reduced variable stress in steel material caused by expansion action.

Regarding (2): In principle, the effects of steel that is constraining shrinkage creep in concrete with respect to all-longitudinal steel arranged in the cross section are to be taken into consideration in the calculation of steel stress intensity. This can be ignored for RC structures, however. The steel stress up to the point that stress in the concrete at the position of the steel becomes 0 is considered to be the reactive force of the steel.

For the stress intensity of steel, the value when the continuous components of permanent action and variable action occur may be used.

The amount of increase in steel stress intensity σ_{se} (or σ_{pe}) must be derived with consideration of both the stress state of concrete and the stress state of steel. In other words, if the stress intensity of the concrete at the same position as the steel is in a compressed state even when cross-sectional force acts, then σ_{se} (or σ_{pe}) is set to 0 even if the stress intensity of the steel increases. When the stress intensity of the concrete changes from a compressed state to a tensile state, the amount of change in the steel stress intensity caused by cross-sectional force when concrete stress intensity changes from compression to 0 is used as the amount of increase σ_{se} (or σ_{pe}). When autogenous shrinkage and dry-shrinkage in concrete cannot be ignored, the amount of change in steel stress intensity caused by cross-sectional force, added to the steel stress intensity until concrete stress intensity changes from tension to 0, is used as the amount of increase σ_{se} (or σ_{pe}). The steel stress intensity until the concrete stress intensity changes from tension to 0 is also dependent on the steel ratio, and may exceed the steel stress intensity equivalent to compressive strain of 300×10^{-6}, but can be set to 200×10^{-6} or lower when curing is performed properly.

When shrinkage-compensating concrete is used, unlike cases involving shrinkage, the compressed state of the concrete is used as the origin. Therefore, the amount of change in rebar stress intensity caused by the cross-sectional force originating in the stress intensity of the rebar when the concrete stress intensity changes from compression to 0 is used as the amount of increase σ_{se} (or σ_{pe}). The degree of rebar stress intensity until concrete stress intensity changes from compression to 0 is the value obtained by multiplying strain that adds chemical prestraining to rebar strain that corresponds to chemical prestressing introduced into the concrete, by the Young's modulus of the rebar. For shrinkage-compensating concrete, calculation of rebar stress intensity until concrete stress intensity changes from compression to 0 may be omitted. In this case, ε'_{csd} may be reduced according to the concrete elastic strain and chemical prestrain introduced by expansion. If the compressive strength is approximately 60 N/mm^2 or higher, then chemical prestress and chemical prestrain may be calculated using linear creep analysis, taking into consideration the mechanical properties of concrete that manifest over time.

Actions for deriving σ_{se} and σ_{pe} must be determined appropriately with consideration of factors including the duration and frequency of variable actions, changes in crack width over time, and the degree to which the crack width persists. If the ratio of permanent actions to total actions is small and the frequency of variable actions is high, then the effect of variable actions must be set to a high level. If the ratio of permanent actions to total actions is large and the frequency of variable actions is low, then the effect of variable actions can be set to a low level.

The examination of crack width in this section does not apply to cases in which only PC steel is used as longitudinal steel. Reinforcing steel is to be used as longitudinal rebar in prestressed concrete.

When steel is arranged in multiple layers, the stress intensity of the tensile steel closest to the surface of the concrete should in principle be used as the steel stress intensity. For simplicity, the value at the center of gravity of the tension steel may be used. In that case, however, the coefficient k_3 that expresses the influence of multi-layer arrangement of reinforcing bars in Equation (2.3.3) must be calculated as 1.0, regardless of the number of layers of steel. For structural members with a few layers of reinforcing bars, the results of the above method may fall on the dangerous side. In such cases, the stress intensity of the steel at the outermost edge should be used.

2.3.5 Calculation of stress intensity

The design stress intensity of a material is to be calculated using the following steps, (1) to (3).

(1) The design stress intensity of the material due to flexural moment or due to flexural moment and axial force is to be calculated based on the assumptions presented in (i) to (iv) below:

(i) Fiber strain is proportional to the distance from the neutral axis of the cross section of the structural member,

(ii) Concrete and steel are generally elastic bodies,

(iii) The tensile stress of concrete is generally ignored, and

(iv) The Young's modulus of elasticity of concrete and steel is based on 5.2.5 and 5.3.4 in "Design: Main Volume".

(2) The design stress intensity of the shear reinforcement steel caused by shear force is to be calculated using:

$$\sigma_{wrd} = \frac{(V_{pd} + V_{rd} - k_r \cdot V_{cd})s}{Aw \cdot z \cdot (\sin\theta + \cos\theta)} \cdot \frac{V_{rd}}{V_{pd} + V_{rd} + V_{cd}} \quad \text{and} \tag{2.3.6}$$

$$\sigma_{wpd} = \frac{(V_{pd}+V_{rd}-k_r \cdot V_{cd})s}{A_w \cdot z \cdot (sin\theta + cos\theta)} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}}, \tag{2.3.7}$$

where σ_{wrd} : design variable stress intensity in shear reinforcing bars;

σ_{wpd} : design stress intensity in shear reinforcing bars due to permanent action;

V_{pd} : design shear force due to permanent action;

V_{rd} : design shear force due to variable action;

V_{cd} : design shear force in rod members that do not use shear reinforcement steel, according to 2.4.3.2 in "Design: Standards" Volume 3; in this case, the structural member coefficient γ_b is generally set to 1.0;

k_r : coefficient for considering effects of the frequency of variable action; this may generally be set to 0.5 but is set to 1.0 for structural members for which repetition of variable action does not matter;

s : spacing in arrangement of shear reinforcing bars;

A_w : total cross-sectional area of shear reinforcing bars in interval s;

θ : angle between shear reinforcing bars and structural member axis;

z : distance from the loading position of the resultant compressive stress to the centroid of the tensile steel; generally $d/1.15$; and

d : effective depth.

(3) The design stress intensity of the torsional reinforcing bars due to torsion is to be calculated using:

$$\sigma_{wpd} = \frac{M_{tpd}-0.7M_{t1}}{M_{t2}-0.7M_{t1}} \cdot f_{wyd}, \tag{2.3.8}$$

where σ_{wpd} : design stress intensity of transverse torsional reinforcing bars due to permanent action;

M_{tpd} : design torsional moment due to permanent action, where

$M_{t1} = M_{tcd} \cdot (1 - 0.8\, V_{pd} / V_{yd})$ and

$M_{t2} = 0.2 M_{tcd} \cdot\ V_{pd} / V_{yd} + M_{tyd} \cdot (1 - V_{pd} / V_{yd})$;

M_{tcd} : design pure torsional capacity when no torsional reinforcing bars are present, according to 2.4.4.2 in "Design: Standards" Volume 3; in this case, the material coefficient γ_c and structural member coefficient γ_b of concrete are generally set to 1.0;

M_{tyd} : design torsional capacity determined by the yield of torsional reinforcing bars, according to 2.4.4.3 in "Design: Standards" Volume 3; in this case, the structural member coefficient γ_b is generally set to 1.0.

V_{pd} : design shear force due to permanent action;

V_{yd} : design shear capacity of rod members, according to 2.4.3.2 in "Design: Standards" Volume 3; in this case, the material coefficient γ_c and structural member coefficient γ_b of concrete are generally set to 1.0; and

f_{wyd} : design tensile yield strength of lateral torsional reinforcing bars.

Commentary: Regarding (1): When the finite-element method or an analytical method such as a fiber model that makes direct use of the stress–strain relationship of materials is applied as the structural analysis method, the cross-sectional force obtained in 2.3.3 may be used to calculate the response values of materials according to this section.

Regarding (2): When vertical stirrups and bent rebar are used together for shear reinforcing bars, the stress intensity of each may be calculated using:

Vertical stirrup:

$$\sigma_{wrd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)^3}{s_b}} \cdot \frac{V_{rd}}{V_{pd}+V_{rd}+V_{cd}} \quad \text{and} \tag{Commentary 2.3.1}$$

$$\sigma_{wpd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)^3}{s_b}} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}}, \text{ and} \tag{Commentary 2.3.2}$$

Bent rebar:

$$\sigma_{brd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s \cdot (\cos\theta_b+\sin\theta_b)^2}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)}{s_b}} \cdot \frac{V_{rd}}{V_{pd}+V_{rd}+V_{cd}} \quad \text{and} \tag{Commentary 2.3.3}$$

$$\sigma_{bpd} = \frac{V_{pd}+V_{rd}-k_r \cdot V_{cd}}{\frac{A_w \cdot z}{s \cdot (\cos\theta_b+\sin\theta_b)^2}+\frac{A_b \cdot z \cdot (\cos\theta_b+\sin\theta_b)}{s_b}} \cdot \frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}}, \tag{Commentary 2.3.4}$$

where σ_{wrd} : design variable stress intensity of the vertical stirrup;

σ_{wpd} : design stress intensity of the vertical stirrup due to permanent action;

σ_{brd} : design variable stress intensity of the bent rebar;

σ_{bpds} : design stress intensity of the bent rebar due to permanent action;

s : spacing in arrangement of the vertical stirrups;

s_b : spacing in arrangement of the bent rebar;

A_w : Total cross-sectional area of the vertical stirrups in interval s;

A_b : Total cross-sectional area of the bent rebar in interval s_b; and

Θ_b : angle between the bent rebar and structural member axis.

Regarding (3): Equation (2.3.8) was derived based on the assumption that the stress intensity of transverse torsional reinforcing bars increases with the occurrence of cracking, and becomes f_{wyd} during failure. Past studies have found that, with respect to repeated torsional moments in the state following occurrence of cracking, experimental values are in good agreement with torsional crack widths derived based on the assumption that the stress intensity of lateral torsion reinforcing bars is less than 70% of cracking moment M_{tc}, becomes 0, and subsequently increases nearly linearly with the increase in torsion moment. The coefficient 0.7 for M_{t1} in Equation (2.3.8) is expressed as 70% of M_{tc}.

The direction of the primary tensile stress due to shear force may perfectly match the direction of the primary tensile stress due to torsional moment. In this case, the stress intensity of transverse torsional reinforcing bars will further increase due to shear force. Therefore, Equation (2.3.8) incorporates the effect of shear force on the stress intensity of lateral torsional reinforcing bars, with consideration of the correlation between load capacity due to shear and torsional moment during cracking and during failure.

In cases in which flexural moment is applied at the locations where torsional cracking occurs on the upper and lower surfaces of beams or in which torsional moment is applied at the locations where flexural cracking occurs on the upper and lower surfaces of beams, the direction of the primary tensile stress due to torsion and the direction of the primary tensile stress due to flexing do not match. Therefore, the combined effect of these is smaller than that of the combination of torsional moment and shear force. In addition, in the case of a vertically long rectangular cross section that is subjected to torsional moment, tensile stress in the upper and lower surfaces is less than tensile stress in the side surfaces. Based on this, it was decided that it is not necessary to examine cases in which flexural moment and torsional moment act simultaneously.

Because torsional moment itself is largely released by initially occurring cracks that are relatively minute under normal use conditions, with crack width that does not tend to increase, and because reinforcing bars with at least minimal resistance to shear are arranged in structural members, thus hindering the progress of torsional cracking, it is not necessary to examine cracks for deformation compatibility torsion.

2.4 Setting design limit values

(1) In the case of girders in a general reinforced concrete structure or PRC structure, the design limit value of crack width with respect to appearance may be set to approximately 0.3 mm.

(2) When verifying torsional cracking and shear cracking with respect to appearance through rebar stress intensity, the limit value of rebar stress intensity due to permanent action may be set to the values shown in **Table 2.4.1**.

Table 2.4.1 The limit value of rebar stress intensity due to permanent action when verifying torsional cracking and shear cracking with respect to appearance through rebar stress intensity (N/mm^2)

Permanently dry environment (e.g. Bottom surface of girders unaffected by rainwater)	Repeated dry and wet environments (e.g. Top surface of girders, humid environments close to the water surface of coasts and rivers)	Permanently wet environment (e.g. Members of underground, etc.)
140	120	140

Commentary: <u>Regarding (1)</u>: In reinforced concrete structures and PRC structures in which the occurrence of flexural cracking under normal use conditions is tolerated, it is necessary to constrain crack width on surfaces primarily to prevent problems with durability and appearance. Based on track record and experience, the design limit value of crack width for appearance may generally be set to approximately 0.3 mm.

Regarding (2): The values in Table 2.4.1 conform to values in Table 3.1.1 presented in "Design: Standards" Volume 2. Because the width of cracks generated by this stress intensity is generally deemed to be equal to or less than the design limit value of crack width for appearance, the design limit value of rebar stress intensity used in verification of appearance is applied correspondingly.

Chapter 3 Verification of Displacement/Deformation

3.1 General

Verification of the functionality and comfort demanded of a structural object using displacement/deformation as a verification metric is performed by confirming that the ratio of the response value for displacement/deformation occurring at a position of interest in the structural object to the limit value of displacement/deformation determined from functionality and comfort, multiplied by the structural object coefficient γ_i, is 1.0 or less:

$$\gamma_i \, \delta_d \,/\, \delta_a \leq 1.0, \tag{3.1.1}$$

where δ_a : design limit value of displacement/deformation;

δ_d : design response value of displacement/deformation; and

γ_i : structural coefficient, set to 1.0.

Commentary: This chapter prescribes the handling of verification of the functionality and comfort demanded of a structural object, when displacement/deformation is used as a verification metric.

3.2 Design action and combinations of design actions

Same as 2.2.

Commentary: Same as 2.2.

3.3 Calculation of design response values

3.3.1 General

(1) In the verification of displacement/deformation, the design displacement and design deformation may generally be calculated as design response values.

(2) The design response value δ_d is to be the sum of response values δ (where δ is a function of F_d) calculated through structural analysis using F_d for the combined design actions, multiplied by the structural analysis coefficient γ_a:

$$\delta_d = \Sigma \gamma_a \, \delta \, (F_d), \tag{3.3.1}$$

where δ_d : design response value;

δ : response value;

F_d : design action; and

γ_a : structural analysis coefficient.

3.3.2 Structural analysis

When verifying the comfort of a structural object using displacement/deformation, the effects of shrinkage and creep that occur during the design service life, as well as the decline in rigidity due to cracks, should in principle be considered.

Commentary: The range over which action varies during normal use is large, meaning that the state of cracking in structural members differs according to the action to be verified and that displacement/deformation varies with rigidity. Therefore, when performing verification using displacement/deformation as a metric, the effect of decline in rigidity caused by cracking must be considered according to the action.

In general, structural members are to be roughly divided into those in which cracking occurs and those in which it does not. For structural members in which flexural cracking does not occur, the theory of elasticity is used with the entire cross section treated as valid. For structural members in which flexural cracking occurs, an analytical method that appropriately considers the decline in rigidity due to cracks is used. The method presented in 3.3.3 may be used for the effect of decline in rigidity due to cracking. However, because this method is for the purpose of calculating the rigidity of a structural member during initial loading, when the limit state to be verified is a limit state for repeated action during normal use, the rigidity of the structural member at the time of re-loading must be used as necessary.

For displacement/deformation, the effects of concrete shrinkage and creep as well as the effects of cracks are in principle to be taken into consideration.

3.3.3 Calculation of displacement/deformation in structural members

3.3.3.1 Calculation of short-term displacement/deformation

(1) Short-term displacement/deformation of concrete structural members in which cracks do not occur may be calculated using the theory of elasticity, with the entire cross section considered valid.

(2) Short-term flexural displacement/deformation of concrete structural members in which flexural cracking has occurred is derived with consideration of the decline in rigidity due to cracking.

(3) Short-term shear displacement or deformation of concrete structural members in which cracking has occurred is derived with consideration of the decline in rigidity due to cracking.

Commentary: Regarding (1): When it is not necessary in practice to derive exact displacement/deformation, both reinforced concrete and prestressed concrete structural members in which cracking is occurring can be regarded as structural members in which flexural cracking is not occurring, and calculation may use the second moment of area I_g with the entire cross section considered valid.

Regarding (2): When calculating short-term displacement/deformation with consideration of decline in rigidity due to flexural cracking, the converted second moment of area presented in the following equations may be used.

(i) When changing cross-sectional rigidity according to the magnitude of flexural moment for each structural member cross section:

$$I_e = \left[\left(\frac{M_{crd}}{M_d}\right)^4 I_g + \left(1 - \left(\frac{M_{crd}}{M_d}\right)^4\right) I_{c_r}\right] \leq I_g \text{ , and}$$

(Commentary 3.3.1)

(ii) When cross-sectional rigidity is constant over the entire length of the structural member:

$$I_e = \left[\left(\frac{M_{crd}}{M_{d\rightleftarrows\rightleftarrows\, max}}\right)^3 I_g + \left(1 - \left(\frac{M_{crd}}{M_{d\, max}}\right)^3\right) I_{c_r}\right] \leq I_g,$$

(Commentary 3.3.2)

where I_e : converted second moment of area;

M_{crd} : limit flexural moment at which flexural cracks occur in the cross section and the flexural moment at which flexural stress intensity at the tensile edge of concrete is the design flexural cracking strength, though in this case γ_b is generally set to 1.0;

M_d : design flexural moment in calculation of displacement/deformation;

$M_{d\rightleftarrows\rightleftarrows\, max}$: maximum value of design flexural moment in calculation of displacement/deformation;

I_g : cross-sectional second moment of gross area;

I_{cr} : second moment of area, excluding concrete subject to tensile stress

The converted second moment of area in a continuous beam may be calculated using Equation (Commentary 3.3.2) for the cross section of positive maximum moment.

Displacement/deformation according to (i) can be derived by numerically integrating the curvatures (M_d/E_cI_e) of the cross sections when Equation (Commentary 3.3.1) is used. Deflection in a statically indeterminate structure such as a continuous beam can be derived through numerical integration for each structural member, using the flexural moment distribution approximated through an elasticity solution.

Calculation of displacement/deformation using Equation (Commentary 3.3.2) in (ii) corresponds to the case in which the distribution of flexural moment is a second-degree parabola. Strictly speaking, the exponent in the equation must be changed according to the distribution profile of flexural moment, but because error is small even with an exponent of 3, this value was approximated and fixed.

When axial force is acting, this may be derived with the flexural moment ratio replaced by the ratio of tensile force generated in rebar in Equations (Commentary 3.3.1) and (Commentary 3.3.2).

Regarding (3): Shear rigidity in concrete structural members decreases even after the occurrence of flexural cracking and tensile cracking. However, the effect on the deformation of structural members is generally less than that of the decrease in flexural rigidity. A method exists to evaluate decrease in shear rigidity in this case using the same method used for decrease in flexural rigidity.

The method presented below may be used to calculate shear deformation in shear-reinforced concrete structural members that are subjected to flexural shear after flexing and shear cracking.

(i) After the occurrence of flexural cracks but before the occurrence of shear cracks, shear deformation δ_s is calculated based on the assumption that shear rigidity GA_e decreases as:

$$GA_e = G\left[A_g\left(\frac{M_{crd}}{M_{d\rightleftarrows max}}\right)^3 + A_{cr}\left\{1 - \left(\frac{M_{crd}}{M_{d\, max}}\right)^3\right\}\right] \leq GA_g$$

and (Commentary 3.3.3)

$$\delta_s = k \int \frac{V_d}{GA_e} dx, \qquad \text{(Commentary 3.3.4)}$$

where G : shear modulus; may be used as shear modulus of concrete G_c;

A_g : total cross-sectional area before flexural cracking;

A_{cr} : cross-sectional area of only the flexural compression zone after flexural cracking;

k : coefficient due to cross-sectional profile; and

V_d : design shear force

(ii) After the occurrence of shear cracking, regardless of whether flexural cracks occur, it is assumed that a truss structure is formed as shown in Commentary Figure 3.3.1, with concrete as the compressive diagonal member (BE in the figure) and shear reinforcing bars as the tensile diagonal member (CE in the figure). The deformation components of the truss structure caused by deformation of the truss structure's compressive diagonal member and tensile diagonal member are defined as the shear deformation of the structural member. This shear deformation can be calculated using:

$$\delta_s = \int \gamma dx = \int \frac{1}{z(\cot\theta + \cot\alpha)^2} \left[\frac{V_{sd}}{E_c b_w \sin^4\theta} + \frac{V_{sd} s}{E_w \left(A_w + \frac{E_c}{E_w} A_{ce} \right) \sin^3\alpha} \right] dx$$

, (Commentary 3.3.5)

where γ : shear strain;

z : distance from the action position of the resultant compressive stress to the centroid of the tensile steel; this can generally be set to $d/1.15$;

θ : angle formed by the long axis of the compressive diagonal member; this can be derived using:

$$\theta = 45° - k \frac{V_d - V_{cd}}{b_w d}, \text{ where} \qquad \text{(Commentary 3.3.6)}$$

$$k = (3.2 - 7800 p_t p_w) \rightleftarrows\rightleftarrows \quad (a/d) \quad ; \qquad \text{(Commentary 3.3.7)}$$

p_t : tensile reinforcement steel ratio, $A_s/(b_w d)$;

p_w : shear reinforcement steel ratio, $A_w/(b_w s)$;

a : shear span, the distance from loading point to bearing frontal face;

d : effective depth;

b_w : web width;

α : angle formed with long axis of shear reinforcement steel;

V_{sd} : design shear force provided by shear reinforcement steel, $V_d - V_{cd}$;

V_{cd} : design shear force other than that provided by shear reinforcement steel; this can be derived according to 2.4.3.2 in "Design: Standards" Volume 3, but the material coefficient γ_c may be set to 1.0;

E_c : modulus of elasticity of concrete;

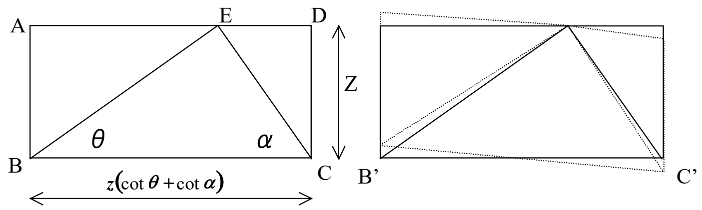
E_w : modulus of elasticity of shear reinforcement steel;

A_w : cross-sectional area of shear reinforcement steel in interval *s*;

A_{ce} : cross-sectional area of the concrete around the tensile reinforcement steel that is valid as the rigidity of the tensile diagonal member; this may be derived using:

$$A_{ce} = A_{ce0}(V_{cd}/V_d)^3; \qquad \text{(Commentary 3.3.8)}$$

A_{ce0} : A_{ce} immediately after occurrence of shear cracking.



Commentary Figure 3.3.1 Shear deformation of the truss structure

(iii) The tensile force of the tensile reinforcement steel increases because of the truss structure that is formed after the occurrence of shear cracking (moment shift). The increase in deformation due to this increase in tensile force may be derived by calculating the flexural deformation, assuming a moment that causes a tensile force of the same magnitude to act on the reinforcement steel. The increase in tensile force can be calculated using: $\Delta T = \left[cot\theta - \frac{sin(\theta+\alpha)}{2sin\theta\cdot \rightleftarrows\rightleftarrows s\rightleftarrows in\alpha}\right]\cdot V_s$. (Commentary 3.3.9).

3.3.3.2 Calculation of long-term displacement/deformation

(1) When calculating long-term displacement/deformation in concrete structural members with high precision, the load; constraints; materials and mixture used for the concrete; the form, dimensions, and reinforcing bar arrangement of the structural object; the sequence of construction of the structural object; the environmental effects such as temperature and humidity and their changes over time should be inputted. Analytical methods that allow appropriate consideration of the progress of the hydration reaction of concrete in the structural object; the moisture transfer, thermal conduction, and associated physical properties of concrete in the structural object; the water content; temporal and spatial changes in the shrinkage; and the effects of creep should be used.

(2) To simply derive long-term displacement/deformation in concrete structural members, the spatial distribution of concrete shrinkage in the structural object based on the temperature and humidity at every interface in the structural object should be evaluated using an appropriate method for prediction of shrinkage. Furthermore, calculation should be performed assuming Navier's hypothesis for the cross section with consideration of creep and of the constraint of shrinkage by rebar or PC steel.

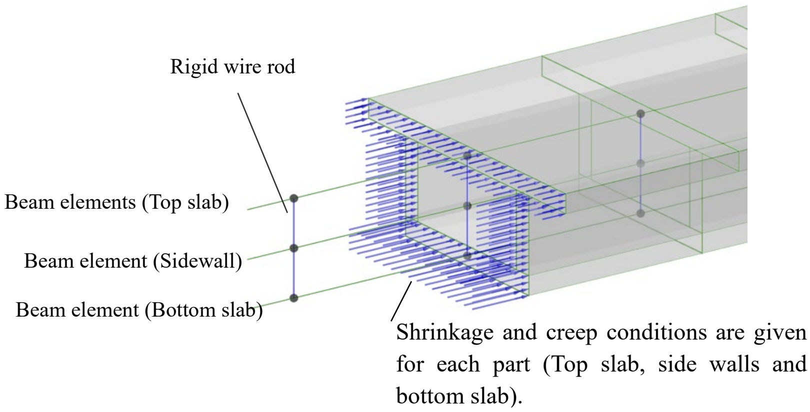
Commentary: Regarding (1): Long-term displacement in a concrete structural object occurs as a result of interactions of physicochemical phenomena that progress in time and space in the structural object. Because precise calculation is difficult using simple methods, it was decided that calculation should in principle be performed using a method that considers these microscopic phenomena as accurately as possible.

Regarding (2): Even when analyzing long-term displacement in a structural object using a general-purpose design equation for prediction of creep, in order to obtain as precise a calculated value as possible, the spatial distribution of contraction in the cross section and the constraint by rebar and PC steel should be considered. Because steel constrains the shrinkage of concrete after hardening, an asymmetrical arrangement of steel causes differences in shrinkage in the cross section. As a result, curvature due to differences in contraction may occur, and may be a factor that increases long-term displacement/deformation.

The method in (2) is a method for extending and applying a general structural analytical method in place of the detailed analytical method presented in (1). It allows easy and relatively accurate calculation of long-term displacement/deformation beginning at the time of construction of static and statically indeterminate structural objects that have standard structural morphologies.

Specifically, a structural member's cross section is divided into multiple parts with consideration of mechanical resistance and environmental conditions (humidity and temperature). The effects of concrete shrinkage and creep according to the environmental conditions of each part are provided as input values, and calculation is performed with consideration of displacement/deformation due to curvature caused by differences in shrinkage in the cross section, according to (i) to (iii) below.

As an example, for a structure with a box-shaped cross section, there are methods involving structural analysis using a fiber model or involving structural analysis that divides the structure into an upper deck slab, side walls, and lower deck slab as shown in Commentary Figure 3.3.2, then applies the effects of concrete shrinkage and creep according to the environmental conditions of each of these three parts to a structural analytical model that uses the respective beam elements of each.



Commentary Figure 3.3.2 Conceptual diagram of a structural analysis model with beam elements

The drying conditions of structural members that are directly affected by rainwater, such as those on the upper surface of girders, should be set so that shrinkage strain is equivalent to the case assuming humidity much higher than the average relative humidity of the outside air, such as when the state is assumed to be close to a constantly wet state. Shrinkage may be assumed to be 0. However, regarding drying conditions for other parts, shrinkage may be calculated from the average relative humidity around the structural object.

(i) Long-term displacement in concrete structural members may be calculated with consideration of the effects of creep and the effects of external force and concrete shrinkage, and generally may be calculated as follows. Each displacement value is obtained with consideration of the effects of constraint by rebar and PC steel, the creep coefficient used to calculate long-term displacement is multiplied for concrete in the cross section, and long-term displacement is calculated from the equilibrium calculation, taking steel in the cross section into account. Expressed in simple terms, the equation is:

$$\delta_t = \delta_L \cdot \varphi_t + \delta_{SH}, \qquad \text{(Commentary 3.3.10)}$$

where δ_t : long-term displacement;

δ_L : displacement due to external force;

δ_{SH} : displacement due to shrinkage; and

φ_t : creep coefficient used in calculation of long-term displacement.

(ii) Displacement due to shrinkage of concrete is calculated through second-order integration of the curvature caused by differences in shrinkage strain according to the material age in the cross section, which results from differences in drying conditions and in constraints on the steel in the parts of the cross section. In general, it may be calculated using:

$$\delta_{SH} = \varphi_{SH} dxdx, \qquad \text{(Commentary 3.3.11)}$$

where δ_{SH} : displacement due to shrinkage strain and

φ_{SH} : curvature due to shrinkage strain.

(iii) For concrete creep, the effects of moisture transfer and dissipation must be taken into consideration according to the structural object. In general, this may be calculated using:

$$\varphi t = \alpha \cdot \varphi, \qquad \text{(Commentary 3.3.12)}$$

where φ_t : creep coefficient used in calculation of long-term displacement;

φ: creep coefficient, according to 2.2 in "Design: Standards" Volume 1; and

α: coefficient for considering long-term progress according to the dry state of the structural object; this should be set to 1.0 or more.

Change over time in the shrinkage strain of concrete is expressed as a superposition of a function that expresses change over time in the dry-shrinkage strain of concrete and change over time in shrinkage under sealed conditions, and may be set according to Equation (Commentary 3.3.13) in the case of general concrete made with normal Portland cement.

Equation Commentary 3.3.13 was obtained through studies conducted using the numerical results of the analytical method of (1) with general concrete mixing conditions, structural specifications, and environmental conditions (temperature and humidity) as parameters, for use in the calculation of long-term displacement/deformation. The equation predicts changes over time for approximately 100 years. The equation is:

$$\varepsilon'_s(t) = \varepsilon'_{ds}(t, t_0) + \varepsilon'_{as}(t, t_s) \geq 0, \qquad \text{(Commentary 3.3.13)}$$

where $\varepsilon'_s(t)$: shrinkage strain of concrete at material age t ($\times 10^{-6}$);

$\varepsilon'_{ds}(t, t_0)$: strain due to dry-shrinkage of concrete at material age t ($\times 10^{-6}$); and

$\varepsilon'_{as}(t, t_s)$: shrinkage strain under sealed conditions of concrete from initial settling to material age t ($\times 10^{-6}$).

Here, strain due to dry-shrinkage of concrete at material age t may be calculated from:

$$\varepsilon'_{ds}(t, t_0) = \kappa_{ds} \cdot \varepsilon'_{dsh\infty}\left[1 - exp\left\{-\alpha_{ds}(t - t_0)^{\beta_{ds}}\right\}\right], \qquad \text{(Commentary 3.3.14)}$$

where $\varepsilon'_{dsh\infty}$: final value of shrinkage strain due to dry-shrinkage in the case of a 100 × 100 × 400 mm square pillar ($\times 10^{-6}$); this can be calculated from:

$$\varepsilon'_{dsh\infty} = \frac{980}{1 + 0.53e^{-0.23 \cdot (W/C - 50)}} \times \left(1 + \frac{RH - 65}{15} \cdot \left(-0.25 - 7400 \cdot e^{-0.214 \cdot W/C}\right)\right), \qquad \text{(Commentary 3.3.15)}$$

W/C : water-to-cement ratio (%) $(35 \leq W/C \leq 60\%)$;

RH : relative humidity (%) $(50 \leq RH \leq 80\%)$;'

T : material age of concrete (days);

t_0 : material age at the start of drying of concrete (days);

α_{ds},β_{ds} : coefficient that expresses the properties of progress of dry-shrinkage strain; this is calculated from:

$$\alpha_{ds} = \frac{1}{(V/S)^2}\left(200 - 2.7 \cdot \frac{W}{C}\right) \quad ただし, \alpha_{ds} \geq 0.0002$$

and (Commentary 3.3.16)

$\beta_{ds} = 0.0074 \cdot \frac{W}{C} + 0.00058 \cdot \frac{V}{S} + 0.25$

(Commentary 3.3.17)

κ_{ds} : coefficient related to the final value of dry-shrinkage strain; this is calculated from:

$\kappa_{ds} = 1 + a_{ds} \cdot (V/S - 25)$;

(Commentary 3.3.18)

V/S : volume-to-surface ratio (mm); and

a_{ds} : coefficient:

$a_{ds} = 0.00018 \cdot W/C - 0.0085$ ($35 \leq W/C < 45\%$), $a_{ds} = -0.00038$ ($45 \leq W/C \leq 60\%$).

The shrinkage strain under sealed conditions of concrete up to material age t may be calculated from:

$\varepsilon'_{as}(t, t_s) = \gamma \cdot \kappa_{as} \cdot \varepsilon'_{ash\infty}\left[1 - exp\{-\alpha_{as}(t - t_s)^{\beta_{as}}\}\right]$,

(Commentary 3.3.19)

where $\varepsilon'_{ash\infty}$: final value of shrinkage strain under sealed conditions in the case of a 100 × 100 × 400 mm square pillar (×10^{-6}); can be calculated from:

$\varepsilon'_{ash\infty} = \frac{1173}{1+2.929e^{-0.192\cdot(50-W/C)}}$; (Commentary 3.3.20)

W/C : water-to-cement ratio (%) ($35 \leq W/C \leq 60\%$);

Γ : coefficient that expresses the effect of the type of cement and admixture (when only normal Portland cement is used, 1.0 may be used);

t_s : initial coagulation (days);

α_{as} , β_{as} : coefficient that expresses properties of progress of concrete shrinkage under sealed conditions; this is derived from:

$\alpha_{as} = 0.020 \cdot \frac{W}{C} - 0.75,\ \ \alpha_{as} \geq 0.1$ and

(Commentary 3.3.21)

$\beta_{as} = -0.0085 \cdot W/C + 0.76$ (Commentary 3.3.22)

κ_{as} : coefficient related to the final value of shrinkage under sealed conditions; this is calculated from:

$\kappa_{as} = 1 + a_{as} \cdot (V/S - 25)$ (Commentary 3.3.23)

a_{as} : coefficient, defined as:

$a_{as} = -0.00014 \cdot W/C + 0.0051$ ($35 \leq W/C < 45\%$) and $a_{as} = -0.0012$ ($45 \leq W/C \leq 60\%$).

When using cement and admixtures other than normal Portland cement, the calculation should take into consideration factors such as differences in the final value of shrinkage strain under the same water-to-cement ratio used in the case of normal Portland cement.

However, the scope of application of Equations (Commentary 3.3.14) and (Commentary 3.3.19) is as follows:

Water-to-cement ratio, W/C (%) : $35 \leq W/C \leq 60$;

Relative humidity, RH (%) : $50 \leq RH \leq 80$;

Temperature, T (°C) : $15 \leq T \leq 25$;

Volume-to-surface ratio, V/S (mm): $25 \leq V/S \leq 300$; and

Minimum structural member thickness d_1 (mm) : $100 \leq d_1$.

When calculating only the final value of long-term displacement/deformation in simple-beam or cantilever-beam structures in which prestress forces or other statically indeterminate forces are not generated, the final value of the strain at the end of the design service life may be calculated from Equation (Commentary 3.3.13), and the curvature φ_{SH} of Equation (Commentary 3.3.11) may be derived according to the shrinkage strain of each part of the cross section and the constraint by steel. When doing so, the creep coefficient may be according to Equation (Commentary 3.3.12).

3.4 Setting design limit values

In order to satisfy functionality and comfort, the design limit values for displacement/deformation must be set according to the purpose of service and the function of the structural object.

Chapter 4 Verification of Watertightness

4.1 General

Verification of watertightness based on the amount of water permeation is to be performed by confirming that, for the parts of the structural object subjected to verification, the value obtained by multiplying the ratio of the design value Q_d for the amount of water permeation per unit of time to the permissible amount of water permeation $Q_{\max}$, multiplied by the structural object coefficient γ_i, is 1.0 or less:

$$\gamma_i \frac{Q_d}{Q_{max}} \leq 1.0, \tag{4.1.1}$$

where γ_i : may generally be set between 1.0 and 1.1;

$Q_{\max}$: permissible amount of water permeation per unit of time (m^3/s); and

Q_d : design value of amount of water permeation per unit of time (m^3/s).

Commentary: In structural objects that require watertightness, it is advisable to avoid cracks in the parts where watertightness is required, and to avoid the use of construction joints. The arrangement of crack-inhibiting rebar and the use of expansive additives are effective in controlling the occurrence and width of cracks. If construction joints are unavoidable, it is essential to apply joint surface treatment or other measures and to use sealing strips in vertical construction joints.

For structural objects such as tanks, the rate of decrease in stored liquid may be used as a verification metric, when targeting the structural object as a whole.

4.2 Design action and combinations of design actions

In verification of watertightness, the acting water pressure and load are to be considered as necessary.

Commentary: The action that has the greatest effect on the amount of water permeation is water pressure. When the occurrence of cracks is permitted, crack width should be calculated with consideration of the effects of permanent actions and variable actions.

4.3 Calculation of design response values

The amount of water permeation can generally be derived using the following equation:

$$Q_d = \gamma_{pn} \left(K_d \cdot A \cdot \frac{h}{L} + Q_{cjd} \right), \tag{4.3.1}$$

where K_d : design value of water permeability coefficient of concrete in structural object (m/s)

$= K_k \cdot \gamma_c$;

K_k : characteristic value of water permeability coefficient of concrete (m/s);

A : total area of concrete corresponding to the cross section of the water permeation path (m^2);

h : difference in hydraulic heads of inner and outer surfaces of the structural object (m);

L : expected value of cross-sectional thickness of the part of the structural object to be verified, corresponding to the length of the water permeation path (m); the design cross-sectional thickness may generally be used;

Q_{cjd} : design value of amount of water permeation from cracks, joints, *etc*. in parts to be verified (m^3/s);

γ_{pn} : safety coefficient that considers unevenness in the design value Q_d for amount of water permeation per unit of time; this may generally be set to 1.15; and

γ_c : material coefficient of concrete; this may generally be set to 1.0.

Commentary: It was decided to evaluate the amount of water permeation in the parts based on the water permeation coefficient of concrete and using Darcy's law. The water permeability coefficient may be derived by performing water permeability testing using the output method of the United States Bureau of Reclamation or the input method of DIN 1048. The water permeability coefficient may also be indirectly calculated from the relationship between water permeability coefficient and the strength of concrete, or from the water-to-cement ratio based on past experience and studies, or from properties related to void structures such as the void ratio of concrete or the air permeability coefficient of concrete. In that case, the accuracy of the predicted value should be evaluated to appropriately determine the safety coefficient. An equation for the relationship between the water permeability coefficient and the water-to-cement ratio based on the results of past studies is:

$$log\, K_k = 4.3 \cdot W/C - 12.5 \quad , \qquad \text{(Commentary 4.3.1)}$$

where W/C : water-to-cement (binding material) ratio.

When using this equation, the safety coefficient γ_c may be set to 1.0.

When it is not possible to prevent water permeation in cracks, joints, and other discontinuous surfaces, the parts to be verified must be set with consideration of their positions, and the effects of these must be incorporated into the evaluation of the amount of water permeation. When the water permeation process in the parts to be verified can be handled one-dimensionally, evaluation can be performed on the sum of the amounts of water permeation from the parts and the amounts of water permeation from cracks and joints, as shown in Equation (4.3.1). The amount of water permeation from cracks and joints is determined from the results of appropriate testing.

4.4 Setting design limit values

The permissible amount of water permeation is to be determined from the functions of the structural object, the treatment capacity of wastewater facilities, and the evaporation of moisture from the surface of the structural object.

Commentary: As an example of a design limit value when verifying watertightness using the rate of reduction of liquid stored in a tank or other structural object as a metric, the reduction in liquid water level over 24 hours in a PC egg-shaped digestion tank might be considered acceptable if it is within 5 mm and permeation and loss of approximately 0.1% of water by volume during the period in which the stored water circulates in a water supply storage container might be considered non-problematic.

However, even in cases in which cracks are permitted, it is advisable to maintain a small crack width so that water permeation from the cracks does not become significant. Commentary Table 4.4.1 shows guidelines for the design limit of crack width with respect to watertightness based on the required degree of watertightness and the type of dominant cross-sectional force. Even when cracks are permitted, it is advisable to keep crack width small so that water permeation from the cracks does not become significant.

Commentary Table 4.4.1 Guideline for design limit of crack width with respect to watertightness（mm）

The required degree of watertightness		For ensuring high watertightness	For ensuring general watertightness
Dominant cross-sectional force	Axial tensile force	——[1]	0.1
	Bending moment[2]	0.1	0.2

1）The stress in concrete due to sectional forces shall be in compression in all sections and the minimum compressive stress shall be not less than 0.5 N/mm^2. In case of detailed analysis, the value shall be specified separately.
2）In the case of alternating loading, the case of predominant axial tensile forces shall be followed.

Past studies used in setting design limit values for crack width all addressed structural members in which penetrating cracks have occurred. These studies have shown that the amount of water leakage is extremely small when the acting water pressure is no more than 0.9 N/mm^2 and crack width is 0.1 mm or less. **Commentary Table 4.4.1** takes into account the fact that a compression zone is secured when flexural moment dominates, which helps ensure watertightness even when the crack width is the same.

Chapter 5 Verification of Fire Resistance

If concrete satisfies the fire resistance that is required according to its covering, then it can be considered that the required performance of the structural object will not be lost due to fire, *etc*.

Commentary: The fire resistance of a concrete structural object depends mainly on the fire resistance of the covering and of the concrete itself. Therefore, in general, if the concrete satisfies the fire resistance required according to its covering, then it can be considered that the fire resistance of the structural object is ensured. However, in structural objects and structural members in which prestress is introduced, heat from fire or other sources may cause a sharp decline in capacity. When such cases are expected, careful examination of fire resistance is required.

If the value of a covering that satisfies durability under general environments shown in "Design: Standards" Volume 2, Chapter 4, augmented by approximately 20 mm, is set as the minimum value, then verification of fire resistance may be omitted.

Part 5 Seismic Design and Seismic Performance Verification

Chapter 1 General

This volume presents standard methods for conducting verification related to earthquake-resistant design and seismic resistance in structures.

Commentary: In light of earthquake damage in recent years, approaches to hazard resistance in the structures that support social infrastructure have become widespread. With the aim of reliably ensuring redundancy and robustness in structures, Chapter 2 of these Standard Specifications clearly describes the basics of aseismic design to ensure that matters to be verified and considered are clearly included in the design. On the premise that the structural planning specified in 2.2 will be properly performed, seismic resistance in structures is ensured by satisfying verification with respect to specified seismic motions, as described in this volume.

This volume also details methods for considering seismic actions as accidental actions, as shown in Chapter 4, on the precondition that performance with respect to accidental actions and major variable actions other than seismic actions is satisfied.

Chapter 2 Basics of Aseismic Design

2.1 General

(1) Aseismic design in a structure must be performed to ensure the safety of the structure during and after earthquakes, in order to prevent catastrophic damage that could result in loss of human life and property and to minimize decline in functionality that would interfere with the livelihoods and productivity of local residents.

(2) In aseismic design, a structure must be designed with excellent seismic resistance based on a thorough consideration of preconditions and the scope of application of verification related to seismic resistance, to ensure that the required seismic resistance of a structure is maintained.

Commentary: Regarding (2): In aseismic design of structures, appropriate structures related to seismic resistance are planned and designed, taking into account the function and purpose of the infrastructure comprising the designed structure. Structural planning related to seismic resistance includes examinations related to the layout plans of structures and the aseismic structure plans for the structures themselves.

The quality of structural planning related to seismic resistance requires thorough consideration of preconditions and the scope of application of the verification methods to be applied. In particular, seismic motions greater than those used in verification may occur, owing to the uncertainty of natural phenomena, and aftershocks on a scale comparable to the strength of an initial earthquake may occur over a short period of time. In aseismic structural planning, it is essential to secure minimum functionality for civil engineering structures and ensure the maintenance of sufficient redundancy and robustness to prevent sudden critical collapse, even in cases such as these.

2.2. Structural planning related to seismic resistance

2.2.1 General

(1) In structural planning related to seismic resistance, the layout plans of structures must be thoroughly examined to ensure that the network and other functions of the structures are impaired as little as possible during and after an earthquake.

(2) In aseismic structural planning, the structural morphology and structural details must be set with damage processes, failure mechanisms, and dynamic response properties of structures taken into consideration, to ensure that the structures satisfy the required seismic resistance and possess redundancy and robustness.

Commentary: Regarding (1): In structural planning related to seismic resistance, a thorough examination must be undertaken from the perspective of route plans (structure layout plans) in the infrastructure constructed linearly from multiple structures, as in roads and railways. In particular, it is advisable to thoroughly consider not only the structures themselves but also redundancy in terms of networks, *i.e.*, non-physical approaches such as substitution of structures. It is also necessary to thoroughly examine the equipment, machinery, *etc*. placed in structures.

Regarding (2): In seismic-resistance-related structural planning, aseismic structural plans for imbuing individual structures with the required seismic resistance must be examined with full consideration of the effects of seismic action on structural frames, on the users of the structures, and on the public, based on the results of the examination in (1). The scope of application of the seismic-resistance-related verification techniques described in Chapter 3 and later Chapters in this volume must also be fully considered.

Verification related to seismic resistance is based on techniques for setting the seismic motion used in verification and confirming that damage to individual structural members remains at or below a certain level. Therefore, seismic motion above the set strength must be given consideration in seismic structural planning for a structure.

The seismic structural planning performed here is a major precondition for ensuring the set seismic-resistance performance using the specified performance verification methods, and for imbuing a structure with redundancy and robustness without performing explicit verification.

2.2.2 Layout planning for structures

(1) Consider the safety of people who use the structure and others nearby during an earthquake.

(2) Investigate and consider the disaster history, *etc*. of the area around the structure.

(3) Consider how to prevent subsoil migration caused by ground liquefaction from adversely affecting the structure.

(4) Consider regional disaster readiness plans related to tsunamis, *etc*.

(5) When the location of an active fault is clear and a structure that spans that location is to be constructed, consider not only the seismic resistance of the structure itself but also countermeasures in terms of non-physical approaches, such as substitutability of systems.

Commentary: Regarding (1): Depending on the behavior of a structure during an earthquake, the safety of people using it and those living in its vicinity may be at risk. In structural planning, full consideration must be given to preventing such risks.

Regarding (2): A landslide is an example of a major disaster associated with an earthquake. It is assumed that landslides not only damage structures but may also cause loss of life and property in users and surrounding residents. Therefore, it is important in the structural planning stage to investigate the disaster history in the vicinity of the structure and the potential dangers in said location, and to use this information in structural planning.

Regarding (3): Ground liquefaction and resulting subsoil migration have significant effects on the seismic resistance of below-ground structures, *etc*. Therefore, in ground where liquefaction may occur due to earthquakes, measures should be taken to combat subsoil migration and liquefaction through means such as foundation improvement work. If such measures cannot be taken, then it is necessary to thoroughly examine the stability of the structure in terms of its design during the occurrence of liquefaction, so that the action of subsoil migration caused by liquefaction will not adversely affect the structure.

Regarding (4): The prediction technologies needed to incorporate the actions of tsunamis into the design of structures have not yet been established. Therefore, taking the case of a bridge as an example, it may be possible to secure space under the girders to accommodate the height of tsunamis, or to integrate upper and lower parts that lack bearings, by using regional disaster readiness plans as a reference.

Regarding (5): If the position of an active fault is known, then potential measures against forced displacement caused by action of the fault include making structures continuous or implementing advanced reinforcement in the case of above-ground structures, and using increased cross sections and flexibility, duplication, and insulation of internal facilities from the structure in the case of below-ground structures. However, when forced displacement is extreme in scale and these noted measures are technically difficult, controlling the decline in functionality of the system as a whole, including in structures, also requires consideration of measures from non-physical approaches, such as the substitution of structures.

2.2.3 Seismic structural planning for structures

(1) Structures must be planned with sufficient energy absorption capacity with respect to seismic action.

(2) Structures must be planned so as to allow easy repair following an earthquake.

(3) Structures must be planned so that dynamic response properties due to an earthquake are not complicated.

(4) Structures must be planned so as to possess redundancy and robustness.

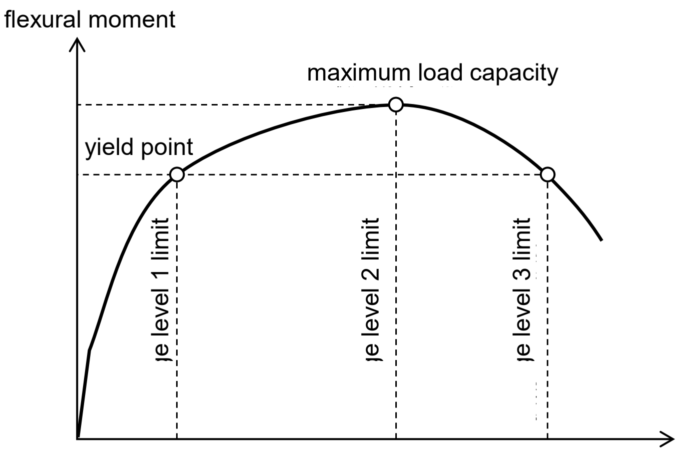
Commentary: Regarding (1): In structures, structural members and parts that permit plasticization are to be arranged and the permissible level of damage is to be set so as to satisfy the required seismic resistance. In order for the structure to be able to properly absorb energy as a system overall, the degree of plasticization (including no damage) must be set for each structural member or part that makes up the structure. In general, a degree of plasticization and absorption of energy with respect to seismic motion of the greatest strength expected at the site of construction should be permitted for some structural members/parts. In this case, however, it may be necessary to repair earthquake damage at an early stage after the earthquake in order to restore the functionality of the structure. Therefore, it is important to set the damage level of structural members/parts with consideration of the ease of repair and the time and cost required, and, having considered combinations of the degree of capacity conferred and the allowable damage level for each structural member/part, in order to properly arrange these throughout the structure.

Commentary Table 2.2.1 presents examples of damage levels and damage states of structural members, along with corresponding repair methods. Commentary Figure 2.2.1 presents examples that associate damage levels with the mechanical properties of general reinforced-concrete rod members subjected to flexural moments.

Because a structure has structural elements arranged at the connections between structural members, the occurrence of deformation or damage to not only structural members but also these structural elements during an earthquake should be taken into consideration in the planning stage. When damage has occurred to soundproof walls, signposts, or other incidental objects installed during the service of the structure, this must also be taken into consideration.

Commentary Table 2.2.1 Examples of damage levels and damage states, and their corresponding repair methods

	Damage state	Examples of repair method
Damage level 1	No damage	No repair (or measures for durability if necessary)
Damage level 2	Damage that may need repair	Filling of cracks and patch repair if necessary
Damage level 3	Damage that is necessary to repair	Filling of cracks and patch repair correction of hoop reinforcement if necessary



Commentary Figure 2.2.1 Example of damage level limits of structural member

It is recommended that a structure be given a fracture morphology with sufficient toughness, and it is recommended in principle that the structural members that compose the structure be given a flexural fracture morphology. Depending on the structure, giving all structural members a flexural fracture morphology may be difficult. In that case, the structure as a whole must be given mechanisms for toughness, even if some structural members undergo brittle failure. Therefore, in structural planning, it is important to give the structure sufficient toughness through means such as arranging a sufficient amount of hoop reinforcement in parts where the occurrence of structural plastic hinges is permitted, to prevent excessive deformation and stress concentration due to earthquake effects and to prevent the collapse of the structure overall. Specifically, approximately twice the flexural-to-shear capacity ratio of the plastic hinge parts should be secured. Even if plastic hinge parts possess toughness, consideration should be given so that shear failure does not occur immediately when the limit value is exceeded and so that repeated seismic motion does not lead to fracture of longitudinal rebar. If not only deformation performance but also flexural capacity is required, then the application of high-strength rebar of SD490 or higher may be considered. In this case, too, it is important to confer sufficient toughness, based on the material properties of high-strength rebar such as its small fracture elongation. Cases in which it is necessary to increase covering for purposes of durability may fall outside the scope of the general seismic resistance verification methods specified in this volume, requiring the implementation of finite-element analysis or other considerations as necessary.

When load capacity or plastic deformation is expected of beams, pillars, or other structural members in contact with junctions, then haunches must be designed into structural members and sufficient shear reinforcing bars must be arranged in the junctions to maintain sufficient rigidity and load capacity in the structural member junctions. In particular, because structural members made with high-strength concrete and high-strength rebar generally have a small cross section relative to their load capacity, the junctions between these structural members require particularly thorough consideration. For structures such as wall-like structures in which seismic force acts mainly as an in-plane force, the inability to expect significant deformation performance must be taken into consideration, and the structures must be designed with sufficient shear capacity to prevent brittle failure.

Care is needed to prevent peeling of the covering concrete from endangering lives and property around the structure. When arranging plastic hinges in structural members, consideration should be given to restriction of damage by appropriately setting the limit states of the structure based on constraints and by taking precautionary measures to prevent peeling of covering concrete when there is risk to the property and the lives of users of spaces beneath girders, surrounding residents, users of intersecting elements, *etc*. due to the peeling of covering concrete during and after an earthquake.

For below-ground structures, plasticization of structural members may cause inundation from the outside or leakage of content. Taking this into account, it is important to consider structures that are capable of absorbing energy. Sufficient consideration must also be given to the degree of deformation of joints in order to prevent the entry of water from the joints between structural members.

Regarding (2): When plastic hinges are built into structural members, the plastic hinge locations should be selected from among structural members that allow easy discovery of damage and prompt repair. In general, structural members in the above-ground portion should be given priority in selection. Because it is difficult to repair structural members that have an outer circumference in contact with the ground, such as the side walls of a cut-and-cover tunnel, selection should be made from among other structural members with consideration of plasticization, such as center pillars and center deck slabs. In the case of bridges with a Rahmen structure in which pillars and beams are rigidly connected, a structure that allows absorption of energy mainly by the pillars should be used, with plasticization of beams kept negligible. In the case of a structure having a foundation, in light of the difficulty of repair of the foundation, primary plasticization should be built into the bridge piers with the foundation limited to secondary plasticization. Therefore, in such cases, limit states should be set appropriately based on constraints.

Regarding (3):

Consideration must be given to aligning the center of rigidity of the structure with the center of action as much as possible to prevent significant horizontal torsion in the structure. When the center of rigidity and the center of action of the structure cannot be aligned, appropriate consideration must be given to the effects of torsion on seismic resistance, such as by enhancing seismic-resistance performance or by setting appropriate design limit values with consideration of the effects of torsion.

Seismic isolation technologies and seismic control technologies should also be considered. Seismic isolation can enhance the damping performance of a relatively short-period structure, while seismic control that include energy absorption mechanisms can control the resonance with seismic motion in a long-period structure. When applying these, the non-linearity of the mechanism itself and its behavior during an earthquake, the possibility of resonance with seismic motion due to lengthening of period, and other factors should be examined.

Coupled actions with adjacent structures and interactions by facilities incidental to the structure are to be considered as necessary. When the dynamic properties, foundation structure, ground conditions, and so on differ among adjacent structures, the response by one structure may affect the response of others, leading to unexpected damage.

In the case of below-ground structures, floating due to ground liquefaction should be considered. The displacement/deformation behavior and stability of surrounding ground during an earthquake are important factors in the seismic design of below-ground structures. In structures such as shield tunnels and utility tunnels, displacement distribution in conduit lines during an earthquake and three-dimensional distribution in not only the horizontal plane but also in the direction of depth are important with respect to displacement of surrounding ground during an earthquake. Therefore, it is necessary to fully assess the seismic response of surface subsoil. In order for a below-ground structure to maintain its specified seismic-resistance performance, it is advisable to actively adopt structures and materials that enhance flexibility.

Regarding (4): In design, seismic actions are conclusively defined for earthquakes, which are probabilistic events. In reality, however, there is a possibility that seismic motion in excess of that used in verification will act on structures. Therefore, structures must be designed so that the structure does not immediately reach critical collapse after major seismic action that exceeds the seismic motion used in verification. For a conclusively defined seismic action, these standards define a limit value that does not lead to sudden failure until the structure reaches the maximum displacement or angle of rotation that maintains a yield flexural moment. In the past, in addition to verification that structural members have sufficient safety with respect to shear failure, the redundancy of structures has been considered for cases in which the seismic motion used in verification is exceeded, from varied standpoints including structural specifications and structural planning. Conferring redundancy on a structure is effective in avoiding the collapse of the structure even when it is subject to multiple major earthquakes. At present, the stage has not been reached at which seismic resistance and residual load capacity can be sufficiently evaluated in a structure that has been subjected to a major earthquake. Therefore, structural planning that confers redundancy on structures is essential in the design stage.

Chapter 3 Principles of Verification Related to Seismic Resistance

3.1 General

In verification related to seismic resistance in structures, performance levels must be set according to the purpose of use of a structure, and appropriate verification metrics must be used to verify that the performance levels are satisfied.

3.2 Actions

(1) In the verification of the seismic resistance of a structure, the permanent actions, variable actions, and accidental actions assumed during the design service life are to be considered in appropriate combinations, according to the required performance of the structure. The accidental actions considered in this volume are seismic actions.
(2) Seismic action refers to design seismic motion and all actions caused by it, and is to be set appropriately according to the response analytical method and the analytical model of the structure.
(3) Actions other than seismic action are to be considered appropriately according to the properties of the actions and their effects on the limit states to be verified.
(4) Combinations of actions are to be appropriately determined according to the type of structure and the required performance.

Commentary: Actions in the seismic design of structures are generally classified into permanent actions, variable actions, and accidental actions, according to their sustainability, degree of variation, and frequency of occurrence. In this volume, seismic action is covered as an accidental action. Permanent actions and secondary variable actions are to be set appropriately, in combination with seismic action.

The following are generally considered to be the effects of earthquakes on structures:

① Inertial force due to the mass of the structure and the mass loaded onto the structure;

② Dynamic interaction between the structure and the ground;

③ Hydrodynamic pressure during an earthquake; and

④ Liquefaction of the ground and subsoil migration caused by liquefaction

Masses that give rise to inertial force through vibration during an earthquake include the mass of the structure and the mass loaded onto the structure. Considering cases in which mass loaded onto the structure acts at the same time as an earthquake, the masses that form the permanent load and the secondary variable load should generally be taken into account.

Actions caused by the dynamic interaction between the structure and the ground arise due to relative differences in the dynamic response properties of the two. Structures for which this effect must be taken into account include abutments, retaining walls, below-ground structures, and foundation structures such as piles and caissons.

In a tank or other structure containing liquid, hydrodynamic pressure associated with the oscillation of liquid and hydrodynamic pressure caused by loaded mass are generated during an earthquake. These must be taken into consideration.

Taking measures to prevent ground liquefaction is basic to seismic design. When such measures are technically difficult or diseconomies are significant, design must fully consider the effects that ground liquefaction has on the improvement of seismic resistance in the structure. If the ground surface is sloped or if asymmetrical soil pressure constantly exerts action, then liquefaction may result in subsoil migration, the effects of which must be taken into consideration.

3.3 Level of seismic resistance

(1) The level of seismic resistance sets seismic-resistance performance as the level of performance that comprehensively takes into account safety and usability during and after an earthquake, as well as post-earthquake reparability. It further determines the structure limit states that satisfy seismic-resistance performance.

(2) The seismic-resistance performance that a structure should possess is in principle determined with consideration of the seismic motion used in verification; the effects of damage to the structure on human life and property; the effects on evacuation, rescue, emergency activities, and secondary disaster prevention activities; the effects on post-earthquake daily life and economic activities in the region; the difficulty of restoration; the construction costs; and other factors. In general cases, a structure should possess the following three levels of seismic-resistance performance:

(i) Seismic-resistance performance 1: The structure is capable of retaining functionality during an earthquake and of supporting use after an earthquake without repair and with sound functionality.

(ii) Seismic-resistance performance 2: Functionality can be restored in a short time following an earthquake, and reinforcement is not required.

(iii) Seismic-resistance performance 3: The structure as a whole does not collapse due to an earthquake.

Commentary: <u>Regarding (2)</u>: For seismic resistance as a required aspect of the performance of a structure, the level of performance is set as seismic-resistance performance ranging from 1 to 3.

Seismic-resistance performance 1 is achieved if the response of a structure during an earthquake remains small and post-earthquake residual deformation of the structure remains within a sufficiently small range. Therefore, the limit state for seismic-resistance performance 1 may generally be set to a state in which the structure remains within the elastic range.

Seismic-resistance performance 2 addresses reparability following an earthquake. Because repair in the shortest time possible is required of structures that are used in regional restoration activities following an earthquake, it is important to set the damage level of structural members and structural elements with consideration of the ease of repair and the time and cost required, and to examine the degree of difficulty of repairing the structure as a whole.

Seismic-resistance performance 3 requires the safety of the structure following an earthquake. If the damage is not critical, however, it is possible to restore functionality by carrying out long-term repair or strengthening. In general, if vertical structural members do not undergo shear failure and if the structure remains self-supporting after the earthquake, then seismic-resistance performance 3 can be considered to be satisfied.

3.4 Verification

(1) Verification related to seismic resistance is performed following consideration of expected actions, by confirming that limit states set according to seismic performance are not reached.

(2) Verification of seismic resistance of structures is performed by calculating design response values under expected seismic motion using a predetermined safety coefficient and by confirming that these do not exceed design limit values:

$$\gamma i \cdot S_d / R_d \leq 1.0, \qquad (3.4.1)$$

where S_d : design response value;

R_d : design limit value; and

γi : structure coefficient.

Commentary: Regarding (1): Because a structure has not only primary structural members such as rod members and plane members, but also bridge bearings, bridge fall prevention mechanisms, seismic isolation mechanisms, and other structural elements arranged at the connections between structural members, the occurrence of deformation or damage to these during an earthquake must be taken into consideration during verification. When damage has occurred to soundproof walls, signposts, or other incidental objects installed during the service of the structure, this must also be taken into consideration.

Regarding (2): From the standpoint of whether functional maintenance or repair/reinforcement is necessary in the event of an earthquake, displacement/deformation of a structure or structural member generally serves as the metric for response values. Depending on the structural model used, stress or strain in the material used may be adopted as a response value. Because constituent structural members behave non-linearly, structural members for which the response value is closest to the limit value will not necessarily remain the same throughout the continuation of seismic motion. Moreover, because the seismic-resistance performance of a structure is not necessarily determined by the single structural member that has a response value closest to the limit value, during verification, response during an earthquake must be considered for all structural members.

3.5 Safety coefficients

Safety coefficients are determined according to seismic-resistance performance, taking into consideration the properties of seismic motion as an action, the precision of the method for calculating response values, and the mechanical performance of the structure when subjected to repetitive major deformation actions, among other factors. In general, the following may be used as safety coefficients used in the verification of seismic resistance.

(1) The safety coefficient used in the verification of seismic-resistance performance 1 is generally set to 1.0. However, the safety coefficient used when verifying shear force is to be set according to the verification of seismic-resistance performance 2 and seismic-resistance performance 3.

(2) The safety coefficients used in the verification of seismic-resistance performance 2 and seismic-resistance performance 3 may be set as follows:

(i) The action coefficient is set to 1.0.

(ii) The material coefficient is generally set to 1.0 when used to calculate response values.

(iii) The structural member coefficient is generally set to 1.0 when used to calculate response values. When used to calculate limit values, the structural member coefficient is increased in order to increase safety with respect to shear in rod members that are subject to positive and negative alternating actions. However, the structural member coefficient for displacement is set to 1.0.

(iv) The structural analysis coefficient is determined appropriately in line with the analytical method.

(v) The structure coefficient is determined based on the importance of the structure.

(3) The safety coefficient when performing verification of seismic resistance using three-dimensional or two-dimensional nonlinear finite-element analysis is to be based on "Design: Standards" Volume 10.

Commentary: Regarding (1): When verifying that seismic-resistance performance 1 does not cause shear failure, the safety coefficient is to be set according to the verification of seismic-resistance performance 2 and seismic-resistance performance 3.

Regarding (2): In general, when performing design to prevent shear failure or torsional failure from occurring in structural members, the structure will exhibit toughness in its behavior with nearly no decline in load-bearing capacity even after flexural yield of structural members.

The response of a structure during an earthquake is affected by seismic motion as well as by the properties of the structural members. The rigidity of a structural member has considerable effect on its response displacement. However, when the response displacement reaches a level corresponding to the yield displacement or flexural capacity of the structural member, the shear force that corresponds to the yield flexural moment or maximum flexural moment of the structural member exerts action. In the calculation of structural member rigidity, yield flexural moment, and maximum flexural moment, the material coefficients, structural member coefficients, and material characteristic values of the materials used in the verification of safety with respect to structural member failure as shown in "Design: Standards" Volume 3 are set for the purpose of calculating the lower limit value of flexural capacity. When the above-mentioned material characteristic values, material coefficients, and member coefficients are used in verification related to seismic resistance, the rigidity, yield flexural moment, and flexural capacity of the structural member will be evaluated on the low side, meaning that, in response analysis, response displacement will be evaluated on the high side but generated shear force will be evaluated on the low side. Moreover, it is known that shear capacity of structural members decreases when subjected to repeated loads with large deformation. Therefore, properly verifying safety with respect to shear force during seismic action requires calculation of shear force action using the design value of the material used based on the actual strength of the material, and requires evaluating the decline in shear capacity caused by repeated large deformations.

Therefore, in verification, the general rule is to perform response analysis multiple times by changing the material characteristic values, material coefficients, structural member coefficients, *etc*. for cases of verifying response displacement and for cases of verifying that shear failure does not occur. Here, the safety coefficient is set as shown in **Commentary Table 3.5.1**, in line with the principles noted below.

Commentary Table 3.5.1 Standard safety coefficients and material correction coefficients

Safety coeff. / correction coeff. — Seismic performance		Material coeff. γm: Concrete γc	Material coeff. γm: Steel γs	Structural member coeff. γb	Structural analysis coeff. γa	Action coefficient γf	Structure coeff. γi	Material correction coeff. of rebar strength ρm
Seismic resistance performance 1	Response value & limit value	1.0	1.0	1.0	1.0	1.0	1.0	1.0 ***
Seismic resistance performance 2, 3	Response value	1.0	1.0	1.0	1.0–1.2	1.0	1.0–1.2	Displacement : 1.0 Shear force : 1.2
	Limit value	1.3	1.0 or 1.05	1.0 * 1.1–1.3 **				1.0

* Limit value of displacement

** For the shear capacity of rod members subject to positive and negative alternating actions, the coefficient is increased by 1.2, which means the contribution from concrete Vcd is set to be 1.3×1.2=1.56, while the contribution from stirrups for shear reinforcement Vsd is set to be 1.1×1.2=1.32. However, if it can be assured that shear failure does not occur after flexural yielding according to experiment or judgement of failure model, the structural member coefficient may not need to be increased. Moreover, in the case of shear capacity calculation for the examination of flexural-to-shear capacity ratio, the structural member coefficient may not need to be increased either.

*** The safety coefficient used when verifying shear force is to be set according to the verification of seismic-resistance performance 2 and seismic-resistance performance 3

(i) Because the seismic motion used in the verification of seismic resistance is an accidental action, the action coefficient is set to 1.0.

(ii) The material coefficients used to calculate the response values of the structure are all set to 1.0. In verification of displacement/deformation of the structure, the characteristic values of the materials used, with lower-limit values assumed, are used as the design values of the materials. For the design value of the tensile yield strength of steel, the lower limit of yield strength under JIS standards is used as the characteristic value.

(iv) Of the structural member coefficients used in the calculation of limit values, the structural member coefficient for the limit value of displacement is set to 1.0. Because no sudden decline in load-bearing capacity occurs in structural members in which flexural behavior dominates, the structural member coefficient related to displacement may be set to 1.0. In addition, in structural members that are in an ultimate state due to shear failure following flexural yield, increasing the structural member coefficient for shear capacity (below) has the effect of increasing transverse rebar ratio and preventing a sudden decline in load-bearing capacity. Therefore, the structural member coefficient related to displacement may be set to 1.0.

When verifying that shear failure does not occur in structural members, the actual strength of the steel is used as the design value. When using steel that conforms to JIS standards, the design tensile yield strength may be set to the value obtained by using the lower-limit value of yield strength under JIS standards as the characteristic value, and multiplying this by the material correction coefficient ρ_m. The material correction coefficient ρ_m may generally be set to 1.2. The characteristic value of the strength of the concrete and the material coefficient may be set to the same values used when verifying displacement/deformation of the structure.

Considering the decline in shear capacity due to repeated large deformation and considering safety with respect to excessive seismic input, in the calculation of shear capacity when there is a possibility of being subjected to repeated large deformation loading that exceeds the maximum load capacity, the structural member coefficient must be increased beyond the value shown in 2.4.3.2 in "Design: Standards" Volume 3. In general, it should be multiplied by approximately 1.2. However, when it has been confirmed through testing or other appropriate means that a sharp decline in load capacity will not occur even if ultimate displacement of the structural member is exceeded, or when it has been determined that a shear failure mode after flexural yield will not occur as the failure mode, it is not necessary to increase the structural member coefficient.

Regarding the effects of the actual strength of rebar, in principle, verification is to be performed based on analysis with the material correction coefficient ρ_m set to 1.2 in the examination of shear failure, and based on analysis with ρ_m set to 1.0 in the examination of deformation.

(iv) Uncertainties related to structural analysis include accuracy and other uncertainties in analytical methods, as well as uncertainties in the ground properties by which seismic motion, set for the engineering bedrock surface, is transmitted. When using the standard analytical methods shown in these Standard Specifications, the structural analysis coefficient may generally be set to 1.0. If the buckling effect of the longitudinal rebar is ignored in response analysis, then measures such as increasing the structural analysis coefficient must be taken.

(v) The structure coefficient may generally be set to 1.0. However, a value larger than 1.0 may be set for particularly important structures.

Chapter 4 Seismic Motion Used in Verification

4.1 General

(1) The seismic motion properties required in verification are to be identified with consideration of factors including the degree of seismic activity around the construction site, seismic source properties, and the propagation of seismic motion from the source to the construction site.

(2) The seismic motion used in verification is generally expressed as a time-history acceleration waveform.

(3) The set position of seismic motion is to be the engineering bedrock surface at the site of construction.

(4) The direction in which seismic motion is allowed to act is to be set in the direction considered most severe with respect to the structure. Depending on the properties of the structure and other factors, however, the vertical direction is also to be taken into consideration, and verification is to be performed using multiple-direction simultaneous input.

Commentary: Regarding (1): In general, the seismic motion used in verification may be set based on a method that considers multiple observed waveforms and the crustal failure process in the focal area. However, creating such a time-history acceleration waveform requires considerable effort, including detailed surveying of ground conditions at the construction site, investigation of the positions and degree of activity of active faults, and investigation of the effects of plate boundary earthquakes. As an alternative method, a simulated seismic-motion waveform containing a vibration component that would have a large effect on the structure may be used.

Regarding (2): Accurately evaluating the safety of a concrete structure during an earthquake requires nonlinear dynamic response analysis with the time-history waveform as an input. The seismic motion waveform can be expressed using displacement, velocity, and acceleration. In the vibration equation, however, because the input seismic motion is generally given as an acceleration waveform, a time-history acceleration waveform is to be used as the waveform representing seismic motion.

Regarding (3): To appropriately evaluate the effects of surface subsoil at the site of a structure, the seismic motion used in the verification of seismic-resistance performance is set at the engineering bedrock surface. The engineering bedrock surface differs according to the definition of seismic motion used in verification. In general, however, it can be set to the top surface of a continuous stratum in which shear elastic wave velocity (in the case of microstrain) is 400 m/s or more, or for which the N value is 50 or higher in sandy soil or 30 or higher in cohesive soil.

When setting seismic motion of the engineering bedrock at the construction site, if sufficient seismic observation records such as vertical array observations have been obtained at that point, then the free bedrock wave may be estimated through identification of the rigidity and damping properties of the ground, and, with the properties of seismic motion at that point assessed and incorporated, may be set as the seismic motion used in verification. The rigidity and damping calculated in identification may also be used as initial values in forward analysis.

Regarding (4): Seismic force is generally dominated by seismic motion in the horizontal direction. Therefore, except for special cases, horizontal seismic motion should be taken into consideration in the verification of seismic resistance performance. Although horizontal seismic motion acts in all directions, if the direction considered to be the most severe for the structure is known, then examination is to be conducted for that direction. If this direction is not known, then examination should address multiple horizontal directions.

The relationship between dimensions of the structure model and the input direction of seismic motion can be considered as shown in **Commentary Table 4.2.1**, with degree of freedom increasing significantly as the dimensions of the design model increase. Along with the need to determine the dimensions of the analytical model with the properties of the target structure taken into account, the direction from which seismic motion is inputted into the analytical model must also be determined, with consideration of the seismic motion properties and the safety of facilities at the construction site.

Commentary Table 4.2.1, Combinations of structure models and input seismic motions

<table>
<tr><td>Model</td><td colspan="7">Direction of seismic motion</td></tr>
<tr><td>One dimension</td><td colspan="7">One horizontal direction</td></tr>
<tr><td rowspan="2">Two dimensions</td><td colspan="3" rowspan="2">One horizontal direction</td><td colspan="4">One horizontal direction＋vertical direction</td></tr>
<tr><td colspan="2">Independent input</td><td colspan="2">Simultaneous input</td></tr>
<tr><td rowspan="2">Three dimensions</td><td rowspan="2">One horizontal direction</td><td colspan="2">One horizontal direction＋vertical direction</td><td colspan="2">Two horizontal directions</td><td colspan="2">Two horizontal directions＋ vertical direction</td></tr>
<tr><td>Independent input</td><td>Simultaneous input</td><td>Independent input</td><td>Simultaneous input</td><td>Independent input</td><td>Simultaneous input</td></tr>
</table>

In seismic-resistance performance 2 and seismic-resistance performance 3, if it is determined that the effects of the vertical component of seismic motion are not negligible, owing to the type, form, distribution of rigidity, and other properties of the structure, then verification must also be performed for seismic force in the vertical direction. The magnitude of the seismic input in this case may be set to 1/2th to 2/3rds that in the horizontal direction, based on records of past seismic motion. However, when strictly evaluating the response behavior of the structure, seismic motion in both the vertical and horizontal directions are to be inputted at the same time.

Curved bridges, structures affected by torsion, or pillars on which an axial force with large eccentricity acts may present multi-axial responses even when seismic motion is unidirectional. In such a case, the effects of simultaneous input in multiple directions should be considered, by selecting an appropriate combination of seismic motions in the two horizontal directions and in the vertical direction.

4.2 Setting of seismic action

(1) In principle, the seismic motion used in verification should be set in two levels.

(2) Level 1 seismic motion should be set to seismic motion that is relatively likely to occur during the design service period.

(3) Level 2 seismic motion should be seismic motion of the highest strength at the point in question, and, of the following seismic motions, should generally be that which has the greatest effect:

(i) Seismic motion due to inland active faults directly below or in the vicinity or

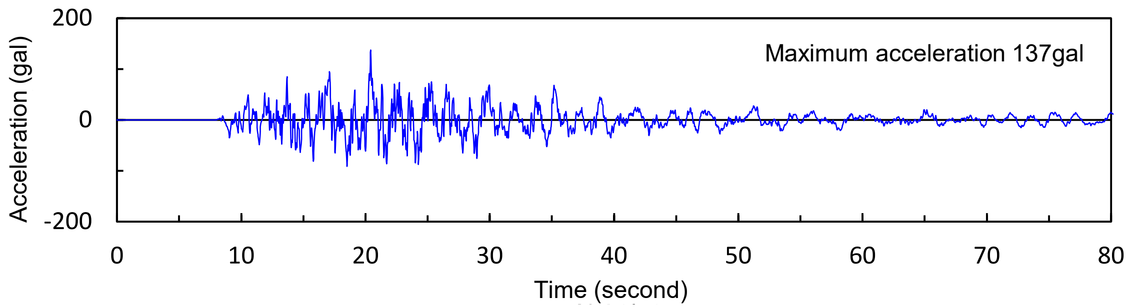
(ii) Seismic motion caused by a large-scale plate boundary earthquake occurring in the vicinity of land.

Commentary: Regarding (1): For general civil-engineering structures, seismic resistance to Level 1 and Level 2 seismic motions should be verified. However, the seismic motion used in verification may be divided into additional stages as needed.

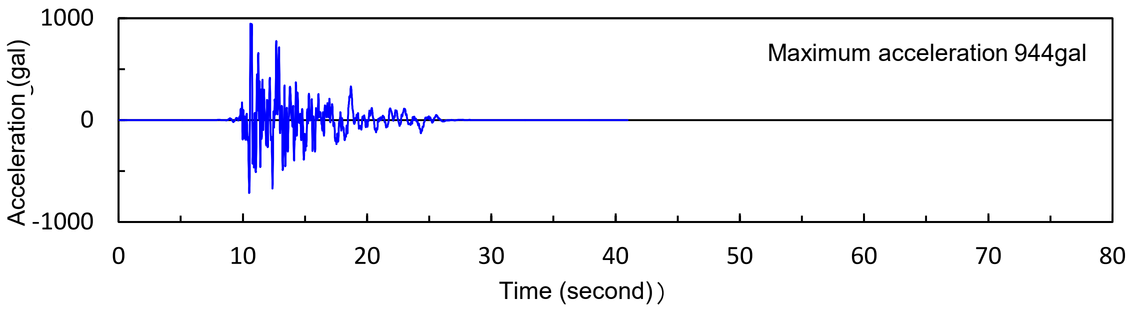
Regarding (2) and (3): Setting an appropriate frequency for the structure and using probabilistic hazard analysis, *etc*. to set Level 1 seismic motion can be considered in line with the aims of this volume. Level 2 seismic motion is to be set by comprehensively evaluating past earthquake occurrences, active fault survey results, and the results of research into seismic tectonics.

As examples of simulated seismic waveforms, Commentary Figure 4.2.1 shows Level 1 seismic motion, **Commentary Figure 4.2.2** shows Level 2 inland seismic motion, and **Commentary Figure 4.2.3** shows Level 2 plate boundary-type (marine-type) seismic motion. Commentary **Figure 4.2.4** also shows the target acceleration response spectra (attenuation constant h = 0.05) of the Level 1 seismic motion waveform and the Level 2 seismic motion waveform. These are examples of seismic motion in engineering bedrock.

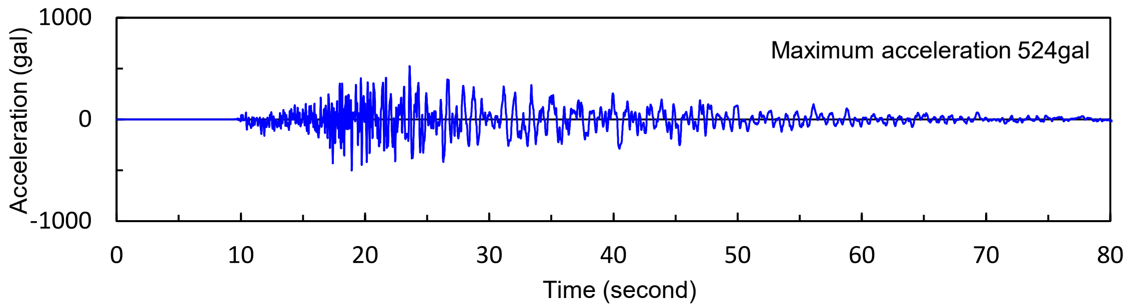
The magnitude and frequency of seismic motion are known to differ by region. When seismic activity in the region of the construction site, the distance from seismic sources, and effects can be appropriately taken into consideration, the set acceleration waveform multiplied by an appropriate attenuation coefficient may be used.



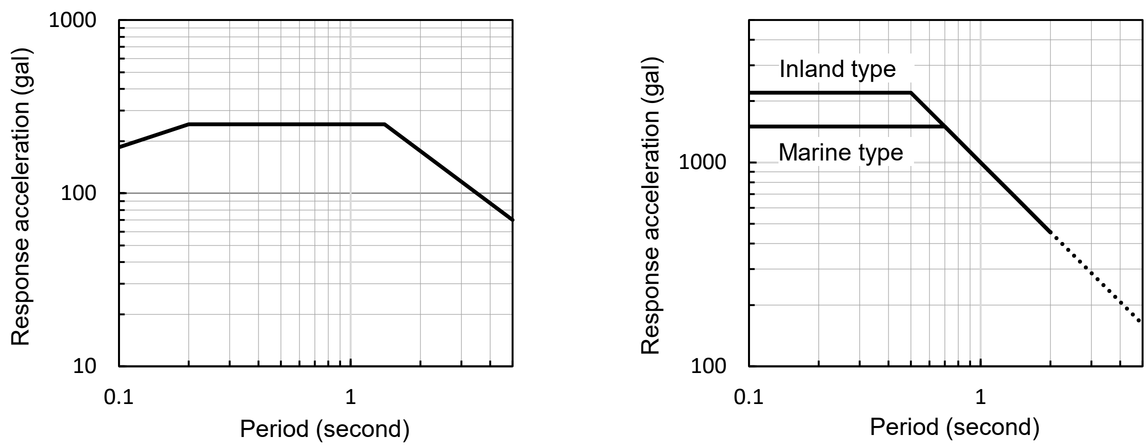
Commentary Figure 4.2.1 An example of time-history acceleration waveform of Level 1 seismic motion



Commentary Figure 4.2.2 An example of time-history acceleration waveform of Level 2 seismic motion (inland type)



Commentary Figure 4.2.3 An example of time-history acceleration waveform of Level 2 seismic motion (marine type)



(a) Level 1 seismic motion (b) Level 2 seismic motion

Commentary Figure 4.2.4 Target acceleration response spectra (attenuation constant h = 0.05)

Chapter 5 Analytical Models

5.1 General

(1) An analytical method with demonstrated validity and scope of application must be selected.

(2) An appropriate model of the structure is set according to the analytical method used for verification.

(3) The scope of analysis and the analytical dimensions are set and the structure and its boundary conditions are modeled, according to the form of the structure, the directions of actions, and its responses to actions.

(4) In modeling, the scope of analysis, being composed of the structure, the ground, boundary elements, *etc*., is set according to the range over which response occurs.

(5) When setting a scope of analysis that includes the ground, modeling that is able to appropriately consider its effects is performed.

Commentary: Regarding (3), (4), and (5): In modeling the structure, the range over which responses are generated by actions must be collectively set as the scope subject to analysis. However, if the effects of interactions among structural elements within the range over which responses occur is small, or if the effects can be taken into consideration by the boundary conditions of the area of analysis, modeling may be performed with the scope of analysis separated into structural elements.

5.2 Modeling of structures

5.2.1 General

(1) The structure must be modeled three-dimensionally or two-dimensionally as an aggregate of structural members and junctions, while taking its form and the directions of actions into account.

(2) Depending on its form, the structure may be analyzed using a simplified structural model consisting of slabs, beams, pillars, Rahmen frames, arches, shells, and combinations of these.

(3) Structural members may be modeled as rod members or plane members.

(4) Rod members or plane members may be modeled using finite elements or beam elements.

(5) Depending on the properties of structural members and the structure, the effects of structural member junctions must be modeled as necessary.

(6) When structural members are in contact with each other via bearings, *etc*., modeling is to take into consideration the effects of the mechanical properties of the bearings, *etc*. on the response of the structure.

(7) When effects of geometric non-linearity are not negligible, this must be taken into consideration.

Commentary: Regarding (1): When modeling a structure, pillars and beams should be modeled as rod members, and walls, floors, and other members with a surface spread should be modeled as slabs, shells, or other plane members, with the structure modeled as an aggregate of these. Because the structure is composed of structural members joined three-dimensionally, strictly speaking, the model should be a three-dimensional one. Depending on the directions of actions and the response characteristics of the structure, however, in some cases it is necessary to only consider response in a two-dimensional plane.

Regarding (2): Structural analysis may be performed using a simplified structural model of the structure consisting of slabs, beams, pillars, Rahmen frames, arches, shells, and combinations of these. Structural analysis of these and related verification methods should be according to "Design: Standards" Volume 1.

Regarding (3) and (4): There are several methods for modeling structures. Modeling bears a close relationship to analytical theory. Therefore, the modeling method must be selected with consideration of the applicability of analytical theory. In these Standard Specifications, the method of modeling three-dimensional and two-dimensional element domains as an aggregate is defined as "modeling with finite elements" and the method of modeling one-dimensional element domains as an aggregate is defined as "modeling with beam elements."

Rod members can be modeled using beam elements or finite elements, with the response values that can be calculated differing for each model. Therefore, modeling must be performed by an appropriate method with consideration of the response values used in verification. Modeling with beam elements is generally effective. Plane members are in principle modeled using finite elements, but modeling these with beam elements by an appropriate method is also possible.

Regarding (5): Damage to junctions between structural members can greatly affect the deformation of the structure as a whole. Therefore, when this damage is not negligible, the junctions must be modeled appropriately.

Because structural members made with high-strength concrete and high-strength rebar generally have a small cross section relative to their load capacity, modeling the junctions between structural members requires particularly thorough consideration.

Regarding (6): When bearings, seismic isolation mechanisms, collision prevention mechanisms, *etc*. are used, boundary elements, *etc*. must be used, with consideration of the mechanical properties of the mechanisms. Depending on the material used, consideration of the effects of strain rate and temperature is advisable.

Regarding (7): When considering buckling in steel members or in structural members with elongated form, the effects of geometrical non-linearity must be considered.

5.2.2 Modeling of structural members using finite elements

When modeling rod members or plane members using finite elements, "Design: Standards" Volume 10 should be followed.

5.2.3 Modeling of members using beam elements

5.2.3.1 General

(1) Members are to be modeled as beam elements having axial rigidity and flexural rigidity.

(2) The mechanical properties of structural members are to be derived directly from the stress–strain relationship of the material. Alternatively, a mechanical model is to be set with consideration of the form and dimensions of the structural members and the mechanical properties of the material.

(3) In general, shear deformation in rod members can be ignored. When this is not negligible, it is necessary to set appropriate mechanical properties and use a model that allows consideration of shear deformation.

(4) When modeling in-plane deformation of a plane member using beam elements, it is necessary to set appropriate mechanical properties and to use an analytical model that allows consideration of shear deformation.

(5) When shear deformation in the out-of-plane direction in plane members is not negligible, the plane members must be modeled using finite elements.

Commentary: Regarding (1): When performing linear analysis of statically determinate structural members, cross-sectional force can be calculated from the equilibrium conditions of force. When analyzing statically indeterminate structural members, compatibility conditions for deformation are necessary, and therefore modeling must take into consideration axial rigidity and flexural rigidity. At that time, consideration of non-linearity is essential if redistribution of stress is expected.

Regarding (2): For the mechanical properties of rod members, it is possible to perform modeling derived empirically from experimental results, as well as modeling using the moment–curvature relationship derived numerically by dividing the cross section into small parts and applying the stress–strain relationship of the material to each small part based on the Navier hypothesis.

A fiber model with the stress–strain relationship of materials applied to beam elements automatically considers the moment–curvature relationship and the axial force–axial strain relationship in the calculation, so it is a method that allows consideration of the effects of axial force variation. The moment–curvature relationship obtained from cross-sectional analysis can also be modeled and used as a mechanical model. When performing cross-sectional analysis, division of the cross section must be set with consideration of the same items as in the fiber model. Rod member mechanical models empirically derived from experimental results include many that are guaranteed to be applicable within a limited range of structural member forms, dimensions, and material properties. Therefore, the scope of application of mechanical models must be considered with care.

Regarding (3): Shear deformation is generally small and negligible in rod members. However, when shear cracking is expected and the amount of shear reinforcement is relatively small, there is a high probability that shear deformation will not be negligible. In such cases, modeling should use finite elements or beam element structural members that allow consideration of shear deformation, such as Timoshenko beams.

Regarding (4): Because of the effects of shear deformation, the in-plane deformation of plane members is not negligible and therefore modeling should in principle use finite elements. When modeling with beam elements is unavoidable, it is necessary to appropriately set the relationship between shear force and shear deformation and to use beam element structural members that allow consideration of shear deformation.

Regarding (5): When flexural deformation of plane members in the out-of-plane direction dominates, modeling may use beam elements. However, when shear deformation in the out-of-plane direction is not negligible, the plane members must be modeled using three-dimensional solid elements or using plate elements or other finite elements that allow consideration of out-of-plane shear deformation, such as Mindlin plates.

5.2.3.2 When the mechanical properties of structural members are directly derived from the stress–strain relationship of materials

(1) Division of elements in cross sections must be performed with consideration of the position of the primary rebar, and must be made particularly fine in peripheral parts.

(2) The element dimension in the longitudinal direction of the structural member should be not more than the effective depth of the cross section of the structural member at the part where deformation is concentrated, such as the end of the structural member where maximum flexural moment acts, and should be approximately 200 mm.

(3) The stress–strain relationship of the concrete and of the steel are to include historical effects. In general, that shown in 5.3 may be used.

Commentary: Regarding (1): Because the strain gradient in the cross section becomes large when flexural moment acts on structural members, analytical precision declines, particularly when division of the peripheral part of the cross section is rough. Therefore, the division of elements in cross sections must be set with consideration of the effects of cross-sectional form, reinforcing bar arrangement, axial force, *etc*. In general, peripheral parts should be divided into widths no greater than the thickness of the covering concrete, taking into account the center of gravity of longitudinal rebar. It is also advisable to confirm that the solution does not change even if the number of cross-sectional divisions is changed to a degree.

When deriving the mechanical properties of a structural member using a fiber model that applies the stress–strain relationship of the material, the applicable range is to be the general case in which compressive rebar is arranged in the compression zone derived from the neutral axis position during flexural capacity, with a shear span ratio of approximately 2.5 or higher, and in which the covering thickness does not become large relative to the cross-sectional dimensions. Material strength in the fiber model is used for the stress–strain relationship. When an effective depth of a structural member's cross section of approximately four or more times that of the concrete cover of the compression edge has been secured, the covering thickness can be deemed to be a general cross section.

Regarding (2): When structural members are greatly deformed and the strain of concrete exceeds the strain that corresponds to compressive strength, a softening zone appears in which intense compressive deformation occurs in a portion of the structural members. The size of this softening zone (200 mm) is allowed to be close to the dimensions of the cylindrical specimen for concrete compression testing (diameter 100 mm × height 200 mm). In the vicinity of the softening zone, stress is instead released and elastic recovery occurs. For this reason, when the element's size is set larger than the softening zone, it is not possible to sufficiently reproduce the deformation state of the actual structural member. The stress–strain relationship of concrete is generally derived from a specimen with a size nearly identical to that of the cylindrical specimen. Therefore, when the element size differs significantly from this size, the stress–strain relationship of concrete in the softening zone must be corrected. Response analysis, however, has confirmed that if the element size is between approximately 0.5 to 2.0 times this size, then the effect on the response values is small.

When targeting a structure with large dimensions, an element size of approximately 200 mm is generally difficult, owing to the relationships with analysis capacity and analysis time. When targeting a structure of small dimensions, division of elements may be rough if using an element size of approximately 200 mm. In such cases, element dimensions can be set arbitrarily by using a model that considers the failure energy of the concrete stress–strain relationship.

Regarding (3): In a fiber model, the stress distribution is derived from the strain distribution in cross sections based on the Navier hypothesis, and the stress distribution is integrated to derive the axial force and flexural moment. The stress–strain relationships of concrete and of steel used at this time are as shown in 5.3. In dynamic response analysis using a fiber model, a correctly modeled hysteresis curve of unloading/reloading must be used for the stress–strain relationship of a material. Conversely, when performing cross-sectional analysis using a fiber model and when modeling the obtained moment–curvature relationship to perform dynamic response analysis, in some cases the stress–strain relationship of the material does not necessarily have to include the hysteresis curve. Even in such cases, however, historical effects must be properly considered in the analysis.

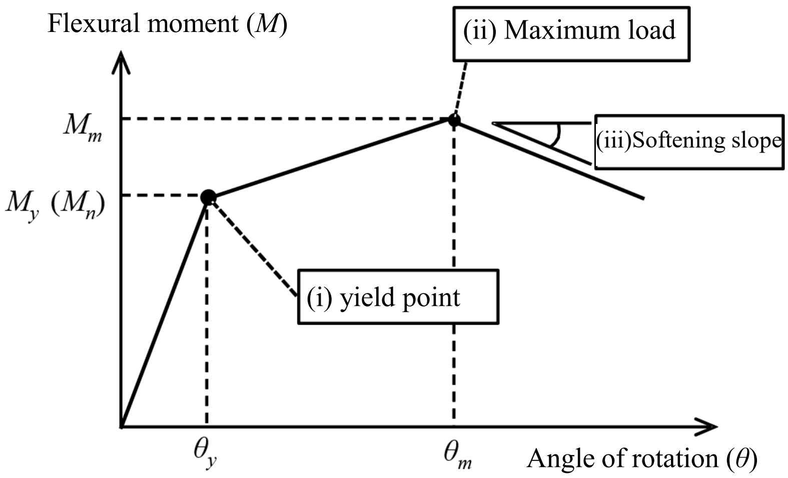
5.2.3.3 Using mechanical models of structural members

(1) In general, a skeleton curve in a dynamic model of rod members may use a trilinear model in which the origin, the structural member's yield point, and the point of maximum load-bearing capacity are connected by a straight line, with the softening gradient after the point of maximum load-bearing capacity taken into consideration.

(2) For structural members for which a response is expected at and beyond the yield point, a hysteresis model must be set. In general, rigidity during unloading decreases as deformation increases. During reloading to the opposite side after unloading, a model that points to the maximum point in the past may be used.

Commentary: Regarding (1): An example of a skeleton curve for a reinforced-concrete rod member is shown in **Commentary Figure 5.2.1**. This skeleton curve was determined with reference to studies based on the results of horizontal unidirectional positive and negative alternating loading testing on reinforced-concrete pillars having normal strength, in which past constant axial force was made to act on the center of gravity of the cross section. A skeleton curve may generally be set according to the following method. For general cases in which the shear span ratio is approximately 2.5 or higher, compressive rebar is arranged in the compression zone obtained from the neutral axis position during flexural capacity, and in which the thickness of covering is not large relative to the cross-sectional dimensions. When responding in multiple horizontal directions, when subjected to axial force variation or torsion, or when axial force is significantly eccentric, the effects of these must be taken into account. When structural member cross section effective depth of approximately four or more times that of the concrete cover of the compression edge has been secured, the cross section can be deemed to be one with general concrete cover.

In the model shown in **Commentary Figure 5.2.1**, the origin and (i) structural member yield point are connected by a straight line, and the decline in rigidity due to cracking is not incorporated. More than just a simplified model, this is intended to indirectly incorporate the effects of various actions including multiple occurrences of Level 1 seismic motions and changes in volume of concrete possibly brought about in service from the construction period onward. When using a model in which the point of cracking is placed between the origin and (i) structural member yield point, the cracking moment and structural member's angle may be set with consideration of varied factors such as the structure's specifications, surrounding environmental conditions, and history of actions.



Commentary Figure 5.2.1 An example of skeleton curve of structural member model (reinforced concrete).

(i) structural member yield point

Flexural moment (M_y): Set to the flexural moment at which the tensile rebar yields.

Structural member angle (θ_y): May be calculated using:

$\theta_y = \delta_{y0} / L_a$, (Commentary 5.2.1)

where δ_{y0} : displacement of the frame part when the structural member yields and

L_a : shear span.

When the effect of extension of the longitudinal rebar from a structural member junction is large, the angle of rotation (θ_{y1}) of the end of the structural member due to said effects should be added to the structural member angle. This may be calculated using:

$\theta_{y1} = \Delta L_y / (d - x_y)$, (Commentary 5.2.2)

$\Delta L_y = 7.4\alpha \cdot \varepsilon_y (6+3500\varepsilon_y)\varphi/(f'_{cd})^{2/3}$,

and(Commentary 5. 2.3)

$\alpha = 1 + 0.9\, e^{0.45(1 - cs/\square)}$, (Commentary 5. 2.4)

where ΔL_y : amount of extension of longitudinal rebar from structural member junction during yield (mm);

d: effective depth of cross section (mm);

x_y: neutral axis (mm) during structural member yield;

ε_y: yield strain of tensile rebar;

ϕ: diameter of tensile rebar (mm);

c_s: rebar spacing (mm); and

f'_{cd}: compressive strength of concrete at structural member junction (N/mm^2).

(ii) Point of maximum load-bearing capacity

Flexural moment (M_m): Maximum flexural moment, according to 2.4.2 in "Design: Standards" Volume 3. However, the structural member coefficient is set to 1.0.

Structural member angle (θ_m): Calculated using:

$$\theta_m = \delta_{m0} / L_a; \qquad \text{(Commentary 5.2.5)}$$

$$\delta_{m0} = \delta_{mb} + \delta_{mp}; \qquad \text{(Commentary 5.2.6)}$$

$$\delta_{mp} = \theta_{mp} \cdot (L_a - L_p / 2); \qquad \text{(Commentary 5.2.7)}$$

$$\theta_{mp} = (0.021 k_{w0} \cdot p_w + 0.013) / (0.79 \cdot p_t + 0.153), \qquad \text{(Commentary 5.2.8)}$$

however, $0.021k_{w0} \cdot p_w + 0.013 \leq 0.04$ and $0.79 \cdot p_t + 0.153 \geq 0.78$; and

$$L_p = 0.5d + 0.05L_a, \qquad \text{(Commentary 5.2.9)}$$

where δ_{m0} : displacement of the frame part at the point of maximum load-bearing capacity;

δ_{mb}: displacement due to flexural deformation of other than the plastic hinge portion;

δ_{mp}: displacement due to flexural deformation of the plastic hinge portion;

θ_{mp}: angle of rotation of plastic hinge portion;

p_w: hoop reinforcement ratio (%);

p_t: tensile rebar ratio (%);

k_{w0}: coefficient that takes into account hoop reinforcement strength, which is 0.85 for SD295, 1.0 for SD345, 1.15 for SD390, 1.4 for SD490, and 1.95 for the equivalent of SD685;

L_p: plastic hinge length; and

d: effective depth of cross section.

When the effect of extension of the longitudinal rebar from a structural member junction is large, the angle of rotation (θ_{m1}) of the end of the structural member due to said effects should be added to the structural member angle. The θ_{m1} added to Equation (Commentary 5.2.5) may be calculated using:

$$\theta_{m1} = (130\theta_{mp} - 0.47)\, \theta_{y1}, \qquad \text{(Commentary 5.2.10)}$$

however, $1.0 \leq 130\theta_{mp} - 0.47 \leq 4.7$.

(iii) Softening zone at and beyond maximum load capacity

In the verification of seismic-resistance performance 2 and seismic-resistance performance 3, the mechanical behavior of structural members in the softening zone beyond the point of maximum load-bearing capacity must also be modeled. At present, accurately evaluating the softening zone at and beyond the point of maximum load-bearing capacity is difficult. Therefore, it was decided that the softening zone may be reduced by a uniform ratio in the skeleton curve and may generally be set to 0.1. In cases such as high axial force or structural member shear capacity close to flexural capacity, η should be set low:

$$\Delta\theta = \eta \left(\frac{M_m - M_n}{M_m}\right), \qquad \text{(Commentary 5.2.11)}$$

where $\Delta\theta$: increment of angle of rotation from θ_m;

η : may generally be set to 0.1;

M_m : maximum flexural moment; and

M_n : may generally be set to M_y.

The failure mode of a structural member at and beyond the angle of rotation or the maximum displacement that maintains yield flexural moment may be determined using the ultimate structural member angle θ_n in Equation (Commentary 7.3.1), through comparison with structural member angle θ_s derived from:

$\theta_n \geq \theta_s$: shear failure mode after flexural yield,

$\theta_n < \theta_s$: flexural failure mode, and

$$\theta_s = (1.125V_{cd} + V_{sd} - V_{mu}) / (25V_{cd}), \qquad \text{(Commentary 5.2.12)}$$

where V_{cd} : design shear force in rod members that do not use shear reinforcement steel, according to 2.4.3.2 of "Design: Standards" Volume 3; however, the safety coefficient is to be set to 1.0 when used in Equation (Commentary 5.2.12);

V_{sd} : design shear capacity provided by shear reinforcement steel, according to Equation (2.4.7) in "Design: Standards" Volume 3; however, the safety coefficient is to be set to 1.0 when used in Equation (Commentary 5.2.12); and

V_{mu} : shear force caused by pure flexural capacity without consideration of axial force; however, the safety coefficient for calculating pure flexural capacity is to be set to 1.0 when used in Equation (Commentary 5.2.12).

<u>Regarding (2)</u>: To accurately reproduce the response of a structure during an earthquake, the mechanical model of structural members must consider the historical properties of the actions. When using a model incorporating decline in rigidity for reinforced-concrete rod members, such as that as shown in **Commentary Figure 5.2.2**, unloaded rigidity may be determined using :

$$k_r = k\left|\frac{\theta_{max}}{\theta_y}||^{-\beta}\right|, \qquad \text{(Commentary 5.2.13)}$$

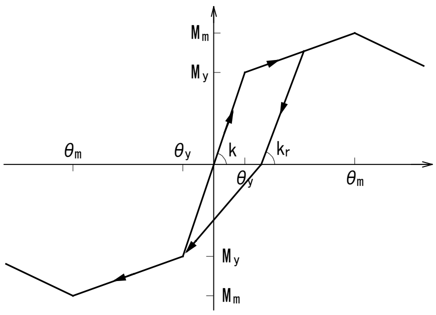
where k_r : unloaded rigidity;

k : yield rigidity;

θ_{max} : response structural member angle of rotation;

θ_y : structural member angle of rotation of structural member yield point; and

β : ratio of decline in rigidity (may generally be set to 0.5).



Commentary Figure 5.2.2 Hysteresis model of a rod member.

5.2.4 Modeling of other structural elements

The mechanical properties of structural elements that connect structural members must be modeled as necessary, taking into account the effects they have on the response of the structure.

Commentary: A structure is generally composed of not only rod members and surface members but also mechanisms such as bridge bearings and expansion/contraction mechanisms. Structural elements, including these mechanisms, affect the response of the structure as a whole, and may suffer damage ahead of structural members during an earthquake. Therefore, the effects of these should be considered as necessary. For such mechanisms and other structural elements, it is necessary to model the mechanical properties of the structural elements according to the mechanisms, the types and properties of constituent materials, the degree of deformation that could occur, and other factor.

5.3 Modeling of materials

5.3.1 Concrete

(1) When modeling structural members using finite elements, mechanical models of concrete are to follow "Design: Standards" Volume 10.

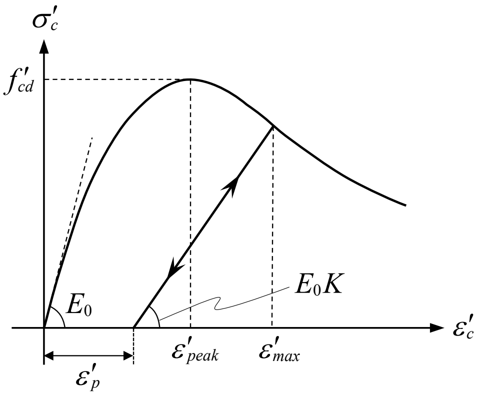
(2) When modeling structural members using beam elements, the skeleton curve of the stress–strain relationship in the compression zone of the concrete must also express the softening zone that exceeds the point of maximum stress. The stress history must express residual plastic strain and the decline in rigidity during unloading and reloading.

(3) In principle, the skeleton curve of the stress–strain relationship in the tensile stress zone is to use the average stress–average strain relationship, with the efficacy of adhesion to rebar taken into account. In this case, the tensile strength in the structure is set with consideration of all factors.

Commentary: A one-dimensional material model of concrete can be divided into three zones: compression, tension, and recontact at and beyond the occurrence of cracking. Ideally, the effects of strain history are to be considered for all of these, and historical absorption of energy be considered in the response analysis.

In the case of rod members, unlike that of plane members, modeling the compression hysteresis loop of concrete generally has a small effect on the response behavior of structural members, and hysteresis damping during unloading and reloading of the compression zone may be ignored. In this case, calculation of the response value related to deformation will be high, and thus is evaluated on the safe side.

It is advisable to consider hysteresis damping during unloading and reloading with reference to "Design: Standards" Volume 10. In general, however, for rod members for which the stress level due to constantly acting axial force is within approximately 10% of the uniaxial compressive strength of the concrete, the simplified material model shown in Equations (Commentary 5.3.1) to (Commentary 5.3.4) and in **Commentary Figure 5.3.1** may be used for the entire area in the structural member cross section. This relationship has been confirmed to be applicable to concrete with compressive strength of up to approximately 50 N/mm^2. In the case of high-strength concrete, this is to undergo compression failure at the peak value and to immediately release the stress.



Commentary Figure 5.3.1 Simplified hysteresis model of concrete in compression.

$$\sigma_c^{'} = E_0 K\left(\varepsilon_c^{'} - \varepsilon_p^{'}\right) \quad \geq 0, \qquad \text{(Commentary 5.3.1)}$$

$$E_0 = \frac{2 \cdot f_{cd}^{'}}{\varepsilon_{peak}^{'}}, \qquad \text{(Commentary 5.3.2)}$$

$$K = exp\left\{-0.73\frac{\varepsilon_{max}^{'}}{\varepsilon_{peak}^{'}}\left(1 - exp\left(-1.25\frac{\varepsilon_{max}^{'}}{\varepsilon_{peak}^{'}}\right)\right)\right\} \quad ,$$

and (Commentary 5.3.3)

$$\varepsilon_p^{'} = \varepsilon_{max}^{'} - 2.86 \cdot \varepsilon_{peak}^{'}\left\{1 - exp\left(-0.35\frac{\varepsilon_{max}^{'}}{\varepsilon_{peak}^{'}}\right)\right\},$$

(Commentary 5.3.4)

where $f_{cd}^{'} = f_{ck}^{'}/\gamma_c$,

$\varepsilon_{peak}^{'}$: strain corresponding to compressive strength (0.002 may generally be used);

$\varepsilon_{max}^{'}$: maximum value of compressive strain undergone in the past;

$\varepsilon_p^{'}$: plastic strain; and

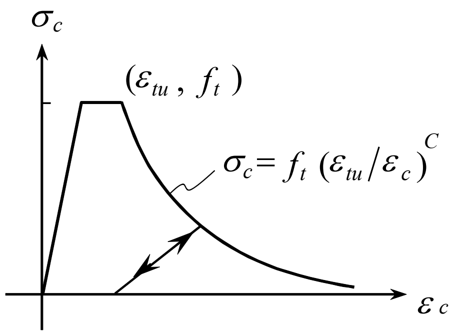
K: elastic rigidity residual ratio.

In the cracked portions of concrete, compressive stress occurs when the crack closes as a result of alternating positive and negative loading. In an actual structure, recontact of crack surface occurs gradually before a crack closes completely, and rigidity gradually increases. Conversely, in the simplified Equation (Commentary 5.3.1) that ignores complex non-linearity during recontact, a sudden change in rigidity is assumed, and in dynamic response analysis, acceleration of high-period components occurs, particularly in the longitudinal direction of structural members. However, the effect on the response value for verification of seismic action is small enough to be negligible.

Due to lateral constraining bars, reinforcing material arranged on the periphery of structural members, *etc*., concrete is subject to constraint stress not only in the axial direction but also in the direction perpendicular to the axis, which enhances strength and deformation performance in the axial direction. Therefore, it is advisable to provide a stress–strain relationship for every part in the cross section, taking into consideration that the constraining effect differs for each part in the cross section according to factors including the arrangement of lateral reinforcing bars. However, for general pillars in which the axial force level is small, Equations (Commentary 5.3.1) to (Commentary 5.3.4) are shown as stress–strain relationships in which the constraining effect is ignored, taking into account evaluation falling on the safe side. Along with the increase in the constraining force due to lateral constraint rebar, the longitudinal stress–strain relationship differs from those in Equations (Commentary 5.3.1) to (Commentary 5.3.4). However, it has been confirmed that the relationship of plastic residual strain with respect to the maximum value of past compressive strain is unaffected by the magnitude of the constraining force, and that the elastic rigidity residual ratio increases instead. If the elastic rigidity residual ratio can be appropriately provided according to the constraining force, it is possible to rationally express maximum strength, the corresponding increase in strain, and change in the unloading/reloading history.

When calculating the response value for the plastic deformation zone after flexural yield of a structural member for which axial force is large, Equations (Commentary 5.3.1) to (Commentary 5.3.4) underestimate the performance of the structural member, which could result in uneconomical design. Therefore, the constraining effect should be evaluated. If a material model that considers the constraining effect is used, it is possible to calculate the response value with high accuracy through three-dimensional analysis with the multiaxial stress field taken into account.

Commentary Figure 5.3.2 and Equation (Commentary 5.3.5) apply to the tensile zone of reinforced concrete with sufficient reinforcement, as an example of the stress–strain relationship in concrete of ordinary strength. The model in this example incorporates the adhesion between the concrete and rebar, with concrete considered to retain tensile stress on average even after cracking occurs. In this case, an adhesion action must be considered for the rebar model (tension) as well, so that it pairs with the concrete model. The average yield strength of rebar after cracking of concrete falls below the yield strength of rebar alone, owing to the adhesive force between the rebar and the concrete. The gradual increase in rigidity when crack recontact occurs should be taken into account. Consideration of hysteresis damping is advisable but may be ignored.



Commentary Figure 5.3.2 An example of the average stress-average strain relationship of concrete in tension

$$\sigma_c = f_t(\varepsilon_{tu}/\varepsilon_c)^C, \qquad \text{(Commentary 5.3.5)}$$

where f_t: tensile strength ($= \alpha_t f_{td}$);

f_{td}: design tensile strength;

α_t: tensile strength reduction coefficient (≤1.0);

ε_{tu} : tensile softening initial strain (may generally be set to 0.0002); and

C: coefficient that expresses the tensile softening property, and is set to 0.4 when sufficient reinforcing steel is arranged.

The average stress–average strain relationship, which is spatially averaged within the finite zone that contains the cracks, is used for the tensile stress–strain relationship of unreinforced concrete or concrete outside the zone affected by adhesion with reinforcing materials. The softening curve after cracking is set with consideration of tensile failure energy and the equivalent length.

The tensile strength of concrete in a structure is affected not only by its dimensions and boundary conditions but also by temperature changes and volume changes during the hardening process, as well as by curing and by surrounding environmental actions. In addition, tensile stress occurs because the shrinkage behavior is constrained by the reinforcing material. It is also expected to be affected by actions, including multiple occurrences of Level 1 seismic motions, during service from the construction period onward. Therefore, in the same manner as the model in **Commentary Figure 5.2.1**, tensile strength in the structure must be set appropriately with consideration of factors including the specifications of the structure, surrounding environmental conditions, and the history of actions. The effects of curing and environmental actions should generally be considered in a simulated manner by setting values lower than the material strength determined through means such as splitting tensile strength test.

5.3.2 Steel

(1) When modeling structural members using finite elements, the mechanical model for steel is to follow "Design: Standards" Volume 10.

(2) When modeling structural members using beam elements, the stress–strain relationship of steel must appropriately model yield, strain hardening, the Bauschinger effect, and energy absorption history after yield. The behavior of the compression zone and the tensile zone may generally be considered to be identical.

(3) The mechanical model for rebar in (2) should be given a stress–strain relationship compatible with the tensile rigidity model of concrete, with consideration of the effect of adhesion to concrete.

(4) When considering the buckling effect of rebar due to compressive force, the stress–strain relationship should be modeled by considering the effect of buckling length.

Commentary: The average stress–average strain relationship of rebar in concrete in a zone containing multiple cracks differs from the stress–strain relationship of rebar alone. Even if plasticization of the rebar starts at the position of cracking, rebar at positions far from the cracking remains in the elastic range, owing to adhesion. The zone plasticized at the position of cracking expands as stress increases, and the ratio of the plastic zone to the elastic zone of rebar in the element is not constant. Therefore, the plastic shoulder that exists in the stress–strain relationship of a single rebar does not appear in the average stress–average strain relationship of rebar in concrete. In addition, he average yield strength of rebar after cracking of concrete falls below the yield strength of rebar alone, owing to the adhesive force between the rebar and the concrete. Because the concrete material model considers the maintenance of the average tensile rigidity after cracking, the decline in average yield strength must be considered in the rebar model as well.

The average yield strength in the average stress–average strain relationship is calculated from Equations (Commentary 5.3.6) and (Commentary 5.3.7). The higher the concrete strength and the lower the rebar ratio, the more the average yield strength falls below the yield strength of rebar alone. However, Equations (Commentary 5.3.6) and (Commentary 5.3.7) are valid within the range affected by adhesion:

$$\overline{f_y} = f_y - \sigma_c\left(\overline{\varepsilon_y}\right) / p \quad \text{and} \qquad \text{(Commentary 5.3.6)}$$

$$\overline{\varepsilon_y} = \overline{f_y} / E_s , \qquad \text{(Commentary 5.3.7)}$$

where $\overline{f_y}$: yield strength of rebar in the average stress–average strain relationship (N/mm^2);

$\overline{\varepsilon_y}$: yield strain of rebar in the average stress–average strain relationship (N/mm^2);

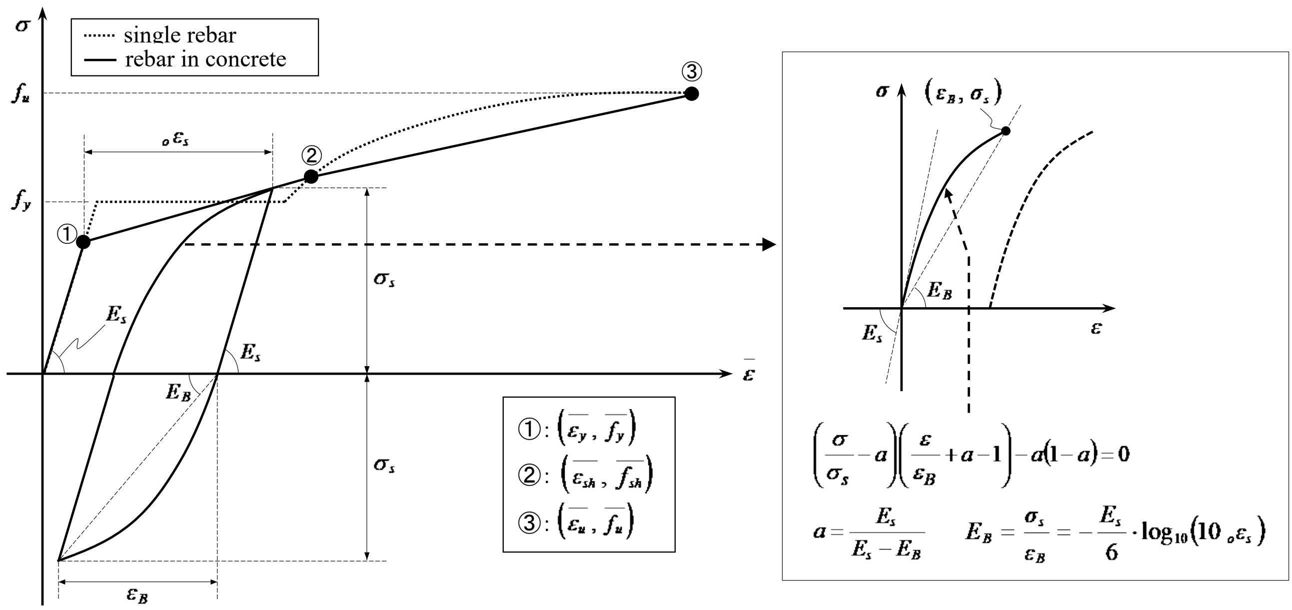
f_y : yield strength of single rebar (N/mm^2);

$\sigma_c\ (\bar{\varepsilon})$: average tensile stress of concrete calculated according to average strain $\bar{\varepsilon}$ (N/mm^2);

p : ratio of rebar; and

E_s : Young's modulus of rebar (N/mm^2).

For the average stress–average strain relationship with the strain hardening properties of rebar in concrete taken into consideration, the stress–strain relationship shown in **Commentary Figure 5.3.3** and Equation (Commentary 5.3.8) may generally be used. For general structures, it is often sufficient to verify performance such that average tensile strain occurring in rebar falls within a range of several percentage points. Therefore, this is modeled using the trilinear models of Commentary Figure 5.3.3. These models are formulated on the assumption of deformed bars with a clear yield point and yield shoulder, and are generally applicable to deformed rebars with a standard strength of not more than 490 N/mm^2 and a rebar diameter of D10–D51. In other cases, the applicable range must be determined by taking note of the stress–strain relationship of rebar.



Commentary Figure 5.3.3 Hysteresis model of steel rebar

$$\overline{\sigma_s} = E_s\,\overline{\varepsilon_s} \qquad \left(\overline{\varepsilon_s} \le \overline{\varepsilon_y}\right)$$

$$= \overline{f_y} + \left(\frac{\overline{f_{sh}} - \overline{f_y}}{\overline{\varepsilon_{sh}} - \overline{\varepsilon_y}}\right)\left(\overline{\varepsilon_s} - \overline{\varepsilon_y}\right) \qquad \left(\overline{\varepsilon_y} < \overline{\varepsilon_s} \le \overline{\varepsilon_{sh}}\right)$$

$$= \overline{f_{sh}} + \left(\frac{\overline{f_u} - \overline{f_{sh}}}{\overline{\varepsilon_u} - \overline{\varepsilon_{sh}}}\right)\left(\overline{\varepsilon_s} - \overline{\varepsilon_{sh}}\right) \qquad \left(\overline{\varepsilon_{sh}} < \overline{\varepsilon_s} \le \overline{\varepsilon_u}\right)$$

(Commentary 5.3.8)

where $\overline{\varepsilon_{sh}} = \alpha_1\alpha_2\alpha_3\varepsilon_y$,

$\overline{f_{sh}} = \overline{f_y} + (f_y - \overline{f_y})\alpha_4$,

$\overline{\varepsilon_u} = \overline{\varepsilon_{sh}} - k_5 \cdot ln\left(1 - \frac{\overline{f_u} - \overline{f_{sh}}}{1.01 f_u - \overline{f_{sh}}}\right)$,

$\overline{f_u} = (0.993 - 0.22 k_1{}^2 k_4{}^{-3}) f_u (\ge\ 1.01\overline{f_{sh}})$,

$\alpha_1 = (2.7 - k_2) + (0.43 + 0.18k_2)k_3$,

$\alpha_2 = (0.45 + 0.055k_3) + (1.0 - 0.1k_3)k_1$,

$\alpha_3 = a - b \cdot k_4 (\ge 1.0)$, $\alpha_4 = c + (1 - c)[1 - exp\{-d(k_4 - 1.09)\}]$,

$a = 3.25 - 0.25k_3 (\ge 2.08)$,

$b = 1.5 - k_3/6\ (\geq 0.72)$,

$c = 0.4k_3^{0.25} + (1 - 0.4k_3^{0.25})[1 - exp\{-0.5k_3^{0.4}(k_1^{-1} - 1.0)\}]$ $\quad d = 7.5 + (k_1^{-1} - 1.0)\ (e - 7.5)/0.75\ (\leq e)$,

$e = 10 + 1.2k_3$,

$p_{cr} = f_t/f_y$,

$k_1 = p_{cr}/p$,

$k_2 = f_y/350$,

$k_3 = \varepsilon_{sh}/\varepsilon_y$,

$k_4 = f_u/f_y$,

and $\quad k_5 = 0.035\left(400/f_y\right)^{1/3}$,

and where:

$\overline{\sigma_s}$: average stress of rebar in concrete (N/mm^2);

$\overline{\varepsilon_s}$: average strain of rebar in concrete (N/mm^2);

$\overline{f_y}$: average yield strength of rebar derived using Equation (Commentary 5.3.6) (N/mm^2);

$\overline{\varepsilon_y}$: average yield strain of rebar derived using Equation (Commentary 5.3.7);

f_y: yield strength of single rebar (N/mm^2);

f_u: tensile strength of single rebar (N/mm^2);

E_s: Young's modulus of single rebar (N/mm^2);

ε_y: yield strain of single rebar;

ε_{sh}: strain hardening initial strain of single rebar;

p: rebar ratio; and

f_t: tensile strength of concrete (N/mm^2).

In general, when no particular examination of historical processes is undertaken for structural members, the simplified model shown in Equations (Commentary 5.3.9) to (Commentary 5.3.11) and **Commentary Figure 5.3.3** may be applied to structural members for which the constantly acting axial force is within approximately 10% of the uniaxial compressive strength of concrete and for which the rebar diameter is sufficiently smaller than the cross-sectional dimension. Equations (Commentary 5.3.9) to (Commentary 5.3.11) express the hysteresis loop of the Bauschinger effect, with (ε_s, σ_s) as the stress inversion position, in the form of a curve passing through the origin. The loop on the opposite side, from tension to compression, may be made symmetrical.

$\left(\frac{\sigma}{\sigma_S} - a\right)\left(\frac{\varepsilon}{\varepsilon_B} + a - 1\right) - a(1 - a) =$ 0,(Commentary 5.3.9)

$a = \frac{E_S}{E_S - E_B}$, (Commentary 5.3.10)

$E_B = -\frac{E_S}{6} \cdot log_{10}(10_o\varepsilon_s)$, (Commentary 5.3.11)

where $o\varepsilon_s$: sum of empirical strain of skeleton curve portion and

σ_s: stress at the start of unloading.

The loop on the opposite side from tension to compression is to be made symmetrical.

In the major deformation zone, the primary rebar buckles laterally under compressive force, the covering concrete peels off, and the ability to transmit compressive force in the axial direction is lost. Therefore, the start of buckling of rebar and the residual compressive force after buckling should be reflected in the analysis, with consideration of the arrangement of the lateral constraint rebar, the diameter of the primary rebar, the material strength, *etc*. In a specimen with a rebar diameter that is large relative to the cross-sectional dimensions, the effect of rebar buckling is not small. In an actual structure, however, the effects of buckling of the primary rebar tend to be small relative to the effects in a scaled-down specimen. If buckling effects are ignored, then the response value of deformation will be calculated on the low side. Therefore, measures such as increasing the structural analysis coefficient are necessary.

When considering the effects of buckling on the material model, it is possible to make the structural analysis coefficient for the response value of displacement smaller than the standard value of 3.2. When doing so, the structural analysis coefficient and structural member coefficient must be set after determining the accuracy of modeling. In general, a method may be used that handles the behavior of buckled rebar as average axial strain with respect to buckling length, using the stress–strain relationship of rebar. However, as the average stress–average strain relationship of rebar after buckling using this method indicates strain softening behavior, sufficient attention must be paid to the averaged length and element size dependence that are preconditions of the model. After buckling and peeling have occurred, concrete stress can be considered negligible. An example of the above modeling is shown in Equations (Commentary 2.3.21) to (Commentary 2.3.24) in "Design: Standards" Volume 10, Chapter 2.

5.3.3 Structural member junction surface

(1) When localized deformation occurs due to the occurrence of cracks on surfaces of reinforced-concrete structural member junctions and when this has a major effect on deformation of the structure as a whole, the structural member junction surfaces must be modeled appropriately.

(2) The opening/closing direction of reinforced-concrete structural member junction surfaces must be appropriately modeled according to the purpose of examination, taking into consideration the opening and contact of localized cracking at the junction surface and the extension and pushing of rebar anchored to the concrete.

(3) When modeling structural members using beam members, deformation in the shear direction of the junction surfaces of reinforced-concrete structural members may be ignored.

(4) When modeling structural members using finite elements, mechanical models of structural member junction surfaces are to follow 2.4 in "Design: Standards" Volume 10.

Commentary: Localized cracking is likely to occur at the junction surfaces of pillars with beams, of pillars with footings, *etc*. This localized deformation may greatly affect deformation of the structure as a whole, and thus should be considered appropriately. For cases in which a mechanical model of structural members is used, models that consider the effects of extension of longitudinal rebar at junction surfaces based on this method are presented in Equations (Commentary 5.2.1) to (Commentary 5.2.11).

When a deformed rebar with a development length 20 times or more longer than the rebar diameter is subjected to tensile force in a state in which no cracking occurs in the anchored portion, the extension displacement of rebar at the junction surface may be derived using Equation (Commentary 5.3.16), and after rebar yield it may be derived using Equations (Commentary 5.3.17) to (Commentary 5.3.19). These equations were derived with reference to testing conducted on elements using deformed rebar exhibiting a clear yield point and yield shoulder, with a concrete compressive strength of 50 N/mm^2 and a rebar yield strength in a range of approximately 685 N/mm^2.

$$S_d = k_1\ k_2\ \varphi\ \varepsilon_s(6+3500\varepsilon_s) \qquad (0<\varepsilon_s\leq\varepsilon_y), \qquad \text{(Commentary 5.3.16)}$$

$$S_d = k_1\ k_2\ \varphi\ s_y \qquad (\varepsilon_y<\varepsilon_s\leq\varepsilon_{sh}), \qquad \text{(Commentary 5.3.17)}$$

$$S_d = k_1\ k_2\ \varphi\{s_y+0.02(f_u-f_y)(\varepsilon_s-\varepsilon_{sh})\} \qquad \left(\varepsilon_{sh}<\varepsilon\leq\varepsilon_{sh}+\frac{0.06-s_y/2}{0.013(f_u-f_y)}\right) \qquad \text{(Commentary 5.3.18)}$$

$$S_d = k_1\ k_2\ \varphi\{s_y/2+0.007(f_u-f_y)(\varepsilon_s-\varepsilon_{sh})+0.06\}, \qquad \left(\varepsilon_{sh}+\frac{0.06-s_y/2}{0.013(f_u-f_y)}<\varepsilon\right) \qquad \text{(Commentary 5.3.19)}$$

where S_d: extension displacement of anchored rebar (mm);

k_1: coefficient that takes into account the effect of close arrangement of rebars (group effect); this may be set to $k_1 = 1.0$ when the center-to-center distance of the rebars is 10 times or more the diameter of the rebar;

k_2: coefficient that takes into account the effect of the strength of the concrete; may generally be set to $k_2 = (20/f'_{cd})^{2/3}$;

φ: diameter of rebar (mm);

ε_s: rebar strain in structural member junction surface;

$s_y = \varepsilon_y(6 + 3500\varepsilon_y)$;

ε_y: yield strain of rebar;

$f'_{cd} = f'_{ck}/\gamma_c$;

f'_{ck}: characteristic value of compressive strength of concrete (N/mm^2); and

γc: material coefficient of concrete.

At present, few results are available from studies on the effect of close arrangement of rebars (group effect). However, in the case of a single-layer arrangement, this may be used:

$$k_1 = 1 + \varphi/c_s \quad \text{(Commentary 5.3.20)}$$

where φ : diameter of rebar (mm); and

c_s: center-to-center distance of rebars (mm).

If the rebars are arranged in two or more layers, then the effect of this should be considered further.

Residual extension displacement may be derived using the following equation until the rebar yields:

$$S_p = 0.15S_{max} , \quad \text{(Commentary 5.3.21)}$$

where S_p : residual extension displacement (mm) at the junction surface of anchored rebar; and

$S_{\max}$: maximum extension displacement at the junction surface of anchored rebar (mm).

Regarding (3): When localized cracking occurs on the joint surface of a reinforced-concrete structural member, it is likely that, in addition to shear resistance arising from crack surface roughness, resistance due to dowel action of the longitudinal rebar passing through the cracking surface is also present. However, when modeling a structural member with wire rods, it is assumed that the effect of this shear deformation on the response of the structure is not large, and thus the deformation in the shear direction can be ignored in the modeling.

Regarding (4): In this case, in addition to junction surface between reinforced concrete structural members, a structural member junction surface may refers to a junction surface in which rebar does not penetrate the surface of the reinforced-concrete structural member junction, and the adhesive surface between differing materials, the joints between concrete casting blocks, *etc*., and localized deformation at the junction surface can be considered to affect the response of the structure in some cases.

Chapter 6 Calculation of Response Values

6.1 General

(1) In principle, time-history response analysis, with the effects of the non-linearity of structural members and the ground considered, is to be used for structural analysis in the verification of seismic resistance. When applying a structural analytical method for which analytical accuracy has been sufficiently demonstrated, the structural analysis coefficient may be set to 1.0.

(2) The effects of the non-linearity of material are generally taken into consideration in the non-linearity of a structural member. Effects of geometrical non-linearity are to be considered as necessary.

(3) In structural analysis of seismic resistance, when the scope of analysis is set to include the ground, the structure and the ground are to be integrated and coupled analysis is to be performed. However, when dynamic interaction between the structure and the ground is negligible or can be modeled appropriately, the method for analyzing a structure individually, separately from the ground, may be used, following 6.3.

(4) Attenuation must be appropriately modeled so that the seismic response obtained from the analysis is not excessively small as a result of attenuation effects, in accordance with the analytical method, analyzed zone, and targeted strain level.

(5) The integral time interval in response analysis must take into consideration both analytical accuracy and the stability of the response value, and must be set in line with the time-integration method used.

Commentary: Regarding (1): A method other than time-history response analysis may be used if its reliability has been confirmed.

Regarding (2): For geometric non-linearity, the effects of additional moments caused by response displacement should generally be considered. When damage to structural members from shear failure, *etc*. is avoided, collapse of the structural system is brought about by geometric non-linear effects. However, when deformation large enough to be affected by geometric non-linearity is not tolerated, this does not need to be taken into consideration.

Regarding (3): For coupled analysis with the structure and the ground integrated, the method shown in **Commentary Table 6.1.1** is generally used.

Commentary Table 6.1.1 Method for coupled analysis integrating the structure and ground

Structure type	Above-ground structure, foundation structure, below-ground structure
Analysis method	Time-history response analysis
Analysis model of structure	Finite element model or beam element model

Analysis model of ground	Finite element model
Input value	Time-history acceleration waveform
Input location	bedrock surface

Regarding (4): It has been confirmed through numerous positive and negative alternating loading tests and dynamic loading tests that hysteresis damping can be reproduced by a mechanical model of hysteresis-dependent materials and structural members. In time-history response analysis, a hysteresis-dependent mechanical model of materials and structural members is to be adopted. However, depending on the analytical model used, it may not be possible to directly consider dissipative attenuation effects caused by energy lost through contact with the ground. Therefore, attenuation effects other than hysteresis damping caused by non-linearity of materials is to be appropriately modeled so that the seismic response obtained from the analysis is not excessively small as a result of attenuation effects, in accordance with the analytical method, analyzed zone, and targeted strain level. Because viscous damping is sometimes taken into consideration for purposes of stability in numerical calculation, in cases such as when the solution diverges, viscous damping may be set after confirming that the effect on the structure will be sufficiently small.

Regarding (5): Explicit and implicit methods exist for time-integration techniques for acceleration of time-history response analysis. Explicit methods include the central difference method and Runge–Kutta methods and implicit methods include the Newmark-β method and the Wilson-θ method. Among these, the Newmark-β method (β = 0.25) and Wilson-θ method are often used for stability of their solutions. These methods require convergence calculations to satisfy the equation of motion at each time step. Care must be taken as, if convergence calculation is not performed, the accuracy of analysis will decline accordingly. By contrast, the central difference method is typical of explicit methods: it does not require the convergence calculation required in implicit methods and it is a highly accurate time-integration method. The stability of its solution is poor, however, and a calculation time interval that is stable with respect to the highest-order natural period of the vibration system must be used.

6.2 Response analysis

(1) When performing coupled analysis with the ground/foundation structure and the superstructure integrated, the ground is to be modeled using finite elements having a necessary and sufficient spread. When there is significant irregularity in the ground structure, this effect must be considered.

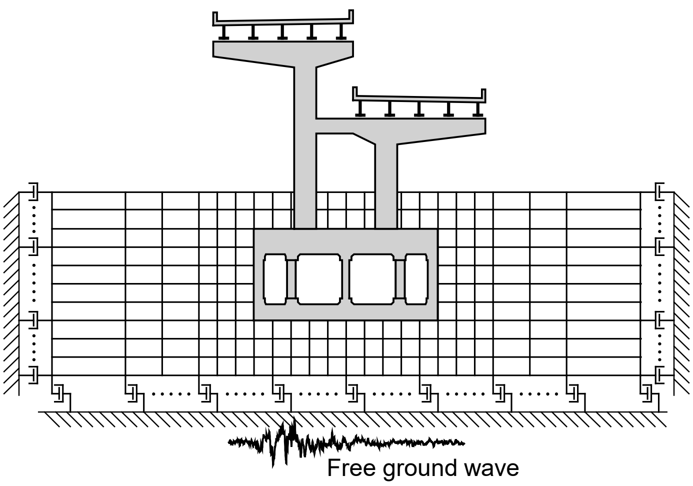
(2) The boundary conditions of the ground model are to be modeled to allow consideration of the propagation of seismic motion at the boundary.

(3) When the ground and peeling or slippage of the structure affect the response of the structure, a joining element should be placed between the structure and the ground to allow consideration of the peeling or slippage.

(4) A constitutive law for the ground must take into consideration changes in the properties caused by liquefaction of the ground and plasticization of the ground due to the effects of seismic motion.

Commentary: Regarding (1): When performing coupled analysis, the scope of modeling of the ground should extend to distant ground where the effects of dynamic interactions between the structure and the ground are sufficiently small. When there is significant irregularity in the ground structure, such as when the engineering bedrock surface or the ground surface is sloped, wave propagation may become complex and seismic motion may be locally amplified, or surface waves and other secondary waves may occur and the duration of seismic motion may become longer. Therefore, when there is significant irregularity in the ground structure, a model that allows consideration of this must be used. When modeling the ground using finite elements, the thickness should be approximately 1/5th to 1/6th of the minimum wavelength (period × shear wave velocity of the target stratum) in the frequency band targeted in the response analysis of the structure and the ground. In modeling the ground, the effects of distant ground must be fully considered. An example of a finite-element model of structures is presented in **Commentary Figure 6.2.1**. In a two-dimensional finite-element model such as that presented in **Commentary Figure 6.2.1**, the solution differs according to how length is considered in the depth direction, which is an effect that modeling must appropriately consider.

Regarding (2): The seismic motion presented in 4.2 is defined at the free rock surface, and does not include the effects of reflected waves in the surface subsoil. Therefore, the boundary condition at the lower end of the ground model should be a viscous boundary that absorbs reflected waves. When the lower end of the ground model is set as a fixed boundary, seismic motion must be input with the effect of reflected waves considered. Modeling of the sides of the ground should extend to distant ground that has sufficient spread, and the boundary condition should be made a condition that absorbs the reflection of waves.



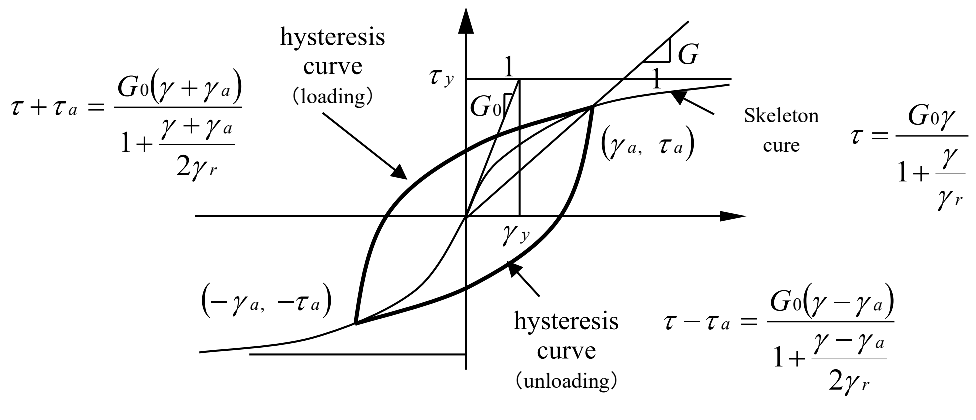
Commentary Figure 6.2.1. An example of a finite element model for coupled analysis of a structure and the ground

Regarding (3): When a Level 2 or other strong seismic motion acts, peeling or slippage may occur between the structure and the ground, owing to the difference in relative motion between the two, which may in turn affect the response of the structure as a whole. One method for considering such phenomena using a finite-element model is to perform the analysis with a junction element placed between the structure and the ground. In doing so, the behavior of the contact surface between the ground and the structure should be clarified and reflected in the properties of the junction element. Peeling occurs when the vertical stress at the contact surface between the structure and the ground exceeds the tensile strength. Slippage occurs when the shear stress at the contact surface exceeds the shear strength. The relationship between vertical stress and vertical strain, and the relationship between shear stress and shear strain, must be evaluated.

Regarding (4): The dynamic shear stress–strain relationship of soil must be appropriately evaluated in line with the magnitude of the shear strain amplitude of the ground that is assumed to occur during an earthquake. The dynamic shear stress–strain relationship of soil is generally handled through division into a skeleton curve and a hysteresis curve as shown in **Commentary Figure 6.2.2**, with the skeleton curve showing significant non-linearity as acting shear strain amplitude increases. As a hysteresis model that expresses such properties, the skeleton curve is often represented by a function such as a hyperbola, with Masing's law applied to the hysteresis curve. In setting the historical model, the strain dependence of the shear elasticity coefficient and of the attenuation coefficient of soil should be derived from such as PS logging and other *in situ* tests, or deformation property indoor test using samples from the ground in question.

When liquefaction of the ground occurs, the rigidity of the ground is affected by the magnitude of excess pore water pressure. In addition to the shear stress–strain relationship, the dilatancy of soil particles, the decline in strength and rigidity associated with the increase in excess pore water pressure, and other factors must be considered. Effective stress analysis is used in considering such phenomena. In addition to test of dynamic deformation properties, test of the liquefaction strength is needed to perform effective stress analysis.

When deriving the response value of a structure or structural member through coupled analysis with the ground, the effects of variations in strength and material constants on the ground should be appropriately considered.



Commentary Figure 6.2.2 An example of the dynamic shear stress-strain relationship of soil

6.3 Methods for analyzing structures and the ground separately

(1) The response of an above-ground structure is calculated using time-history response analysis by inputting the time-history response acceleration waveform of the ground at a predetermined depth calculated using response analysis of the ground.

(2) The dynamic interaction between the above-ground structure and the foundation structure may be modeled using springs with consideration of the strain dependence between the foundation structure and the ground that supports the above-ground structure. In general, vertical springs, horizontal springs, and torsion springs are to be used.

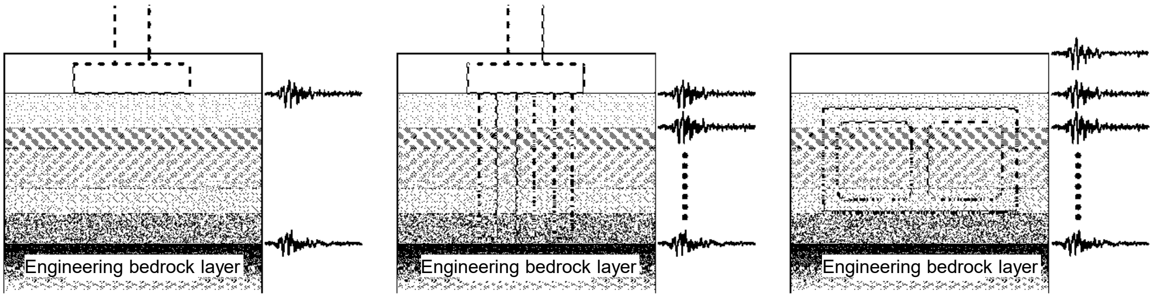
(3) In principle, the responses of the basic structure and the below-ground structure are to be calculated using time-history response analysis. The interaction between the ground and the foundation structure or the below-ground structure may be modeled using springs that take into account the strain dependence of the surrounding ground. In general, normal springs, tangential springs, and torsion springs are to be used for the surface at which the structure is in contact with the ground.

(4) Regarding the response of the ground, response analysis of the ground is performed and the time-history response waveform at a predetermined depth is calculated from the acceleration waveform at the engineering bedrock surface. The analytical model for the ground in dynamic analysis should be a finite-element model or a one-dimensional continuum model divided by strata.

Commentary: Regarding (1): Commentary Table 6.3.1 summarizes methods of analyzing a structure separately from the ground. In the case of an above-ground structure, as shown in **Commentary Figure 6.3.1**, the seismic response of the ground at the position where seismic motion acts is calculated in the analytical model of the structure by performing response analysis of the surface subsoil at the location in question, and dynamic response analysis of the structure is performed using this seismic response.

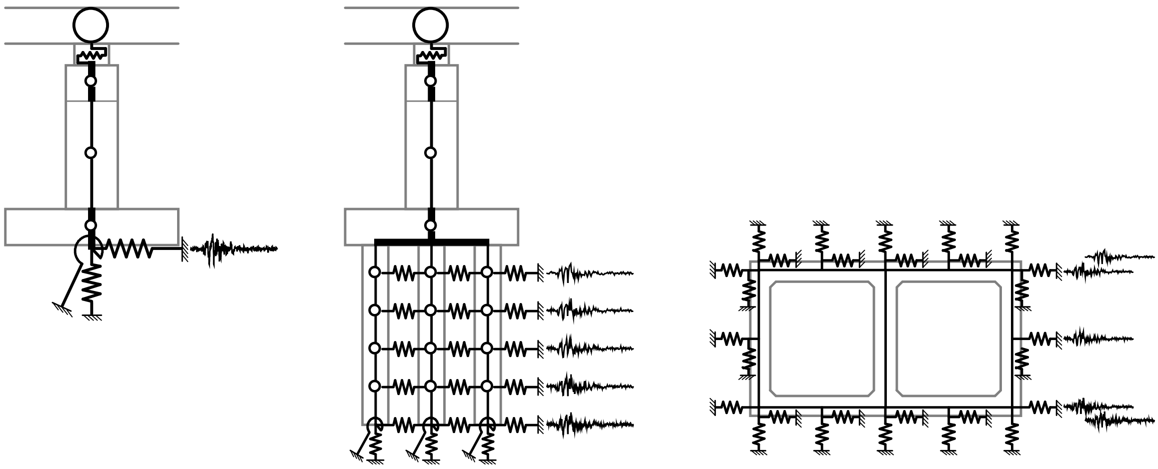
Commentary Table 6.3.1 Methods for analyzing structure and ground separately.

Type of structure		Above-ground structure	Below-ground structure
Analysis of structure	Method	Time-history response analysis	Time-history response analysis
	Model of structure	Finite element model or beam model	
	Model of interaction	Spring model	
	Input value	Time-history response acceleration waveform calculated from ground analysis	Response of ground calculated from ground analysis
	Input location	Below the structure	Surrounding the structure
Response analysis of ground	Method	Time-history response analysis or response analysis based on frequency band	
	Model of ground	One-dimensional discrete model divided by strata or finite element model	
	Input value	Time-history response acceleration waveform	
	Input location	Engineering bedrock surface	



(a) Direct foundation (b) Pile foundation (c) Below-ground structure

(1) When deriving the seismic response of stratums where the structure is located by applying free ground wave



(a) Seismic response of above-ground structure (b) Seismic response of above-ground structure and foundation (c) Seismic response of below-ground structure

(2) Models of analyzing a structure and the ground separately

Commentary Figure 6.3.1 Methods for analyzing an above-ground structure and the ground separately.

Regarding (2): This model, with the above-ground structure supported by replaced non-linear springs, assumes that the primary mode of vibration will dominate as the response of the structure. In this case, vertical springs, horizontal springs, and torsion springs are generally to be used as the non-linear springs, and must be set appropriately according to ground conditions, foundation form, and load-bearing capacity properties. In setting the restoring force properties to be given to the springs, the appropriate hysteresis characteristics must be considered by using the relationship between force and displacement (with the non-linearity of the ground or of the ground and the foundation structure considered) as the skeleton curve. In this case, the effects of dissipative attenuation or internal attenuation may be considered for the load-bearing springs, in accordance with the foundation structure. The modeling shown here is for the purpose of calculating response values for the above-ground structure. Therefore, when the response values of the structure foundation are calculated, the ground, the foundation structure, and the above-ground structure must be modeled as an integrated unit through a finite-element model or a model of equivalent analytical accuracy that combines springs and mass points, to enable consideration of the effects of the mode of vibration of the foundation.

Regarding (3): The response of the foundation structure and the below-ground structure may be calculated using time-history response analysis, while inputting the ground response derived separately through springs with interaction between the structure and the ground considered. However, because box culverts and other general below-ground structures exhibit behavior that follows the deformation of the surrounding ground as a whole during an earthquake, the effects of inertial force based on the mass of the structure are small. Therefore, when the modes of vibration of the ground and the structure match, the deformation (response displacement) of the ground during an earthquake can be derived and analysis can be performed, with the deformation acting statically on the structure used as forced displacement during an earthquake. In this case, the peripheral shear force and the inertial force of the frame should be considered as necessary. The behavior of the structure is greatly affected by the rate of change of horizontal displacement in the vertical direction, not by the magnitude of the absolute displacement of the ground at the position of the structure. The displacement that is inputted should be the displacement of the ground at the time that relative displacement of the ground at the upper and lower ends of the structure is at a maximum. For long structures such as piles, however, the time at which relative displacement of the ground is maximum may differ for each depth, depending on geological conditions. Multiple ground displacements that have large effects on each interval of the piles must be selected, from among the time-history displacements.

Non-linear springs with consideration of the effects of the interaction between the ground and the foundation structure and below-ground structure are to be normal springs and tangential springs, or torsional springs as necessary, at the surface where the structure is in contact with the ground, and must be set appropriately according to surrounding ground conditions, their bearing capacity properties, and the structural form.

<u>Regarding (4)</u>: The time-history response analysis method for the time zone and the response analysis method in the frequency zone are generally used as the ground response analysis methods. The time-history response analysis method is a method for sequentially tracking the non-linear stress–strain relationship of soil along its time history. The method requires considerable time to calculate response values. At the same time, it has been noted that, because the response analysis method in the frequency zone approximates the non-linear stress–strain relationship of soil using the equivalent linearization method, compatibility with the behavior of ground is poor in the major strain zone at which the strain level of soil exceeds 10^{-3}. Therefore, dynamic analysis requires the selection of an appropriate analytical method according to the strain level of the soil that is of interest in the targeted ground. In order to consider the non-linearity of the ground, it is advisable to derive the dynamic properties of soil through *in situ* test and laboratory test.

The equivalent linearization method is a method of response analysis in the frequency zone. It is used when the magnitude of the shear strain amplitude of the ground during an earthquake is relatively small at approximately 10^{-3} or less. In the equivalent linear model, soil is a viscoelastic body with rigidity and viscosity considered. The equivalent shear modulus and viscous damping constant during seismic motion are set according to the shear strain amplitude (effective strain) that acts during the earthquake. In the equivalent linearization method, linear analysis is repeated using an equivalent linear model until the shear strain amplitude converges. Because it allows a stable solution to be obtained, it is often used in practical design.

In order to derive the time history response acceleration waveform at a specified depth, the ground around the structure is generally divided into strata or finite elements, down to the engineering bedrock surface. When the ground structure around the structure is a horizontally layered ground, the response analysis can be simplified by applying a one-dimensional ground model.

6.4 Calculation of design response values

(1) Design response values are calculated by using an appropriate method to convert the response values obtained from analysis into verification metrics.

(2) Calculation of design response values when modeling a structure using finite elements is to follow "Design: Main Volume" Chapter 7 and "Design: Standards" Chapter 10.

(3) Calculation of design response values when modeling a structure using beam elements is to follow "Design: Main Volume" Chapter 7.

Commentary: For mechanisms and other structural elements, design response values converted into appropriate verification metrics in accordance with "Design: Main Volume" Chapter 7 must be calculated.

Chapter 7 Verification of Seismic Resistance

7.1 General

(1) Verification related to the seismic resistance of a structure confirms that the structure does not reach the limit states set in accordance with seismic performance, in order to maintain the required seismic resistance performance with respect to assumed seismic motion. In general, the following should be examined:

(i) Seismic-resistance performance 1 is satisfied with respect to Level 1 seismic motion.

(ii) Seismic-resistance performance 2 or seismic-resistance performance 3 is satisfied with respect to Level 2 seismic motion.

(2) Limit values must be set appropriately in accordance with the seismic-resistance performance of the structure. In principle, limit values for seismic-resistance performance 1 and seismic-resistance performance 2 are to be set with consideration of the effects of the damage states of constituent structural members and other structural elements on the overall behavior of the structure. In principle, limit values for seismic-resistance performance 3 are to be set with consideration of the relationship between the stability of the structure and the resistance force of the constituent structural members and other structural elements.

(3) Limit values of structural members modeled using finite elements are determined based on "Design: Standards" Volume 10.

Commentary: Regarding (1): Level 1 seismic motion and Level 2 seismic motion are according to 4.2. When performing verification related to the seismic resistance of a structure, the seismic-resistance performance that the structure should possess must be set according to the magnitude of the seismic motion and its recurrence interval.

Regarding (2): When verifying the seismic resistance of a structure, limit states that satisfy the set seismic-resistance performance must be set. As specified in 2.2.3, the limit states of a structure are generally set through combinations of the damage levels of individual structural members and other structural elements that compose the structure. Combinations of limit values for the respective damage states must be determined from relationships between factors including necessity and ease of repair of individual structural members and other structural elements, and the maintenance of the structural object's functions during an earthquake.

Because a structure has not only primary structural members such as rod members and plane members but also structural elements arranged at the connections between structural members, the occurrence of deformation or damage to not only primary structural members but also to these structural elements and their surroundings during an earthquake is taken into consideration during verification. In the same manner as structural members, limit states for damage to structural elements that have effects on the response of the structure are to be set in accordance with the required seismic performance and the magnitude of seismic motion, with verification performed to confirm that these are not exceeded. When damage has occurred to soundproof walls, signposts, or other incidental objects installed during the service of the structure, this must also be taken into consideration as the damage could affect users of the structure.

Regarding (3): When analyzing surface members using a finite-element model, instead of verifying that response displacement does not reach ultimate displacement, it should be verified that in-plane principal compressive strain on the surfaces of structural members does not reach twice the strain with respect to compressive strength. Moreover, instead of verifying that vertical structural members do not undergo shear failure, it should be verified that in-plane principal compressive strain on the surfaces of structural members does not reach three times the strain with respect to compressive strength. In rod members, major strain is concentrated locally, with minor damage in other areas. By contrast, in surface members, strain is distributed more evenly over a wide area, and in areas subjected to in-plane force, nearly identical strain occurs in the direction of the thickness of the shell element. Therefore, compressive principal strain is suitable for expressing the limit state.

When modeling structural members using finite elements, the limit values of other structural elements should be set according to (2) and 7.2 to 7.4.

7.2 Verification related to seismic-resistance performance 1

(1) For limit values of structural members used in the verification of seismic-resistance performance 1, the yield displacement or yield angle of rotation of the structural members, or equivalent limit values, are used.

(2) The yield displacement or the yield angle of rotation of structural members may be derived as the displacement or the angle of rotation when rebar yields at the position of resultant tensile force generated in the rebar in a structural member cross section.

(3) The limit values of other structural elements used in the verification of seismic-resistance performance 1 may generally be used as the force or deformation by which the structural elements remain within the elastic range following an earthquake.

Commentary: Regarding (1): If rebar does not yield during an earthquake, then seismic-resistance performance 1 is generally satisfied. Therefore, the yield displacement or the yield angle of rotation of a structural member was set as the limit value of the structural member with respect to seismic-resistance performance 1, on the assumption that shear failure does not occur. If the rebar has not yielded, then the stress in concrete will generally not have reached compressive strength. However, when the amount of rebar is large, when high-strength rebar is used, or in similar cases, the compressive strength of the concrete must be used as the limit value. When structures are close to each other and a risk of collision exists, or when the safety of operating vehicles must be examined, displacement determined according to the maintenance of functionality with respect to the maximum response displacement of the structure during an earthquake should also be set as a limit value. In any of these cases, it is essential that the response of the structure properly considers the effects of the response of mechanisms and other structural elements, not only of structural members.

Regarding (2): When longitudinal rebar is arranged in multiple layers in the cross section, because rebar in each layer gradually yields as flexural moment increases, no clear yield phenomenon is observed in the relationship between force and displacement in the structural member. The yield displacement of a structural member is determined using the method presented in this section. The yield angle of rotation may generally be derived using Equation (Commentary 5.2.1).

Regarding (3): Structural elements that affect the response of the structure during an earthquake, such as bridge bearings, must be verified by determining limit states in accordance with the mechanical properties and the form of damage. In general, force or deformation by which a structural element remains within the elastic range after an earthquake may be used as the limit value of the structural element with respect to seismic-resistance performance 1.

When response occurs in the structural element, localized damage could occur at structural members in the vicinity of the mounting portion of the structural element even if the structural element itself is undamaged. Therefore, the soundness of the mounting portion is also to be verified, according to the magnitude of the response of the structural element.

7.3 Verification related to seismic-resistance performance 2

(1) Shear capacity, torsional capacity, ultimate displacement or ultimate angle of rotation of the structural member, or equivalent limit values are used as the structural member limit values used in the verification of seismic-resistance performance 2.

(2) The ultimate displacement or angle of rotation of a structural member may be derived as the maximum displacement or angle of rotation at which the flexural moment of the structural member does not fall below the yield flexural moment.

(3) The shear capacity and torsional capacity of structural members are determined according to "Design: Standards" Volume 3.

(4) In the case of deformation compatibility torsion and a structural member for which the effect of torsional moment is small, verification related to torsion may be omitted, provided that verification concerning shear force of the structural member is satisfied.

(5) In general, the limit values of other structural elements used in the verification of seismic-resistance performance 2 may generally be used as the force or deformation by which excessive residual deformation or failure does not occur in the structural elements.

Commentary: Regarding (1): Seismic-resistance performance 2 is to be set to the state in which usability can be restored in a short time after an earthquake and reinforcement is not required. Therefore, limit values for structural members are set appropriately between the points at which ultimate displacement or ultimate angle of rotation (maximum displacement or angle of rotation that is able to maintain yield flexural moment) beyond the yield displacement or yield angle of rotation of the structural members is reached, while preventing shear failure, torsional failure, *etc*. and taking into consideration the effects of the damage status of the constituent structural members and the difficulty of repairing the structure. In other words, in order to ensure seismic-resistance performance 2 in a structure (meaning that function can be restored in a short time after an earthquake, with no reinforcement required), the limit values for structural members may be appropriately set between the ultimate displacement and ultimate angle of rotation beyond yield displacement or angle of rotation, taking into account conditions of the structure such as whether inspection is possible and whether repair is possible in light of technical or construction conditions.

In the case of general pillar members, the peeling of concrete cover occurs at a displacement close to the ultimate displacement or ultimate angle of rotation. As the vicinity of the point of maximum load-bearing capacity (the maximum point at which decline in load-bearing capacity is not significant) can be considered displacement for which repair in a short time is possible, the vicinity of the point of maximum load-bearing capacity should be used as the limit value for normal structural members.

For structural members that are located in the below-ground portion and for which inspection or repair is not possible in a short period after an earthquake, or for structural members which are in a state of supporting vertical weight and for which repair methods have not been established, the limit value for seismic-resistance performance 2 should be set to not more than the maximum displacement or angle of rotation at which decline in load capacity is not significant.

In addition, when it is possible to appropriately determine metrics and limit values (stress, strain, *etc.*) for damage states that correspond to shear capacity, torsional capacity, ultimate displacement, or ultimate angle of rotation of structural members, those metrics and limit values may be used to perform verification related to seismic-resistance performance 2.

Residual displacement may be specified for special cases. It must be calculated using an appropriate method and must be confirmed to be within the set permissible limit. Particularly in cases of soft ground, the residual displacement of a foundation with ground included may be larger than the residual displacement of the structure. Therefore, the residual displacement of the foundation must be appropriately evaluated.

In any of these cases, however, it is essential that the response of the structure properly considers the effects of the response of mechanisms and other structural elements, not only structural members.

Regarding (2): After maximum capacity has been reached in a structural member in which flexural properties dominate, the structural member generally exhibits a gradual decline in load capacity along with an increase in displacement, owing to the peeling of concrete cover and the buckling of longitudinal rebar. If load capacity falls below yield load capacity, then compression of core concrete or fracture of longitudinal rebar may occur, resulting in excessive damage. The ultimate displacement of structural members is determined as described in the text, with safety taken into consideration. However, even if the angle of rotation is such that the flexural moment is lower than the yield flexural moment, when flexural moment can be stably maintained without a sudden decline in load capacity with respect to repeated actions, *etc.*, the ultimate displacement or the ultimate angle of rotation may be determined separately.

Because a variety of factors affect the structural member angle at the time of ultimate displacement of the structural member, the effects of these factors must be appropriately evaluated according to the structural member. When no particular examination of the effects of these factors is performed, derivation may follow Equation (Commentary 7.3.1). However, for structural members for which the effect of torsion cannot be ignored, the angle of rotation and ultimate angle of rotation may exhibit a tendency to become smaller at the point of maximum load bearing capacity. Therefore, appropriate limit values must be set separately, as necessary.

$$\theta_n = \theta_m + \eta\{1 - (M_n/M_m)\}, \quad \text{(Commentary 7.3.1)}$$

where θ_m : structural member angle at the point of maximum load capacity, determined from Equation (Commentary 5.2.5);

M_m : maximum flexural moment, determined from 2.4.2 in "Design: Standards" Volume 3 with structural member coefficient set to 1.0;

M_n : flexural moment when tensile rebar yields; and

η : may generally be set to 0.1, with consideration of the softening gradient of rod members.

In the verification of seismic-resistance performance 2, the mechanical behavior of structural members in the softening zone beyond the point of maximum load-bearing capacity must also be modeled. At present, accurately evaluating the softening zone at and beyond the point of maximum load-bearing capacity is difficult. Therefore, the softening zone may be reduced by a uniform ratio in the skeleton curve. In the case of high axial force or when shear capacity is close to flexural capacity, or when the effects of torsion cannot be ignored, η should be set small. When used to determine failure mode, η is not reduced.

When analyzing rod members using a fiber model, instead of verifying that response displacement does not reach ultimate displacement, it should be verified that the average elastic rigidity residual ratio of the structural member cross section does not correspond to the ultimate displacement of the structural member. In general, when the average elastic rigidity residual ratio reaches 50% of the initial value, it corresponds to the ultimate displacement of the structural member. The average elastic rigidity residual ratio is the average of the elastic rigidity residual ratios of concrete for the elements in the cross section of the structural member. The elastic rigidity residual ratio of concrete is the ratio of the residual elastic rigidity when reloading occurs beyond the peak with respect to initial elastic rigidity in the compression-side stress–strain relationship, and is defined in 5.3.1.

Regarding (3): The shear capacity and the torsional capacity of structural members are known to decline after flexural yield or under positive and negative alternating action. It is advisable to confirm that load capacity does not decline sharply, even if the ultimate displacement of the structural member is exceeded, and to determine that the failure mode that occurs is not a shear failure mode after flexural yield, through either test or other appropriate methods. As shown in 3.2, it is advisable to ensure shear capacity in the large deformation zone by increasing the structural member coefficient of shear capacity, and to take care to prevent the occurrence of a shear failure mode after flexural yield or a sudden decline in load capacity.

Regarding (4): When rebar yields as a result of flexural moment or shear force, the torsional rigidity of the structural member declines significantly. If the toughness of the structural member is sufficiently secured even after yield of the rebar, because the deformation compatibility torsion will also decline in line with the decline in torsional rigidity, then verification of torsional moment may be omitted. However, in verification related to ultimate displacement or ultimate angle of rotation of structural members in which torsional moment exceeds 20% of the design pure torsional capacity and in which the effect of torsion is considered to be non-negligible, attention must be paid to maintaining a low damage level, or to appropriately determining design limit values with the effect of torsion considered.

Regarding (5): Bridge bearings and other structural elements that affect the response of the structure during an earthquake must be verified by determining limit states in accordance with the mechanical properties and the form of damage. In particular, when the structure has been subjected to the seismic motion used in the verification of seismic-resistance performance 2, it is possible that damage occurs to not only the primary structural members that constitute the structure, but also to mechanisms and other structural elements; or that damage to structural elements occurs first in structural members; or that the response of structural elements exacerbates damage in other structural members. Therefore, appropriate limit states are to be determined and verified with consideration of the mechanical properties of the structural elements themselves, the form of damage, the position of installation, the ease of repair after an earthquake, *etc*. In general, force or deformation by which excessive residual deformation or failure does not occur in a structural element after an earthquake may be used as the limit value of the structural element with respect to seismic-resistance performance 2.

When response occurs in a structural element, localized damage could occur at structural members in the vicinity of the mounting portion of the structural element, even if the structural element itself is undamaged, and the structural element may not respond as intended. Verification should also confirm that mounting portions are sound and that damage is limited to the degree that structural elements still function properly, according to the magnitude of the structural element's response.

7.4 Verification related to seismic-resistance performance 3

(1) The limit values used in the verification of seismic-resistance performance 3 are to be the shear capacity of vertical structural members and self-weight-bearing capacity of the structure.

(2) The shear capacity and the torsional capacity of structural members are determined according to "Design: Standards" Volume 3.

(3) For deformation compatibility torsion, in the case of a structural member for which the effect of torsional moment is small, verification related to torsion may be omitted, provided that verification concerning the shear force of the structural member is satisfied.

(4) In the case of surface members, when verification targets a structural form in which in-plane shear dominates, verification of out-of-plane shear force may be omitted.

(5) Regarding the self-weight bearing capacity of a structure, if input of seismic motion and performance of time-history response analysis are able to confirm that the structure does not reach collapse, then said analytical results may replace verification.

Commentary: <u>Regarding (1)</u>: In the case of concrete structures, if a vertical structural member is sufficiently safe with respect to shear failure, then seismic-resistance performance 3 is generally satisfied. However, there may be cases in which displacement of the structural system as a whole is excessive, axial deformation and additional moment increase in structural members under their own weight, and the structure is unable to stand on its own and collapses. Analysis of the response of the structure to seismic motion must confirm that the structure does not collapse under its own weight.

Design seismic motion is given deterministically during design. However, the uncertainty of natural phenomena means that seismic motion in excess of design seismic motion may occur. Fracture of longitudinal rebar, crushing of internal concrete, *etc*. occur in structural members in which ultimate displacement or ultimate angle of rotation is exceeded as a result of excessive seismic force. In addition, as displacement or angle of rotation increases from ultimate displacement or ultimate angle of rotation, there is also a possibility of shear capacity falling below the shear capacity that corresponds to yield flexural moment. If fracture of longitudinal rebar or shear failure occurs, then sudden declines in load capacity and energy absorption capability will occur. Therefore, cases in which excessive seismic force occurs should be assumed, and whether sudden failure states occur in those cases should be investigated.

In the case of a bridge in which girders are supported by bearings, damage must be restricted so that bridge piers and other vertical structural members are able to support the girders, *etc*. after an earthquake, and so that the girders themselves do not fall.

<u>Regarding (2)</u>: The shear capacity and the torsional capacity of structural members are known to decline after flexural yield or under positive and negative alternating action. Therefore, it is advisable to confirm that load capacity does not decline sharply even if the ultimate displacement of the structural member is exceeded, and to determine that the failure mode that occurs is not shear failure mode after flexural yield, through test or other appropriate methods. As shown in 3.2, it is advisable to ensure shear capacity in the large deformation zone by increasing the structural member coefficient of shear capacity, and to take care to prevent the occurrence of a shear failure mode after flexural yield or sudden decline in load capacity.

<u>Regarding (3)</u>: When rebar yields due to flexural moment or shear force, the torsional rigidity of the structural member declines significantly. If the toughness of the structural member is sufficiently secured even after yield of the rebar, because the deformation compatibility torsion will also decline in line with the decline in torsional rigidity, verification of torsional moment may be omitted. However, in verification related to ultimate displacement or ultimate angle of rotation of structural members in which torsional moment exceeds 20% of design pure torsional capacity and in which the effect of torsion is considered to be non-negligible, attention must be paid to maintaining a low damage level, or to appropriately determining design limit values with the effect of torsion considered.

<u>Regarding (4)</u>: When the primary shear resistance mechanism is due to the in-plane shear resistance mechanism of structural components, as in an underground tank, even if localized out-of-plane shear occurs, the parts in which out-of-plane shear occurs are generally not the same parts that act as the primary shear resistance mechanism. In such a case, because the out-of-plane shear mechanism is stable, verification may be omitted.

<u>Regarding (5)</u>: For seismic-resistance performance 3, by which the structure as a whole does not collapse, it is necessary to confirm that vertical structural members do not undergo shear failure and that they remain self-supporting after an earthquake. If seismic motion is inputted and time-history response analysis is performed, and if the response calculation converges properly even in a state in which seismic motion is no longer inputted (state of free vibration) and no collapse of the structure is observed, then these analytical results may replace verification. In the case of a statically determinate structure, it is thought that the structure will collapse when the load-bearing capacity of one structural member is lost. Therefore, seismic-resistance performance related to the stability of the structure can be verified using the response value of one structural member. Conversely, in the case of a statically indeterminate structure, even if the load-bearing capacity of one structural member is lost, the cross-sectional force is redistributed and collapse does not immediately occur. Therefore, the state in which the load-bearing capacity of structural members is lost should be appropriately modeled and verified. Because the effect of additional flexing and the effect of creep deformation may be large when residual displacement is large, these should be taken into consideration. In addition, in zones in which load-bearing capacity declines, safety with respect to seismic motion of the same level as design seismic motion

should be confirmed, to prevent collapse due to subsequent seismic motion.

In the event of an earthquake in excess of that assumed in design, if fracture occurs in longitudinal rebar in structural members, a sudden decline in load capacity and energy absorption capability will occur. While it is unlikely that longitudinal rebar will fracture under general specifications, when deformation is excessively localized, it is advisable to confirm through an appropriate method that fracture of longitudinal rebar does not occur.

Chapter 8 Structural Specifications Related to Seismic Resistance

8.1 General

When performing verification of the seismic resistance of a reinforced concrete structure based on this volume, the structural specifications regarding seismic resistance presented in this chapter must be satisfied.

8.2 Concrete cover

Concrete cover must be set appropriately with consideration of the seismic resistance required of the concrete structure.

Commentary: For cover, the larger of the diameter of the rebar or the depth of concrete cover that satisfies durability, with construction work error considered, is to be used as the minimum value. Because concrete cover is determined based on environmental conditions, the percentage of structural member thickness accounted for by cover may become extremely large. When such a structural member is subjected to repeated plastic deformation due to the effects of earthquakes, buckling of rebar may be prevented and deformation performance may increase in some cases, but the cover concrete may be prone to peeling and durability may decline sharply. In a vertical structural member in which the structural member thickness is relatively small and the cover is large, hoop reinforcement is arranged only near the center of the structural member cross section, and the probability of intersecting a diagonal crack is relatively small. In a structural member in which the cover is large, hoop reinforcement may not sufficiently function as shear reinforcement and shear capacity may fall below the calculated value. Therefore, with respect to seismic resistance, it is important that cover does not become excessive with respect to the structural member thickness. Moreover, the evaluation equations presented in this volume cannot be considered to have been sufficiently confirmed for cases of excessive cover. Therefore, for structural members in which a plastic hinge is formed, cover must be determined with consideration of the effect of excessive cover on structural member thickness, within the range from the structural member junction to the pillar width and beam height.

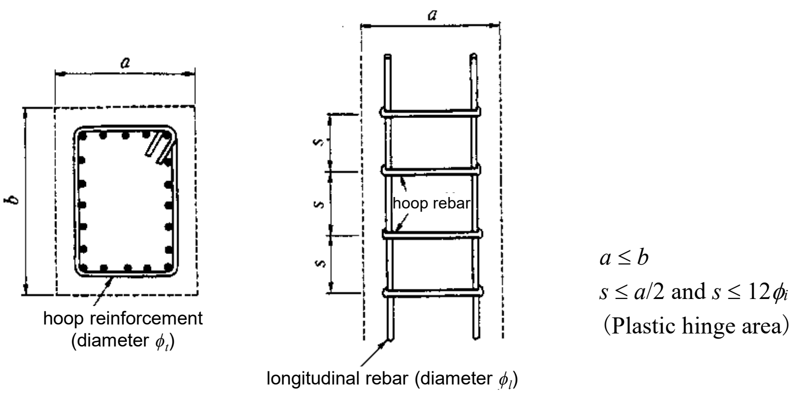
Based on the above points, it should generally be confirmed that an effective structural member cross section height of approximately 4 times the cover of the compression edge is secured at locations subjected to a severe salt damage environment. Otherwise, it is advisable to confirm the presence of required toughness through finite-element analysis, according to "Design: Standards" Volume 10. Alternatively, rather than making cover excessively great with respect to structural member thickness, measures such as the use of epoxy resin-coated rebar, stainless steel rebar, concrete surface coating, or electrolytic protection should be examined, following 2.2.4.1 in "Design: Standards" Volume 2.

8.3 Arrangement of hoop reinforcement

(1) The spacing of hoop reinforcement in the axial direction of the structural member is generally to be not more than 12 times the diameter of the longitudinal rebar and not greater than the minimum dimension of the structural member’s cross section. The area that becomes the hinge is to be not more than 12 times the diameter of the longitudinal rebar and not more than 1/2 the minimum dimension of the structural member's cross section. In principle, hoop reinforcement is arranged so as to surround the longitudinal rebar.

(2) When using hoop reinforcement with a rectangular cross section, the length of one side of the hoop reinforcement is to be not more than 48 times the diameter of the hoop reinforcement and not more than 1 meter. Hoop reinforcement must be arranged so that the length of one side of the reinforcement does not exceed these values.

Commentary: <u>Regarding (1)</u>: Hoop reinforcement, spiral reinforcement, and other transverse rebar constrain the progress of diagonal cracks and enhance shear capacity, while also preventing the buckling of longitudinal rebar and binding core concrete. Therefore, from the viewpoint of ensuring shear reinforcement or required toughness, an amount of rebar that satisfies verification in Chapter 7 must be arranged, and, as shown in this section, the spacing in the structural member axial direction must be no greater than the specified value (**Commentary Figure 8.3.1**).



Commentary Figure 8.3.1 Spacing of hoop reinforcement arranged in the axial direction surrounding longitudinal rebar

<u>Regarding (2)</u>: It is known that when the cross-sectional dimensions of a structural member with a rectangular cross section are large, the constraint effect of hoop reinforcement declines at locations distant from the corner sections of the cross section.

8.4 Anchoring of rebar

When high-stress repetitive action has acted on the anchored portion, owing to the action of an earthquake, it should be confirmed in principle that the structure possesses the required seismic resistance, through test and analysis that reproduce the stress state and the reinforcing bar arrangement of the target structural members.

Commentary: It should be confirmed that the anchored portion of the longitudinal rebar is set so that the structure possesses the required load capacity and deformation performance with respect to the repetitive actions assumed in an earthquake. When no special examination of required load capacity and deformation performance is conducted, then this is to be carried out according to "Design: Standards" Volume 7. In recent years, mechanical anchorage has been applied in some cases. This is an effective anchorage method for structural member junctions, *etc*. in which the anchorage zones for the rebar from structural members intertwine.

8.5 Rebar joints

(1) It is necessary to verify through appropriate test, analysis, *etc*. that joints have the required high-stress repetition performance, taking into consideration the effects of the confidence level originating in the actual construction work and inspection.

(2) It is necessary to confirm through appropriate structural member test, *etc*. that excessive stress concentration, fissures, or fractures do not occur in the base material in the vicinity of joints, even when the joints are subjected to high-stress repetitive action that exceeds the yield strength of the base material.

(3) When the rigidity and load capacity of structural members increase because of the arrangement of joints in the structural members' longitudinal rebar, it is necessary to confirm the mechanical properties of structural members through full-scale testing, *etc*. in order to appropriately consider the increase in rigidity and load capacity of the structural members in verification of seismic resistance.

(4) When seismic design permits damage to structural members in which joints are arranged in longitudinal rebar, it must be confirmed through full-scale testing, *etc*. that the damage can be appropriately repaired.

Commentary: Regarding (1): When a joint that directly joins rebar is used in a plastic hinge zone that is subjected to alternating stress, the structural member properties are to be examined by setting structural specifications based on test according to 8.6, and construction and inspection are to be carried out using methods by which defects do not occur in joints. When no special examination of the properties of joints under alternating stress is performed, this is to be done according to "Design: Standards" Volume 7. In particular, when joints are arranged in all of the longitudinal rebar in the same cross section, it must be set as a precondition that the confidence level of all of the joints will be confirmed through inspection, and it must be confirmed through appropriate structural member test, analysis, *etc*. that the performance of structural members is secured by a sufficient margin with respect to repetitive action caused by earthquakes.

8.6 Setting of structural specifications based on test

When setting structural specifications through test-based verification, the performance of specimens fabricated using those structural specifications must equal or exceed the performance of specimens fabricated according to "Design: Standards" Volume 7. As a general rule, test to confirm performance is to be performed according to the following method:

(1) The form and dimensions of structural members, the types of materials used, the dimensions and spacing of steel, the concrete cover, *etc*. should in principle be the same as those of the actual structural members.

(2) Positive and negative alternating loading is to be performed through displacement control using a force application method in which the force acting on the structural member cross sections is nearly identical to that in actual structural members.

Commentary: When full-scale test is not possible because of constraints on the test apparatus, scaling down is acceptable to an extent that does not affect failure behavior. However, the selection of the rebar diameter and rebar arrangement, the treatment of concrete cover, and other structural details in the scaled-down model must be properly modeled by the responsible engineer.

Because it is also important that a structure possess redundancy that ensures its functions to the degree possible even under the action of seismic motion greater than design seismic motion, the range of loading should extend to displacement beyond the ultimate state of structural members. Loading is to be based on displacement at the time that longitudinal rebar yields, and is to be repeated positively and negatively, using integral multiples of said displacement. The number of repetitions for each of the set displacements should be set appropriately within a range of 1 to 10 times in order to conduct evaluation through relative comparison, but approximately three repetitions may be used.

Part 6 Thermal Cracking Verification

Chapter 1 General

1.1 Scope of application

This volume presents methods for verifying the effect of cracks caused by hydration of cement on the performance of concrete structures, and presents standard methods for the thermal analysis and stress analysis required for this verification.

Commentary: Verifying the occurrence of thermal cracking caused by hydration of cement and measuring the width of cracks require accurately deriving the thermal generation caused by hydration of cement, the accompanying change in concrete temperature, and changes in concrete volume due to changes in temperature and due to autogenous shrinkage. It also requires accurately calculating concrete stress caused by these changes in volume. This volume presents standard analytical methods for confirming the presence of effects of initial cracking caused by hydration of cement on the performance of concrete structures. For the CP method, "Design" in the 2007 Standard Specifications for Concrete Structures should be followed. However, the values for physical properties of materials shown in this volume are only standard values. When the thermophysical and mechanical properties of materials actually used in construction can be obtained through testing, those test values should be used.

Chapter 2 Methods of Verification

2.1 Verification of cracking

(1) The safety coefficient γ_{cr}, which corresponds to the probability of cracking $P(\gamma_{cr})$ set as a target, is determined from **Figure 2.1.1** and:

$$P(\gamma_{cr}) = \left[1 - exp\left\{-\left(\frac{\gamma_{cr}}{0.92}\right)^{-4.29}\right\}\right] \times 100. \tag{2.1.1}$$

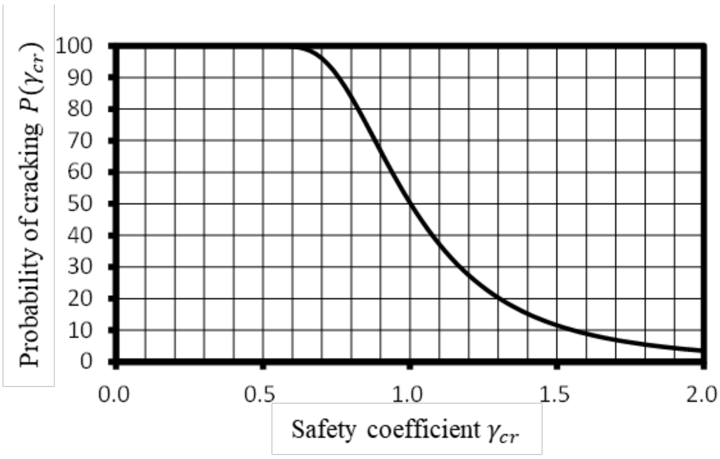


Figure 2.1.1 Relationship between the probability of cracking and the safety coefficient.

(2) The safety coefficients and probabilities of cracking shown in Table 2.1.1 are set as target values, according to the level of measures taken against cracking.

Table 2.1.1 The safety coefficients and probabilities of cracking in structures with general arrangement of rebar.

Level of measures taken against cracking	Probability of cracking	Safety coefficient γ_{cr}
The case that the occurrence of cracks is prevented.	5（%）	1.85 以上
The case that the occurrence of cracks is limited as much as possible.	15（%）	1.40 以上
The case that the occurrence of cracks is permitted but preventing the width of cracks from becoming excessive	50（%）	1.0 以上

Commentary: Regarding (1): Verification of cracking can be performed using Equation (Commentary 2.1.1), using the minimum value of the thermal cracking indices during the period of examination.

$$I_{cr}(t) \geq \gamma_{cr}, \tag{Commentary 2.1.1}$$

where

$I_{cr}(t)$: thermal cracking index;

$I_{cr}(t) = f_{tk}(t)/\sigma_t(t)$;

$f_{tk}(t)$: concrete tensile strength at material age of t days;

$\sigma_t(t)$: maximum principle tensile stress in concrete at material age of t days; and

$_{cr}$: safety coefficient related to the probability of cracking.

When the calculation methods presented in this chapter and in Chapters 3 and 4 are used, the relationship between the safety coefficient γ_{cr} and the probability of cracking is obtained according to **Figure 2.1.1** and Equation (2.1.1). The cracking probability curve shown in **Figure 2.1.1** was derived by comparing analytical results with the state of cracking in actual structures. Care must be taken as this cracking probability curve should be applied to cross sections in which change in stress is gradual, such as wall-like or slab-like structures, and is not appropriate for corner sections, *etc*.

Regarding (2): Because the safety coefficient is large when the goal is to prevent cracks, the level of cracking countermeasures should be set appropriately according to the required performance of the structure, in order to avoid excessive costs for cracking countermeasures.

When the occurrence of cracks is permitted but preventing the width of cracks from becoming excessive is a goal, it is advisable to set a limit value for crack width and to perform verification by deriving crack width through calculation. When calculation of crack width is difficult, it may be evaluated in simple fashion from thermal cracking index, as shown in 2.2. In Table 2.1.1, the safety coefficient when the crack width limit value is 0.3 mm and the rebar ratio is 0.25% is derived from Equation (Commentary 2.2.1). When steel corrosion, watertightness, and airtightness present problems, a target value for the safety coefficient should be determined separately.

2.2 Verification of initial crack width

In principle, verification of initial crack width due to hydration of cement should confirm that the maximum value of crack width during the period of examination is not more than the limit value for crack width.

Commentary: Methods for deriving crack width include those that incorporate the finite-element method, with behavior after cracking considered. When direct derivation through calculation of crack width is difficult, thermal cracking indices may be used for confirmation. For the relationship between thermal cracking indices and maximum crack width when external constraints dominate, **Commentary Figure 2.2.1** and the following equation may be used:

$$w_c = \gamma_a \left(\frac{-0.141}{p} + 0.0938\right) \times (I_{cr} - 1.965)$$

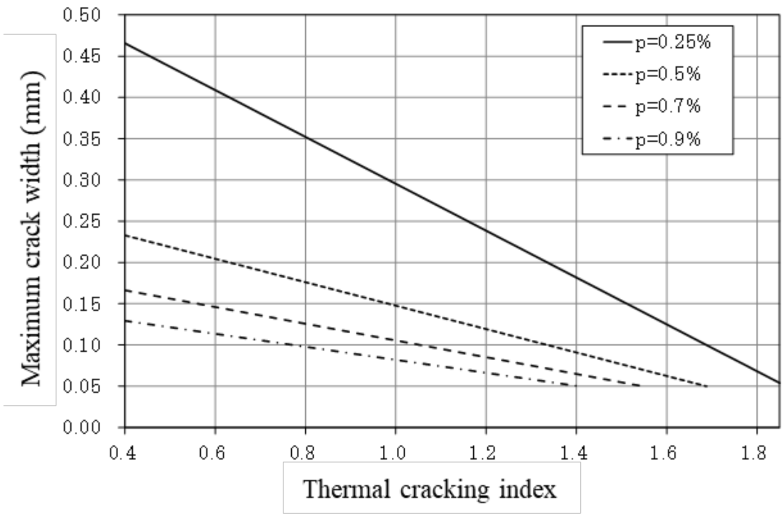
(Commentary 2.2.1)

where

w: maximum crack width (mm);

p: rebar ratio (%), with an applicable range of 0.25%–0.9%; and

γ_a: safety coefficient for evaluating the width of thermal cracks; this may generally be set to 1.0.



Commentary Figure 2.2.1 Relationships between maximum crack width, thermal cracking index and rebar ratio.

Commentary Figure 2.2.1 and Equation (Commentary 2.2.1) show the relationships between crack width, thermal cracking indices, and rebar ratio created based on testing of wall-like structures subjected to internal constraints and drying, with external constraints dominant. The thermal cracking indices can be calculated from analytical results and substituted into Equation (Solution 2.2.1) to calculate the width of thermal cracks.

When the minimum thermal cracking index inside the structural member (excluding the structural member's surface sections and corner sections) is used, the correlation with maximum crack width during the period of examination is relatively good. Using this minimum thermal cracking index, maximum thermal crack width can be derived from Equation (Commentary 2.2.1) and can be used for verification of initial crack width by confirming that the value satisfies the limit value of crack width.

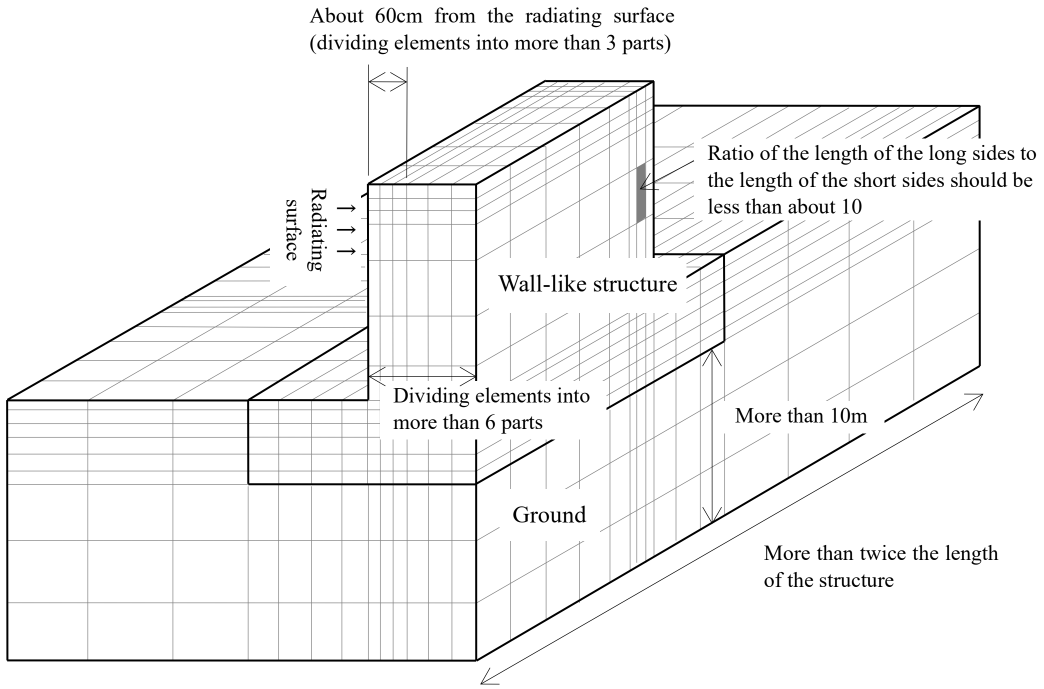
However, when the thermal cracking index and the rebar ratio are both small, the safety coefficient γ_a must be set appropriately based on factors including the importance of the structure, environmental conditions, and past experiences.

Chapter 3 Thermal Analysis

3.1 Method of analysis

Thermal analysis of concrete must be performed using analytical models that are appropriate to the forms, dimensions, and other particulars of the structure (a constrained body) and constraining bodies.

Commentary: When using the finite-element method, the difference method, or other numerical analysis methods, for those parts for which thermal generation due to hydration of cement is to be considered, it is advisable to divide elements into at least six parts in each direction, as shown in **Figure 3.1.1**. In principle, as a guideline, this division should be into at least three parts with a depth of approximately 60 cm from the heat dissipation surface. The ratio of the lengths of the long sides to the lengths of the short sides of the elements composing the structural member to be analyzed should be not more than approximately 10. The depth of the ground to be modeled is to be not less than 10 m, with a width not less than twice that of the structure. The calculation of temperature should target the period until temperature changes in the structural member are nearly in equilibrium with the outside air temperature. This requires approximately one month when the structural member thickness is 2 m or less and approximately three months when the structural member thickness exceeds 2 m.



Commentary Figure 3.1.1 Example of mesh division in thermal analysis (cross-sectional view of a wall-like

3.2 Boundary conditions and initial temperature conditions

The boundary conditions and initial temperature conditions used in thermal analysis of concrete must be determined with consideration of the form of the structure and constraining bodies, heat dissipation conditions, concrete temperature during pouring, and other factors.

Commentary: Irradiation may generally be ignored in thermal analysis of concrete. For the ground, the side surfaces are considered the adiabatic boundary, and the lower surface is set to the fixed temperature boundary based on the annual average temperature of the region subjected to analysis.

The heat transfer coefficient, which expresses the properties of the heat transfer boundary, must be determined with consideration of factors including surrounding wind speed, concrete curing method, and presence or absence, type, thickness, and stripping time of concrete forms.

Regarding the effect of wind speed on the heat transfer coefficient, the coefficient is 12–14 W/m^2°C at the exposed surface of ordinary concrete when the wind speed is 2–3 m/s. As wind speed increases, as a guideline, the heat transfer coefficient increases by approximately 2.3–4.6 W/m^2°C per 1 m/s of wind speed. Determination of the heat transfer coefficient, with effects of concrete forms, curing method, *etc.* considered, should be according to:

$$\eta = \frac{1}{\frac{1}{\beta}+\sum\frac{d_{Fi}}{\lambda_{Fi}}} \qquad \text{(Commentary 3.2.1)}$$

where

η: corrected heat transfer coefficient (W/m^2°C);

β: heat transfer coefficient of surface exposed to outside air (W/m^2°C) (12–14 W/m^2°C may generally be used);

d_{Fi}: thickness of curing material (m); and

λ_{Fi}: heat transfer coefficient of curing material (W/m°C).

Reference values for heat transfer coefficient are shown in **Commentary Table 3.2.1.**

Commentary Table 3.2.1 Reference values for heat transfer coefficientη.

No.	Type of forms, curing methods	η (W/m^2°C)
1	Steel form	14
	Spray (the flooding depth is less than 10mm)	
2	Flooding (the flooding depth is between 10mm and 50mm)	8
3	Flooding (the flooding depth is between 50mm and 100mm)	8
4	Plywood	8
5	Plastic sheet	6
6	Curing mat (including the methods of flooding + curing mat and flooding + plastic sheet)	5
7	Styrene form (thickness of 50mm) + plastic sheet	2
8	Air bag with plastic sheet; 2, 3, 4 layers	6,4,2
9	Exposed surface of concrete, ground and bedrock	14

Concrete pouring temperature is to be the average atmospheric temperature on the day of pouring plus 5 °C, with temperature rise during mixing and transport taken into consideration. It is advisable to derive the initial temperature of parts such as the ground and existing concrete by performing a transient thermal analysis of only the existing parts. The period of analysis is to be approximately three months, with initial temperature derived by performing transient analysis up to the day on which the concrete of the structural member to be examined is poured. The initial temperature of existing parts during transient analysis may be set to the outside temperature during the month in which analysis begins.

Chapter 4 Stress Analysis

4.1 Method of analysis

Changes in the volumes of concrete, steel, bedrock, ground, foundation, and other constraining bodies are to be derived based on temperature changes in thermal analysis. Stress associated with change in the volume of concrete is to be calculated so as to satisfy compatibility conditions for boundaries and deformation, and equilibrium conditions for force.

Commentary: It is preferable to use an analytical method that satisfies both the compatibility conditions and the equilibrium conditions for deformation and that allows consideration of change in concrete volume and transient mechanical properties, as finite-element analysis does. In this stress analysis, the maximum value of tensile principal stress in the examined zone at each material age t is defined as $\sigma_t(t)$.

4.2 Consideration of autogenous shrinkage

When autogenous shrinkage cannot be ignored, in principle, stress is to be calculated with consideration of both autogenous shrinkage and change in volume due to hydration thermal generation. Autogenous shrinkage strain can be determined using experiments or estimation equations that have demonstrated applicable range and accuracy.

Commentary: Depending on the type and formulation of the cement and admixture, autogenous shrinkage associated with the progress of hydration may not be negligible.

Assuming thermal expansion and autogenous shrinkage to be deformations independent of stress, stress may be calculated from the effective strain, which is calculated, with consideration of transient mechanical properties of concrete, using:

$$\varepsilon_{ij,ef} = \varepsilon_{ij} - \varepsilon_{ij,T} - \varepsilon_{ij,ag} \quad \text{(Commentary 4.2.1)}$$

where

$\varepsilon_{ij,ef}$: effective strain;

ε_{ij}: total strain;

$\varepsilon_{ij,T}$: temperature strain = $\alpha\, \Delta T \delta_{ij}$; α is the thermal expansion coefficient and δ is the Kronecker’s delta; and

$\varepsilon_{ij,\ ag}$: autogenous shrinkage of concrete; may generally be a function of time.

At present, incorporating all material and construction factors to predict autogenous shrinkage strain is difficult. Therefore, either an estimation equation or testing is selected, according to design and construction work conditions. An estimation equation should take into consideration the composition of the cement, the mixing of the concrete, and the temperature of the concrete. When using only ordinary Portland cement or Type-B blast furnace cement as a binding material, the following autogenous shrinkage strain prediction equations may be used:

$$\varepsilon_{ij,ag} = -\beta\varepsilon'_{as\infty}[1 - exp\{-a(t' - t_s)^b\}]$$

, (Commentary 4.2.2)

where

β: coefficient that expresses the effect of the type of cement and admixture (may be set to 1.0 for ordinary Portland cement, Type-B blast furnace cement, or Type-B fly ash cement; 0.50 for moderate-heat Portland cement; 0.40 for low-heat Portland cement; or 1.20 for high-early-strength Portland cement);

t': effective material age (days);

t_s: start of settling (days) (may be set to an effective material age of 0.30 days for ordinary Portland cement, moderate-heat Portland cement, Type-B blast furnace cement, or Type-B fly ash cement; 0.35 days for low-heat Portland cement; or 0.20 days for high-early-strength Portland cement);

$\varepsilon'_{as\infty}$: final value of autogenous shrinkage strain ($\times 10^{-6}$): Ordinary Portland cement, moderate-heat Portland cement, low-heat Portland cement, high-early-strength Portland cement, Type-B fly ash cement:

$$\varepsilon'_{as\infty} = 3070\ exp\{-7.2(W/C)\} + \varepsilon'_{asT};$$

(Commentary 4.2.3)

Type-B blast furnace cement:

$$\varepsilon'_{as\infty} = 2350\ exp\{-5.8(W/C)\} + \varepsilon'_{asT};$$

(Commentary 4.2.4)

W/C : water-to-cement ratio;

ε'_{asT} : amount of increase in autogenous shrinkage due to high temperature history, according to: Ordinary Portland cement, moderate-heat Portland cement, low-heat Portland cement, high-early-strength Portland cement, Type-B fly ash cement:

$$\varepsilon'_{asT} = 50[1 - exp\{-1.2 \times 10^{-6} \times (T_{max} - 20)^4\}]$$

(20 °C ≤ T_{max} ≤ 70 °C); (Commentary 4.2.5)

Type-B blast furnace cement:

$$\varepsilon'_{asT} = 80[1 - exp\{-1.2 \times 10^{-6} \times (T_{max} - 20)^4\}]$$

(20 °C ≤ T_{max} ≤ 70 °C); (Commentary 4.2.6)

T_{max}: maximum temperature (°C) of concrete, derived through thermal analysis;

a, b: coefficients that express properties of the progress of autogenous shrinkage; these are derived depending on the type of cement: Ordinary Portland cement, moderate-heat Portland cement, high-early-strength Portland cement, Type-B fly ash cement:

$a = 3.7 \times exp\{-6.8 \times (W/C)\}$ and

$b = 0.25 \times exp\{2.5 \times (W/C)\}$; (Commentary 4.2.7)

Low-heat Portland cement:

$a = 2.4 \times exp\{-6.5 \times (W/C)\}$ and

$b = 0.12 \times exp\{2.7 \times (W/C)\}$; (Commentary 4.2.8)

Type-B blast furnace cement:

$a = 3.7 \times exp\{-6.8 \times (W/C)\} \times g$ and

$b = 0.25 \times exp\{2.5 \times (W/C)\} \times h$; and

(Commentary 4.2.9)

g, h: coefficients that express the effect of high temperature history, derived from:

$g = 0.060T_{max}$ and

$h = -0.0075T_{max}$ (20°C ≤ T_{max} ≤ 70°C).

(Commentary 4.2.10)

Equations (Commentary 4.2.2) to (Commentary 4.2.10) are prediction equations specialized for application to thermal stress analysis and may not be used for other purposes.

When using a binding material other than those shown above, the autogenous shrinkage value must be determined through separate testing.

When the minimum dimension of the structural member is no greater than approximately 400 mm, it is advisable to consider the effect of drying shrinkage strain, in the same manner as autogenous shrinkage strain. When effects of autogenous and drying shrinkage present problems, it is advisable to enact measures such as the use of an expansive additive.

4.3 External constraining bodies

In principle, the range that should be considered for external constraining bodies is up to the point at which the stress generation that occurs in the constrained body due to change in the volume of the constraining body is negligible.

Commentary: The range that should be considered for an external constraining body should be confirmed through preliminary test calculations. While the range depends on the calculation method, it requires dimensions two to five times greater than those of the constrained body in both the horizontal direction and depth direction, and is dependent on the rigidity ratio between the concrete and the constraining body. Three-dimensional stress analysis must be performed for the system as a whole, including constraining bodies.

When the external constraining body is hardened concrete, bedrock, *etc*., the constraint effect should be calculated as an effect by which slippage does not occur at the interface between the constraining body and the concrete. Even when an external constraining body in contact with the structure has high rigidity, deformation or displacement may occur at the interface. However, to perform evaluation on the safe side, a complete constraint that permits no slippage may be assumed.

4.4 Effects of constraint by rebar

Because autogenous and drying shrinkages are both constrained by steel, analysis should take steel into consideration when the effects of said constraint cannot be ignored.

Commentary: Because rebar constraint has a small effect on stress caused by change in volume due to temperature change, it is generally not necessary to model the rebar. However, in structures in which internal constraint dominates, generated stress may be affected by the presence or absence of rebar constraint, necessitating consideration of the steel in the structures. When autogenous shrinkage is significant or when an aggregate with a thermal expansion coefficient significantly different from that of the steel is selected, the difference between the thermal expansion strain of the concrete and that of the steel cannot be ignored, necessitating modeling of the steel.

Chapter 5 Physical Property Values

5.1 Mechanical properties

5.1.1 Tensile strength of concrete

(1) The tensile strength of the concrete in a structure must be used as the tensile strength of concrete used to calculate the thermal cracking index.

(2) The tensile strength of the concrete in the structure must be derived by appropriately estimating the difference from tensile strength determined from split tensile strength testing on specimens, taking into consideration the effects of construction conditions and the construction work.

Commentary: Unlike the value obtained in tensile testing using small and wet specimens, the tensile strength of the concrete in a structure is greatly affected by unevenness during its production, owing to differences in the dryness state, loading speed, dimensions, *etc*. The tensile strength of the concrete in a structure under general construction conditions and standard construction work can be considered to be the value of the split tensile strength of the specimen reduced by approximately 20%.

Changes associated with the tensile strength of concrete in a structure subjected to the effects of general construction conditions and standard construction work are given by:

$$f_{tk}(t') = c_1 \cdot f_c'(t')^{c_2} \qquad \text{(Commentary 5.1.1)}$$

where

$f_{tk}(t')$: tensile strength of concrete at an effective material age of t' days (N/mm^2);

$f'_c(t')$: compressive strength of concrete at an effective material age of t' days (N/mm^2);

t': effective material age (days) derived from Equation (Commentary 2.2.7) in "Design: Standards" Volume 1; and

c_1, c_2: constants determined by the curing method, *etc*., with $c_1 = 0.13$ and $c_2 = 0.85$ set as standards.

Because these coefficients are determined based on specimens cured in water, appropriate correction must be performed when a curing method equivalent to curing in water is not possible. In particular, in the case of blended cement such as blended blast furnace and fly ash cement, the development of tensile strength is sensitive to the quality of curing. Therefore, it is necessary to appropriately reduce the tensile strength in line with curing conditions. The tensile strength of concrete made with expansive additives may be set to that of concrete made without expansive additives but with the same ratio of water to bonding material.

The compressive strength can be derived using an effective material age with the effects of temperature evaluated as equivalent material age:

$$f_c'(t') = \frac{t' - S_f}{a + b(t' - S_f)} f_c'(i) \qquad \text{(Commentary 5.1.2)}$$

where

$f'_c(t')$: compressive strength of concrete at an effective material age of t' days (N/mm^2);

$f'_c(i)$: compressive strength of concrete at an effective material age of i days (N/mm^2);

i: standard material age of design benchmark strength (days);

a, b: constants for type of cement and benchmark material age; and

S_f: effective material age corresponding to the hardening origin for the type of cement (days).

The constants a, b, and S_f that express the development properties of compressive strength can be derived from **Commentary Table 5.1.1**. Compressive strength at the standard material age of i days can be derived from **Commentary Table 5.1.2**.

Commentary Table 5.1.1 Constants of formula of compressive strength development.

Cement type	Standard material age i (days)	$a=\alpha_1+\beta_1(C/W)$		$b=\alpha_2+\beta_2(C/W)$		S_f
		α_1	β_1	α_2	β_2	
Ordinary Portland cement	28	6.31	−1.36	0.771	0.0494	0.37
	56	6.94	−1.54	0.875	0.0278	
	91	7.37	−1.67	0.946	0.0138	
Moderate-heat Portland cement	28	15.8	−3.44	0.428	0.125	0.42
	56	20.2	−4.79	0.637	0.0862	
	91	24.3	−6.09	0.844	0.0399	
Low-heat Portland cement	28	21.9	−3.94	0.203	0.143	0.50
	56	32.8	−6.92	0.410	0.125	
	91	42.0	−9.72	0.612	0.086	
High-early-strength Portland cement	7	3.27	−0.816	0.512	0.122	0.30
	14	3.96	−1.04	0.711	0.0759	
	28	4.39	−1.19	0.841	0.0428	
	91	4.79	−1.32	0.966	0.0096	
Type-B blast furnace cement	28	14.4	−3.86	0.477	0.140	0.42
	56	17.4	−4.88	0.687	0.0877	
	91	19.2	−5.44	0.787	0.0757	
Type-B fly ash cement	28	13.4	−3.20	0.514	0.116	0.47
	56	16.2	−4.12	0.708	0.0739	
	91	18.4	−4.80	0.850	0.0456	

Commentary Table 5.1.2 Estimation formula of compressive strength at each standard material age (N/mm^2)

Cement type	Standard material age i (days)	$f'_c(i)=p_1+p_2(C/W)$	
		p_1	p_2
Ordinary Portland cement	28	−14.5	28.1
	56	−12.8	28.7
	91	−11.6	29.1
Moderate-heat Portland cement	28	−17.6	27.5
	56	-12.9	28.8
	91	−7.28	29.1
Low-heat Portland cement	28	−17.6	25.2
	56	−13.4	28.7
	91	−6.44	29.4
High-early-strength Portland cement	7	−22.6	30.5
	14	−18.2	31.0

	28	−14.9	30.9
	91	−11.5	30.5
Type-B blast furnace cement	28	−10.2	24.3
	56	−3.38	23.6
	91	−1.43	24.5
Type-B fly ash cement	28	−27.2	31.8
	56	−24.2	32.9
	91	−22.4	34.0

5.1.2 Effective Young's modulus of concrete

Determination of the effective Young's modulus of concrete is set as a standard for the calculation of temperature stress, with the effects of material age and dryness state taken into consideration.

Commentary: For temperature stress, the mechanical properties and change in volume of the constrained concrete during hardening must be evaluated using sequential stress analysis. Calculation of temperature stress using the effective Young's modulus is one method for handling the average change in rigidity associated with the progress of hydration and the stress relaxation associated with creep.

An approximate value of the effective Young's modulus may be easily derived with:

$$E_e(t') = \Phi_e(t') \times 6.3 \times 10^3 f_c'(t')^{0.45} \qquad \text{(Commentary 5.1.3)}$$

Where

$E_e(t')$: effective Young's modulus (N/mm^2) at effective material age of t' days

$f'_c(t')$: compressive strength (N/mm^2) at effective material age of t' days according to Equation (Commentary 5.1.2); and

$\Phi_e(t')$: reduction coefficient of the Young's modulus for consideration of the effect of creep: Until the effective material age at which maximum temperature is reached (however, when temperature fluctuations occur multiple times due to multiple lifts, *etc*., then until the effective material age at the first peak temperature): $\Phi_e(t') = 0.42$;

Effective material age (days) at which maximum temperature is reached + 1 day, and later: $\Phi_e(t') = 0.65$; and

After the effective material age at which maximum temperature is reached, linear interpolation is used up to effective material age (days) at which maximum temperature is reached + 1 day.

The effective Young's modulus of concrete mixed with expansive additives may be the same as that of concrete that is made without expansive additives but has the same ratio of water to binder material.

5.2 Thermal properties

5.2.1 Thermal properties of concrete

Determination of the properties of adiabatic temperature rise of concrete with consideration of constituent materials, mixture, pouring temperature, *etc*. is set as a standard. Heat transfer coefficient, thermal diffusivity, specific heat, and other thermal properties of concrete should be determined based on the mixture.

Commentary: The properties of adiabatic temperature rise of concrete can change due to factors such as the type and mixture of the cement and the rock type of the aggregate. Therefore, it is advisable to use adiabatic temperature rise properties that were measured in the actual materials to be used. When measured values cannot be obtained, the standard values shown in this section can be used instead. The design values for the properties of adiabatic temperature rise when admixture material with significant hydration delay effect is not used can be expressed using:

$$Q(t) = Q_\infty(1 - e^{-rt}) \quad \text{(Commentary 5.2.1)}$$

where

$Q(t)$: the amount of adiabatic temperature rise (°C) at a material age of t days;

Q_∞: the amount of the ultimate adiabatic temperature rise; and

r: a constant related to the rate of temperature rise - all of these are constants determined through testing- and t is the material age (days).

If the type of cement, unit cement content, and temperature during pouring are known, then Q_∞ and r can be estimated from the regression equation shown in **Commentary Table 5.2.1**. An equation that more faithfully expresses thermal insulation temperature rise properties is:

$$Q(t) = Q_\infty\left(1 - e^{-r(t-t_0)^s}\right) \quad \text{(Commentary 5.2.2)}$$

where

t_0, s: the parameters related to the origin and rate of temperature rise, respectively: These were introduced as regression equations based on past data, and vary with pouring temperature: With the exception of low-heat Portland cement, $s = 1$.

Commentary Table 5.2.1 Standard values of Q_∞ and r in Equation (Commentary 5.2.1).

Cement type	$Q(t) = Q_\infty(1 - e^{-rt})$			
	$Q_\infty = a + b \times T_a$ *1)		$r = g + h \times T_a$ *1)	
	a	*b*	*g*	*h*
Ordinary Portland cement	17.5+0.113×C*2)	−0.146+0.000308×C	−0.426+0.00207×C	0.0471+0.0000188×C
Moderate-heat Portland cement	8.0+0.118×C	0.0709−0.00016×C	−0.101+0.000811×C	0.00679+0.0000631×C
High-early-strength Portland cement	15.9+0.135×C	−0.106+0.0000257×C	−0.601+0.0031×C	0.0989−0.0000688×C
Low-heat Portland cement*3)	12.2+0.0912×C	0.0946−0.000159×C	0.218+0.0003×C	−0.00179+0.0000598×C
Type-B blast furnace cement *4)	17.9+0.115×C	−0.149+0.000314×C	−0.325+0.00156×C	0.0216+0.000039×C
Type-B fly ash cement *5)	3.03+0.138×C	0.0741−0.00016×C	−0.0212+0.00033×C	0.00762+0.00013×C

*1) T_a : Pouring temperature (ºC)

*2) C : Unit amount of cement (kg/m^3) , 250kg/m^3 ≤ C ≤ 400kg/m^3

*3) When using low-heat Portland cement, the adiabatic temperature rise curve is approximated by $Q(t)=Q_\infty\{1-\exp(-rt^s)\}$. S is a coefficient related to the adiabatic temperature rise rate and is obtained from the following equation.

$$S = (0.302+0.00104\times C)+(0.00293-0.0000216\times C)\times T_a$$

*4) This is for the case that the content of blast furnace slag is 40% (Blaine value: 4200 cm^2/g). The case that the content of blast furnace slag isn't 40% should be obtained from past data or tests.

*5) This is for the case that the content of fly ash is 18%.

The thermophysical properties of concrete may be assumed to be constants. It is advisable to determine the value of the thermophysical properties of concrete with consideration of the properties of the concrete mixture and of the aggregate properties, unit aggregate content, and wet state of the concrete in particular.

In general concrete, the heat transfer coefficient λ is 2.6–2.8 W/m°C, the specific heat c_c is 1.05–1.26 kJ/kg°C, and the thermal diffusion rate h_c^2 is (0.83 – 1.1) × 10^{-6} m^2/s.

If the heat transfer coefficient and the density ρ are determined, then the thermal diffusion rate and specific heat can be estimated from, respectively:

$h_c^2 = 3.34\times10^{-7}\lambda$ and (Commentary 5.2.3)

$c_c = 3.03\times10^{3}/\rho$. (Commentary 5.2.4)

5.2.2 Thermal properties of ground, bedrock, *etc*.

If thermophysical properties are required for ground, bedrock, *etc*., then using existing data as a reference and determining the properties so that the evaluation will be on the safe side are set as standards.

Commentary: Generally, the density of bedrock is 2600–2700 kg/m^3, the thermal conductivity is 1.7–5.2 W/m°C.

Part 7 Reinforced Concrete Premise and Structural Details

Chapter 1 General

This volume presents a method concerning the details of reinforcing bar arrangement, forms of structural members, *etc*. that satisfy "Design: Main Volume" Chapter 13.

Commentary: Structural specifications are important prerequisites for methods used in the verification of performance.

In this volume, a standard method is presented for a structural specification that satisfies "Design: Main Volume" Chapter 13. When examining the provisions of structural specifications, the scope of application of the verification method and the intent of the structural specifications must be fully understood. As an example, if the dimensions, form, and structural details of a structure, along with the actions and their range of variability, are made clear; if the validity of the verification method has been confirmed through loading tests on specimens that faithfully simulate actual items; and if the method's validity can be assured throughout the design service life, then the examination of structural specifications will be relatively easy. However, if testing is conducted under conditions that differ from actual conditions, then the validity of structural specifications will require a high degree of judgment and examination will not be easy. It must be noted that structural specifications include items that are difficult to address in verification, such as rules for guiding design to reduce the risk of deterioration in quality in construction work. These must be fully examined when considering the provisions of structural specifications.

"Chapter 2 Preconditions for Reinforced Concrete Structures" in this volume corresponds to "Design: Main Volume" Chapter 13. When using the verification method presented in "Design: Standards" Volume 3, these provisions in Chapter 2 must be satisfied. "Chapter 3 Structural Specifications of Structural Members" addresses cases in which design methods based on the linear analysis method presented in "Design: Standards" Volume 1 are applied. "Chapter 4 Other Structural Specifications" determines structural specifications common to various structures, and determines methods, including ways of compensating for structural weaknesses not directly related to the method of verification.

This volume presents standard methods for using rebar that conform to JIS standards. When using rebar not covered by JIS standards, it is necessary to return to "Design: Main Volume" Chapter 13 and conduct separate examinations, including of whether the content of this volume is applicable.

In the case of a prestressed concrete structure, the structural specifications presented in "Design: Standards" Volume 8, in addition to those in this volume, must be satisfied.

2.5.3 Development length of rebar

(1) The basic development length l_d of the rebar is to be the value calculated using Equation (2.5.1), corrected according to (i)–(iii) below. The corrected value l_d is to be not less than 20φ.

$$l_d = \alpha \frac{f_{yd}}{4 f_{bod}} \varphi, \quad (2.5.1)$$

where φ : diameter of rebar;

f_{yd} : design tensile yield strength of rebar

f_{bod} : design bond strength of concrete, with γ_c set to 1.3; this may be derived from f_{bok} in Equation (Commentary 5.2.2) in "Design: Main Volume" with $f_{bod} \leq 3.2$ N/mm^2

α = 1.0 ($k_c \leq 1.0$)

= 0.9 ($1.0 < k_c \leq 1.5$)

= 0.8 ($1.5 < k_c \leq 2.0$)

= 0.7 ($2.0 < k_c \leq 2.5$)

= 0.6 ($2.5 < k_c$),

where $k_c = \frac{c}{\varphi} + \frac{15 A_t}{s\varphi}$

c : the smaller of the value of the covering on the lower side of the rebar and the value of half of the spacing between the rebars to be anchored;

A_t : cross-sectional area of transverse rebar perpendicular to the envisioned split failure cross section; and

s : center-to-center distance of transverse rebar.

(i) The basic development length l_d of the tensile rebar is to be the value calculated from Equation (2.5.1). When providing standard hooks, this calculated value may be reduced by only 10φ.

(ii) The basic development length l_d of the compressive rebar is to be 0.8 times the value calculated from Equation (2.5.1). Even when providing standard hooks, this must not be reduced further.

(iii) When the rebar to be anchored is located at a depth less than 300 mm from the end surface of the concrete during pouring and is arranged at an angle of 45° or less from the horizontal, and when the concrete pouring height below the rebar is not less than 300 mm, the basic development length of the tensile rebar or compressive rebar is to be 1.3 times the value calculated in (i) or (ii).

(2) If the amount of rebar to be arranged, A_s, is greater than the calculated required amount of rebar A_{sc}, then the reduced development length l_o may be derived using:

$$l_o \geq l_d \cdot (A_{sc}/A_s) \quad (2.5.2)$$

$l_o \geq l_d/3$, $l_o \geq 10\phi$,

where ϕ : rebar diameter.

(3) The development length of rebar with a bent anchored portion is to be understood as follows (see **Figure 2.5.2)**.

(i) When the inside radius of the bend is not less than 10 times the diameter of the rebar, the total length of the rebar, including the bent portion, is to be considered effective.

(ii) When the inside radius of the bend is less than 10 times the diameter of the rebar, the rebar up to the intersection of the extension of the linear part and the extension of the linear part after the bend is to be considered effective as the development length, but only when the rebar is extended straight after the bend for a length of not less than 10 times the diameter of the rebar.

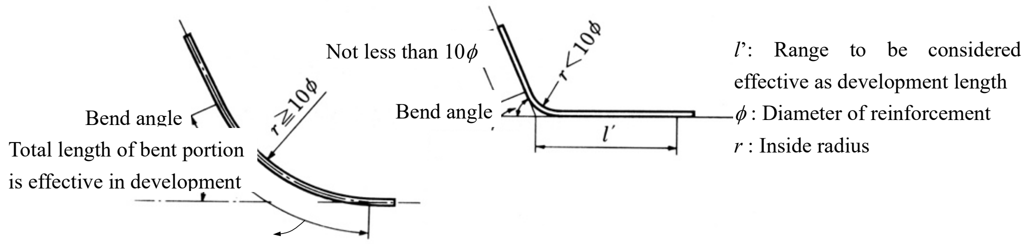


Figure 2.5.2 Development length of reinforcement with bend

Commentary: Regarding (1): The development length of rebar depends on factors such as the type of rebar, the strength of the concrete, the thickness of the covering, the state of the transverse rebars, and others. These factors must be appropriately considered when determining basic development length.

The required development length of rebar when using transverse rebar for reinforcement may be derived using:

$$l_0 = \frac{\left(\frac{f_{yd}}{1.25\sqrt{f'_{cd}}} - 13.3\right)\varphi}{0.318 + 0.795\left(\frac{c}{\varphi} + \frac{15A_t}{s\varphi}\right)}, \qquad \text{(Commentary 2.5.1)}$$

where f_{yd} : design tensile yield strength of rebar (N/mm^2); and

f'_{cd} : design compressive strength of concrete (N/mm^2), where γ_c is to be set to 1.3.

Here:

$$f'_{cd} = \frac{f'_{ck}}{\gamma_c} \text{ and} \qquad \text{(Commentary 2.5.2)}$$

$c/\phi \leq 2.5$.

The effects of the type of rebar, the strength of the concrete, the covering, the state of the transverse rebars, and other factors were incorporated into the calculation equation by multiplying these by the coefficient α.

If hooks are provided at the anchored portion of the tensile rebar, then the hook portion of the rebar is added to the development length. Because transmission of force by the bearing pressure of concrete on the inner side of the hooks can be expected, the development length was shortened accordingly. The amount of reduction in the development length of tensile rebar in which standard hooks are provided differs by the type of rebar, the strength of the concrete, and other factors. However, with reference to the standards of various countries, this was set to a uniform 10ϕ. Reduction for hooks is not performed in the case of compressive rebar.

Regarding (2): The development length of the rebar is determined by taking the basic development length l_d, which is determined by the type and arrangement of rebar, the strength of the concrete, and other factors, and by modifying this in line with the use conditions.

When the amount of rebar actually used is greater than the calculated required amount of rebar, the basic development length should be reduced in line with that ratio. If the development length is made too short, then safety with respect to additional force will deteriorate. Therefore, the lower limit of the value was set to l_o.

Part 8 Prestressed Concrete

Chapter 1 General Provisions

(1) This volume presents standards particularly necessary for matters concerning threshold states set in verification of the safety, usability, etc. of prestressed concrete structures or structural members using PC steel material, as well as methods for the examination of these limit states, preconditions for verification, and other matters.

(2) The degree of prestress must be set so that the structures or structural members satisfy the required performance.

Commentary: Prestressed concrete has a structure that improves cracking performance in concrete structural members and reduces the cross sections of structural members, among other effects. It enables selection from a wide range of structural formats and construction work methods. Designers of prestressed concrete structures must follow the provisions presented in these Standard Specifications as well as the provisions of this volume, which presents matters particular to prestressed concrete. Designers must further ensure that structures satisfy the required performance in terms of safety, usability, etc.

The handling of prestress force introduced into concrete structural members in design calculations generally varies with the limit states of items to be verified. In the verification of usability, prestress may be considered an external force in design calculations. To include the effects of prestress in the calculation of cross-sectional failure yield strength in the verification of safety, design calculations must consider only the statically indeterminate force caused by prestress as an action.

In calculating the response value, the redistribution of the moment must be considered and prestress must be modeled using a method that fits the goals of the verification and the analytical method. In general, applicable methods include those that treat prestress as a fixed external force and those that directly model tendons as chord members (one-dimensional wire rod elements). Modeling must enable appropriate consideration of the prestress introduced into concrete structural members, with the anchoring position of tendons, the presence or absence of bond between tendons and concrete structural members, and other factors taken into account.

Regarding (1): The provisions of this volume apply to prestressed concrete made with PC steel material. In cases including the use of concrete with a compressive strength characteristic value in excess of 100 N/mm^2 or the use of high-strength materials outside the scope of Chapter 5 in "Design: Main Volume," factors including the design values of materials must be considered separately. The provisions of this volume may not be applied to the design of the following structures or structural members:

1) Structures or structural members that are prestressed by a method other than the application of tensile force to tendons. Examples include the introduction of prestressing in arch structural members or concrete pavement using jacks, prestressing through the use of expansion materials, and prestressing by pouring concrete on the tension side of steel girders to which flexural moment has been applied and then releasing the flexural moment after hardening.

2) Prestressed steel-frame concrete structures or structural members, and composite structures or structural

members made of steel and concrete.

3) Prestressed concrete structures or structural members that are constantly subjected to temperatures that differ significantly from normal temperatures. Here, "normal temperatures" may generally be considered ambient temperatures of greater than 0°C and less than 40°C around a structural object.

4) Factory products including prestressed concrete piles and prestressed concrete pipes.

Means for introducing permanent prestress into concrete structural members may be classified into two types according to the time at which tendons are tensioned: pre-tensioning methods and post-tensioning methods. Post-tensioning methods may be considered as falling under three types: the internal tendon method, the external cable method, and the use of both of these in combination.

In the internal tendon method, tendons are arranged inside the concrete structural member. The method can be further categorized according to structures made with internal tendons in which concrete structural members and PC steel material are integrated using PC grout, and structures made with unbonded PC steel material. Structures or structural members made with unbonded PC steel material are distinguished from the external tendon system by factors including differences in the form of the arrangement of tendons and the fact that the effective height of the tendons does not change even in a fractured state. However, increases in flexural cracking width caused by lack of adhesion to the concrete structural member, decrease in flexural yield strength, minimum amount of steel material, fatigue resistance of anchoring devices, and other matters must be considered separately.

Conversely, in the external tendon system, tendons subjected to permanent anti-rust and anti-corrosion treatment are arranged outside the concrete structural member, and the structural member is subjected to permanent prestressing through the anchored part or deflecting part. This volume refers to tendons to which the external tendon system can be applied as "external tendons," and addresses cases in which the effective height of the external tendon is no more than the structural member height. Therefore, when stress fluctuations caused by variable action are large and it is necessary to set strict limits on the introduced tension, as in the case of diagonal bracing cables in cable-stayed bridges or additional external tendons arranged in advance for the purpose of unplanned repairs and reinforcement or temporary prestressing during construction work, or when the effective height of the tendon is greater than the height of the structural member and the tendon is directly subjected to the effects of wind with the result of vibration-induced fatigue, any points of difference from the external tendon system covered by this volume should be examined separately and the provisions indicated in this volume should be applied.

For structures or structural members that use the external tendon system, verification must pay attention to factors including increases in flexural crack width caused by non-integration of the concrete structural member and the external tendon; inapplicability of the assumption that plane sections remain plane is not adopted to external tendons ; changes in tension and in effective height of external tendons associated with deformation of concrete structural members; the minimum amount of tendon content; fatigue failure of external tendons; and protection, rust-proofing, etc. of external tendons.

Regarding (2): The degree of prestress should be determined for the limit state of the tensile edge in its service state. In general, structures should be categorized following Chapter 2, and proper setting of the degree of prestress should be verified.

Chapter 2 Classification of Prestressed Concrete

(1) Prestressed concrete is to be classified into PC structures and PRC structures, and verification of the satisfaction of required performance throughout the design service life is stipulated.

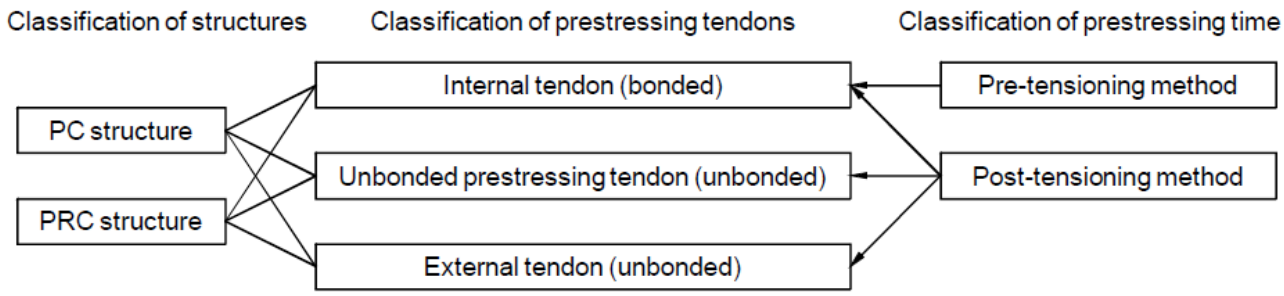
(2) A PC structure is to have a structure by which the edge stress intensity of concrete is controlled through the introduction of prestress, on the assumption that cracking is not tolerated in verification of usability.

(3) A PRC structure is to have a structure by which crack width is controlled through the arrangement of reinforcing steel and the introduction of prestress, with cracking tolerated in verification of usability.

Commentary: Regarding (1): The behavior of prestressed concrete may change significantly before and after the occurrence of cracking. Therefore, depending on whether or not cracking is tolerated in the normal service conditions, the specific calculation method for stress intensity of materials, etc. and the reinforcement method (e.g., the presence or absence of rebars arranged for dispersion of cracks) will differ. Therefore, this volume categorizes prestressed concrete into PC structures and PRC structures on basis of the definitions in (2) and (3), and stipulates verification of the proper setting of degree of prestressing.

At the same time, flexural behavior in prestressed concrete during failure differs according to whether tendons are arranged inside or outside concrete structural members. Flexural behavior similarly differs according to whether the tendons used are bonded to concrete structural members. Therefore, it is necessary to classify the type of structures into PC structures and PRC structures, and to classify the verification method according to the method for arrangement of tendons and the presence or absence of bond . The decision was made to classify tendons into three types: internal tendons, unbonded prestressing tendons, and external tendons, and, in combination with classification according to type of structures, to classify prestressed concrete based on the concept indicated in **Commentary Figure 2.1**. After the introduction of prestressing, PC steel under the pre-tensioning method may be treated in the same manner as internal tendons under the post-tensioning method, and therefore may be classified as internal tendons. Pre-grouting PC steel material should be classified as unbonded PC steel material until effective bond strength has developed, and should be treated as internal tendons after effective bond strength has developed.

Classification of prestressed concrete should be performed for each structural member or cross section. Rational prestressed concrete structures can be constructed by using a mixture of PC structures and PRC structures, according to the purpose of use and the required performance of the structures.



Commentary Figure 2.1 Classification of prestressed concrete structures

Regarding (2): A PC structure is a structure that, in accordance with the provisions indicated in 7.2, limits edge stress intensity and diagonal tensile stress intensity in concrete through the introduction of prestress, on the assumption that cracking will not be tolerated in verification of usability. The material stress intensity may be calculated with the entire cross section of the concrete considered valid. As a general rule, the constraint action of rebars should be considered in the effects of concrete creep and shrinkage.

Regarding (3): A PRC structure is to have a structure by which crack width is controlled through the arrangement of reinforcing steel and the introduction of prestress, on the assumption that cracking is tolerated in verification of usability. Specifically, crack spacing is controlled by the crack dispersion action of the reinforcing steel in the same manner as in reinforced concrete, and the amount of increase in rebar stress intensity is controlled by prestress. Therefore, examination of flexural cracking is to be performed in principle on the tensile rebars arranged in the position closest to the concrete surface, and flexural cracking width is to be calculated by setting the coefficient k_1 that represents the effects of the surface form of steel material on crack width to 1.0 in Equation (2.3.3) in 2.3.4 in "Design: Standards" Volume 4.

In PRC structures, the states of actions to be verified can be set in steps according to factors including the proportion of permanent actions among all actions and the frequency and duration of variable actions and accidental actions. As a matter of principle, the degree of prestress can be arbitrarily set by properly setting the tensile edge threshold state for the state of each action. A PRC structure is a structure that encompasses the entirety of the ground between reinforced concrete and PC structures. Adopting a precise verification method that takes the effects of cracking into account as necessary generally enables an extremely broad range of application for PRC structures.

When a special verification method that enables consideration of the effects of cracking over time is not followed, a limit state in which the concrete edge tensile stress intensity does not exceed the limit value should be set on the assumption that cracking is not caused by permanent actions, and proper setting of the degree of prestress should be verified by using a formula for calculation of the amount of reduction in the introduced PC steel tensile stress intensity on the assumption that the entirety of the concrete cross section is valid. The limit value of edge tensile stress intensity may be determined with the flexural cracking strength of concrete as a guideline.

Setting such a limit state enables prevention of cracking early in the material age of the concrete, and therefore is able to inhibit expansion over time in the width of cracks caused by variable actions. Moreover, by selecting a limit state for which the limit value of edge tensile stress intensity is 0 (i.e., a decompression state which does not cause tensile stress on the tensile edges of structural members), any cracks occurring under variable actions may be judged to be cracks that effectively close under permanent actions. Therefore, the limit value of the edge tensile stress intensity of concrete should be appropriately set according to the frequency and duration of variable actions and accidental actions and according to the proportion of permanent actions among all actions, as noted earlier. However, when the effects of constraints on concrete shrinkage are large or when the effects of variable actions are large and may also affect the state of permanent actions, these effects should be considered.

Part 9 Precast Concrete

Chapter 1 General

(1) This volume presents standards for vital matters regarding the limit states set for the verification of the safety, usability, *etc*. of precast concrete structures or structural members manufactured in factories or in on-site production facilities, methods for the examination of these, preconditions for verification, and other matters.

(2) Precast concrete must be designed to ensure the required performance of singular units or of structures made with these.

Commentary: In this volume, the characteristic value of the compressive strength of concrete is 80 N/mm^2 or less. This volume covers precast prestressed concrete and precast reinforced concrete that satisfy "Preconditions and Structural Specifications for Reinforced Concrete" in Volume 7 and "Preconditions and Structural Specifications for Prestressed Concrete" in Volume 8, Chapter 10 of "Design: Standards." Precast concrete may be used in singular units, in joined structural members, or joined with poured-in-place concrete. The structure as a whole, including precast concrete singular units and interfaces between precast concrete structural members, must exhibit appropriate safety with respect to actions during construction and during service, and must exhibit sufficient functionality during normal use. Precast concrete structural members and their interfaces must also possess sufficient durability over the targeted design service life of the structure. In other words, guaranteeing the durability of the materials, as well as the setting and verification of the durability of structural members and of limit states at locations where structural members interact, such as the interfaces between structural members, is essential to maintain resistance with respect to the performance of the structure. Because events that exceed the conditions set in design may occur in real-world environments, it is advisable to incorporate redundancy and robustness into the design. Toward that end, the limit states of structural members and structural member interfaces must be made clear. To implement engineering measures against states that reach or exceed limits, considerations including the control of damaged locations and of the types of failure in structural members are necessary during design.

In the design of factory products, specified values such as cracking strength and failure strength must be satisfied. In such cases, specifications may be determined appropriately with consideration of the manufacturing method and concrete mixture. Rules regarding durability may be appropriately determined on the condition that replacement is performed for easily replaced factory products.

Part 10 Performance Verification by Nonlinear Material Finite Element Analysis

Chapter 1 General Provisions

1.1 Scope of application

(1) This volume applies to the use of nonlinear finite element analysis in the verification of performance related to the mechanical properties of concrete structures.

(2) When verifying performance using nonlinear finite element analysis, it is necessary to select an analytical method based on an analysis plan formulated in advance, then to appropriately model the target structure, and perform response analysis. **Figure 1.1.1** shows the steps in verification of performance using nonlinear finite element analysis.

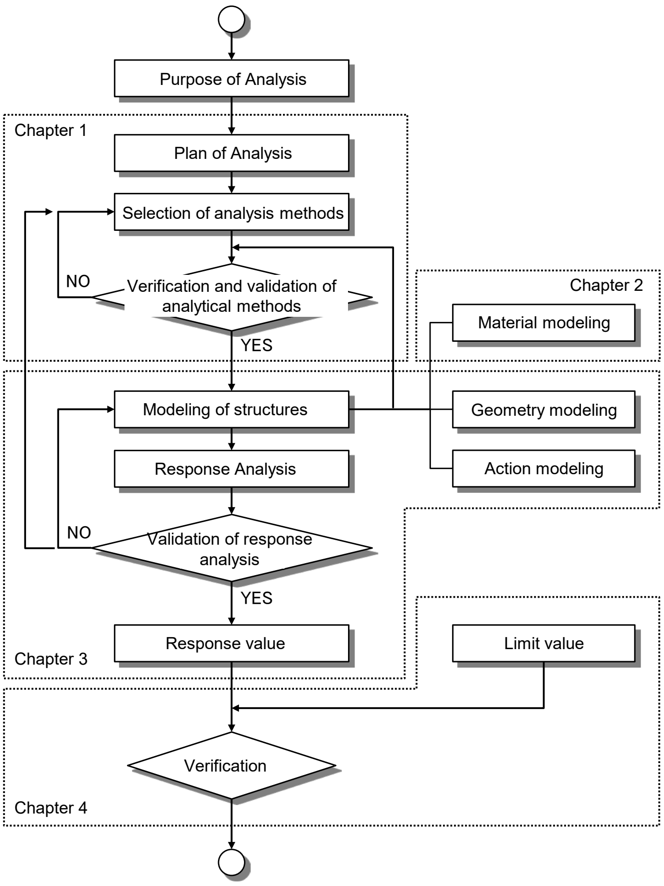


Figure 1.1.1 Steps in the verification of performance using nonlinear finite element analysis.

(3) When using nonlinear finite element analysis, an analytical method, which has been demonstrated and validated and for which the scope of application is clear, must be selected according to the goals of the analysis. When the analytical method has already been demonstrated and validated and is used within its scope of application, the process of demonstration and validation of the analytical method may be omitted.

(4) When response analysis is performed using nonlinear finite element analysis, the validity of the response analysis results must be evaluated appropriately.

(5) Nonlinear finite element analysis may, within its scope of application, be used for the calculation of design response values, the calculation of design limit values, or the verification of performance.

(6) Methods for the verification of performance indicated in this volume must be used on the premise that Volume 2 of "Design: Standards" is satisfied.

(7) This volume does not cover performance evaluations of structures in which the properties of constituent materials change over time, such as analysis of material deterioration caused by environmental actions or thermal stress analysis of concrete.

Commentary: Regarding (1): In these standards, methods that have been used in practical work and for which applicability and reliability have been confirmed are positioned as the current standard methods for nonlinear finite element analysis. Matters to be observed when using methods for verification of performance are also described. Modeling is performed using a distributed rebar model that assumes the integration of rebars and concrete. It is premised on analysis using a method that introduces averaged constitutive law in regions around rebars where the compounded action of concrete and rebars is dominant, and introduces the concept of concrete fracture mechanics in regions to which the compounded action of concrete and rebars does not extend.

This volume presents a standard method for using three-dimensional finite element analysis for verification of performance related to the mechanical properties of concrete structures. Depending on the form of the structure, the properties of actions, and the properties of the resulting response, modeling of the structure in two dimensions may be possible. The finite element method yields approximate solutions to complex problems by dividing the region of interest into minute elements and expressing solutions in each region through relatively simple functions. Methods of modeling are broadly divided into those in which each structural member in the structure is modeled as a set of small elements, and those in which each structure is modeled as a single element. This volume covers methods for modeling three-dimensional or two-dimensional element regions as assemblies.

Unless otherwise specified, this volume indicates material nonlinear finite element analysis that considers nonlinearity in the stress-strain relationship of materials for cases of nonlinear finite element analysis or nonlinear analysis. Regardless of whether materials are linear or nonlinear, depending on the problems to be addressed, it may be necessary in some cases to perform analysis that considers geometric nonlinearity. This volume will specify this analysis in such cases.

Nonlinear analytical methods remain a field under development. It is advisable to make active use of up-to-date knowledge in this field, following the basic concepts presented in this volume.

Regarding (2): Nonlinear finite element analysis must be applied following the formulation of an analysis plan in advance, in line with the goals of the analysis. In the analysis plan, analytical methods with functionality suited to the goals of the analysis must be selected and the analytical method must be demonstrated and validated, taking into consideration the combinations of verification metrics and limit values to be used in verification of performance of the structure. Here, "demonstration" refers to the process of confirming element division dependence and convergence in discretization and convergent calculations, confirming the setting of analysis conditions, and otherwise confirming that the analysis is performed properly. "Validation" refers to the process of using comparisons with experimental results to confirm that the numerical calculation model reproduces actual phenomena with predetermined accuracy.

When modeling a target structure based on the formulated analysis plan, modeling of materials, modeling of the form of the structure (including boundary conditions), and modeling of the actions that affect the structure are performed, based on the properties and functions of the selected analytical method. Following response analysis using these models and evaluation of the validity of the response analysis results, response values are calculated according to the goals of the analysis and verification is performed on the basis of comparisons with the limit values. Whether the response analysis yields a valid solution, and the extent of that validity, must be determined on the basis of engineering knowledge. Evaluation of the validity of the response analysis here refers to interpretation and judgment of the results of the response analysis from an engineering and technical viewpoint.

Regarding (3): Linear analytical methods have yielded many achievements, and may be considered sufficiently applicable to regions in which the nonlinearity of the constituent materials of structures is not significant. Analytical methods not covered in this volume must be used for verification of performance after demonstration and validation of the methods (including modeling), and after determination of the appropriate scope of application.

When nonlinear finite element analysis is used, demonstration and confirmation of validity of the division of elements and the mechanical model of the materials are vital. Prior to the analysis, the accuracy, applicable range, *etc*. of the analytical method must be assessed through the selection of reliable demonstration experiments (structural member experiments, *etc*.) or through preliminary analysis of demonstration experiments conducted separately. When appropriate experimental data is difficult to obtain, the analytical method must be validated through means such as reference to past examples of similar analysis.

Following this, in the modeling of the materials, forms, and actions of the structure to be verified, items used in the analysis of demonstration experiments can be applied mutatis mutandis. When changes must be made to element dimensions because of constraints in the analytical environment or other constraints, the effects of size dependency and of the coarseness of the element division must be considered in advance.

A variety of modeling and material constitutive laws have been proposed for the nonlinear finite element analysis of reinforced concrete structures. These include modeling of cracks, modeling of the interaction between concrete and rebars after the onset of cracking, and modeling of stress transmission on the cracked surface. It is important to select appropriately from among these in line with the target of analysis.

Because nonlinear finite element analysis can express the planar and spatial breadth of structures in a manner that approximates reality, it is also applicable to structures to which preconditions for the application of wire rod models do not hold, or to the verification of performance of deteriorated structures. The use of the fixed crack model and the average stress-to-average strain relationship with rebars and concrete integrated were set as standards, with the degree of difficulty of application to verification of performance in actual structures and past results from actual use taken into consideration.

Here, the calculation of crack width was premised on a method of indirectly evaluating average crack width from crack spacing and average strain.

Analytical models and constitutive rules not indicated in this volume may be applied after demonstration and validation are performed according to section 1.3.

Regarding (4): Even when demonstration and validation are performed on the basis of an analysis plan formulated in advance and response analysis is performed using an analytical method for which the scope of application is fully considered, the response values obtained may not always be appropriate because of factors including differences in conditions between the structure subjected to verification and the experiments used in demonstration. When calculating response values for the structure, the validity of the obtained results must be fully evaluated before being used in verification.

Regarding (5): Verification of the performance of structures is performed through comparison of design response values and design limit values. Nonlinear finite element analysis can be used in cases involving individual tasks such as calculation of design response values or design limit values, or cases of blanket verification extending to the collapse of the entire structural system. Nonlinear finite element analysis may be used for any of the tasks covered in this volume.

Regarding (6): The methods indicated in this volume address verification of performance based on the mechanical properties of structures. The methods for modeling the stress-strain relationship of materials, division of elements, and other matters are premised on the satisfying of structural details such as the steel material and assumptions concerning changes over time caused by environmental actions. Therefore, it must be separately confirmed that the provisions in Volume 2 of "Design: Standards" are satisfied.

Regarding (7): Analysis of the penetration of chloride ions or permeation of moisture, thermal stress analysis performed in verification of initial cracking in concrete, or other analyses that consider changes over time in constituent materials are not covered by this volume. In the evaluation of the performance of existing structures based on inspection findings, changes over time in the mechanical properties of materials such as strength or stress-strain relationships must be appropriately evaluated and modeled. The use of finite element analysis in such performance evaluations is to be according to the "Maintenance" volume of the Standard Specifications for Concrete Structures. However, this volume may be used as reference for general items such as analysis planning, demonstration and validation, and modeling of structures.

1.2 Analysis plans

(1) When nonlinear finite element analysis is applied to the calculation of response values of structures, matters including the selection of analytical method, the modeling of materials, forms, and actions, the setting of conditions for response analysis, the evaluation of the validity of response analysis results, and the setting of verification metrics must be considered.

(2) Selection of the analytical method is to be performed in line with the goals of verification, the verification metrics, and mechanical properties including the hysteresis properties and failure modes of the concrete structure to be verified.

(3) The accuracy and scope of application of the selected analytical method must be confirmed through demonstration and validation of the method. Validation of the analytical method is to be performed using reliable experimental results and the scope of application of the analytical method is to be determined based on said results. The accuracy of the analytical method is also to be examined.

(4) In the modeling of materials, the applicability and combinations of mechanical models of materials are to be taken into consideration when selecting the models to be used and when setting their parameters.

(5) Modeling of the form of the structure must be performed with consideration of matters including scope of analysis, analytical dimensions, modeling of structure units, and boundary conditions, based on the division of elements, analytical accuracy, and other information obtained through demonstration and validation of the analytical method.

(6) Modeling of actions is to take combinations, directionality, scale, time dependence, and other factors into consideration.

(7) Response analysis is to be performed by setting conditions related to the numerical calculations of the analytical method and to the categories of properties of actions in the analysis, in line with the goals of the verification.

(8) The validity of the results of response analysis must be evaluated. When the required accuracy is not satisfied, the analytical method and modeling must be reviewed.

(9) In verification, appropriate verification metrics and limit values must be set in line with the analytical method.

(10) The safety coefficient used in verification is to be set according to Chapter 4 of "Design: Main Volume."

Commentary: Regarding (1): When nonlinear finite element analysis is applied to the verification of performance of a structure, the analysis plan must be sufficiently examined to ensure the reliability of analytical results and the validity of the verification. In the analysis plan, consideration is to be given to the modeling of actions, forms, materials, and other factors in order to appropriately model the target structure. To obtain reliable solutions, the conditions for the response analysis must be set appropriately and the results of the response analysis must be evaluated for validity. In verification of performance, it is important to set appropriate verification metrics matched to the goals of the verification and the properties of the analytical method, and to use the results of the response analysis within the scope for which validity has been confirmed. In order to achieve reliable verification using nonlinear finite element analysis, it is advisable that an analytical engineer who possesses the appropriate ability perform the response analysis.

Regarding (2): In verification of performance using nonlinear finite element analysis, an analytical method capable of appropriately evaluating the hysteresis properties, failure modes, *etc*. of the structure must be selected.

In principle, the analytical method should use a method that directly obtains response values for the verification metrics. When these cannot be obtained directly, they may be replaced with verification metrics through an appropriate method. **Commentary Table 1.2.1** and **Commentary Figure 1.2.1** show the relationships between the finite element method and other analytical methods and the limit values of the mechanical properties of rod members.

Commentary Table 1.2.1 Relationships between analytical methods and the limit values of the mechanical properties of rod members.

Analytical method	Modeling		Rule of construction	Limit values of mechanical properties of members									
				①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩
Finite element method	Material Elements	Three dimensional	$\sigma-\varepsilon$	◎	◎	◎	◎	◎	◎	◎	△	×	△
		Two dimensional	$\sigma-\varepsilon$	◎	◎	◎	◎	◎	◎	◎	△	×	△
Fiber model		One dimensional	$\sigma-\varepsilon$	◎	◎	◎	△	◎	○	○	○	×	△

Wire rod model	Cross-section element	$M-\varphi$	◎	◎	◎	△	◎	○	○	○	×	△
	Member element	$M-\theta$	◎	◎	◎	△	◎	○	○	○	×	○

Note) 1) ◎ : Limit values obtained directly from the analysis

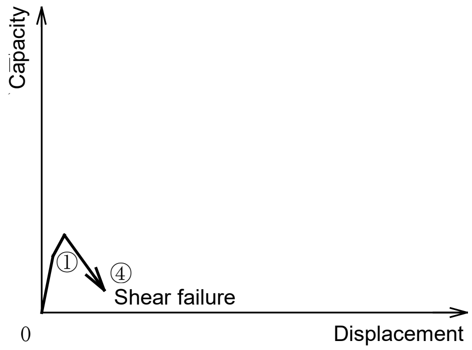
○ : Limit values that can be obtained directly from the analysis if based on empirical engineering formulas with a range of applicability

△ : Limit values that are the indicators for which only response values can be obtained from the analysis and for which the limit values need to be calculated separately.

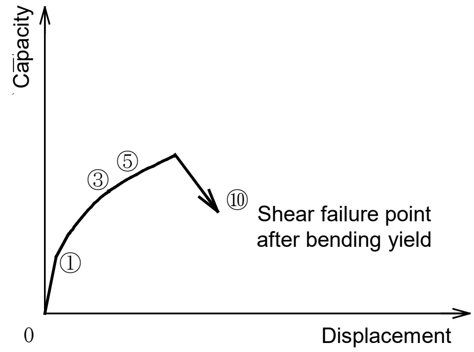
× : Limit values that cannot be verified

2) Numbers with a circle in the table refer to **Figure 1.2.1**.

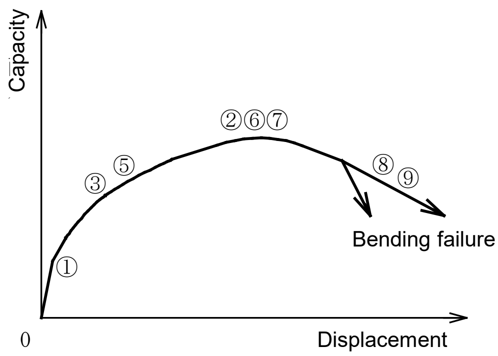
3) The symbols in the table are σ: stress, ε: strain, M: bending moment, φ: curvature, θ: angle of rotation.



(a) Shear failure



(b) Shear failure after bending yield



(c) Bending failure

① Crack initiation point
② Point where the concrete reaches its compressive strength
③ Point at which steel reaches yield strength
④ Point of shear failure before flexural yield
⑤ Yield point of member
⑥ Buckling start point of axial rebar
⑦ Spalling point of cover concrete
⑧ Yield strength
⑨ Crush point of core concrete
⑩ Shear failure point after bending yield

Commentary Figure 1.2.1 Example of a force-displacement relationship envelope for a bar member by fracture mode type

Regarding (3): The selected analytical method must be validated through comparison with reliable experimental results. When doing so, the characteristics of the structure that is the subject of verification should be taken into consideration, and experiments for which information on action conditions, boundary conditions, material properties, *etc*. is clearly indicated should be used. Experiments that enable confirmation of the reproducibility of all failure modes of the structure must be targeted. Here, the applicability of the analytical method to the structure that is the subject of verification should be confirmed, together with the scope of reliable analysis results.

Regarding (4): A mechanical model of materials that is capable of expressing varied types of damage to the constituent materials must be selected so that the failure modes of the target structure can be accurately understood. When a combination of multiple material models is used, attention should be paid to overlap among phenomena to be reproduced and differences in the scope of application of the model. The recommended values should in principle be used as parameters in material models.

Regarding (5): The structure to be subjected to analysis must be modeled by appropriately setting the scope and the dimensions of the analysis to achieve the goals of the verification. Boundary conditions, division of elements, and other factors should be set based on the analytical accuracy obtained through demonstration of the analytical method. Modeling at the level of the structure, structural members, parts, and so on must be performed so that failure modes are appropriately reproduced without loss of the characteristics of the structure that is the subject of verification.

Regarding (6): In combinations of different types of design actions, initial actions must be distinguished from the main variable actions in analysis. In the case of seismic response analysis in particular, analysis must be performed under conditions in which dead load, temperature, shrinkage, creep, earth pressure under ordinary conditions, water pressure, live load, and other factors are separately loaded in advance as initial action states.

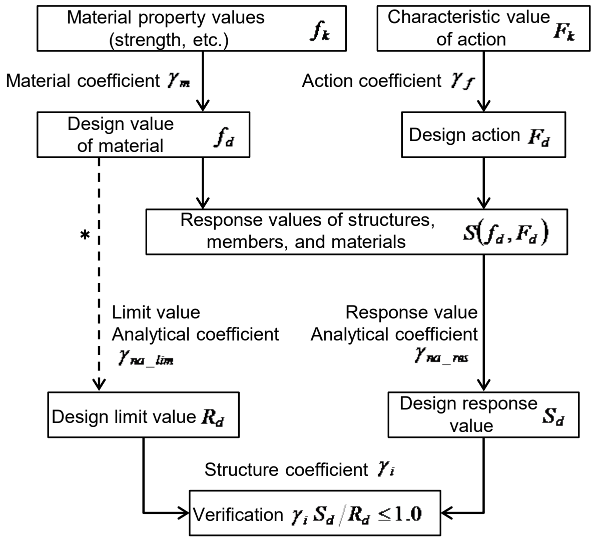
Regarding (7): In response analysis, selection of the solution method, setting of convergence conditions, and other conditions related to numerical analysis must be appropriately set in line with the goals of the verification. In some cases, the analytical solution may be affected by parameter settings related to the solution, and its validity may not be guaranteed across the entire scope until the end of the analysis. The reliability of the analytical solution must be considered through means such as demonstration of the analytical method.

Regarding (8): In evaluation of validity, it should be confirmed that the planned analysis was performed correctly and that the results of response analysis are valid in terms of engineering within the allowed scope.

Regarding (9): When using nonlinear finite element analysis, information expressing various mechanical properties can be obtained, according to the selected analytical method. When using nonlinear finite element analysis for verification, those items appropriate as verification metrics must be selected from among this information and corresponding limit values must be set. "Design: Main Volume" presents a method for setting strain, stress intensity, cross-sectional force, crack width, displacement, and other items as verification metrics. When using verification metrics other than these, demonstration must be performed in advance in accordance with reference materials, and the scope of application of the metrics must be confirmed.

Regarding (10): Because the effects of material constants are reflected in response values when nonlinear finite element analysis is used, the safety coefficient differs from the coefficient when linear analysis is used. Chapter 4 of "Design: Main Volume" mainly describes the handling of safety coefficients when cross-sectional force, displacement, and other items are used as verification metrics. When calculating response values and verifying performance using verification metrics other than these, the applicability of the analytical model and the analytical accuracy within the scope of verification must be ensured in accordance with this volume, and safety coefficients must be set so that the results of verification are able to maintain necessary and sufficient accuracy.

As shown in **Commentary Figure 1.2.2**, the action coefficient, material coefficient, analytical coefficient, and structure coefficient should be set as the four partial safety coefficients. The analytical coefficient considers both the structural member coefficient and the structural analytical coefficient as factors. The action coefficient and structure coefficient are to be in accordance with Chapter 4 of "Design: Main Volume." The material coefficient may be set to 1.0 on the premise that the characteristic value is set with consideration of inconsistency in strength and other material properties. This means that the characteristic values of materials are set so that the response values of verification metrics become unfavorable for reasons of inconsistency in strength, *etc*. Verification must be performed by setting the states of multiple material properties as required. For the analytical coefficient used to calculate the design response value S_d, the effects of analysts' knowledge, experience, *etc*. with respect to the analytical method must also be considered in addition to the structural member coefficient and the structural analytical coefficient. The analytical coefficient is generally set between 1.1 and 1.5. Setting this requires that the responsible engineer judge the knowledge and experience of analysts with respect to the analytical method. The analytical coefficient used to calculate the design limit value R_d should be set with consideration of the accuracy of the setting of characteristic values of the limit values and with consideration of uncertainty factors accounted for by the structural member coefficient.



∗ is shown here as a dotted line because the design limits are not only determined by the design value of the material or by the structural analysis with design value input, but may also be independent of the design value of the material.

Commentary Figure 1.2.2 Safety factor when nonlinear analysis is used.

1.3 Demonstration and validation of the analytical method

1.3.1 General

(1) When applying an analytical method to the verification of performance of a structure, demonstration and validation of the analytical method selected on the basis of the analysis plan must be performed, and the accuracy and scope of application of these must be confirmed in advance.

(2) In demonstration of the analytical method, sensitivity analysis is to be performed in order to assess matters including the effects of the setting of material constants and material models and the effects of the division of elements and various parameters on the results of analysis.

(3) Validation of the analytical method is to be comprehensively evaluated through comparison with the results of independent methods such as theoretical solutions, alternative analyses, and experimental results for which reliability

has been confirmed.

Commentary: Regarding (1): The accuracy of the analytical method that was selected based on the analysis plan is to be confirmed through demonstration and validation of the analytical method. At the same time, it is possible to assess the degree of the effects that the set analytic model and various parameters have on the results of analysis, to judge the scope of application of the analytical method in the subsequent verification of performance the structure, and to calculate the response values for the verification items set as goals. Even when the selected analytical method does not yield sufficient results, if the accuracy and scope of application of the analytical method are correctly understood, the analytical method can be used in calculating the response values and limit values of the structure. In the subsequent response analysis of the structure, however, care must be taken to ensure that usage does not exceed the scope of application determined here. When the demonstration and validity of the analytical method have been performed through analysis at the structural member level, the analytical method conditions used when calculating the response values of the structure assume the use of the analytical conditions that has been confirmed at the structural member level.

Regarding (2): The characteristics of the analytical method should be assessed by performing sensitivity analysis in advance to consider the effects of material constants and material modeling; the effects of modeling of element dimensions, division of elements, and other forms; and the degree of the effects of various parameters on the results of analysis.

Regarding (3): In validation of the analytical method, it is necessary to appropriately select the results of independent methods such as the experiments to be used in comparisons, theoretical solutions, solutions based on empirical engineering formulas, and alternative analyses.

When performing validation of the analytical method, reliable experiments for which information has been disclosed must be selected as the subjects of comparison. Experiments to be used in validation should have clearly indicated goals, range of parameters handled, and scope of application. Boundary conditions and material constants necessary for constructing an analytical model should be sufficiently presented, and information on the inconsistency, accuracy, and reproducibility of experimental results should be available.

In validation, the results of analysis must be comprehensively evaluated. As an example, appropriately judging the maximum yield strength and form of failure from the load-displacement relationship alone is difficult if information on fracture properties (crack diagrams, *etc.*) is not also used.

1.3.2 Improvement of analytical models

If the results of analysis do not satisfy the required accuracy, improvements must be made to the analytical model and demonstration and validation of the method must be performed again.

Commentary: When errors are present in the analysis conditions and the expected results cannot be obtained from the analysis or when more accurate results of analysis must be obtained, the analytical model must be improved to better match actual phenomena. In doing so, the analysis conditions must not be arbitrarily modified to make the results of analysis match the results of experiments. Improvements should be considered in combination with the setting of the scope of application of the analytical method.

1.3.3 Consideration of the scope of application

When using a demonstrated and validated analytical method in the analysis of a structure, the scope of application of the results of response analysis must be set in advance.

Commentary: The scope of application of the analytical method must be judged based on comprehensively analyzing the results of analysis. This permits the use of the selected analytical method within a certain scope of application even when the results of analysis do not perfectly match the experimental results. The judgment should be made rationally through cross-checking against the goals of analysis, cost of analysis, *etc*. Regarding the scope of application, not all metrics obtained through structural analysis are necessarily required to satisfy the results of verification, and it is possible to set a different scope of application for each metric. However, when calculating response values of the structure, care must be taken to ensure that usage does not exceed the set scope of application. The scope of application of the selected analytical method can also be determined by using the analytical coefficient to set the relationship between the accuracy of the selected analytical method and required accuracy to certain ranges.

Part 11 Design by Strut-Tie Model

Chapter 1 General Provisions

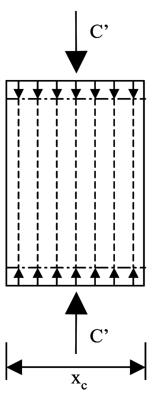
1.1 Scope of application

(1) This volume applies to the design of cross-sectional failure limit states through the application of strut-and-tie models.

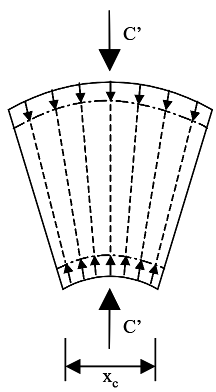
(2) Strut-and-tie models are used in the design of discontinuous areas in concrete structures, including corners, openings, steel anchorage zones, sections with sudden changes in the flow of force, areas with a sudden change in cross section, and other areas in which theories of beams and plates are difficult to apply. In principle, the validity of models is to be verified through experiments or nonlinear finite element analysis.

(3) When calculating yield strength, the compressive force borne by struts and the tensile force borne by ties must satisfy equilibrium conditions at all nodes.

Commentary: A strut-and-tie model is a structural model in which load-bearing mechanisms and force flows with respect to design actions are clearly set. In the model, concrete structures or structural members are discretized into one-dimensional struts, ties, and nodes connecting these struts and ties. The ultimate capacities with respect to set flows of forces are calculated from static equilibrium conditions and from the strength of the struts and ties. Ties are usually modeled as the tensile resultant force of rebars or prestressing steel. Struts are modeled as uniform compressive stress fields such as the compressive chord or a diagonal compression strut in webs of beams, or are modeled as a fan-shaped compressive stress fields such as those above a supporting point or below a loading point (**Commentary Figure 1.1.1**). Nodes are expressed as constant volumes of concrete in sections where struts and ties intersect or where the direction of a flow of force changes because of a tie.



(a) Uniform compressive stress field



(b) Fan-shaped compressive stress field

Commentary Figure 1.1.1 Concrete struts

For beams, columns, slabs, and other structural members for which results from past design and construction work are abundant and for which structural member cross sections and reinforcement arrangement methods that satisfy the required performance are widely understood empirically, verification may be performed by applying Chapter 2 of "Design: Standards" Volume 3. For structural members that include corners, openings, and sections with sudden changes in cross section, methods are to be used by which load-bearing mechanisms are derived for the forms, dimensions, and materials of the assumed structural members based on experiments or high-precision nonlinear analysis, followed by verification of safety.

Using a method which sets load-bearing mechanisms with respect to design actions in advance and achieves these by determining the arrangement of rebars and the strength of materials enables the setting of a design solution that is reasonable overall. Strut-and-tie models are effective in such cases.

Strut-and-tie models in this volume apply only to examination of the limit state of cross-sectional failure. Strut-and-tie models do not strictly consider compatibility conditions for deformation. Therefore, struts and ties must be arranged in a manner that is able to satisfy deformation compatibility conditions without difficulty. Imparting sufficient ductility to structural members to reliably achieve transition to set load-bearing mechanisms is also a condition for the application of strut-and-tie models.

In these standards, strut-and-tie models are treated as one design method. In principle, the appropriateness of the models should be confirmed through experiments or nonlinear finite element analysis. If applicability can be demonstrated through experiments or nonlinear finite element analysis, the models can also be used, within their scope of application, as a verification method.

Materials and Construction

Standard Specifications for Concrete Structures -2018

“Materials and Construction”

CONTENTS

General Requirements

Construction Standards

Inspection Standards

Special Concrete

General Requirements

Chapter 1 General Rules

1.1 General

(1) **"Materials and Construction"** in the Standard Specifications for Concrete Structures applies to construction work of concrete structural objects.
(2) **"Materials and Construction: General Requirements"** presents basic approaches to construction work in accordance with performance criteria principles for the construction of concrete structural objects indicated in design drawing documents.
(3) **"Materials and Construction: Construction Standards"** presents standard matters that builders should implement concerning structural formats, materials, construction equipment, and construction work conditions, within the scope used in general civil engineering construction work.
(4) **"Materials and Construction: Inspection Standards"** presents standards for inspections performed under the responsibility of the responsible engineer with the commissioning party for each stage of construction work of general new civil engineering structural objects and for completed structural objects.
(5) **"Materials and Construction: Special Concrete"** presents information on particularly necessary matters in the manufacture or construction work using concrete not covered in **"Materials and Construction: Construction Standards"**

Commentary: Regarding (1): The **"Materials and Construction"** volume of the Standard Specifications for Concrete Structures presents general basic principles concerning the construction of concrete structural objects. In other words, it presents approaches to construction work for the creation of concrete structural objects that satisfy the required performance specified in the design drawing documents.

Regarding (2): **"Materials and Construction: General Requirements"** assumes the introduction of new technologies, etc., and in principle does not stipulate specific production specifications or construction work methods as long as the completed concrete structural objects satisfy the characteristic values set in design and as long as the cross sections and structural specifications of constructed structural objects conform to blueprints.

Regarding (3): **"Materials and Construction: Construction Standards"** deals with general concrete construction work.

1.2 Basics of construction work

(1) A builder drafts an appropriate construction work plan based on design drawing documents, constructs the structural object while managing the construction work according to the construction work plan, and confirms that the structural object was constructed in line with the design drawing documents.
(2) The responsible engineer for the commissioning party checks the construction work plan drafted by the builder prior to construction work and performs inspections to ensure that the structural object is being constructed in accordance with the design drawing documents.
(3) If it is determined during formulation of the construction work plan that rational construction work is not possible, the responsible engineer for the builder and the responsible engineer for the commissioning party discuss changes to the content of the design drawing documents.

Commentary: Regarding (1): In addition to presenting structural drawings and reinforcement arrangement drawings, design drawing documents must clearly indicate the characteristic values of the concrete, reference values for the materials and mixes planned by the designer to satisfy those characteristic values, and other matters necessary for the construction work, thereby accurately communicating the designer's intent to the builder.

Construction work refers to work that realizes the structural object envisioned in the design. The builder selects the appropriate construction work methods and concrete, and, after using test construction or other means to confirm that the structural object can be constructed in accordance with the design drawing documents, begins the construction work. However, when formulating a construction work plan, the builder must be mindful that all of the information required for the construction work is not necessarily included in the design drawing documents. Tasks in construction work are performed in line with the construction work plan, with quality controlled appropriately at every stage.

Regarding (2): After the responsible engineer for the commissioning party has checked the construction work plan that was drafted on the basis of the design drawing documents, the builder begins the construction of the structural object. Therefore, when checking the construction work plan, the responsible engineer for the commissioning party must confirm that the construction work can be reliably performed and must then independently draft an inspection plan. Quality control is performed under the responsibility and self-direction of the builder. However, the content of the quality control plan should be checked by both the commissioning party and the builder. Delivery of the structural object to the commissioning party takes place after the commissioning party performs and approves inspections confirming that the structural object has been constructed in line with the design.

Chapter 2 Constructability of Structural Objects

2.1 General

Builders must set appropriate construction work methods in accordance with principles for performance criteria and must use appropriate means to verify that structural objects are constructed according to their design drawing documents.

Commentary: The constructability of a concrete structural object is affected by the workability of the fresh concrete, as well as by construction work conditions including the location of the construction site; environmental conditions dependent on the topography and construction work period; structural conditions such as the structural object's purpose, form, and arrangement of reinforcement; working conditions such as processes and labor; and conditions related to materials and equipment. Therefore, in the construction of concrete structural objects, appropriate construction work methods and materials must be selected with consideration of constructability.

Construction work methods for the construction of a concrete structural object may be set freely as long as the principles for performance criteria are followed. **"Materials and Construction: Construction Standards"** assumes construction work performed using general methods, and describes steps regarding selection of materials, mix design, and setting of construction work conditions, as appropriate to the general methods. By contrast, this volume further assumes the introduction of new technologies, etc., and prescribes setting the construction work methods first and then performing concrete mix design to match.

2.2 Setting of construction work methods

Appropriate construction work methods must be set based on a comprehensive consideration of factors including on-site construction work conditions, environmental conditions, and economic efficiency.

Commentary: In concrete construction, it is important to thoroughly examine the workability of concrete structural objects; to perform construction work rationally, economically, and safely; and to construct structural objects that possess the required performance.

Chapter 3 Construction Work Planning

3.1 General

Builders draft appropriate construction plans and prepare construction work plan documents to construct the concrete structural objects indicated in the design drawing documents.

Commentary: In the drafting of a construction work plan for erecting the concrete structural object indicated in the design drawing documents, the structural conditions of the structural object and the on-site environmental and construction work conditions are taken into account. In addition, taking work safety and environmental load into consideration, an examination is undertaken of the overall process, the construction work methods, the materials to be used, the concrete mix, the concrete production method, and quality control plans. In general, construction work plan documents include an overview of construction; requirements for the construction and the structural object; processes; labor and organizations; key machinery; important materials; temporary facilities; construction work methods; quality control (including construction work management); frameworks for the occurrence of emergencies; traffic management; occupational safety and health management; environmental protection measures; and other matters and details related to the overall construction.

The information on environmental protection planning and occupational health and safety planning provided in these Standard Specifications is not always sufficient, so it is necessary to carefully check environmental laws and standards related to construction and to refer to related documentation and past examples when engaging in planning.

Concrete structural objects in civil engineering often have large cross sections, which can lead to issues with thermal cracking caused by the heat of hydration of cement. Specific methods of temperature regulation, transport, placement, and curing of concrete are to be incorporated into construction work planning to obtain full effectiveness from thermal cracking control measures that were considered prior to construction work.

3.2 Setting of concrete construction work methods

Builders set appropriate construction work methods with thorough consideration of the effects of concrete construction work methods and environmental conditions on the post-hardening performance of the concrete used in the structural object.

Commentary: The performance of concrete after hardening is affected by the concrete transport method, the placement position, the deployment of workers and the content of the work they perform, the compaction method, the finishing method, the curing method, and other construction work methods, as well as the atmospheric temperature and other environmental conditions at the time of placement. Accordingly, the adoption of appropriate construction work methods is important.

3.3 Concrete mix planning

3.3.1 Performance of concrete after hardening

Builders select concrete that satisfies the characteristic values specified in the design drawing documents.

Commentary: The characteristic values that express the performance of concrete after hardening should generally be confirmed through tests using specimens, with the requirement that the values satisfy the values specified in the design drawing documents.

3.3.2 Concrete performance required for construction work

Builders set the workability and strength development properties of concrete as appropriate for the selected work construction methods.

Commentary: The workability of concrete must be set appropriately to facilitate tasks including concrete transport, placement, compaction, stacked placement, and finishing, in line with factors including the concrete placement locations, the cross-sectional form and dimensions of structural members, and the arrangement of steel materials. When setting the workability, changes in the fresh concrete properties over time are to be fully considered, and the pumpability, filling ability, compactability, settling properties, and other properties required at each stage of the concrete work are to be set. When the required workability cannot be ensured, the construction work methods must be reviewed.

Appropriate strength development properties are required to ensure structural safety with respect to actions that are expected to occur before completion of the structural object and to ensure rational construction work as well as the performance of the structural object immediately after completion. Concrete strength during construction work is affected by factors including the placement temperature, the material quality of the formwork, the curing method, and the ambient temperature, so the timing of the removal of formwork and shoring must be determined considering these factors.

3.3.3 Selection of materials

The materials used in concrete are to be checked for quality.

Commentary: The use of appropriate materials is extremely important in production concrete that possesses the required performance. The suitability of materials may be judged through testing or based on past results. In construction work following the principles of the performance criteria, any materials may be used so long as the concrete meets the characteristic values specified at the time of design. However, the builder must show by appropriate means that the quality of the materials has been ascertained and that the concrete made with those materials satisfies the characteristic values for concrete that were set in the design.

3.3.4 Mix design

The concrete mix is to be designed in consideration of the constraints on the production plant, the availability of materials, and economic efficiency (including that of transportation costs) to ensure that the concrete satisfies the required performance.

Commentary: In mix design, trial mixing is performed using materials and a mix that are thought to yield fresh concrete that satisfies the performance specified for concrete during construction work, and to yield concrete after hardening that achieves the required performance values that were set based on the characteristic values and their inherent variability. This work is repeated until the concrete is confirmed to satisfy all aspects of the required performance. Combinations of concrete materials and mixes are to be selected from among multiple combinations that satisfy required performance.

3.3.5 Verification of the performance of concrete

(1) Verification is performed to confirm that the concrete made using the selected materials and mix proportions satisfies the characteristic values specified by the design.

(2) Verification is performed to confirm that the concrete made using the selected materials and mix proportions possesses workability and strength development properties in line with the construction work method.

Commentary: Regarding (1) and (2): In principle, the performance of concrete is to be confirmed through testing. However, when sufficient past results are available, verification of performance can be deemed to have been achieved based on those past results, with responsibility borne by the builder.

Characteristic values to be examined when confirming the performance of concrete after hardening include the strength, shrinkage, and carbonation diffusion coefficient that were set in the design of the structural object. At present, there is no established method for the appropriate and quick verification of resistance to alkali-silica reaction. Verification of resistance to alkali-silica reaction may be replaced by specific control measures to prevent alkali-silica reaction from reaching a hazardous level during the service period of the structural object.

Performance of concrete specified in construction work is to be verified by performing construction work under the set methods using specimens, etc., that are of sizes capable of simulating the structural object to enable confirmation that there are no unfilled locations and to confirm the strength development properties that were set. If these verifications reveal that even one of the specified aspects of performance is not satisfied, the selection of materials and the mix are to be changed and verification is to be performed again. When concrete mix design cannot be rationally performed, the construction work method is to be reviewed.

3.4 Concrete production planning

The builder creates a production plan to ensure that the concrete possesses the required performance.

Commentary: To manufacture concrete that satisfies the required performance within a specified range of variability, it is important that the production equipment possesses the required production capacity, that the production method is appropriate, and that quality control is performed by an engineer who possesses the quality control capabilities required for the stable manufacture of concrete having the specified performance. To prevent interruptions in placement work, it is also necessary to create a transport plan that considers the capacity of the production plant, transport capacity, and the transport distance between the production plant and the construction site.

3.5 Concrete work planning

Builders create plans for the transport, placement, compaction, finishing, curing, jointing, etc. of concrete.

Commentary: Concrete work encompasses a series of construction work tasks that include concrete transport, placement, compaction, finishing, curing, and jointing. Generally, any method may be selected for each of the tasks involved in concrete work. However, in the series of tasks from transport to the start of curing, which involve the handling of fresh concrete, the ability to perform tasks smoothly with no interruption in the supply of concrete is important.

3.6 Reinforcement work planning

Builders create plans to enable the processing, arrangement, assembly, etc., of the reinforcing materials indicated in the design drawing documents.

Commentary: Rebar and other reinforcing materials, and the quality of construction work performed using these, greatly affect safety, durability, and other performance aspects of a structural object. Therefore, the type and diameter of reinforcing materials indicated in the design drawing documents must be confirmed, and the materials must be arranged at the specified positions without error. In addition, the spacing of rebars also affects the pourability and compactability of the concrete work. Concrete work and reinforcement work are closely related; therefore, their mutual relationship should be taken into consideration in the planning of reinforcement work.

In addition to rebar, other reinforcing materials include PC steel used in prestressed concrete, stainless steel rebar with enhanced resistance to degradation, epoxy resin-coated rebar or PC steel, and continuous fiber reinforcing materials made with resin-hardened organic fibers. When using reinforcing materials such as steel fiber and organic short fiber, the construction work plan is to be created with a thorough understanding of the characteristics of these materials so that the required reinforcement effect can be obtained.

3.7 Planning of formwork and shoring

Builders create plans for formwork and shoring to ensure that concrete structural objects will have the form and dimensions indicated in the design drawing documents.

Commentary: Formwork and shoring are important for the construction of structures having specified forms and dimensions. In the design of construction work for formwork and shoring, plans are drafted to ensure the specified form and dimensions of the structural object while considering the safety of the construction work and grounded in a solid understanding of occupational health and safety regulations. It is assumed that formwork and shoring materials will be reused, so any changes in the quality of the materials associated with their reuse are to be considered in the planning. Formwork and shoring enable the attainment of the specified form and dimensions, and, with safety also taken into consideration, facilitate the application of special formwork/shoring construction methods; large-size formwork; techniques for the unitizing of formwork, shoring, and scaffolding; and other technologies held by individual builders.

3.8 Quality control planning

To construct structural objects in line with the design drawing documents, builders create efficient and effective quality control plans for every stage of the construction work.

Commentary: To construct structural objects in line with the design drawing documents, builders follow construction work plans when performing construction work. Because it is difficult for the commissioning party to directly inspect the performance of a structural object after completion, inspection is normally performed for every process occurring during the construction work. In addition, from the standpoint of the builder, it would be impractical if pass-fail judgments on completed work could not be performed until the entire structural object has been completed. Therefore, it is important to subject the status of construction work to quality control at every stage indicated in the construction work plan.

Quality control is a self-directed activity performed by the builder, who may use discretion in selecting and implementing what is deemed necessary. However, efficient and effective quality control is facilitated by specifying the parties responsible for and in charge of quality control for all tasks in production and construction work, and by clarifying the items subject to control, the methods for control, and the measures to take in the event of anomalies.

3.9 Construction work plan documents

Builders describe the outline of construction work, the construction work schedule, construction work instructions, occupational safety and health management, environmental measures, construction work systems, and other matters in construction work plan documents.

Commentary: The results of construction work planning are compiled into construction work plan documents and are submitted to the commissioning party for approval. In general, construction work plan documents include an overview of construction; requirements for the construction and the structural object; processes; labor and organization; key machinery; important materials; temporary facilities; construction work methods; quality control (including construction work management); frameworks for the occurrence of emergencies; traffic management; occupational safety and health management; environmental protection measures; and other matters and details related to the overall construction.

3.10 Confirmation of construction work planning

The commissioning party uses the construction work plan documents to confirm that the concrete construction work planning meets the construction requirements and that the structural object can be constructed according to the design drawing documents.

Commentary: Construction work plans must satisfy construction requirements, and the concrete structural object constructed in accordance with the construction work plans must satisfy the required performance. The builder submits the construction work plan documents to the commissioning party, who confirms that the structural object indicated in the design drawing documents can be constructed according to the construction work plan documents. When adopting new technologies or new construction methods, the reliability of the construction work is to be confirmed based on reliable data, actual-scale test construction work, or other means.

Chapter 4 Construction Work

4.1 General

(1) A builder follows construction work plans in performing construction work on concrete structural objects.

(2) At the construction site, the builder appoints an engineer who possesses sufficient knowledge of and experience with construction work on concrete structural objects and performs construction work under the direction of the engineer.

(3) When it is not possible to conform to the construction work plans in the construction work, the builder takes appropriate measures following the instructions of the responsible engineer to ensure that the performance required by the design of the structural object is achieved.

Commentary: Regarding (1): The foundation of construction work is the economic and efficient implementation of work using appropriate construction work methods, with safety in construction as a prerequisite. Because construction work on concrete structural objects consists of diverse types of work including formwork and shoring construction, reinforcement work, and concrete work, it is advisable to sufficiently coordinate related types of work to enable efficient construction work.

Regarding (2): In general, the quality of construction work is greatly affected by human factors such as the experience and aptitude of the builder. Therefore, it is vital to appoint an on-site engineer who possesses sufficient knowledge and experience concerning the construction of concrete structural objects, and to perform the construction work under the direction of the engineer.

Regarding (3): Unexpected situations not envisioned at the planning stage commonly occur in actual construction work, meaning that the work cannot always be performed according to the construction work plan. For cases in which it is difficult to follow construction work plans, appropriate measures are to be taken to ensure required performance under the direction of the responsible engineer with the commissioning party.

Chapter 5 Quality Control

5.1 General

A builder performs quality control at each stage of construction work following a quality control plan.

Commentary: A builder performs quality control based on a quality control plan to construct concrete structural objects that satisfy the required performance. When quality control indicates that the planned permissible control thresholds have been exceeded and the possibility exists that the performance of the concrete structural object will not be satisfied in the future, the causes of the issue must be investigated, countermeasures must be taken, and the quality control plan must be reviewed

Chapter 6 Inspection

6.1 General

(1) The commissioning party drafts a reliable inspection plan based on the design drawing documents and construction work plans, with consideration of the structural object's importance, intended use, and purpose.

(2) Inspections are conducted under the responsibility of the commissioning party, following the inspection plan.

(3) When the findings of inspection are judged to be not acceptable, countermeasures are to be considered.

(4) The commissioning party records and stores the details of the performed inspections.

Commentary: Regarding (1) and (2): Inspections may deem a completed structural object to possess the required performance by confirming that the structure was constructed according to the design drawing documents and through appropriate methods at every stage of construction work. However, in structural objects built through construction work based on the principles of performance criteria, the concrete mix and the construction work methods may differ from standard mixes and methods. Taking the effects of this into consideration, the inspection plan must be able to appropriately confirm that the characteristic values required by the design are satisfied.

The commissioning party drafts an appropriate inspection plan after sufficiently confirming matters including the reliability of the materials proposed by the builder, the construction work methods applied, and the methods for verifying the performance of the concrete. The inspection plan must describe specific matters to be inspected, as well as their associated testing methods, frequencies, and pass/fail criteria. The builder receives the inspection plan prior to the construction work and checks its content. The content of the inspection plan must be as agreed upon between the commissioning party and the builder. The builder incorporates the inspection plan into the construction work plan documents. Inspections are conducted under the responsibility of the commissioning party of the structural object in line with the inspection plan.

Regarding (3): If the findings of an inspection at any stage of the construction work are judged to be not acceptable, the causes of failure to pass inspection are to be investigated and appropriate remedies are to be considered. For cases in which the matter can be dealt with through partial remedies while construction is in progress, quality is to be confirmed through re-inspection after the remedies have been implemented. If appropriate remedies cannot be taken, reconstruction is to be considered.

Regarding (4): Inspection records are the records of the inspection plan and the findings of inspections performed according to the plan. These materials constitute a guarantee that the concrete structural object was constructed according to the design drawing documents, and serve as initial values for maintenance of the concrete structural object following its completion. In particular, if inspections are deemed to have failed at any stage of the construction work, it is important to create detailed records that include information on any remedial measures.

Chapter 7 Construction Work Records

7.1 General

(1) A builder records the details of construction work that has been performed.

(2) During the service period, the commissioning party stores records related to the construction for the purpose of maintenance of the concrete structural object.

Commentary: Regarding (1): Construction work records consist of the construction work plan and records of the details of construction work performed based on the plan. These records are stored to aid in quality assurance for the constructed structural object and to enhance the quality and constructability of structural objects constructed in the future.

Regarding (2): "Records" here are design drawing documents and inspection records. These records contain information necessary for the maintenance of the structural object and serve as initial data used in maintenance. Therefore, it is important that the records comprehensively cover construction work-related information required for maintenance. To clarify the roles and positions of the engineers involved in construction work, the documents should clearly state the companies, personnel names, and other information related to the persons who performed specific work. These records constitute the only invariant information from the period of construction; therefore, they must be stored during the service period of the structural object.

7.2 Structural object labels

Structural object labels, etc., are affixed to the structural object.

Commentary: A structural object label, etc., bearing information including the structural object's name, load, names of the designing and constructing organizations, construction starting date, construction completion date, design, material supply, and names of persons responsible for construction work and construction supervision should be affixed to the structural object. This is important for maintenance and is also a way to honor the engineers involved. As such, it can be expected to contribute to improving the quality of structural objects

Construction Standards

Chapter 1 General Rules

1.1 General

(1) The Standard Specifications in **"Materials and Construction: Construction Standards"** present the regulations that builders should implement with respect to structural forms, materials, construction equipment, and construction work conditions, within the scope of general civil engineering construction work.

(2) **"Materials and Construction: Construction Standards"** addresses AE concrete with a design specified strength of less than 50 N/mm^2 and a minimum placement slump of 16 cm or less.

Commentary: Regarding (1) and (2): Concrete construction varies greatly with a variety of factors, including the type of structural object, materials, construction work conditions, construction work environment, and the equipment used in construction work. However, the majority of concrete construction projects are general projects that do not involve special considerations. For such general concrete construction, **"Materials and Construction: Construction Standards"** presents regulations for the construction of concrete structural objects having a specified level of quality.

The general construction addressed by **"Materials and Construction: Construction Standards"** assumes several conditions, including a concrete design specified strength of less than 50 N/mm^2, a minimum placement slump of 16 cm or less, agitator trucks for off-site transport, on-site transport by a concrete pump with equivalent horizontal pumping distance of less than 300 m, and compaction using rod-shaped vibrators. However, the minimum placement slump is set to 16 cm in cases in which reinforcing bar arrangement and other conditions are particularly difficult. Otherwise, the general minimum placement slump is assumed to be approximately 8 cm to 12 cm.

1.2 Construction work plans

Construction work plans should be properly drafted with sufficient understanding of the contents of design drawing documents.

Commentary: Prior to the implementation of concrete work, construction work plans are drafted in consideration of constructability to enable the trouble-free execution of construction work while taking into account the conditions at individual sites, as well as quality assurance, the construction work period, safety, economy, and environmental impacts. Builders must thoroughly examine the contents of contract documents presented by the ordering party (contracts, design drawings, specifications documents, manuals related to the construction site, question-and-answer documents, *etc.*) and must draft construction work plans after confirming points of difference from the construction site and any ambiguous matters.

Depending on the scale of the construction work, preparation often includes detailed construction work plan documents as indicated below, in addition to overall construction work plans describing the construction as a whole:

(a) Construction work outline

(b) Construction schedule

(c) Construction work plans

(d) Construction work instructions

(e) Detailed occupational safety and health management plans

(f) Detailed plans for environmental measures

(g) Construction work organizational chart

Chapter 2 Concrete Quality

2.1 General

(1) Concrete must possess workability and strength development suited to its role in construction work, as well as the required strength, resistance to deterioration, and other properties after hardening.

(2) Concrete should exhibit little variation in the quality of its materials and production, with both fresh concrete and hardened concrete exhibiting stable quality.

Commentary: Regarding (1): Creating concrete structural objects that possess the necessary performance requires the use of concrete of the appropriate kind and that enables proper construction work. Workability, strength (strength development), resistance to deterioration, resistance to permeation by substances, watertightness, resistance to cracking, and wear resistance are treated as the basic qualities required in concrete.

Regarding (2): A large degree of variability in the materials used for concrete and in concrete production creates difficulties in stably supplying concrete of a specified quality, which may adversely affect the performance of concrete structural objects. In the production of concrete, it is important to perform thorough quality control of materials and production control of concrete, and to ensure the ability to consistently supply concrete of stable quality with little variation between batches.

2.2 Workability and strength development

(1) Concrete must possess workability suited to the transport, placement, compaction, finishing, *etc.* of the concrete and suited to construction work conditions, structural conditions, and environmental conditions.

(2) Concrete must possess the strength development required at each stage of construction work.

Commentary: Regarding (1): **"Materials and Construction: Construction Standards"** addresses filling ability, pumpability, and setting properties as important aspects of workability in performing trouble-free construction work in general concrete construction.

2.3 Strength

(1) Test values for the strength of concrete at a specified material age must not fall below the design specified strength at a probability greater than that specified.

(2) The strength of concrete is generally expressed using the test value for a standard cured specimen at 28 days of material age.

(3) Concrete compressive strength testing and tensile strength testing should be in accordance with JIS A 1108 and JIS A 1113, respectively. The preparation of specimens should also be in accordance with JIS A 1132.

Commentary: Regarding (1): Concrete must satisfy the design specified strength specified in the design of the structural object. Like other materials, however, concrete is subject to variations in quality. Therefore, in civil engineering structural objects, taking into account economy and other factors, a value of 5% or less is generally used as the probability of the test value of concrete compressive strength falling below the design specified strength.

Regarding (2): Although the strength of concrete increases as material age progresses when concrete is properly cured in a wet state, curing efficacy equivalent to that of standard curing cannot always be expected in actual structural objects. However, taking into account that a post-curing increase in strength can also be expected in civil engineering structural objects composed of relatively large structural members, it was decided that the strength of concrete at the start of service of a structural object may be evaluated using test values from specimens at a material age of 28 days subjected to standard curing.

In the case of structural objects on which load acts at a relatively early stage, the strength of the concrete must be based on test values from specimens at a material age of less than 28 days. When using concrete for the long-term increase in strength is large and the curing period before loads act is long, the strength of the concrete should be based on test values from specimens at a material age of greater than 28 days.

In high-strength concrete in which the unit cement content is high, early-stage exposure to high temperatures may hinder strength development over time.

In quality control of concrete, compressive strength is generally used as an indicator, as the testing method is easy and changes in quality are easy to assess. Because strengths other than compressive strength can be roughly judged through conversion based on compressive strength, compressive strength may be used as an indicator to express the strength of concrete.

2.4 Concrete durability

2.4.1 General

Concrete must possess sufficient resistance to deterioration caused by physical and chemical actions during the service life of the structural object, as well as resistance to permeation by substances.

Commentary: Deterioration of concrete includes freezing damage, chemical erosion, and the alkali–silica reaction. Corrosion of steel material caused by permeation of substances into concrete primarily involves salt damage, carbonation, and cracking that can accelerate these phenomena.

2.4.2 Resistance to deterioration

Concrete must possess resistance to deterioration.

Commentary: The resistance of concrete to deterioration is determined by the materials used, the concrete mix, and the quality of the construction work. The rate of deterioration of concrete is also affected by environmental actions. Therefore, the materials used must not inhibit the concrete’s resistance to deterioration. Materials and the concrete mix must be selected on the basis of environmental actions. Materials should satisfy the provisions of "**Chapter 3 Materials.**"

2.4.3 Resistance to permeation by substances

Concrete must possess resistance to permeation by substances to ensure that the steel material arranged inside the concrete is able to exhibit its intended functions throughout its service life.

Commentary: For concrete to fully demonstrate its function of protecting the steel material inside the concrete, the carbonation depth of the concrete must not progress to the position of the steel material during the service period and the chloride ion content of the concrete must not exceed the threshold at which the passive film at the position of the steel material is destroyed. Because the rate of rusting and corrosion in the steel material is also affected by the amount of supplied water and oxygen required for steel corrosion to occur, cover that is resistant to permeation by substances must be secured.

2.5 Other aspects of quality

When creating records of inspections, prediction of deterioration progress, evaluations, judgments, remedial measures, and so on, matters specific to steel corrosion associated with carbonation and water penetration shall also be recorded.

Commentary: Depending on the type of the structural object, other aspects of quality may be required. Appropriate quality must be set for the concrete, with comprehensive consideration of matters including the design drawing documents, construction work methods, the required performance of the structural object, and the content of verifications to be carried out at the time of design. Other approaches concerning quality are shown in **Table C2.5.1**.

Table C2.5.1 Way of thinking of other aspects of quality

Other aspects of quality		Remedial measures
Cracking resistance	Cracks resulting from settlement	Reduction of the water content by using admixtures Tamping or revibration at appropriate timing
	Cracks resulting from plastic shrinkage	Prevention of rapid drying
	Cracks resulting from drying shrinkage	Reduction of the water content Selection of aggregates with good quality Use of expansive agent or shrinkage reducing chemical admixtures
	Cracks resulting from autogenous shrinkage	Selection of appropriate materials and mix proportions
Water-tightness		The water to cement ratio not more than 55% Prevention of cracking Remedial measures after cracking (Water stop, Sealing, etc.)
Abrasion resistance		Use of aggregates with good quality Lowering the water to cement ratio Strong vibration Adequate curing

Chapter 3 Materials

3.1 General

Materials for which quality has been confirmed should be selected.

3.2 Cement

(1) Cement should be selected as appropriate for the intended use of the concrete.

(2) Cement should conform to JIS R 5210, JIS R 5211, JIS R 5212, JIS R 5213, and JIS R 5214 as standard practice.

(3) Cement should be stored in a manner that does not affect its quality.

(4) When the temperature of cement is excessively high, its temperature should be lowered before use.

3.3 Mixing water

(1) As standard practice, mixing water should be tap water or water that conforms to JSCE-B 101 or to JIS A 5308 Appendix C.

(2) Recovered water must conform to JIS A 5308 Appendix C.

(3) In general, seawater must not be used as mixing water.

3.4 Aggregate

3.4.1 Fine aggregate

(1) Fine aggregate should be clean, hard, resistant to deterioration, and chemically and physically stable, and should not contain organic impurities, chlorides, *etc*. at or above hazardous levels.

(2) Sand should conform to JIS A 5308 Appendix A as standard practice.

(3) Crushed sand should conform to JIS A 5005 as standard practice.

(4) Fine aggregate using blast-furnace slag should conform to JIS A 5011-1, that using ferro-nickel slag to JIS A 5011-2, that using copper slag to JIS A 5011-3, and that using electric-arc-furnace reducing slag to JIS A 5011-4 as standard practice.

(5) Recycled fine aggregate should conform to JIS A 5021 as standard practice.

(6) Fine aggregate should combine large and small particles in proper amounts, with particle sizes should be within the ranges shown in **Table 3.4.1** as standard practice.

Table 3.4.1 Standard grading of fine aggregate

Nominal openings of sieve（mm）	10	5	2.5	1.2	0.6	0.3	0.15
Percentage of mass passing through sieve	100	90– 100	80–100	50–90	25–65	10–35	2–10[1)]

1) When only crushed sand or slag fine aggregate is used as fine aggregate, this range may be 2-15%. When mixed fine aggregate is used and most of passing through 0.15 mm sieve are crushed sand or slag fine aggregate, 15% may be accepted.

2) It is desirable that the percentage of aggregate remained between two successive sieves may not be larger than 45 %

3) When the air content is larger than 3% and the cement content is 250km^3, minimum of the percentage of passing through the 0.30mm and 0.15mm sieve may be decreased to 5% or 0% in case of supplement of fine grain by use of powdery material with good quality.

(7) When mixed fine aggregates are used, their quality prior to mixing should conform to the requirements of (2), (3), (4), and (5). With regard to chloride content and particle size, however, it is sufficient for fine aggregate quality after mixing to conform to the requirements of (1) and (6).

3.4.2 Coarse aggregate

(1) Coarse aggregate should be clean, hard, resistant to deterioration, and chemically and physically stable, and should not contain organic impurities, chlorides, *etc*. at or above hazardous levels. When fire resistance is required, fire-resistant coarse aggregate should be used.

(2) Gravel should conform to JIS A 5308 Appendix A as standard practice.

(3) Crushed stone should conform to JIS A 5005 as standard practice.

(4) Coarse aggregate using blast-furnace slag should conform to JIS A 5011-1, that using ferro-nickel slag to JIS A 5011-2, and that using electric-arc-furnace reducing slag to JIS A 5011-4 as standard practice.

(5) Recycled coarse aggregate should conform to JIS A 5021 as standard practice.

(6) Coarse aggregate should combine large and small particles in appropriate amounts, and particle sizes should be within the ranges shown in **Table 3.4.2** as standard practice.

Table 3.4.2 Standard grading of coarse aggregate

Nominal openings of sieve (mm)		Percentage of mass passing through sieve (%)									
		50	40	30	25	20	15	13	10	5	2.5
Max size of coarse aggregates (mm)	40	100	95～100	–	–	35～70	–	–	10～30	0～5	–
	25	–	–	100	95～100	–	30～70	–	–	0～10	0～5
	20	–	–	–	100	90～100	–	–	20～55	0～10	0～5
	10	–	–	–	–	–	–	100	90～100	0～15	0～5

(7) When mixed coarse aggregates are used, their quality prior to mixing should conform to the requirements of (2), (3), (4), and (5). With regard to particle size, however, it is sufficient for coarse aggregate quality after mixing to fall within the range in (6).

3.4.3 Storage of aggregates

(1) Fine aggregates, coarse aggregates, and aggregates that differ by type, place of origin, and grading should be divided and stored separately.

(2) The acceptance, storage, and handling of aggregates must be performed with care using machinery and equipment and using appropriate structures for preventing the segregation of large and small particles, contamination by debris and foreign matter, damage to coarse aggregates, *etc*.

(3) Aggregates should be stored with suitable drainage facilities provided so that surface moisture is evenly distributed.

(4) During cold weather, aggregates should be stored in a manner that prevents freezing and contamination by ice or snow.

(5) Aggregates should be stored away from direct sunlight to prevent drying and temperature rise during hot weather.

3.5 Admixtures

3.5.1 General

Mineral admixtures and chemical admixtures used as admixtures must be of confirmed quality.

3.5.2 Mineral admixtures

(1) Fly ash used as a mineral admixture should conform to JIS A 6201 as standard practice.

(2) Expansion admixtures used as mineral admixtures should conform to JIS A 6202 as standard practice.

(3) Blast-furnace-slag fine powder used as a mineral admixture should conform to JIS A 6206 as standard practice.

(4) Silica fume used as a mineral admixture should conform to JIS A 6207 as standard practice.

(5) For mineral admixtures other than (1) to (4), quality should be confirmed and methods of use should be thoroughly examined.

(6) Mineral admixtures should be stored in a manner that does not affect quality.

3.5.3 Chemical admixtures

(1) AE agents, water-reducing agents, AE water-reducing agents, high-performance AE water-reducing agents, high-performance water-reducing agents, plasticizers, and hardening accelerators used as chemical admixtures should conform to JIS A 6204 as standard practice.

(2) Reinforced concrete rust inhibitors used as admixtures should conform to JIS A 6205 as standard practice.

(3) For chemical admixtures other than (1) and (2), quality should be confirmed and methods of use should be thoroughly examined.

(4) Chemical admixtures should be stored in a manner that does not affect quality.

3.6 Reinforcing materials

3.6.1 Reinforcing bars

(1) Reinforcing bars should conform to JIS G 3112, JSCE-E 121, JIS G 4322, or JSCE-E 102 as standard practice.

(2) Reinforcing bars should be stored in a manner that does not affect their quality.

3.6.2 Structural steel material

(1) Structural steel material should conform to JIS G 3101, JIS G 3106, or JIS G 3136 as standard practice.

(2) Structural steel material should be stored in a manner that does not affect its quality.

3.6.3 Other reinforcing materials

For other reinforcing materials, quality should be confirmed and methods of use should be thoroughly examined.

Chapter 4 Mix Design

4.1 General

(1) In mix design, the maximum size of the coarse aggregate, the slump, the air content, the water-to-cement ratio, the sand aggregate ratio, and other mix conditions are clearly set and the unit content of each material is determined to satisfy properties such as the workability, design specified strength, resistance to deterioration, and resistance to permeation by substances.

(2) In the mix design of concrete, unit water content should be set as low as possible while still within the range that enables the necessary workability.

Commentary: Regarding (1): This chapter presents concrete mix design methods that satisfy the workability, design specified strength, resistance to deterioration, resistance to permeation by substances, and other properties of the concrete.

In design drawing documents, reference values are presented for the maximum size of coarse aggregate, minimum slump for placement, water-to-cement ratio, type of cement, unit cement content, air content, and other mix conditions to be incorporated into specific mix design. If the reference values in the design drawing documents have been determined on the basis of the **"Design"** volume in these Standard Specifications, then the reference values may be used in mix design.

4.2 Procedures for mix design

(1) In mix design, the characteristic values for concrete strength, resistance to deterioration, *etc*. described in design drawing documents are checked, along with reference values for maximum size of coarse aggregate, minimum slump for placement, water-to-cement ratio, type of cement, unit cement content, air content, and other properties.

(2) Mix conditions are set based on the reference values for concrete described in design drawing documents, as shown in (1) above.

(3) Based on the mix conditions that were set, a provisional mix that will serve as a standard for trial mixing is set.

(4) Based on the provisional mix that was set, trial mixing is conducted using materials intended for actual use to check whether the concrete possesses the necessary quality. When trial mixing reveals that the concrete does not possess the necessary quality, the materials used are changed and the mix is corrected to determine the mix that will yield the specified quality.

Commentary: Regarding (1): The design drawing documents describe the concrete design specified strength, the coefficient of the rate of carbonation, the coefficient of the rate of water permeation, the coefficient of the penetration of chloride ions, the relative dynamic modulus of elasticity in freezing and thawing testing, the shrinkage strain, and other characteristic values that were set based on the required performance for the structural object. As a reference when using a concrete mix that has a proven track record or when designing specific mixes at the construction work stage, reference values including the maximum size of coarse aggregate, the minimum slump for placement, the water-to-cement ratio, the type of cement, the unit cement content, and the air content are described. Therefore, these characteristic values and reference values described in design drawing documents must first be checked when performing mix design.

Regarding (2): Based on reference values described in design drawing documents, the maximum size of coarse aggregate, the minimum slump for placement, the water-to-cement ratio, the air content, and other mix conditions are set. Here, when it has been determined that the reference values in design drawing documents are not suited to the actual materials to be used and the construction work conditions, the reference values must be changed to appropriate values suited to the actual conditions, after confirming that these values will satisfy the characteristic values of concrete described in design drawing documents.

Figure C4.2.1 shows approaches to mix selection that satisfy workability, design specified strength, resistance to deterioration, resistance to permeation by substances, and other properties of the concrete. For concrete to smoothly flow between steel material items without segregation of materials and densely fill the cover portions, corners, and other parts of the formworks, the concrete must also possess resistance to material segregation appropriate to its fluidity. In this **"Construction Standards"** unit powdery material content is used as an indicator of resistance to material segregation. The necessary unit powdery material content must be secured and appropriate resistance to material segregation must be provided as appropriate to the size of the slump.

Regarding (3) and (4): Based on the mix conditions that were set, a provisional mix that uses the materials intended for actual use is set, and whether the mix satisfies the necessary quality is confirmed through trial mixing. When trial mixing reveals that the mix does not satisfy the necessary quality, the mix is corrected and mixing is performed again. When the necessary quality cannot be met through correction of the mix alone, the materials used are changed and mixing is repeated until the necessary quality is obtained.

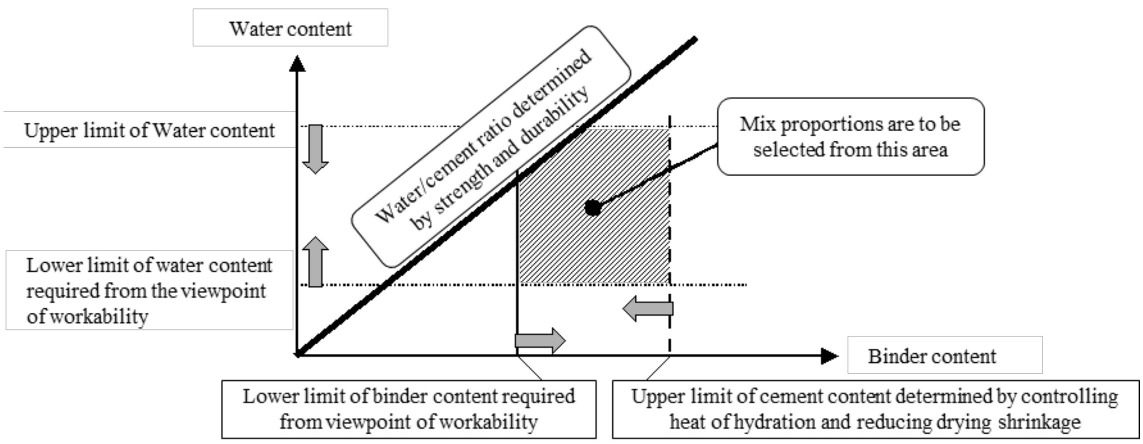


Figure C4.2.1 Concept of mix proportioning (in the usual case)

4.3 Confirmation of the characteristic values of concrete

4.3.1 General

Prior to mix design, characteristic values of concrete concerning design specified strength, resistance to deterioration, resistance to permeation by substances, and other aspects of quality described in design drawing documents are confirmed.

4.3.2 Design specified strength

The design specified strength described in the design drawing documents is confirmed.

Commentary: In the case of concrete structural objects designed in accordance with the **"Design"** volume in these Standard Specifications, the design specified strength that was set on the basis of the structural object’s performance is described in design drawing documents. Based on this specified strength, the required strength is set based on past variation in the quality of the concrete and materials, and the upper limit for the water-to-cement ratio is determined based on the strength. The specific method for setting this is described in **4.5.3 Required strength**.

4.3.3 Deterioration of concrete and resistance to permeation by substances

(1) Characteristic values and reference values regarding resistance to permeation by substances and resistance to deterioration in concrete described in the design drawing documents are confirmed.

(2) Appropriate mix conditions and materials are set based on the reference values described in design drawing documents so as to satisfy the necessary durability of the structural object.

(3) If the reference values described in design drawing documents are not followed, then appropriate mix conditions are set using reliable data or mixes with a demonstrated record of performance as a reference, or after confirming through preliminary testing that characteristic values described in design drawing documents are satisfied.

(4) Appropriate measures are taken to inhibit the alkali–silica reaction.

Commentary Regarding (4): At present, no method has been established that enables fast and appropriate verification of the alkali–silica reaction, and it is often difficult to adequately address the alkali–silica reaction using only the characteristic values and reference values described in design drawing documents. In general, resistance to degradation caused by the alkali–silica reaction can be deemed to be satisfied through implementation of any of the following three control measures:

(i) Inhibition of the total alkali content in concrete

(ii) Use of mixed cement with an alkali–silica reaction inhibitory effect

(iii) Use of aggregates classified as Category A (“harmless”) in alkali–silica reactivity testing

4.3.4 Other characteristic values and reference values

(1) Together with characteristic values including watertightness, adiabatic temperature rise properties, and shrinkage properties described in design drawing documents, the described reference values are checked so as to obtain these characteristic values.

(2) The upper limit of the water-to-cement ratio is set so as to obtain the necessary watertightness.

(3) The type of cement and upper limits on the unit cement content are set based on reference values described in design drawing documents so that the adiabatic temperature rise properties of the concrete are equal to or less than their design values.

(4) Appropriate materials and the upper limit of unit water content are set based on characteristic values for concrete shrinkage strain described in design drawing documents or on reference values that have been verified as satisfying shrinkage properties.

(5) When characteristic values for shrinkage strain are not described in design drawing documents, appropriate mix conditions are set using reliable data or past construction work performance as a reference, or after confirming through testing that the value of shrinkage strain does not affect the necessary performance of the structural object.

Commentary Regarding (3): Thermal cracking caused by hydration of cement is verified in **"Design: General Requirements" (Chapter 12 Verification of Initial Cracking)**. When environmental conditions, construction work conditions, and other conditions are expected to change and factors not assumed in the design are expected to exert effects, said effects must be taken into consideration in the selection of the concrete materials, mix, and construction work methods.

Regarding (4) and (5): In design, verification is performed using the characteristic values of shrinkage strain of concrete to calculate response values of structural objects. Characteristic values of shrinkage strain by which response values satisfy threshold values are presented in design drawing documents. Therefore, appropriate mix conditions and materials must be set so as to satisfy the shrinkage strain.

It is important to confirm in advance that the concrete mix and materials to be used do not present a problem in terms of cracking associated with shrinkage.

4.4 Workability of concrete

Based on the minimum slump for placement that is presented as a reference value in design drawing documents, the workability of concrete is set in line with the environmental conditions, construction work conditions, and materials associated with actual construction.

Commentary: To construct concrete structural objects that possess the necessary performance, slump and unit powdery material content are determined so that the concrete possesses filling ability, pumpability, and setting properties suitable for transport, placement, compaction, finishing, and other tasks.

4.4.1 Filling ability

(1) Filling ability is determined appropriately within a range that does not interfere with placement and compaction work, taking into account the methods used in placement and compaction as well as the type of the structural object, the type and size of structural members, the amount of steel material, the minimum spacing of steel material, and other conditions of the reinforcing bar arrangement.

(2) Filling ability is determined based on the fluidity and resistance to material segregation of the concrete.

(3) The fluidity of the concrete is ensured by appropriately setting the minimum slump for placement.

(4) The resistance to material segregation of the concrete is ensured by appropriately setting the unit cement content or unit powdery material content.

Commentary: Regarding (1) and (2): The filling ability required of concrete refers to the performance by which poured concrete is able to smoothly flow between reinforcing bars by vibration compaction without the occurrence of material segregation and is able to densely fill cover portions, corners, and other parts. Appropriate filling ability must be set with consideration of varied construction work conditions. In **"Construction Standards,"** filling ability is determined from the balance between fluidity and resistance to material segregation.

In **"Construction Standards,"** the use of slump to express flowability and the use of the unit powdery material content as an indicator for resistance to material segregation are set as standard practices, taking practical convenience into consideration. Because the necessary filling ability differs with work conditions, the specific concrete mix is determined on the basis of **"4.2 Procedures for mix design."**

Regarding (3): Ensuring dense filling for concrete requires achieving the necessary slump during placement. Therefore, **"Materials and Construction: Construction Standards**" sets minimum slump for placement as the basis for the fluidity needed for filling ability.

As shown in **Figure C4.4.1**, slump changes as a result of factors including transport and the passage of time between production and placement. Satisfying the specified minimum slump for placement requires that the target slump at the times of mixing and of unloading be determined with consideration of factors including the transport method, the time between mixing and placement, and the temperature. The procedure for setting slump at each of the stages of work is detailed in **"4.5.2 Slump."**

As noted above, slump must be made greater during mixing and unloading than during placement. When the target slump for mixing and unloading is expected to be considerably greater than the minimum slump for placement, appropriate mix and construction work methods must be considered to prevent problems such as pipe clogging during pumping and material segregation. When using ready-mixed concrete, a target slump for unloading that satisfies the minimum slump for placement should be set. As in conventional practice, this unloading target slump should be set as the specified slump.

Regarding (4): Ensuring the resistance of material segregation of concrete requires unit cement content or unit powdery material content at or above a specified level. If there is concern over thermal cracking as a result of heat of hydration, then low-activity fly ash, fine limestone powder, *etc*., should be used in addition to cement to secure the unit powdery material content. Setting the sand aggregate ratio at an effective level is also effective in enhancing resistance to material segregation. When setting the sand aggregate ratio, the values presented in **“4.5.6 Sand-aggregate ratio”** can be used as reference.

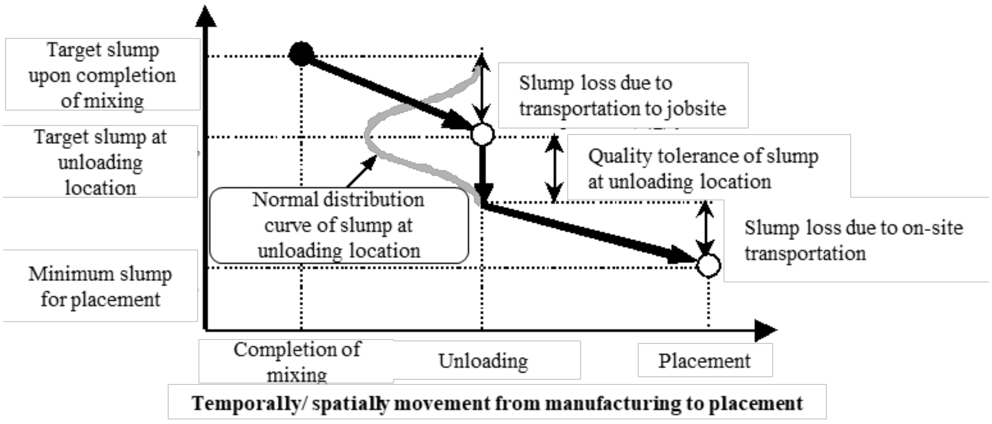


Figure C4.4.1 Relationship between target slump at different stages of construction and change in slump over time

4.4.2 Pumpability

When using a concrete pump, the fresh concrete must possess fluidity suited to the pumping work as well as an appropriate level of resistance to material segregation.

Commentary: When transport is performed using a pump, the specified pumping volume under planned pumping conditions must be achievable without clogging of the pipe. It is also advisable that the workability of fresh concrete before and after pumping does not change significantly. To meet these conditions, it is necessary to not only change the target slump at the time of cement unloading but also to consider the type of pump, the diameter of the transport pipe, the transport distance, and other construction work conditions to comprehensively determine appropriate conditions.

Because the pumpability of concrete is determined by its fluidity and resistance to material segregation, it is standard practice to set appropriate slump, unit powdery material content, *etc*. In slump management during pumping, it is typical to perform management of slump at the time of unloading, not during placement.

The amount of decrease in slump associated with pumping should be set with reference to **Table 4.5.6**.

4.4.3 Setting properties

The setting properties of fresh concrete must be suitable for work such as two placing lifts and finishing.

Commentary: Setting properties are related to factors including the allowable time interval for two placing lifts concrete, the timing for finishing, and the lateral pressure acting on formworks. In hot-weather concreting and cold-weather concreting, attention must be paid to setting properties, which change depending on factors including the timing and temperature of placement.

Setting properties are generally evaluated using the starting and ending times for setting, which are obtained according to JIS A 1147.

4.5 Setting of Mix Requirements

4.5.1 Maximum size of coarse aggregate

(1) It should be confirmed that the maximum size of the coarse aggregate specified in design drawing documents has been determined with consideration of structural member dimensions, reinforcing bar spacing, and cover thickness.

(2) As standard practice, the maximum dimension of coarse aggregate should not exceed 1/5th the minimum dimension of structural members in the case of reinforced concrete, or 1/4th the minimum dimension of structural members in the case of plain concrete.

(3) The maximum dimension of coarse aggregate must not exceed 3/4th the minimum horizontal reinforcing bar spacing in beams and slabs. In the case of columns and walls, the maximum size must not exceed 3/4th of the minimum reinforcing bar spacing in the axial direction.

(4) As standard practice, the maximum dimension of coarse aggregate should not exceed 3/4th of the cover thickness.

(5) **Table 4.5.1** shows standards for the maximum dimension of coarse aggregate.

Table 4.5.1 Maximum size of coarse aggregate

Structural conditions	Maximum size of coarse aggregate
The smallest dimension of the cross section is large※ And three-fourths of the smallest reinforcing bar spacing and the concrete cover > 40mm	40 mm
In other cases	20 mm or 25 mm

※500mm or larger as a guide

Commentary: When the amount of steel material is large or the steel material spacing is small, and if the maximum dimension of the aggregate is too large, then coarse aggregate will not be able to easily pass between steel materials. This increases the risk of defects such as honeycombing and locations with incomplete filling.

4.5.2 Slump

(1) Slump should be made as low as possible within the range suitable for transport, placement, compaction, and other work, and should be such that material segregation does not occur.

(2) As a guideline, the minimum slump for placement should be selected from **Tables 4.5.2** to **4.5.5**, based on the type of structural object, the types and sizes of structural members, reinforcement bar conditions such as the amount of steel material and the minimum spacing, and construction work conditions such as compaction work height.

(3) The target slump at the time of unloading and the target slump for mixing are set based on the minimum slump for placement, with consideration of the decrease in slump associated with on-site transport and the passage of time between unloading and placement, the decrease in slump associated with transport to the site, and the tolerances for quality at the production stage.

(4) When there are multiple structural members to be poured and concrete can be poured for each, the minimum slump for placement should be set for each structural member. When changing the slump is not possible in mid-process, such as when continuously placement multiple structural members, the standard practice should be to use the largest of the minimum slump values for placement found among the structural members.

(5) When using a concrete pump to perform pumping, the amount of decrease in slump should be predicted in line with conditions including environmental conditions, minimum slump, and the pumping conditions shown in **Table 4.5.6**, with consideration of the decline in slump associated with pumping.

Tables 4.5.2 Typical values of minimum slump for placement of slabs

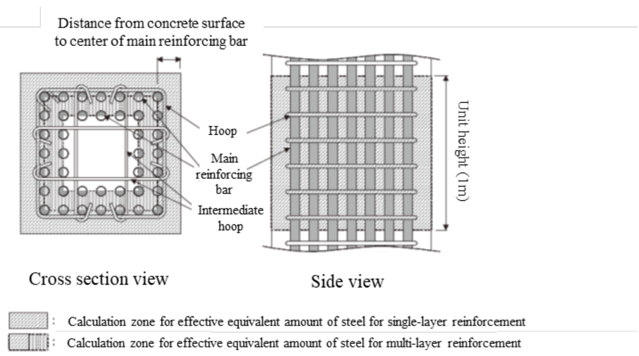
Compaction height	Less than 0.5 m	0.5 m to less than 1.5 m	3.0 m or less	
Placement interval	At any location	At any location	2~3m	3~4m
Minimum slump at placement (cm)	5	7	10	12

1) The minimum slumps at placement in this table is assumed that the amount of steel shall be 100 to 150 kg/m^3 and the minimum steel spacing shall be 100 to 150mm. Thus, in case of 100mm or less of the spacing, it is recommended that 2~3 cm be added to each of the values shown above.
2) The drop height of concrete shall be 1.5m or less as standard.

Tables 4.5.3 Typical values of minimum slump for placement of columns

Effective equivalent amount of steel in the concrete cover zone 1)	Cover concrete or minimum steel spacing	Compaction height		
		Less than 3m	3m to less than 5m	5m or more
Less than 700 kg/m^3	50 mm or more	5	7	12
	Less than 50 mm	7	9	15
700 kg/m^3 or more	50 mm or more	7	9	15
	Less than 50 mm	9	12	15

1) The effective equivalent amount of steel in the concrete cover zone is the amount of steel per unit volume in the regions shown below.



Tables 4.5.4 Typical values of minimum slump for placement of beams

Minimum steel spacing	Compaction height		
	Less than 0.5 m	0.5 m to less than 1.5 m	1.5 m or more
150 mm or more	5	6	8
100 mm to less than 150 mm	6	8	10
80 mm to less than 100 mm	8	10	12
60 mm to less than 80 mm	10	12	14
Less than 60 mm	12	14	16[1)]

1) If it is judges that adequate compaction can be done, a minimum slump for placement of 14cm is used

Tables 4.5.5 Typical values of minimum slump for placement of walls

Amount of steel	Minimum steel spacing	Compaction height		
		Less than 3m	3 m to less than 5 m	5 m or more
Less than 200 kg/m^3	100 mm or more	8	10	15
	Less than 100 mm	10	12	15
200 kg/m^3 to less than 350 kg/m^3	100 mm or more	10	12	15
	Less than 100 mm	12	12	15
350 kg/m^3 or more	—	15	15	15

Tables 4.5.6 Typical slump loss under different conditions

Pumping conditions		Slump loss	
Equivalent horizontal pipe length	Connection condition of transportation pipe	When the minimum slump is smaller than 12cm	When the minimum slump is 12cm or greater
Less than 50 m (including placement by buckets)		-	-
50 m to less than 150 m	—	-	-
	Connecting tapered pipe of less than 100A(4B)	0.5–1.0 cm	0.5–1.0 cm
150 m to less than 300 m	—	1.0–1.5 cm	1.0 cm
	Connecting tapered pipe of less than 100A(4B)	1.5–2.0 cm	1.5 cm
In other special conditions		To be determined on the basis of past project data or field trial results	

1) When daily average temperature exceeds 25°C, it is recommended that 1cm be added to each of the values shown above.

When pumping distance in the upper part or lower part is 20mm or more, it is recommended that 1cm be added to each of the values shown above.

Commentary: Regarding (1): Slump is set with consideration of the type and dimensions of structural members and the arrangement of reinforcing materials (reinforcing bars and steel material) as structural conditions, and with consideration of on-site transport methods (pump types, pumping distances, and pipe orientation), placement methods (drop height and height per layer of placement), and compaction methods (types of rod vibrator, insertion interval, insertion depth, and vibration time) as construction work conditions.

Regarding (2): Constructing concrete structural objects that possess the necessary performance requires that the concrete be provided with workability matched to the structural and construction work conditions. **"Construction Standards"** presents guidelines for the minimum slump for placement in line with the height of compaction work and structural conditions such as the amount and spacing of steel material, for each structural member.

Tables 4.5.2 to **4.5.5** were determined on the basis of past construction performance and other information, on the assumption that the standard construction work will be implemented. Because the table addresses concrete made with crushed stone and crushed sand, when high-quality natural aggregate is used, the minimum slump for placement should be set lower than the values shown. The definition of compaction work height, which is the work height related to the difficulty of compaction work, is shown in **Figure C4.5.1**.

Regarding (3): **"Construction Standards"** specifies that the minimum slump necessary for placement is first set, then is used as a standard for the target slump used in quality control at the time of unloading and for the target slump during mixing used in quality control at the production stage. In mix design, the slumps for placement, unloading, and mixing are set in line with the following steps, taking into account the change in slump at each stage of work as shown in **Figure C4.5.2**.

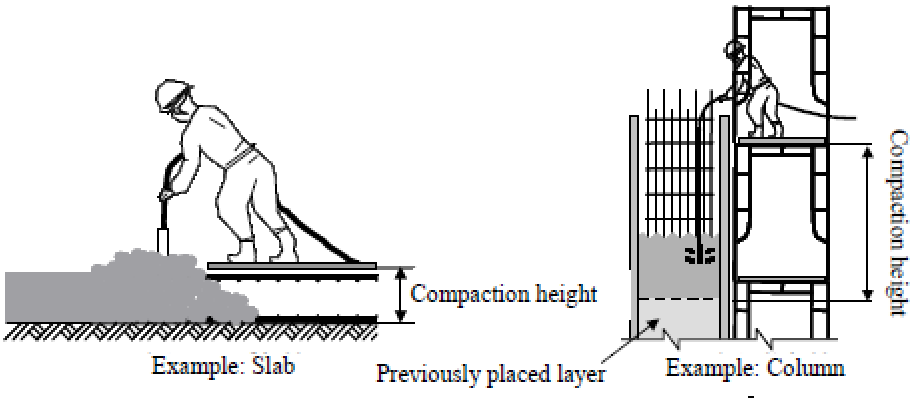


Figure C4.5.1 Examples of compaction height

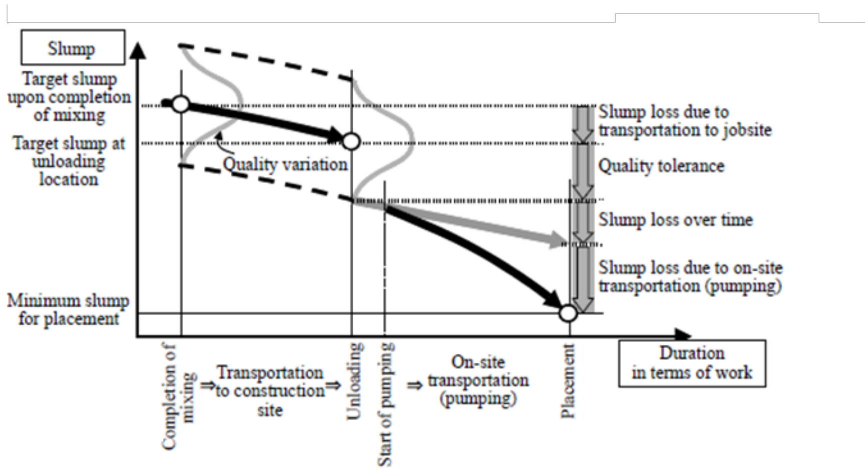


Figure C4.5.2 Changes in slump over time at different stages of construction

Step 1: Setting the minimum slump for placement

The minimum slump necessary to densely fill formworks (see **Tables 4.5.2** to **4.5.5**) is selected in line with structural conditions such as the type of structural member and the amount and spacing of steel material and in line with construction work conditions such as the placement method (free fall height and height per layer of placement) and the compaction method (height of compaction work, type of rod vibrator, insertion interval, insertion depth, and vibration time).

Step 2: Setting the target slump at the unloading point

The target slump at the point of unloading is determined from the minimum slump for placement, with consideration of the decrease in slump associated with pumping or other on-site transport, the change in slump associated with the passage of time between unloading and placement, and variations in quality at the production stage. The amount corrected for the decrease in slump associated with pumping is determined from **Table 4.5.6**, in line with construction work conditions such as pumping distance. Variation in quality at the production stage is appropriately determined in line with the actual production equipment and quality control conditions, within the tolerance for JIS-certified ready-mixed concrete (slump 8–18 cm: 2.5 cm; slump 21 cm: 1.5–2.0 cm).

Step 3: Setting the target slump for mixing

The target slump for mixing is determined with consideration of the decrease in slump associated with off-site transport to the location of unloading. According to studies of ready-mixed concrete, a decrease in slump of approximately 1.0 cm per 30 minutes of transport time to the construction site can be used as a guideline during standard seasons (when the temperature of concrete is within the range of 20±7 °C). In winter (when the temperature of concrete is 12 °C or lower), the decrease in slump associated with transport time is relatively small, at approximately 1 to 1.5 cm for a transport time of up to 60 minutes as a guideline. In summer (when the temperature of concrete is 28 °C or higher), the decrease in slump over time tends to increase to approximately 1.5 cm per 30 minutes of transport time, and thus requires special attention.

When the time required for transport to the site or from unloading to placement is long and a large decrease in slump associated with time is expected or when a large decrease in slump associated with pumping is expected, it is assumed that the target slump at the point of unloading or for mixing will be large relative to the minimum slump for placement. In such cases, a mix by which mixing slump or slump at the location of unloading is simply made greater is prone to segregation of materials during production or off-site transport, resulting in quality changes that interfere with quality control at the location of unloading as well as the risk of pipe clogging associated with pumping. In such cases, the mix should be determined such that the concrete will possess good workability, by means such as switching to materials that are less susceptible to changes in quality.

Regarding (4): The minimum slump for placement is set for each structural member. However, when continuously placement multiple structural members such as pillars, beams, walls, and slabs, the use of several types of concrete possessing different slumps may complicate work and management or even create obstacles to the trouble-free performance of construction work. In such cases, it is rational to select the concrete that applies to the structural members for which structural and construction work conditions are the most severe and for which minimum slump for placement is the greatest. However, it is important to first fully consider whether the issue might be dealt with through refinements to aspects of construction work, such as the placement method or compaction method, rather than through the simple selection of a large slump.

4.5.3 Required strength

(1) The required strength of concrete is set with consideration of the design specified strength and variation in concrete quality.

(2) The required strength f'_{cr} of concrete is generally determined so that the probability of the test value of the compressive strength test of concrete on site falling below the design specified strength f'_{ck} is 5% or less.

Commentary: Regarding (1): The quality of concrete at the construction site varies with the variation in the quality of aggregates and cement, measurement errors, variability in mixing work, *etc*. To ensure that the compressive strength of concrete used in any part of the structural object is not excessively small with respect to the compressive strength used as a standard in structural design, the required strength of the concrete must be set higher than the design specified strength, in line with the variation in the quality of concrete at the site.

Regarding (2): Test values for the compressive strength of concrete will unavoidably vary to some extent. It is known from experience that the variation in compressive strength of concrete under ordinary quality control is distributed almost normally. Treating this stochastically, a 5% or lower probability of a test value falling below design specified strength was set as a condition for concrete used in general structural objects. A test value for the compressive strength of concrete on site is the average value of compressive strength obtained through standard curing of three concrete specimens sampled from the same batch at the site.

To ensure that the probability of a test value for compressive strength falling below the design specified strength is not higher than the specified probability, a strength representing an appropriate increase from the design specified strength, in line with the degree of variation in test values, must be selected as the target value for compressive strength. **Figure C4.5.3** shows the relationship between the coefficient of variation and the overdesign coefficient when the specified probability is 5%. Based on this figure and as general practice, the overdesign coefficient that corresponds to the coefficient of variation of the compressive strength of concrete on site should be derived, and the product of this overdesign coefficient and the design specified strength, or an appropriate value not less than this product, should be selected as the required strength.

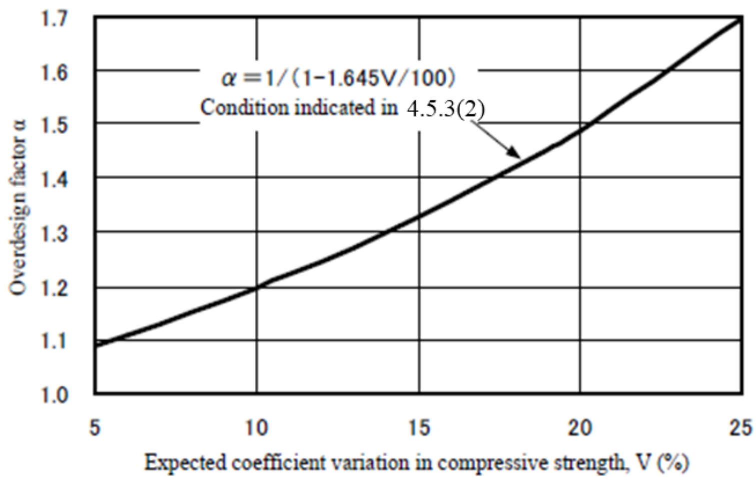


Figure C4.5.3 Overdesign factor in a normal case

4.5.4 Water-to-cement ratio

(1) The water-to-cement ratio should be set to no more than 65% and should be set to the lowest water-to-cement ratio determined with consideration of factors including the necessary strength for the concrete, resistance to deterioration of the concrete, and resistance to permeation by substances.

(2) When determining the water-to-cement ratio based on the compressive strength of concrete, the following method can be used.

(a) As a general rule, the relationship between compressive strength and water-to-cement ratio should be determined through testing. The standard material age to be used in testing is 28 days. However, a different material age for testing may be set, taking characteristics of the cement used into account.

(b) The water-to-cement ratio used in the mix is the reciprocal of the cement-to-water ratio that corresponds to the required strength f'_{cr} in the relational expression between the cement-to-water ratio (C/W) at the reference material age and the compressive strength f'_c.

(3) When reference values are provided in design drawing documents, the water-to-cement ratio set with consideration of the resistance of the concrete to deterioration, the resistance to the migration of substances, *etc*. should be no higher than said reference values.

Commentary Regarding (2)

(a) When determining the relationship between the compressive strength of concrete and the water-to-cement ratio, the fact that the relationship between the cement-to-water ratio (C/W) and compressive strength (f'_c) is linear within a certain range can be used. However, determination of the relationship between the compressive strength of concrete and the cement-to-water ratio through testing was set as a general rule.

The relationship of C/W to f'_c is derived as follows:

Testing is performed on concrete with three or more different cement-to-water ratios within a range considered appropriate, and $C/W–f'$ curves are generated. Because the relationship between C/W and f'_c varies with the air content, specimens are prepared using concrete with the necessary air content. It is advisable that the value of f'_c for each C/W be the average for specimens produced from two or more batches of concrete to reduce errors in mix testing. Here, when using blast-furnace-slag fine powder, fly ash, or other mineral admixtures that can be expected to act as binders, the binder-to-water ratio is used.

When the material age of the design specified strength is a value other than 28 days, the above relationship is derived using strength at that material age. However, the water-to-cement ratio may be set based on an age other than 28 days if the relationship between the material age and strength at 28 days of material age is clear.

(b) Creating homogeneous concrete requires using homogeneous materials, measuring these accurately, and mixing them thoroughly. However, even when these tasks are performed with care, the quality of concrete will unavoidably vary to some extent. Therefore, to ensure the degree of safety that was considered in the design of the structural object, required strength is used as the value of compressive strength when the relationship between compressive strength and the cement-to-water ratio described in (a) is used.

Regarding (3): When it is necessary to ensure the resistance to deterioration of concrete and resistance to permeation by substances in design, the upper limit for the necessary water-to-cement ratio, verified according to **"Design: General Requirements" (Chapter 8**

Verification of Durability), should be indicated in design drawing documents as a reference value. When watertightness must be taken into consideration, too, the upper limit of the necessary water-to-cement ratio should be indicated in design drawing documents as a reference value. In the mix design, the water-to-cement ratio is set to be no higher than the reference value in design drawing documents, but consideration must be given to the possibility that, by making the water-to-cement ratio excessively small, the test values for Young's modulus and concrete strength may be excessively large relative to design values, and the constructed concrete structural object may exhibit response different from the design.

4.5.5 Air content

(1) As standard practice, the air content of concrete is 4% to 7% of the concrete volume at the time of mixing, depending on the maximum dimension of the coarse aggregate and other factors.

(2) Testing of concrete air content should be according to JIS A 1116, JIS A 1118, or JIS A 1128.

Commentary: Regarding (1): The standard air content in concrete is generally set to around 4% to 7% of the concrete volume at the time of mixing (**Table C4.5.1**) so as to obtain the necessary workability and resistance to freezing damage within a range that does not have adverse effects on strength, resistance to deterioration, or resistance to permeation by substances.

Entrained air contributes to improvement of the workability of concrete and can considerably reduce the unit water content required to obtain the necessary workability. It is extremely effective in ensuring the necessary resistance to freezing damage. However, as the air content increases, the strength of concrete decreases while variation in the quality of the concrete tends to increase. Therefore, care should be taken to avoid excessively increasing the air content if the concrete is not subject to freezing and thawing actions.

Table C4.5.1 Approximate values of the unit bulk volume of coarse aggregate, sand percentage and water content of concrete

Maximum size of coarse aggregate	Unit bulk volume of coarse aggregate	Air-entrained concrete				
		Air content	With air-entraining agent		With air-entraining water-reducing agent	
			Sand percentage s/s	Water content W	Sand percentage s/s	Water content W
(mm)	(m^3/m^3)	(%)	(%)	(kg)	(%)	(kg)
15	0.58	7.0	47	180	48	170
20	0.62	6.0	44	175	45	165
25	0.67	5.0	42	170	43	160
40	0.72	4.0	39	165	40	155

The values shown above are average values determined with reference to the standard mix proportions adopted by ready-mixed concrete industrial associations in Japan and are for concrete made by using sand of an ordinary size distribution (fineness modulus: about 2.80) and crushed stone.

4.5.6 Sand aggregate ratio

The sand aggregate ratio is determined through testing so that the unit water content is as small as possible, within the range that enables the necessary workability.

Commentary Because the sand aggregate ratio affects pumpability, when on-site transport of concrete is performed by pumping, the appropriate sand aggregate ratio is set based on past data and performance, in line with pump performance, piping, pumping distance, and other factors (see **"7.3 Transport"**).

In the case of concrete made with high-performance AE water-reducing agents, better results can often be obtained by increasing the sand aggregate ratio by 1% to 2% above that of concrete made with normal AE water-reducing agents and having the same water-to-cement ratio and slump.

Methods for determining the ratio of fine and coarse aggregates in concrete include methods based on the bulk density of coarse aggregate, in addition to those based on the above-noted sand aggregate ratio. In particular, because the relationship between the sand aggregate ratio and the quality of workability can become less clear as slump increases, first determining the bulk density of the coarse aggregate may enable the selection of a more appropriate mix.

4.5.7 Admixtures

The types and amounts of admixtures are determined through testing or reference to past performance and data so as to obtain the necessary efficacy.

4.6 Determination of unit contents

4.6.1 Unit water content

(1) Unit water content is determined through trial mixing so as to be as low as possible while still within the range that allows workability.

(2) The upper limit of the unit water content of concrete is set to 175 kg/m^3 as a standard. When the unit water content exceeds this upper limit, the necessary resistance to deterioration of concrete and resistance to permeation by substances must be confirmed to be satisfied.

Commentary: Regarding (1): The unit water content of concrete necessary to obtain a specified slump varies with factors including the maximum size of the coarse aggregate, the aggregate particle size and shape, the type of admixture, and the air content of the concrete. Therefore, it was decided that the value should be determined through trial mixing using the materials intended for use in the actual construction work.

The use of AE agents, AE water-reducing agents, high-performance AE water-reducing agents, *etc.* can considerably reduce unit water content. JIS A 6204 may be used as a reference for the water reduction rate.

Regarding (2): Unit water content in excess of 185 kg/m^3 greatly affects the cracking resistance of concrete as a result of excessive shrinkage or other factors. The upper limit for unit water content was set at 175 kg/m^3 as standard practice, taking into consideration the variability in materials and in mix conditions. When the effects of drying shrinkage do not need to be considered, as in cases of unreinforced concrete or poured-in-place piles, the unit water content may be set in excess of 175 kg/m^3.

Note that the unit water content of concrete that is set here is the unit water content that was determined in the mix design. This does not mean that the upper limit of unit water content, reflecting variation in unit water content caused by surface moisture on aggregates, correction of the mix according to season, or other considerations in actual construction work, should be set to 175 kg/m^3 or lower.

4.6.2 Unit powdery material content

(1) The unit powdery material content is set so as to obtain resistance to material segregation appropriate to the size of the slump.

(2) The unit powdery material content is set within a range appropriate for placement and pumping.

(3) When the lower limit or upper limit of the unit powdery material content is specified, said specification must be satisfied.

Commentary: Regarding (1) and (2)

Figure C4.6.1 shows an example of the relationship between slump for placement and unit powdery material content. If unit powdery material content appropriate to the slump is not secured, then material segregation is likely to occur, resulting in defects such as honeycombing and incomplete filling. **Figure C4.6.2** shows an example of the relationship between unloading slump and unit powdery material content. Performing pumping smoothly without clogging of pipes requires securing unit powdery material content at a certain level or higher. Therefore, as in the lower limits for unit powdery material content shown in the diagrams in **Figure C4.6.1** and **C4.6.2**, to ensure good filling ability and pumpability it is recommended that a unit powdery material content of no less than 270 kg/m^3 be secured when the maximum size of coarse aggregate is 20 to 25 mm (or no less than 250 kg/m^3 when the maximum size of coarse aggregate is 40 mm) or, ideally, no less than 300 kg/m^3. In **"Construction standards,"** fine particles in aggregate are not considered powder. Because the viscosity of the concrete will increase and workability will decrease when the fine particle content in the aggregate is large, the unit powdery material content should be reduced as necessary.

Regarding (3): When the upper limit or lower limit of the unit binder content is specified in design drawing documents, these must be compared with the unit powdery material content derived in (1) and (2) above, and unit powdery material content must be set so that the conditions of both are simultaneously satisfied. If the conditions of both cannot be satisfied, then the intended materials and mix must be changed.

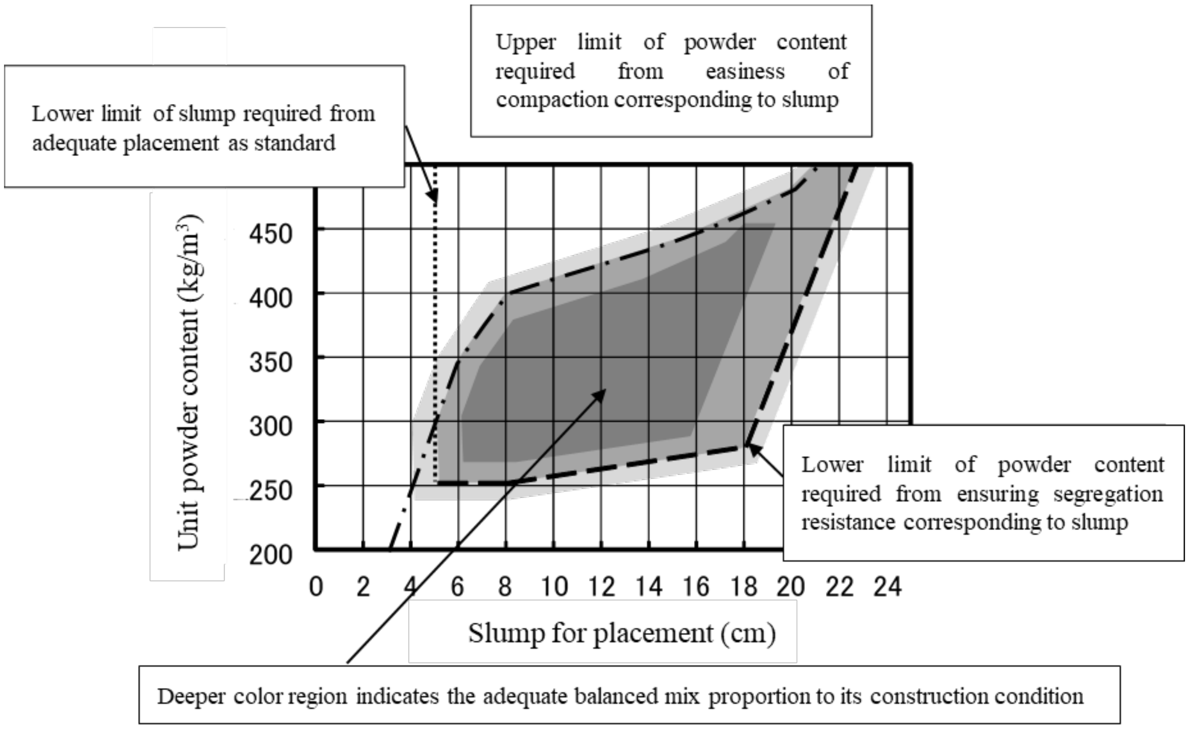


Figure C4.6.1 Example of relationship between slump for placement and unit powdery material content

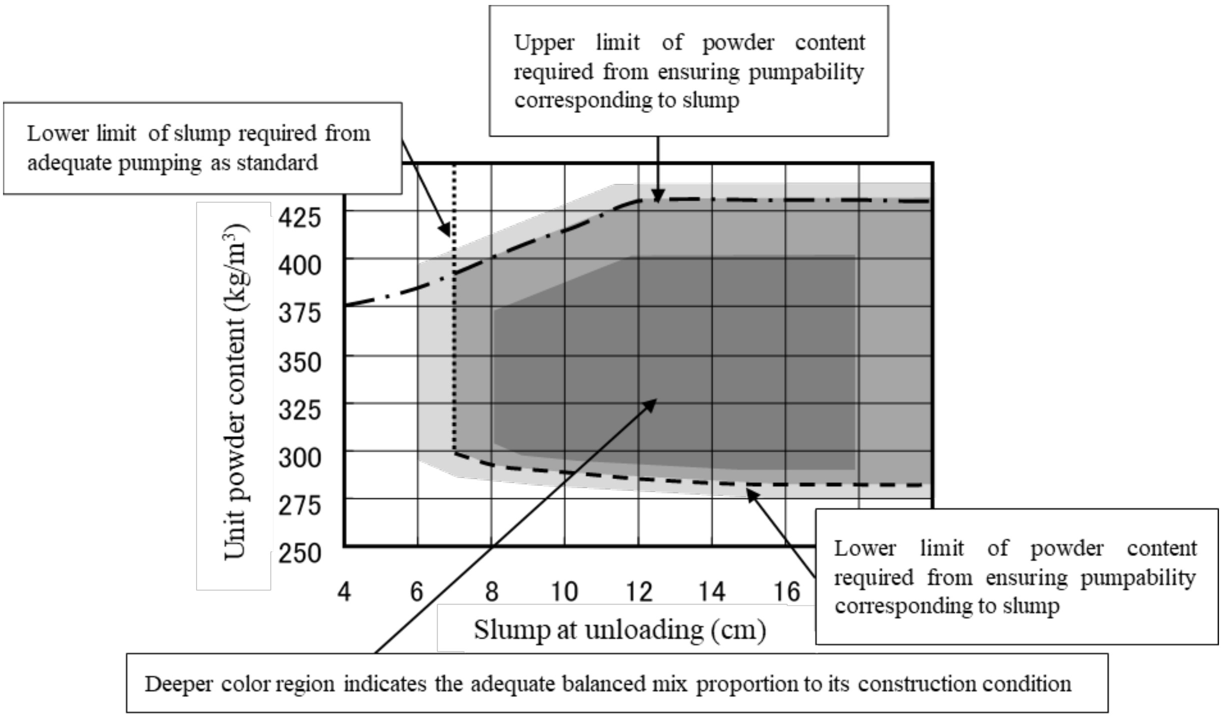


Figure C4.6.2 Example of relationship between slump at unloading to ensure pumpability for placement and unit powdery material content

4.6.3 Unit cement content

The unit cement content is set based on the reference value described in design drawing documents. When the lower limit or upper limit of the unit cement content is specified, said specifications must be satisfied. If they are not satisfied, then the intended materials and mix are changed.

4.7 Trial mixing

4.7.1 General

(1) The concrete mix must be determined through trial mixing to obtain concrete that satisfies the mix conditions.
(2) As standard practice, trial mixing of concrete should be conducted through laboratory testing.
(3) When it can be confirmed from performance records or other means that the planned mix satisfies the mix conditions, trial mixing may be omitted.

Commentary: Regarding (1): Concrete quality is affected by a number of factors. In particular, the quality of fresh concrete varies greatly with the passage of time after mixing, the ambient temperature, and the method of transport on site. In concrete mix design, it is important to set target quality at the stages of both mixing and unloading to ensure the necessary workability of concrete during placement.

4.7.2 Trial mixing method

(1) When trial mixing is performed in laboratory testing, the workability during mixing is judged with consideration of factors including the difference in slump from that of actual production conditions, the concrete temperature at the time of construction work, the mixing performance of the mixer, and the transport time.
(2) As standard practice, trial mixing of concrete should be performed at a room temperature of 20±3°C. When it cannot be performed under these test conditions, the mix should be determined with consideration of the temperature difference.
(3) Concrete slump, air content, compressive strength, and other factors are to be confirmed in trial mixing.

Commentary: Regarding (1): In trial mixing through laboratory testing, modification of the mix must be corrected through iteration to ensure the target slump at the point of unloading, the target slump for mixing, and the specified minimum slump for placement are all obtained, taking into account the decrease in slump not only immediately after mixing but also after the passage of time. When correcting the mix, **Table C4.5.1** and C**4.7.1** should be used as a reference.

Table C4.7.1 Approximate amounts of correction for the sand percentage and the water content due to variations in the quality of materials used or concrete

Category	Correction of s/a (%)	Correction of W
For every 0.1 in fineness modulus of sand greater (smaller) than reference value	Increase (decrease) by 0.5	No correction
For every 0.1 cm in slump greater (smaller) than reference value	No correction	Increase (decrease) by 1.2%
For every 1% in air content higher (lower) than reference value	Decrease (increase) by 0.5 to 1	Decrease (increase) by 3%
For every 0.05 in water/cement ratio higher (lower) than reference value	Increase (decrease) by 1	No correction
For every 1% in s/a higher (lower) than reference value	-	Increase (decrease) by 1.5 kg
When river gravel is used	Decrease by 3 to 5	Decrease by 9 to 15 kg

Chapter 5 Production

5.1 General

(1) Production of concrete must be performed in a manner that yields concrete of the necessary quality.

(2) Equipment used for the storage, weighing, and mixing of materials must be confirmed to possess the necessary performance.

(3) In the production of concrete, methods of storing, weighing, and mixing materials must be determined in advance.

(4) Engineers with extensive knowledge of and experience in the production of concrete must be assigned and must properly perform quality control for concrete materials and concrete and perform maintenance of production equipment.

5.2 Production plant

5.2.1 Storage facilities

(1) Storage facilities for cement and admixtures must be silos that have damp-proof structures and that are capable of storing cement and admixtures separated by type.

(2) Silos for storing cement and admixtures must be constructed so as to avoid the accumulation of dead stock materials.

(3) Storage facilities for aggregates must be capable of storing aggregates by type and by particle size while preventing fluctuations in aggregate particle size.

(4) Storage facilities for aggregates must be provided with appropriate drainage facilities, *etc*. to minimize fluctuations in surface moisture on the aggregates.

(5) Storage facilities for aggregates must be capable of preventing freezing and contamination by ice and snow in cold weather.

(6) Storage facilities for aggregates must be capable of preventing drying and temperature rise in aggregates through means such as avoidance of direct sunlight in hot weather.

(7) Storage facilities for chemical admixtures must be capable of storing chemical admixtures separated by type and capable of preventing contamination by impurities, alteration of properties, separation in liquid chemical admixtures, moisture absorption by powdered chemical admixtures, *etc*.

5.2.2 Weighing equipment

(1) Weighing equipment must be suitable for the concrete production conditions and must possess the accuracy for weighing target materials within the specified weight value tolerances.

(2) Weighing equipment for materials must be inspected before use and regularly during use, and must be calibrated as necessary.

5.2.3 Mixers

(1) As a general rule, batch mixers must conform to JIS A 8603-1.

(2) Continuous mixers must have been confirmed to possess the specified mixing performance through testing in accordance with JSCE-I 502.

5.3 Weighing

(1) To obtain concrete of the necessary quality, the weighing of materials must be conducted using a modified mix that takes into account factors including the state of management of the materials, the concrete temperature, and the slump retention time.

(2) The amount per batch must be determined with consideration of factors including the type and performance of the concrete, the performance of the mixing equipment, the transport method, the type of construction work, and the amount of concrete to be poured.

(3) As a general rule, the weight of each material should be measured per batch. For each weighing, tolerances for the weighed values must not be greater than the values shown in **Table 5.3.1**. The weight of each material used in mixing must also be recorded.

(4) When a continuous mixer is used, materials may be measured by volume. In this case, the value of the amount weighed per specified unit of time (predetermined according to the capacity of the mixer), when converted into mass, must be within the ranges shown in **Table 5.3.1**. The amount weighed per specified unit of time must be determined appropriately based on factors including the type of mixer and the mixing time.

Table 5.3.1 Permissible errors in weighing

Material	Error in weighing (%)
Water	1
Cement	1
Aggregate	3
Mineral admixture	2[1)]
Chemical admixture	3

1) In case of finely ground blast furnace slag, the error in batching shall be taken as 1.

5.4 Mixing

(1) Concrete materials must be mixed thoroughly until the mixed concrete is homogeneous.

(2) The order in which materials are placed into the mixer must be determined in advance.

(3) As a general rule, the mixing time should be determined through testing.

(4) Mixing must not be performed for a length of time over three times the predetermined mixing time.

(5) As a general rule, the interior of the mixer should be coated with mortar before the start of mixing.

(6) New materials must not be placed in the mixer until after all concrete in the mixer has been discharged.

(7) The mixer must be thoroughly cleaned before and after use.

(8) When using a continuous mixer, the first concrete discharged after the start of mixing must not be used.

Chapter 6 Ready-Mixed Concrete

6.1 General

(1) As a general rule, ready-mixed concrete should conform to JIS A 5308 and should bear a JIS Mark indicating JIS certification. (Such concrete is hereinafter referred to as a "JIS certified product").

(2) When purchasing ready-mixed concrete, a production plant must be selected and the specified items provided in JIS A 5308 must be determined through consultation with the producer to obtain concrete of the necessary quality.

6.2 Selection of a plant

(1) When selecting a plant, consideration must be given to factors including transport time to the work site, concrete production capacity, the number of transport vehicles, the production equipment in the plant, and the state of quality control.

(2) Ready-mixed concrete plants must be selected from among plants that produce JIS certified products and that have been approved for the ㊜ mark by the Ready-Mixed Quality Control Inspector Conference.

6.3 Specifications of quality

When purchasing JIS-certified products, the type of ready-mixed concrete and other necessary matters must be specified to obtain concrete of the necessary quality. This also applies when purchasing ready-mixed concrete that is not JIS certified.

(a) Types of ready-mixed concrete

The type of ready-mixed concrete should be selected based on the maximum size of coarse aggregate, the nominal strength, the target slump at the point of unloading or the target slump flow, and the type of cement, with consideration of factors including the quality necessary for fresh concrete and changes in quality during transport and between unloading and placing.

(b) Specified matters

The type of cement, the type of aggregate, the maximum size of the coarse aggregate, and measures to inhibit the alkali–silica reaction should be specified through consultation with producers. The material age that guarantees nominal strength, the target values for the upper limits of the water-to-cement ratio and unit water content, the upper or lower limits of the unit cement content, the air content, and other matters are also determined through consultation with producers as required, with consideration of the material age, resistance to deterioration, resistance to cracking, and other factors that are criteria for the specified strength.

6.4 Confirmation of the mixing plan

When placing orders for ready-mixed concrete, a mixing plan must be requested from the producer and the content of the plan must be checked.

6.5 Acceptance

(1) In order to perform concrete placement without trouble, matters including the type of ready-mixed concrete, quantity, delivery date and time, location of unloading, and rate of delivery must be determined in advance through consultation with the producer.

(2) Planning and management must be performed to avoid long periods of standby for transport vehicles at the work site.

(3) Close communication with the producer must be maintained during placement to prevent any interruption of the work.

(4) Unloading should be performed at a location where transport vehicles are able to enter and exit safely and without hindrance, and where unloading work can proceed easily.

(5) Unloading should be performed by means that prevent segregation of materials.

(6) At the time of acceptance, the purchaser must confirm that the statement of delivery for the ready-mixed concrete matches the content of the order, and must conduct an acceptance inspection by an appropriate method.

Chapter 7 Transport, Placement, Compaction, and Finishing

7.1 General

Transport, placement, compacting, and finishing must be performed so as to obtain concrete structural objects that possess the necessary performance.

Commentary: To account for changes in the quality of fresh concrete and to prevent defects in placement, compacting, and finishing to achieve a concrete structural object that satisfies the necessary quality, it is necessary to consider transport in advance and carry it out using the planned methods, with full understanding of the information in this chapter.

7.2 Time from mixing to the completion of placement

A time of up to 2 hours from mixing to the completion of placement when the outside air temperature is up to 25 °C and a time of up to 1.5 hours when the temperature exceeds 25 °C is set as standard. When placement time will be longer than this, it is necessary to confirm in advance whether the necessary concrete quality can be ensured and to set limits on time.

Commentary: Concrete construction faces transport constraints that include the location of the structural object, plants that can be reached by transport, and transport routes. Constraints on placement and compacting work include the form of the structural object, the arrangement of reinforcement, and the available personnel and equipment. It is important that these constraints be taken into consideration, that planning and management of the time necessary for each task be performed to facilitate trouble-free progress, and that changes in the quality of fresh concrete over time be confirmed in advance. The limit on time from mixing to completion of placement varies with factors including the concrete mix, materials, temperature, humidity, and transport method. Taking these conditions into account, the limit on time should be set appropriately for each construction task.

7.3 Transport

7.3.1 Transport to the construction site

(1) Transport to the work site must employ methods by which unloading is easy, material segregation is unlikely to occur during transport, and changes in slump, air content, *etc*. are small.

(2) When using ready-mixed concrete, the transport-related provisions of JIS A 5308 must be followed.

Commentary: Regarding (1): When the transport distance is long or when the slump of the concrete is large, transport should be conducted using a agitator truck or a mixer truck equipped with an agitator or other stirring capability. When no record of performance exists for the combination of concrete quality and transport method, a testing method that assumes the actual construction work should be performed to confirm effects on changes in concrete quality.

Regarding (2): These Standard Specifications set the time from mixing to the completion of placement to 1.5 hours or less as standard practice when the outside air temperature exceeds 25 °C. The transport time from the plant to the work site should be set through consultation with the producer, with 1 hour or less as a guideline. Because concrete placement and other tasks may take time under some site conditions, measures to address this should be discussed in advance.

7.3.2. On-site transport

7.3.2.1 Concrete pumps

(1) When pumping concrete, the pumpability of the concrete and the quality of the concrete after pumping must be considered, and the types and number of concrete pumps, the diameters of transport pipes, piping routes, discharge volumes, and other matters must be determined.

(2) As a general rule, mortar with a water-to-cement ratio less than that of the intended concrete should be pumped before the concrete is pumped. As a general rule, this mortar pumped in advance should not be poured into formworks.

(3) Pumping must be carried out as continuously as possible. When a long interruption is unavoidable, appropriate measures must be taken so that the pumpability and quality of the concrete are not impaired after pumping is resumed.

Commentary: Regarding (1) It is important that not only slump but also air content, unit mass, temperature, and other properties of the concrete fall within the required ranges following pumping.

Concrete pumping must be performed by persons who possess sufficient knowledge and experience to adapt to the pumping conditions. In particular, when high-altitude pumping, low-altitude pumping, long-distance pumping, or other advanced techniques are necessary, the pumping plan should be prepared through consultation with registered concrete pumping engineers who are well-versed in the capabilities and performance of concrete pumps and who have extensive practical experience. Even construction work overseers who do not directly engage in pumping work must have knowledge of concrete pumps and the ability to manage pumping workers and must fully understand the content of these Standard Specifications; otherwise, difficulties will arise in carrying out construction work according to plans.

Particular attention should be paid to the following points when planning concrete pumping:

(i) Diameter of transport pipes

Transport pipes with nominal dimensions of 100A (4B) or 125A (5B) are commonly used in the pumping of ordinary concrete, while 150A (6B) transport pipes are sometimes used in large-scale construction. The nominal size is the inner diameter of the transport pipe, expressed in millimeters and inches. As an example, 100A (4B) indicates an inner diameter of approximately 100 mm or 4 inches.

Near bent pipes and tapered pipes, velocity distribution fluctuates inside the pipes, in-pipe pressure loss increases, and clogging readily occurs. To address this, bent pipes with a large bending radius and tapered pipes with a gradual taper should be used.

(ii) Placement of concrete pumps and routing of piping

The placement of concrete pumps and the routing of piping should be determined in such a manner that the piping distance is as short as possible and the number of bends is as low as possible. Stable pumping requires that appropriate selections be made for matters including piping routes, means of support for pipes, work scaffolding, checking of transport pipe interiors for deposits and wear, and the ease of tasks including vehicle assignment, assembly of transport piping, cleaning of pipe interiors after pumping, and relocation and removal of transport pipes.

(iii) Type and number of concrete pumps

Selection of the type of pump is the most important matter in ensuring trouble-free concrete pumping work. Methods for selecting the type and number of concrete pumps are shown below:

(a) Calculation of the maximum pumping load on a concrete pump

The maximum pumping load (P_{max}) on a concrete pump can generally be derived from the following equation:

$$P_{max} = \text{(Loss in pressure in pipe per 1 m length of horizontal pipe)} \times \text{(horizontal equivalent distance)}. \quad \text{(C7.3.1)}$$

The loss in pressure per 1 m of length of horizontal pipe is determined by the type and quality of the concrete, the discharge volume, and the diameter of the transport pipe. The loss is greater when slump is smaller, the diameter of the transport pipe is smaller, and the rate of discharge is greater. Standard values in the case of concrete with a maximum coarse aggregate size of 20 to 25 mm are shown in **Figure C7.3.1** and C**7.3.2**.

When a horizontal pipe is used for transport, its length is used as the horizontal equivalent distance. When non-horizontal transport pipes are included, the length is converted to that of horizontal pipes using the horizontal equivalent coefficient shown in **Table C7.3.1**. The horizontal equivalent factor is the ratio of the loss of pressure per 1 m of the transport pipe to the loss of pressure per 1 m of horizontal pipe. The total distance derived through the conversion of all transport pipes to the horizontal pipe equivalent is the horizontal equivalent distance.

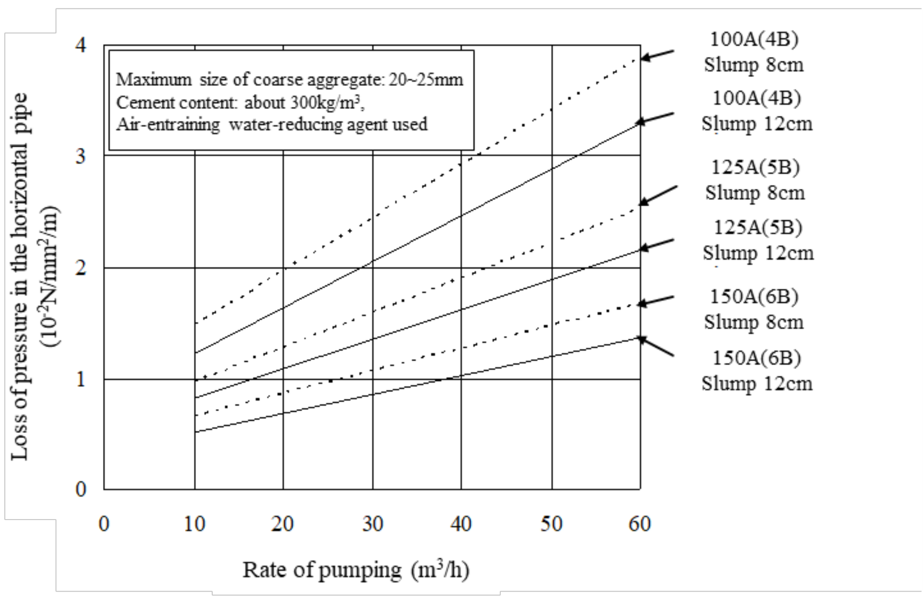


Figure C7.3.1 Standard values of the loss in pressure of air-entraining water-reducing agent concrete

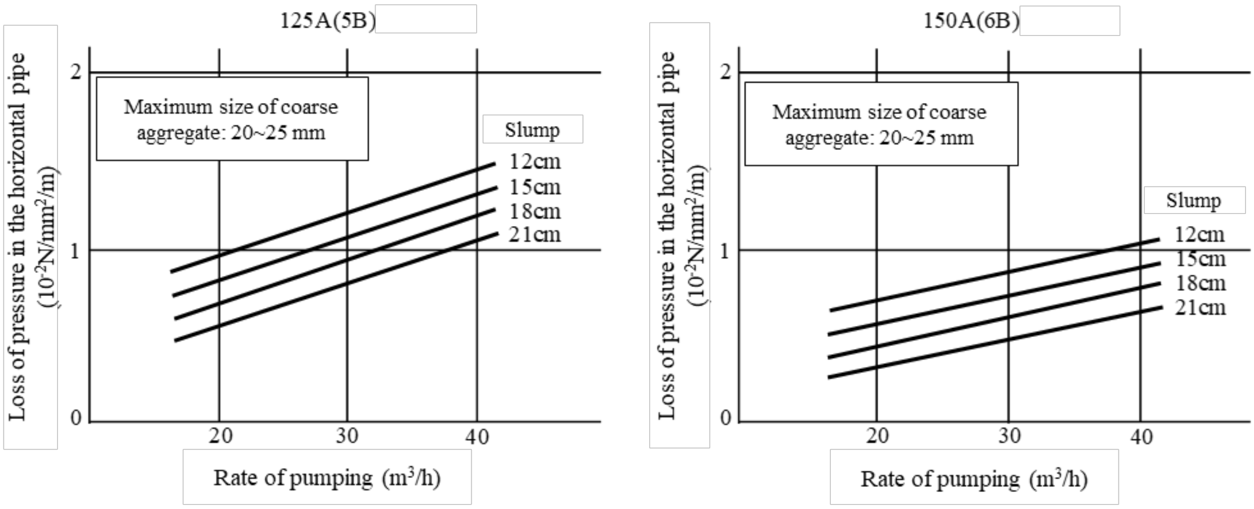


Figure C7.3.2 Standard values of the loss in pressure of air-entraining high-range water-reducing agent concrete

Table C7.3.1 Equivalent horizontal pipe length

Item	Unit	Nominal diameter of pipes	Equivalent horizontal pipe length (m)
Vertical pipes	1 m	100A(4B) 125A(5B) 150A(6B)	3 4 5
Tapered pipe		175A→150A 150A→125A 125A→100A	3
Bent pipe		90° rr=0.5 m or1.0 m	6
Flexible hose		—	$\frac{20}{L}$

(b) Selection of type of concrete pump

In selecting a concrete pump type, the necessary theoretical discharge pressure ($1.25P_{max}$) is first calculated by multiplying the maximum pumping load (P_{max}) derived above from Equation (C7.3.2) by a coefficient (1.25) that takes into account mechanical loss and quality fluctuations in the concrete. In addition, following Equation (C7.3.3), the necessary theoretical discharge volume (Q_{max}) is calculated by dividing the per-hour necessary discharge volume required in the construction plan by the mechanical efficiency. This is done to account for the fact that actual discharge volume is less than the theoretical discharge volume, owing to the pump's efficiency of suction and discharge. The efficiency of these varies with factors including the concrete mix, the properties of the fresh concrete, and the performance of the machinery. As a reference, **Table C7.3.2** presents guidelines for the relationship between slump at the point of unloading of the concrete and the mechanical efficiency of concrete pump:

Required theoretical discharge pressure ($1.25\ P_{max}$)

$$= 1.25 \times P_{max} \qquad \text{(C7.3.2)}$$

and:

Required theoretical discharge volume (Q_{max}) = (per-hour necessary discharge volume) / (mechanical efficiency). (C7.3.3)

Using the calculated necessary theoretical discharge pressure and necessary theoretical discharge volume, a comparison is performed with the performance curve (*P-Q* curve) that expresses the pumping capacity of the concrete pump. Then, a type of pump capable of performing the pumping is selected. If an appropriate type is unavailable or cannot be procured, then the piping route, discharge volume, and other pumping conditions must be reviewed.

As the concrete placement speed set in the construction work plan is an average speed that incorporates tasks including movement of the pipe tip and compaction, it is less than the actual discharge volume emitted from the concrete pump. Therefore, the discharge volume that is assumed in the calculation of pipe internal pressure must be set to a value (*i.e.*, necessary theoretical discharge volume per unit of time) that takes these additional tasks into account, and a plan that enables trouble-free pumping even at that discharge volume must be created.

Table C7.3.2 Machinery efficiency of concrete pumping as standard

Slump at unloading (cm)	Types of concrete pumping	
	Piston	Squeeze
6~11	0.65~0.70	-
12~17	0.70~0.90	0.75~0.90
18~21	0.85~0.90	0.85~0.90

<u>(c) Selection of the number of concrete pumps</u>

The number of concrete pumps should be determined with consideration of factors including the pumping volume per unit of time, the discharge volume of the planned pump type, the size of concrete placement sections, the total volume to be poured, the placement order, the placement speed, the status of concrete supply, the compaction capacity, the number of placement locations, and the work time. In construction work in which continuous placement of concrete is essential, spare concrete pumps should be prepared.

<u>(iv) Pumping testing</u>

When difficulties in concrete pumping are expected, pumping testing must be conducted in advance under piping conditions that are close to the actual construction work conditions, and the operational status of the concrete pump, the pumping load, the state of the discharged concrete, and other matters must be checked.

Work and environmental conditions affecting concrete or construction that must be considered during pumping include the following:

(a) Pumping of a lean concrete mix with unit cement content of less than approximately 270 kg/m^3 or a rich concrete mix with unit cement content of over approximately 350 kg/m^3.

(b) Pumping of concrete with a pre-pumping slump of less than 8 cm.

(c) Pumping of concrete in a construction work environment described in **"Chapter 12 Cold Weather Concreting"** or **"Chapter 13 Hot Weather Concreting"**.

(e) Pumping to a high point or a low point, or over a long distance. The horizontal equivalent distance in general concrete construction work is typically less than 150 m. When the distance exceeds 150 m, the decrease in slump associated with pumping must be fully considered.

(e) Pumping of flowing concrete, lightweight aggregate concrete, high-strength concrete, high-fluidity concrete, short-fiber-reinforced concrete, underwater concrete, or sprayed concrete.

When test pumping conducted under conditions close to those of construction work is impossible as a result of conditions at the work site or other constraints, pumpability may be evaluated based on past performance under similar conditions. Alternatively, test pumping may be performed using short pipes and pumpability may be evaluated based on the conditions at this time.

Regarding (2): Before the start of concrete pumping, mortar must be pumped to ensure lubricity in the concrete pump and on the inner surface of transport pipes and to prevent clogging.

The quality of the mortar pumped in advance may undergo changes as a result of adhesion to transportation pipes, *etc*., or as a result of contamination by residual water, grease, residue, *etc*. in the pipes. When poured into formworks, large amounts of mortar may be present locally in the concrete, resulting in a loss of homogeneity in the concrete structural object. For this reason, as a general rule, mortar should not be poured in advance into formworks. Because mortar pumped in advance may be mixed into the first concrete that is discharged during construction work, the general rule was set that the water-to-cement ratio of mortar pumped in advance should be no greater than that of the concrete.

Regarding (3): If pumping is interrupted and the concrete is allowed to remain in pipes for a long time, then pumpability will deteriorate as a result of segregation of materials and decline in workability, which may result in clogging or other issues. The quality of the concrete may also be affected. Therefore, it is advisable that the concrete supplied to the hopper of the concrete pump be pumped continuously and that placement and compacting be performed quickly. When interruption is unavoidable, the time until resumption should be predicted as quickly as possible and communicated to parties concerned. When long-term interruption is expected, interval operation should be implemented to prevent clogging. When there is a likelihood of clogging as a result of a long interruption, the concrete in the pipes should be discharged.

7.3.2.2 Buckets

Buckets must have a structure that does not readily cause material segregation, that facilitates concrete discharge, and that does not leak concrete or mortar when closed.

Commentary: The work method by which concrete discharged from the mixer is received in an appropriately structured bucket and is promptly transported to the placement site is convenient in that it allows easy transport to the placement site both vertically and horizontally. The structure of the bucket should have the discharge port placed directly under the center, as off-center placement may easily result in material segregation during discharge. Because transport by bucket often requires more time than concrete pumping and because buckets typically do not have stirring functions, material segregation and change in workability may occur if concrete is left in the bucket for a long time.

7.3.2.3 Chutes

(1) When chutes are used, the use of vertical chutes is set as standard practice.

(2) When the use of an inclined chute is unavoidable, the inclination of the chute should be such that the concrete flows smoothly and material segregation does not occur. An inclination with a ratio of vertical to horizontal of approximately 1:2 should be set as standard practice.

(3) The structure of the chute and its method of use must be such that material segregation does not readily occur in the concrete.

Commentary: Regarding (1) and (2): Material segregation readily occurs when concrete is transported by an inclined chute. Because concrete with a large slump is often used to improve concrete flow, material segregation may become even more pronounced. When construction work conditions make the use of an inclined chute unavoidable, the inclination of the chute should be determined with consideration of both the smooth flow of concrete and the prevention of material segregation in the falling concrete. In these Standard Specifications, an inclination with a ratio of vertical to horizontal of approximately 1:2 is set as standard practice.

Regarding (3): In a vertical chute, joints must possess sufficient strength so as to prevent them coming apart under the impact of falling concrete. When using an inclined chute, a funnel tube or baffle plate must be installed at the discharge port to inhibit material segregation.

The chute should be washed well with water before and after use, and mortar should be allowed to flow through prior to use. However, care must be taken to prevent water or mortar from flowing down the chute and into the concrete or formworks.

The bottom end of the funnel tube should be kept as close as possible to the concrete placement surface. The interval between concrete inlets, the order of injection, and other matters must be considered. When material segregation is observed in concrete transported through an inclined chute, a receptacle must be provided at the discharge port of the chute and the concrete must be remixed before use.

7.3.2.4 Other transport machinery

(1) When using a belt conveyor, appropriate measures should be taken against sunlight, wind, rain, and so on as necessary. As a general rule, a baffle plate and a funnel tube should be installed at the end of the conveyor to prevent material segregation of the concrete.

(2) When using a wheelbarrow, trolley, *etc.* for transport, the transport distance should be shortened to the extent possible to prevent material segregation of the concrete.

Commentary: Regarding (1): In the placement of belt conveyors, differences in level between belt conveyors must be minimized to the extent possible, with inclines set to prevent material segregation. As long transport distances will result in longer exposure to sunlight and air, which may result in drying of the concrete and loss of slump, measures such as the provision of covering must be taken. When a large amount of mortar adheres to the end of the belt conveyor and the mortar content of the concrete is insufficient, measures such as scraping off the mortar are necessary. Providing a baffle plate and funnel tube is also effective in preventing material segregation. The end of the belt conveyor must be moved appropriately to keep concrete from collecting at only one location in formworks.

Regarding (2): Because concrete may undergo material segregation as a result of vibration during transport, the transport path should be made as flat as possible and the transport distance should be restricted to 50 m to 100 m or less as a guideline.

7.4 Placement

7.4.1 Preparation

(1) Before concrete is poured, it must be confirmed that reinforcement bars, formworks, *etc*. are arranged and firmly fixed in place as specified in design and construction work plans and will not interfere with the placement and compacting of the concrete.

(2) Before placement is performed, information on precipitation and strong winds must be collected and measures to address these must be prepared.

(3) Immediately before concrete placement, the transport equipment, the placement equipment, and the interior of formworks must be cleaned to prevent the concrete from becoming contaminated with wood chips, debris, *etc*. Areas that could potentially absorb water when in contact with concrete must be wetted in advance.

(4) Any water pooled in formworks must be removed before placement. Appropriate measures must also be taken to prevent water from flowing into the formworks and over newly poured concrete.

Commentary: Regarding (1): When placement is difficult, the position of the horizontal reinforcement bars can be temporarily shifted to secure an opening for placement concrete. The shifted reinforcement bars must be returned to their specified positions afterward. It is also important to take measures to properly secure an opening for placement so that falling concrete does not directly contact reinforcement bars, *etc*.

Regarding (2): Rainfall occurring during concrete placement may increase the water-to-cement ratio or wash away mortar, potentially causing deterioration of the quality of the concrete and impairing the performance of the structural object. To avoid construction work during poor weather, it is important to check the hourly or daily weather forecast for the area of the work site before the start of placement.

Regarding (3) and (4): Because vacuuming up debris and water from above is difficult when formworks are high, a discharge port should be provided near the bottom of the formworks in advance and later closed up after cleaning with compressed air, *etc*.

Areas that may absorb water must be wetted in advance, but care must be taken to prevent excessive wetting that would result in standing water. If concrete is poured when standing water is present in formworks as a result of rainfall, inflow of groundwater, spray curing, or bleeding from concrete already poured, then the quality and integrity of the concrete may be impaired. Therefore, standing water must be removed before placement.

7.4.2 Placement

(1) When placement concrete, care must be taken to prevent reinforcement bars and formworks from moving from their specified positions.

(2) Poured concrete must be prevented from moving laterally within formworks.

(3) When significant material segregation is observed during placement, measures must be taken to control the segregation.

(4) Concrete must be poured continuously until completed, except at planned construction joints.

(5) As a general rule, concrete should be poured so that the placement surface is nearly level. A single layer of concrete should be no more than 40 to 50 cm as standard practice.

(6) When placement concrete divided into multiple layers, the work must be performed so that upper and lower layers are integrated. In addition, the surface area of the construction work section, the concrete supply capacity, the time interval for layering concrete, and other matters must be decided to prevent cold joints from occurring. **Table 7.4.1** presents standards for the allowable layering time intervals.

Table 7.4.1 Standard for the allowable time lag between two placing lifts

Temperature in the environment	Allowable time lag between two placing lifts
25°C or less	2.5 hours
Over 25°C	2.0 hours

(7) When the height of formworks is significant, concrete must be poured by providing an inlet in the formworks or by lowering the vertical chute or transport pipe's discharge port to close to the placement surface. In this case, the free-fall height from the discharge outlet of the chute, transport pipe, bucket, hopper, *etc*. to the placement surface should be set to 1.5 m or lower as standard practice.

(8) Bleeding water pooled on the surface of the concrete must be removed by an appropriate method before the concrete is poured.

(9) The rate of placement should generally be set to about 1.0 to 1.5 m per 30 minutes as standard practice.

(10) When the concrete of a slab or beam is continuous with the concrete of a wall or column, placement the concrete of the slab or beam after the subsidence of the concrete for the wall or column has nearly completed should be set as standard practice to prevent subduction cracking.

(11) When concrete is poured directly onto the ground, leveling concrete should first be poured as a general rule.

Commentary: Regarding (1): It is advisable that reinforcement bar workers be stationed during the placement work in preparation for a possible disturbance of the reinforcement bar arrangement during placement work and that formwork workers be stationed in preparation for damage to formworks during placement.

Regarding (2): Because the potential for material segregation to occur increases every time that concrete is moved, it is important to unload and pour the concrete at the desired location. Placement intervals must be set with consideration of factors including the form of structural members, the fluidity of the concrete to be used, and the height of the compaction work. When the thickness of the structural members is great, the steel material is densely arranged, and the locations for placement concrete are limited, the placement interval should be 2 to 4 m as a guideline.

Regarding (3): When significant material segregation is observed during placement, it will be difficult to remix the concrete to be homogeneous. Therefore, it is necessary to interrupt the placement, investigate the causes of the material segregation, and implement remedial measures. In areas where the coarse aggregate in poured concrete has undergone segregation and the mortar content is low, it is advisable to eliminate segregation by scooping up the segregated coarse aggregate, embedding it in concrete with a large mortar content, and compacting it.

Regarding (5): When there are few placement locations and rod-shaped vibrators are used to move the concrete laterally, the management of compacting interval and time becomes difficult when compacting work cannot keep up with the rate of placement, and the possibility that dense concrete cannot be poured increases. Therefore, it is important to coordinate a good balance between the number of placement locations and rate of placement.

When the height of a layer is no more than approximately 40 to 50 cm, the height will be less than the length of the vibrating part of a rod-shaped vibrator and lateral movement of the concrete can be controlled. For that reason, these Standard Specifications set that range as a standard.

Regarding (6): The allowable layering time interval is the time interval during which integration between a lower layer and an upper layer of concrete can be maintained by first placement the lower layer and then placement the upper layer before the lower layer has begun to harden. If the upper layer of concrete is poured after the lower layer has begun to harden, then cold joints may form. To prevent this, it is important to set and manage the allowable layering time interval with consideration of factors including the type and quality of concrete, the elapsed time from the start of mixing to the completion of placement, the temperature of the concrete, and the compaction method. When placement the upper layer of concrete, a rod-shaped vibrator should be inserted into the lower layer as well and compaction should be performed, following **7.5 (4)**.

For general concrete, conducting layering within the time shown in **Table 7.4.1** may be set as standard practice. However, to enable layering at time intervals as short as possible, it is important to carefully consider factors including the partitioning of placement sections, the placement height per layer, and the placement order.

When it is expected that layering concrete within the time shown in **Table 7.4.1** will be difficult, it is possible to extend the time until setting by means such as the use of a retardant chemical admixture. It has been shown that when the penetration resistance as measured in testing using a penetration resistance tester according to JIS A 1147 exceeds 0.1 N/mm^2, compaction will be difficult and the risk of cold joints is high. In this case, the setting time should be confirmed in advance through testing and the allowable layering time interval in construction work should be set.

When placement concrete past the standard time from mixing to the end of placement shown in 7.2, the upper surface of the concrete may begin hardening. Therefore, the concrete to be used must be tested in advance and the allowable layering time interval must be set.

Regarding (7): Standards for the drop height from the discharge port to the placement surface (*i.e.*, the concrete's free-fall height) are presented to prevent material segregation caused by the dropping of concrete from a high location. When this drop height is exceeded, an inlet must be provided at an appropriate location in the formworks, or the discharge port outlet of the vertical chute, transport pipe, *etc*. must be lowered close to the placement surface.

Regarding (8): If water is not removed from the concrete placement surface, then it may wash over the surface in contact with formworks and cause sand streaks or the formation of a weak layer near the placement surface.

Regarding (9): If the rate of concrete placement is excessively increased in high walls or columns, then the pressure acting on formworks will increase (see 11.2.4 "Lateral pressure in concrete" in "Chapter 11 Formworks and Shoring"). While it is advisable to change the rate of placement in line with cross-sectional size, the concrete mix, compaction method, *etc*., the rate should generally be set to approximately 1.0 to 1.5 m per 30 minutes as standard practice.

Regarding (10): The time required for subsidence of concrete to settle is generally 1 to 2 hours.

Regarding (11): When placement concrete directly onto the ground, the placement of leveling concrete was set as general practice to ensure integrity of the ground with the structural object, prevent unevenness in the ground, ensure the specified structural member thickness and cover, inhibit moisture transfer with the ground, and ensure the quality of the concrete.

7.5 Compaction

(1) The use of a rod-shaped vibrator for compaction of concrete should be set as a general rule. However, when the use of a rod-shaped vibrator is difficult at locations close to formworks, a formwork vibrator must be used to ensure compaction.

(2) To prevent the planned compaction work height from being exceeded, it is necessary to consider the installation of scaffolding and the method of compaction.

(3) Concrete in contact with end-plates must be poured and compacted so as to obtain as flat a surface as possible.

(4) When placement concrete, a rod-shaped vibrator must be inserted approximately 10 cm into the lower layer of concrete to achieve integration between the upper layer and lower layer.

(5) The insertion interval of the rod-shaped vibrator and the vibration time at each location must be determined so as to sufficiently compact the concrete. In addition, the rod-shaped vibrator must be gradually pulled out from the concrete to prevent a hole from remaining.

(6) When revibration is performed, it must be done at an appropriate time within the time span during which compaction is possible.

Commentary: Regarding (1): Because civil engineering structural objects often feature relatively thick structural members and make use of concrete with stiff consistency, the use of a rod-shaped vibrator was set as standard practice for compaction. However, at locations close to formworks where the use of a rod-shaped vibrator is difficult, such as at the locations of cover over reinforcement bars, a formwork vibrator must be properly used to enhance the filling ability of the concrete.

Regarding (2): In these Standard Specifications, the height of compaction work is assumed in **"Chapter 2 Concrete Quality"** and **"Chapter 4 Mix Design"** and the appropriate slump is determined. It is important to consider the installation of scaffolding and the method of construction work to prevent the compaction work height from exceeding planned values.

Because the mix, workability, and other aspects of the concrete are determined in line with the form and dimensions of structural members and the arrangement of reinforcement, the concrete must be carefully compacted to an extent that does not impair workability at locations where reinforcement is densely arranged or where it is otherwise difficult for concrete to adequately fill all areas.

Regarding (4): When some time has passed since placement, lower layers of concrete will often become harder than upper layers even if the allowable layering time interval has not been exceeded. To prevent insufficient compaction of the lower-layer concrete into which the rod-shaped vibrator is inserted or the occurrence of material segregation due to excessive compaction of upper-layer concrete, before placement the upper-layer concrete, it is necessary to check the condition of the lower-layer concrete with a rod-shaped vibrator or other method and the placement height of the upper layer concrete, *etc*. should be adjusted.

Regarding (5): To provide uniform vibration to poured concrete, the insertion interval for the rod-shaped vibrator and the vibration time per location must be determined and communicated to workers in advance. Key points of note when using a rod-shaped vibrator are as follows:

(i) The rod-shaped vibrator should be inserted vertically at intervals as uniform as possible. The interval should be less than the diameter of the range over which vibration is recognized as effective, which should generally be less than 50 cm for concrete of average fluidity and viscosity.

(ii) One indicator of sufficient compaction is the appearance of cement paste lines at the interface between the concrete and end-plates. Sufficient compaction can also be confirmed by the surface becoming nearly flat and taking on a glossy appearance, with no reduction in concrete volume. A general guideline for compaction time is 5 to 15 s.

(iv) Lateral movement must not be made using the rod-shaped vibrator.

(v) The type, rod diameter, form, and number of rod-shaped vibrators must be selected to suit factors including the cross-sectional thickness and surface area of the structural members, the maximum amount poured per hour, the maximum dimension of coarse aggregate, the mix, and, in particular, the sand aggregate ratio and concrete slump.

Regarding (6): Revibration refers to the repeated application of vibration at an appropriate time after the initial compacting of concrete. When revibration is performed at an appropriate time, the concrete once again achieves fluidity, which reduces surplus water and voids in the concrete. This is effective in increasing the strength of the concrete and its adhesion to reinforcement bars and in preventing subduction cracking. The appropriate time for revibration is the latest time possible within the time span during which fluidity is restored by compaction. The timing must be determined properly; however, the concrete may be damaged or its adhesion to reinforcement bars may decline if revibration is performed on concrete that has already begun to harden.

7.6 Finishing

(1) After the upper surface of the concrete has been compacted and leveled to nearly the specified height and form, finishing must not be performed until water no longer seeps out and water on the upper surface has been removed.

(2) After finishing work, any cracks occurring before the concrete begins to harden must be repaired through tamping and refinishing.

(3) When a smooth and dense surface is required, the concrete surface must be finished by applying strong force using a steel trowel, at as late a time as possible within the time span during which work is still possible.

Commentary: Regarding (1): When water seeping from the surface of the concrete is not removed, laitance, fine cracking, or the formation and detachment of a weak layer may occur.

Performing excessive leveling while bleeding is still occurring may cause cement paste to accumulate near the surface and make shrinkage cracking more likely to occur, or may reduce resistance to abrasion by allowing a weak layer to form on the concrete surface.

Regarding (2): Around the time that bleeding water disappears from the surface of the concrete, cracking may readily occur as a result of shrinkage caused by rapid drying of the surface and other external forces. Cracks should be repaired by tamping with a trowel, followed by refinishing.

Regarding (3): The final finishing time must be determined with consideration of the setting time, drying conditions, and other matters concerning the concrete. The time for performing trowel work varies with the concrete mix, the weather, the temperature, and other factors. As a guideline, it can be performed when the concrete has hardened enough to resist indentation when pressed by a finger. The cement paste is tamped and a dense surface is finished using a steel trowel and strong pressure.

Chapter 8 Curing

8.1 General

Methods must be established to keep concrete wet and at the temperature required for hardening for a specified period of time after placement without subjecting the concrete to adverse actions. Curing must be performed in a manner that ensures the required quality in the concrete.

Commentary: Quality and crack resistance near the surface of concrete, which have effects on the durability of a structural object, are readily affected by curing. Therefore, it is important that specific methods and periods of curing be appropriately determined in accordance with the individual conditions for the construction, following applicable stipulations concerning the methods and periods, conditions concerning the structural object (structural form, the types, forms, and dimensions of structural members, etc.), conditions concerning the concrete (required performance, materials used, and mix), and conditions concerning the construction work environment (ambient temperature, humidity, etc.). The builder should allocate engineers who possess sufficient knowledge and experience about curing and should determine curing methods with consideration of the importance of the structural object, the efficiency of construction work, economic efficiency, and other factors.

Curing is classified into three categories according to purpose: "keeping wet state," "control temperature," and "protect against adverse actions."

8.2 Moist curing

(1) After placement, concrete must be maintained in a sufficiently wet state for a specified period of time by an appropriate curing method, according to the location.

(2) The moist curing period must be appropriately determined in accordance with the type of cement used, the ambient temperature during the curing period, and other factors. **Table 8.2.1** is to be used as standards for the moist curing period of concrete in ordinary concrete construction.

Table 8.2.1 Standard curing duration

Daily mean temperatures	High-early strength cement	Ordinary Portland cement	Blended cement B
More than 15°C	3 days	5 days	7 days
More than 10°C	4 days	7 days	9 days
More than 5°C	5 days	9 days	12 days

Commentary: Regarding (1): After concrete has been poured, drying from the surface must be prevented so that the hydration reaction of the cement is not hindered. Moreover, if the surface alone rapidly dries because of direct sunlight, wind, or other causes, cracking may result. For these reasons, it is advisable to provide a sunshade and windbreak using sheets on the concrete placement surface or other means. In this way, preparation work for the curing of concrete should be performed in parallel with placement and finishing so that curing can be started immediately after placement.

Spraying the exposed surface of the concrete, covering the surface with sheets, or similar actions before the concrete hardens may degrade quality and finish near the concrete surface. Therefore, these actions are to be started only after the concrete has hardened to an extent that work can be performed without damaging the surface.

Regarding (2): The speed of the cement hydration reaction varies with the type of cement used and the ambient temperature during curing. These factors must be taken into account in determining the moist curing period. **Table 8.2.1** in these Standard Specifications shows standards for the moist curing period for concrete in ordinary concrete construction. The numerical values in the table are standard moist curing periods set with reference to the results of experiments on strength development in concrete when adequate water was supplied. However, when enhancement of the quality of concrete can be expected, it is advisable to lengthen the moist curing period as much as possible, to an extent that does not adversely affect the efficiency of construction work or economic efficiency. Therefore, even when the compressive strength required for removal of the formwork and shoring shown in **Table C11.8.1** in **"Chapter 11 Formwork and Shoring"** is attained quickly, the concrete should be kept in a wet state for the period indicated in **Table 8.2.1**.

When using moderate heat Portland cement, low heat Portland cement, or other cement not noted in **Table 8.2.1**, a moist curing period must be appropriately set on the basis of reliable materials and testing.

8.3 Thermal-controlled curing

(1) Concrete must be kept at the temperature conditions required for hardening until hardening has sufficiently progressed, and the temperature during curing must be controlled as necessary to prevent adverse effects from low temperature, high temperature, sudden temperature changes, etc.

(2) The method of temperature control and the curing period and its management method must be determined on the basis of the type of concrete, the form and dimensions of the structural object, the construction method, and environmental conditions.

Commentary: Regarding (1) and (2): In general, the effects of curing temperature and material age on compressive strength can be expressed using the concept of integrated temperature based on the temperature dependence of the hydration reaction. The period necessary to obtain the required compressive strength is long when the curing temperature is low and short when the curing temperature is high. Moreover, the hydration reaction of ground granulated blast-furnace slag and fly ash is highly temperature-dependent, and temperature history and strength development within structural members may vary significantly with factors including the dimensions of structural members and the thermal insulation performance of formwork. Therefore, caution is required. Checking the strength of specimens that have been cured under conditions as identical as possible to those of the concrete at the site facilitates determination of whether the assumed curing period is appropriate.

Commentary Table 3.2.1 in **"Design: Standards methods" Part 6 Thermal Cracking Verification** shows reference values for heat transfer rates used in thermal analysis for each curing method and type of formwork.

8.4 Protection against adverse actions

Concrete must be protected from adverse actions expected during curing, such as vibration, shock, loads, and seawater.

Commentary: Concrete that has not yet hardened sufficiently is susceptible to cracking and other damage caused by factors such as vibration, shock, and excessive load. Therefore, during the curing period, adequate consideration must be given to work such as the removal of Shoring and the temporary placement of materials and equipment to minimize the effects of vibration, shock, and loads on the concrete. Other adverse actions include heavy rainfall, overheating by curing heaters, and seawater during the curing period after placement. Based on a good understanding of the properties of concrete at a young material age, these adverse actions must be prevented from occurring; when this is not possible, the concrete must be protected from their effects.

Chapter 9 Joints

9.1 General

(1) Joints must have the structure indicated in design drawing documents and must be installed at the specified positions.
(2) When installing joints that are not indicated in the design drawing documents, the positions, orientation, and construction work method of the joints must be set out in construction work plan documents to avoid any impairment of the performance of the structural object.

Commentary: Regarding (1): Joints include construction joints installed out of necessity in concrete construction work, and masonry joints installed out of necessity in design for the purpose of controlling the occurrence of cracks and the position of their occurrence. These joints exert considerable effect on the strength, durability, appearance, and other aspects of structural objects. In general, the structure of joints and their positions of installation are set at the time of design, with consideration of the performance, workability, etc. of the structural object. Therefore, construction work must be performed with care, following construction work plan documents prepared based on the design drawing documents.

Regarding (2): Construction joints must be planned with detailed consideration of the amount of concrete to be supplied, the time to perform placement, and other construction work conditions. These conditions may not be indicated in design drawing documents. Construction work plans must be examined in detail so that expansion joints and Joints to control cracking specified in the design drawing documents, construction joints not specified in the design drawing documents, etc. are all compatible with actual construction work conditions.

9.2 Construction joints

9.2.1 General

(1) Construction joints must be planned with consideration of the structural form of the structural object, environmental conditions, construction work conditions, and other factors.
(2) In principle, construction joints are to be installed at positions where shear force is as small as possible, and construction joint surfaces are to be perpendicular to the direction in which the compressive force of structural members acts.
(3) The positions of joints must be determined with consideration of potential cracking caused by thermal stress, drying shrinkage, and other causes.
(4) In principle, construction joints should not be placed in marine, harbor, or other concrete structural objects that may be subjected to damage from external salinity. When the installation of construction joints in such structural objects is unavoidable, sufficient consideration must be given so that the construction joints do not affect durability.

Commentary: Regarding (1): The presence, position, orientation, structure, and construction method of construction joints are important factors in ensuring the quality of a structural object. For cases in which placement concrete all at once is difficult because of construction work conditions such as constraints on the amount of concrete supplied and the arrangement of concrete pumps, construction joints may be installed. In addition to the structural form and environmental conditions indicated in the design drawing documents, it is important to organize construction work conditions in advance and to take these into consideration in the planning of appropriate positions, orientations, structures, and other construction work methods for the construction joints.

In the planning of construction joints, the construction site, the form of the structural object, the division of blocks and lifts, the daily supply of concrete, transport plans, the amount of concrete that can be poured at once, and other factors must be considered.

Regarding (2): Construction joints tend to be weak points with respect to shear force and thus caution is required.

Regarding (3): When a large section of concrete is poured as one piece, large cracks may occur because of drying shrinkage and thermal stress, which can result in failure to satisfy required performance in a structural object that requires water-tightness. The locations of construction joints must be determined with consideration of not only reasons related to construction work but also the heat of hydration of the cement, thermal stress caused by fluctuations in ambient temperature, and cracking caused by drying shrinkage. It is also effective to use thermal stress analysis to examine the number and the timing of jointing of successive pours that are effective in controlling cracking.

Regarding (4): In marine and harbor concrete structural objects, construction joints should be avoided as external salt may permeate the joints and accelerate the corrosion of rebars.

9.2.2 Horizontal construction joints

(1) When jointing concrete, laitance on already-poured concrete surfaces, poor-quality concrete, loosened aggregate particles, etc. must be completely removed. The concrete surface must be roughened and then allowed to absorb sufficient water.

(2) Construction work must be performed on reverse-cast concrete so that construction joints are integrated, with consideration of concrete bleeding and subsidence.

Commentary: Regarding (1): Methods of treating construction joint surfaces of already-poured lower-layer concrete include removing a thin layer from the surface of the concrete using high-pressure air or water after the concrete has settled to expose the coarse aggregate particles. If the strength of the already-poured lower-layer concrete is not very high, thorough rinsing with high-pressure air and water even after hardening, or roughening the surface with a wire brush while spraying water, are also available treatment methods. When the lower-layer concrete has hardened and its strength is high, the most reliable methods are scraping the surface with a wire brush or sandblasting the surface and then rinsing it with water. Water on the upper surface of the already-poured lower-layer concrete should be removed before placement new concrete.

When laying mortar immediately before jointing new concrete, the water-to-cement ratio of the bed mortar should be lower than the water-to-cement ratio of the concrete to be used.

Regarding (2): In reverse-cast concrete, construction joints always form the bottom surface of already-poured concrete, and construction joint surfaces do not normally integrate because of bleed water and subsidence in the newly poured concrete jointed below. Therefore, in reverse-cast concrete, the integrity of construction joints is to be ensured by the construction work methods shown in **Figure C9.2.1**.

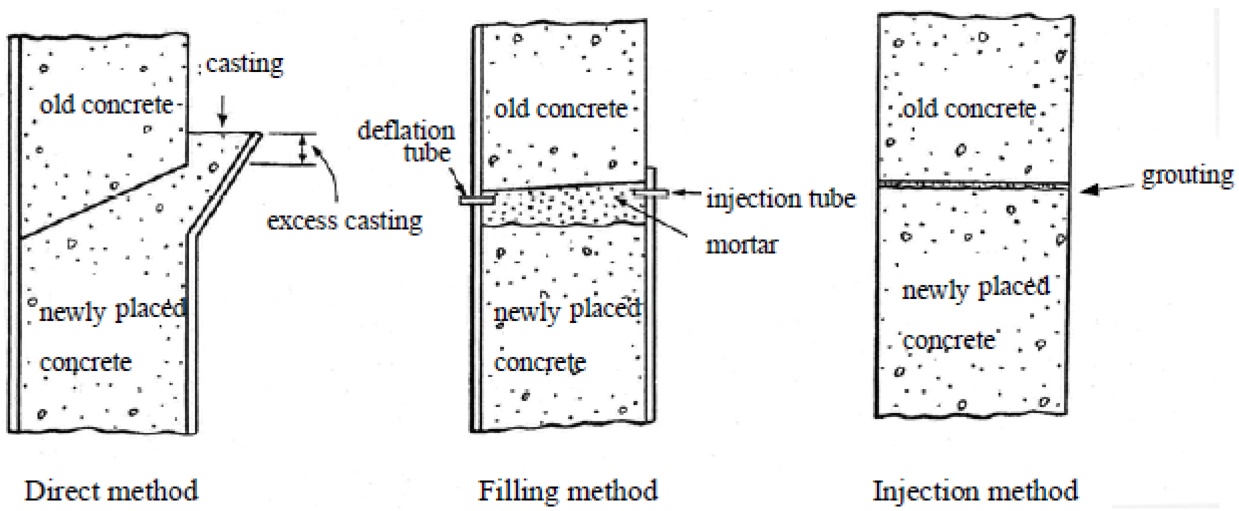


Figure C9.2.1 Construction joint of inversely placed concrete

9.2.3 Vertical construction joints

(1) When forming a vertical construction joint, the formwork on the construction joint surface must be firmly supported.

(2) The construction joint surface of concrete that has already been poured and hardened must be scraped with a wire brush or roughened by chipping, etc. and allowed to absorb sufficient water, after which the new concrete must be jointed.

(3) Placement and compaction must be performed so that the poured concrete spreads over and adheres to the construction joint surface.

(4) In principle, water sealing plates should be used for the vertical construction joints of concrete structural objects that require water-tightness.

Commentary: Regarding (1): The formwork used for vertical construction joints must be resistant to leaks of mortar and must be firmly supported.

Regarding (2): Just before jointing new concrete after the hardened concrete surface has been roughened, the surface can be coated with cement paste, mortar, epoxy resin for wet surfaces, or other materials to enhance integrity. Other construction work methods for roughening the vertical construction joint surface include using a wire mesh, etc. as shown in **Figure C9.2.2** on the formwork on the construction joint surface, or affixing a resin sheet with an uneven surface as shown in **Figure C9.2.3** to the formwork surface. The efficacy and workability of these methods must be confirmed prior to their use.

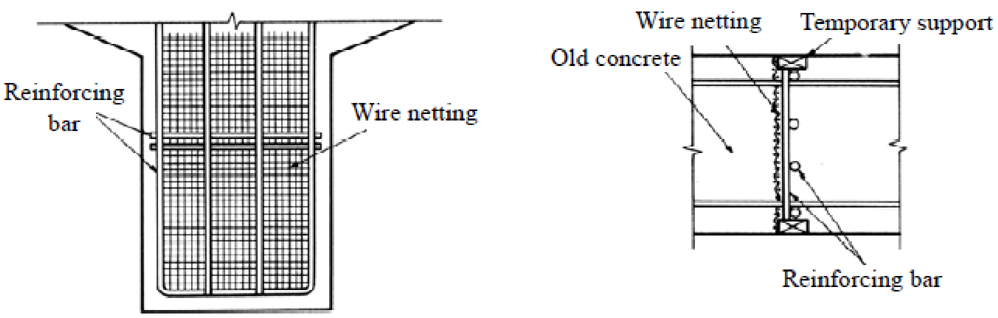


Figure C9.2.2 Vertical joint using wire net

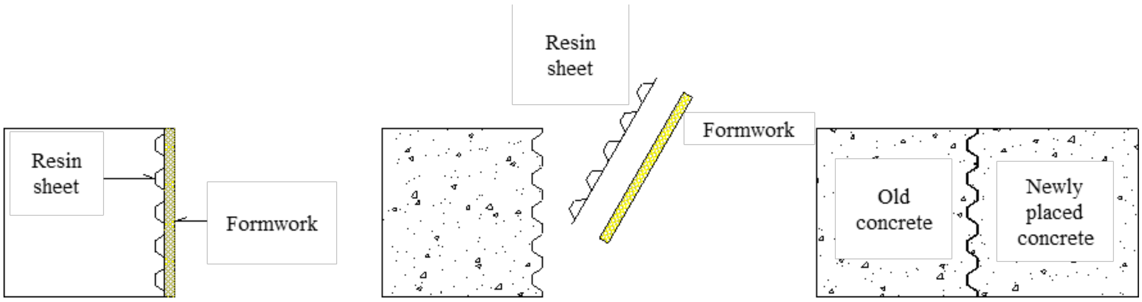


Figure C9.2.3 Construction joints using rugged shaped resin sheet

Regarding (4): In locations where water pressure acts, water leakage may occur even in construction joints that have been properly treated and achieving water-tightness can be difficult without the use of water sealing plates. Therefore, water sealing plates are in principle to be used for the vertical construction joints of structural objects that possess water-tightness.

When placement concrete around a water sealing plate, the plate may be prone to bending, sinking, or shifting position, or issues may readily occur in concrete filling. Construction work must be performed with care to prevent these defects from occurring.

Water sealing plates should also be installed on the horizontal construction joints of concrete structural objects that require water-tightness, as necessary.

9.2.4 Construction joints between the floor system and integrated columns or walls

(1) As standard practice, construction joints in floor systems are to be provided near the center of the spans of beams or slabs.

(2) When the center of the span of a beam intersects with a small beam, reinforcement against shear force must be provided by providing a construction joint for the beam at a distance of about twice the width of the small beam, and by running diagonal tensile rebar through the construction joint.

(3) As standard practice, the construction joints of columns or walls that are integrated with the floor system are to be provided near the boundary with the floor system.

(4) For haunches, concrete must be poured consecutively in conjunction with the floor system.

Commentary: Regarding (1) and (2): Construction joints are provided near the center of the span of beams or slabs for the reason that shear force is generally small in this area, compressive stress acts perpendicularly to vertical construction joints, and the yield strength of slabs or beams decreases little even when construction joints are provided. However, when the middle portion of a beam's span is intersected by a small beam, it is appropriate to provide the construction joint at a distance of about twice the width of the small beam, to avoid placing the construction joint at a point of sudden transition in stress. In this case, as large shear force acts on the construction joint, the construction joint should be reinforced with tensile rebar passing through the construction joint at an inclination of 45° (see **Figure C9.2.4**).

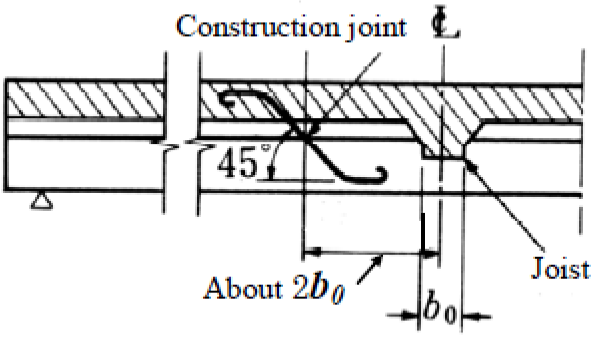


Figure C9.2.4 Reinforcement for construction joint

Regarding (3) and (4): Haunches installed on the underside of the floor system are supported directly by formwork, and thus are not able to shrink or to subside together with the concrete of columns or walls, or they are integrated with and move together with slabs. For such reasons, concrete must be poured continuously in conjunction with the floor system. Construction work is to be performed in the same manner in the case of a structural object with an overhanging portion.

9.2.5 Construction joints in arches

In principle, construction joints in an arch should be provided perpendicular to the arch axis.

Commentary: If a construction joint in an arch is not provided perpendicularly to the arch axis, shear force will act in line with the construction joint surface and will potentially create a structural vulnerability. When providing a vertical construction joint that is parallel to the arch axis, shear force will act on the construction joint surface because of unbalancing in the live load, etc. Therefore, construction joints are to be placed with thorough consideration of reinforcement methods, etc.

9.3 Masonry joints

9.3.1 General

(1) Masonry joints must have the structure indicated in design drawing documents and must be installed at the specified positions.
(2) When installing masonry joints that are not indicated in the design drawing documents, the appropriate structure and positions must be determined through discussion with the designer.

Commentary: Regarding (1): Masonry joints are installed out of necessity in design. Expansion joints, Joints to control cracking, and other types are available, and are indicated in the design drawing documents.

Regarding (2): When installing new masonry joints that are not indicated in the design drawing documents, the positions, orientations, spacing, and other matters concerning the joints must be determined through discussion with the designer, following consideration of the positional relationships between masonry joints and construction joints, the structural form, environmental conditions, constraints on construction work, and other factors, to avoid impairment of the performance of the structural object.

9.3.2 Expansion joints

Expansion joints must have the structure specified in the design drawing documents and must be installed at the specified positions.

Commentary: An expansion joint may completely insulate both opposing surfaces of the structural members or structural objects on both sides of the joint, or, depending on the type of structural object and the location of insulation, may insulate only the concrete and allow rebars to pass through. In both cases, however, the structure does not restrain the structural object or structural member on either side of the joint.

When there is a risk of sediment or other substances entering the gaps between expansion joints, expansion mortar joint fillers should be used. Expansion mortar joint materials include joint plates of asphalt, rubber foam, resin foam, or other materials, as well as sealing materials and filling materials. For expansion mortar joints in structural objects that require water-tightness, water sealing plates with an appropriate level of elasticity should be used. Water sealing plates include copper plates, stainless steel plates, polyvinyl chloride resin, rubber, and other materials. Considerations that apply to construction work on vertical construction joints must also be applied to water sealing plates.

When it is necessary to avoid level differences in expansion mortar joints, tenons or grooves should be created, or dowel bars should be used. When placement concrete in the area around dowel bars provided in expansion joints, construction work must be performed with care to prevent defects such as sinking or misalignment.

9.3.3 Joints to control cracking

Joints to control cracking must have the structure specified in the design drawing documents and must be installed at the specified positions.

Commentary: In a concrete structural object, deformation may be caused by factors other than external forces, such as drying shrinkage or temperature changes caused by heat of hydration of cement and external temperature. When such deformation is restrained, cracking may occur. Therefore, cross-sectional defects may be provided at specified intervals for the purpose of concentrating cracking at predetermined positions by providing joints to control cracking to artificially induce cracking. Consideration of joints to control cracking must be performed at the design stage. When these are not specified in the design drawing documents, joints of appropriate position and structure must be provided following discussion with the designer.

When installing joints to control cracking, the spacing of the masonry joints and the cross-sectional defect rate must be set, and adequate consideration must be given to matters including methods for preventing corrosion of rebars in the masonry joint portions, methods of maintaining the specified covering, and the selection of filling material to be used in the masonry joints.

Chapter 10 Reinforcement

10.1 General

Rebars must be processed to meet the dimensions and forms specified in the design drawing documents using appropriate methods that do not damage the materials. The rebars must be arranged in prescribed locations inside the formwork and must be solidly assembled.

Commentary: The main items to be considered in reinforcement work are summarized below.

(a) Overall plan

When performing reinforcement work, it is necessary to confirm the cover, rebar spacing, form of bends, anchorage, arrangement of joints and precaution reinforcement, and other items indicated in the design drawing documents, and to make preparations so that the rebars can be assembled according to the design. In plans for reinforcement work, the reinforcement work processes, necessary personnel, rebar storage location, rebar processing machinery, and other matters are to be thoroughly considered.

(b) Ordering, delivery, and storage of rebars

The type, diameter, length, quantity, etc. of rebars must be confirmed before ordering and after delivery of the rebars. The quality of rebars and other reinforcing materials is discussed in 3.6.1 "Rebar" in "Chapter 3 Materials." In the storage of rebars and other steel materials, measures must be implemented to protect rebars from corrosion, dirt, etc., and storage must employ means that do not harm the quality of rebars, using 3.6.1 "Rebars" as reference.

(c) Processing of rebars

In the processing of rebars, workers are to be instructed in details of the processing work following 10.3 to ensure against mistakes in the types of rebars to be processed, the inside radius of bends, etc. After processing, the types, numbers, and processing accuracy of the rebars are to be checked.

(d) Arrangement and assembly of rebars

In the arrangement and assembly of rebars, anchorage using spacers and binding wires, fitting with formwork, specified positioning/center-to-center spacing, effective height, securing of cover and development length, appropriate jointing methods, end surface treatment of rebars through pressure welding, and other matters are to be confirmed, following **10.4**.

10.2 Preparation

(1) It must be confirmed in advance that the assembly of rebars is possible using the forms and dimensions indicated in the design drawing documents.

(2) It must be confirmed that necessary space has been secured for placement and compaction work.

Commentary: Regarding (1): Correct understanding of the design drawing documents, with both engineers and technicians involved in reinforcement work thoroughly checking the documents before starting work, is vital.

Bar arrangement drawings depict rebars using a combination of dots and lines. In the rebar assembly stage, however, it may be difficult to perform assembly according to the bar arrangement drawing because of factors including the diameter of the rebars, the presence of ribs and knots on the rebars, the presence of lap joints and anchors, and the precision of the rebars and formwork. In addition, because the arrangement of rebars is complicated at structural member junctions, such as the junctions of beams and columns, of below-ground beams and columns, and of beams/columns and slabs, it may not be possible to assess the state of rebar arrangement from the rebar arrangement drawings for individual structural members. Therefore, it is necessary to redraw the drawings to take into account the actual conditions for the rebar arrangement and to confirm in advance whether assembly of the rebars is possible, based on the assembly procedures for the rebars.

Regarding (2): For structural members in which the height of formwork is large, a flexible hose, etc. at the end of the vertical chute or concrete pump must be inserted between rebars to control the free-fall height of the concrete. In addition, when compacting concrete, compacting must be performed by a specified method with a rod-shaped vibrator inserted between the rebars. At the preparatory stage of reinforcement work, it is vital to anticipate the details of concrete placement and compaction work and to confirm the details in advance. As an example, the position of insertion of the hose or rod-shaped vibrator must be made clear in the drawings, and space for the insertion must be confirmed prior to assembly of the rebars.

10.3 Processing of rebars

(1) Rebars must be processed to match the forms and dimensions indicated in the design drawing documents, using methods that avoid damage to the materials.

(2) When the bending radius of rebar is not indicated in the design drawing documents, the rebar must not be bent to a radius smaller than the inside radius of the bend.

(3) In principle, rebars should be processed at normal temperature.

(4) Epoxy resin-coated rebars must be processed using a method appropriate to their properties.

(5) In principle, bent rebars should not be straightened again.

Commentary: Regarding (1) and (2): When bending welded rebars, it is advisable to avoid the welded part and to perform the bending at a distance from the welded part equal to not less than 10 times the rebar diameter.

Regarding (4): Processing and assembly of epoxy resin-coated rebars must be performed using an appropriate method that does not damage the coating.

Regarding (5): In principle, bent rebars should not be straightened again as this may damage the material. When temporarily bending rebars at construction joints or other locations out of necessity in construction work and then unbending the rebars to predetermined positions, the bending and unbending must be performed with the largest radius possible or performed with the rebars heated to approximately 900°C to 1,000°C.

10.4 Assembly of rebars

10.4.1 General

(1) Before assembly, rebars must be cleaned to remove loose rust and other matter that could hinder the adhesion between the rebars and the concrete.
(2) Rebars must be arranged in the correct position and must be solidly assembled to prevent movement of the rebars when the concrete is poured. Steel assembly materials should be used as necessary. The intersections of rebars must be bound at key points using annealed steel wire with a diameter of 0.8 mm or more or using appropriate clips. The annealed steel wire or clips must not remain inside cover.
(3) To properly maintain rebar cover, spacers made of a material suitable for the location of use must be arranged at the necessary intervals.
(4) In principle, spacers in contact with formwork should be made of mortar or concrete.
(5) When any part of assembled rebars is to be exposed to air for a long period, the rebars should in principle be rust-proofed or protected by sheets or other means.
(6) When a long time has passed since the assembly of rebars, the surface of the rebars must be cleaned again before placement the concrete to remove loose rust or other matter that could hinder adhesion.

Commentary: Regarding (2): Deviations in the positions of rebars from the positions indicated in the design drawing documents will affect the yield strength of the reinforced concrete structural members. If cover is insufficient, the durability of the structural object will be impaired. Therefore, rebars must be anchored to prevent them from moving from their specified positions. In some design drawing documents, the positions of rebars are indicated only by the center lines of the rebar, with cover not properly indicated. To accurately secure cover, effective depth, or space between rebars, a rebar assembly drawing should be produced with the precision of the rebar arrangement confirmed in advance, taking into consideration factors including the outer diameter and bending radius of the rebars and the order of assembly.

In order to anchor the relative positions of rebars, the intersections of the rebars are commonly bound with annealed steel wires with a diameter of at least 0.8 mm. Various types of clips may be used in addition to steel wire to anchor the intersections of rebars, or spot welding may be performed. However, spot welding presents the risk of damaging the rebar material because of localized heating and may significantly reduce fatigue strength in particular.

If the annealed steel wire used for binding is left inside the cover, the steel wire may corrode and induce corrosion of the rebars. Therefore, the wire should be bent inward toward the rebars.

Regarding (3) and (4): Spacers made of materials suitable to their location of use should be appropriately arranged to hold rebars in their proper positions and to secure the required cover. Spacers made of mortar, concrete, steel, plastic, ceramic, and other materials are available. The appropriate spacers should be selected according to the location and environment in which they will be used. Spacers installed at the bottom of formwork must directly support the load of the rebars. This increases the area in contact with the formwork and causes spacers to become exposed at the surface of the concrete on the underside of upper deck slabs, etc. Therefore, these construction work standards suggest that spacers made of mortar or concrete are to be used in principle, with consideration of factors including strength, durability, and appearance. Spacers made of mortar or concrete must possess quality equal to or higher than that of the main body concrete. In addition, appropriate dimensions must be selected for the spacers to ensure minimal cover and to restrict the positions of rebars to within the margin of error. Generally, the number of spacers should be four or more per square meter for beams, decks, etc., and two to four per square meter for webs, walls, and columns. The spacers should be arranged at intervals as even as possible so that significant localized deflection of rebars does not occur at any location. As an example, when arranging four spacers per square meter, these should be arranged in a staggered pattern at 50 cm intervals.

10.4.2 Rebar joints

(1) The positions of rebar joints and the jointing method must follow the design drawing documents. When it is necessary to provide rebar joints not indicated in the design drawing documents, the positions of the joints and the jointing method are to follow **"Design."**

(2) As a general principle for rebar lap joints, a predetermined length should be superposed and binding should be performed at several locations using annealed steel wire with diameter of at least 0.8 mm.

(3) When lap joints, gas pressure welded joints, welded joints, or mechanical joints are used for rebar joints, appropriate methods are to be used.

(4) Exposed rebars protruding from the structural object for the purpose of splicing must be protected from damage, corrosion, etc.

Commentary: Regarding (1): The jointing method must be selected according to factors including the rebar type, diameter, stress state, joint positions, and performance demanded of the joints. Therefore, in the design stage, the positions of joints and jointing method are stipulated in the design drawing documents, taking the preceding into consideration. The positions of rebar joints and the jointing method must generally follow the design drawing documents. In the construction work stage, it may be necessary to provide rebar joints not indicated in the design drawing documents. In such cases, the positions of the joints and the jointing method are to be decided through discussion between the builder and designer, following **"Design."**

Regarding (2): The superposed portion is to be firmly bound using annealed steel wire. Because the adhesive strength between the concrete and the rebar declines and the strength of the joint decreases, the winding length of the annealed steel wire should be suitable to allow secure binding, without being longer than necessary.

Regarding (4): Methods to prevent corrosion of rebars include coating with cement paste and wrapping in polymer material film. In any case, when performing splicing, anything that would impede adhesion with concrete must be completely removed. Covering the spliced portion of rebars with airtight caps as close as possible to the rebars to prevent moisture and air from entering and to provide protection from direct contact with rain is also effective in preventing corrosion in rebars.

10.4.3 Installation of pre-assembled rebars

(1) Pre-assembled rebars must be accurately installed in their specified position within the formwork.

(2) The connecting of rebars in each pre-assembled unit must follow methods capable of yielding the specified joint performance.

Commentary: Regarding (1): "Pre-assembled rebars" refers to rebars that have been assembled in advance. When pre-assembled rebars are installed in a specified position, an appropriate hoisting method must be set with thorough consideration of safety. This may include the arrangement of reinforcement steel for hoisting and suspension frames as necessary, to prevent residual adverse effects caused by hoisting such as disturbance or excessive deformation of the form and dimensions of the pre-assembled rebars. To properly install the pre-assembled rebars in the specified position, it is advisable to place guideline marks at necessary positions on the upper part of formwork, etc.

Regarding (2): Lap joints must be arranged so that the rebars of superposed portions are in contact with each other. However, when it is difficult to arrange rebars in contact with each other as in an underground continuous wall, testing or other means must be used to confirm in advance that the jointing method to be adopted yields the specified joint performance.

Chapter 11 Formwork and Shoring

11.1 General

(1) Formwork and shoring must be designed and constructed following prepared construction work plan documents so that the concrete structural object will have the form and dimensions indicated in the design drawing documents.

(2) The formwork and shoring must be designed and constructed so as to possess the strength and rigidity required to withstand the load specified in 11.2, and so that deviations do not occur in the form and dimensions of the structural object.

(3) When assembling formwork and shoring the precision of assembly must be confirmed before the concrete is poured to ensure that required precision is met.

Commentary: In the construction of concrete structural objects, the time from concrete placement to hardening is short. Therefore, it is generally not necessary to build an excessive safety factor into the design of formwork and shoring. However, this must not in any way have adverse effects on the structural object or compromise safety.

11.2 Load

11.2.1 General

Formwork and shoring must be designed so as to ensure safety with respect to the loads shown in **11.2.2** to **11.2.5**, taking into consideration the type, scale, and importance of the structural object, the construction work conditions, and the environmental conditions. In the design of shoring, deformation as well as strength should be taken into consideration.

Commentary: **11.2.2** to **11.2.5** indicate the minimum values to be considered for each acting load. These values must be increased appropriately according to conditions concerning the structural object.

11.2.2 Vertical loads

(1) Mass and shock loads from formwork, shoring, concrete, rebars, workers, construction machinery and equipment, temporary installations, *etc*. must be taken into account as loads in the vertical direction.

(2) The standard calculated unit weight of concrete used in calculation of formwork and shoring is 23.5 kN/m^3. In the case of reinforced concrete, the addition of 1.5 kN/m^3 as the unit weight of rebar is set as a standard for general structural objects.

(3) In principle, the unit volume mass of concrete made wholly or partly with aggregate of a density that differs significantly different from that of normal aggregate is to be confirmed through trial mixing and set appropriately.

(4) When heating materials, the water or aggregates should be heated. However, the cement itself should not be directly heated under any circumstance. The heating of aggregates must employ methods by which temperature is uniform and drying does not occur.

Commentary: Regarding (1) and (2): In a general structural object, vertical load may be calculated based on the values shown in the text. If the amount of steel, *etc*. is greater than usual or if the actual mass is known, calculations are to take these into consideration. In **Commentary Table 6.4.1** in **6.4.2 Dead load** in **Chapter 6 Actions** of **Design: General Requirements**, the unit weight of hardened concrete is 22.5 to 23.0 kN/m^3 and the unit weight of reinforced concrete is 24.0 to 24.5 kN/m^3. However, safer values were set as the standard values for use in calculations for formwork and shoring.

Regarding (4): Setting load values for working and shock loads is generally difficult. Therefore, for convenience of calculation, the replacement of these with a uniformly distributed load set to 2.50 kN/m^2 or more was made standard practice.

11.2.3 Horizontal loads

(1) Horizontal loading that must be considered includes loading caused by inclination of formwork, shock, vibration during work, normally expected eccentric loads, and construction work error, as well as wind pressure, running water pressure, and other loading as necessary.

(2) In principle, acting horizontal loading is to be taken into consideration in design. However, if the acting horizontal loading is smaller than the reference horizontal loading, safety is to be examined using the reference horizontal loading.

(3) Regarding the reference horizontal loading, in general, it is assumed that horizontal loading acts on the top portion of the shoring, and that this loading is equivalent to 5% of the design vertical loading when the formwork is nearly horizontal and the shoring is custom-assembled on-site using pipe supports, single pipe support columns, assembled steel columns, support beams, *etc*.; or is equivalent to 2.5% of the design vertical loading when shoring is assembled through steel pipe frame columns with shop fabrication precision. In the case of a formwork for a retaining wall or other wall, a lateral load of no less than 500 N/m^2 is to be considered to act on the side surface of the formwork.

Commentary: Regarding (1): The load acting on shoring in the horizontal direction is considered to act in the horizontal direction at the top surface or side surface of the shoring.

Regarding (3): This item is based on experience indicating that collapse-related accidents often occur because of insufficient consideration of horizontal loading. In general cases, horizontal loading may be determined based on values in this text. However, it should be examined carefully with consideration of the importance of the structural object, construction work conditions, the construction period, and other factors.

11.2.4 Lateral pressure of concrete

(1) Formwork must be designed with consideration of lateral pressure from fresh concrete.

(2) The lateral pressure of concrete varies with structural conditions, concrete conditions, and construction work conditions. Therefore, its value must be determined with consideration of the effects of these factors.

Commentary: Regarding (1) and (2): Main factors affecting lateral pressure are as follows:

① Structural conditions: Cross-sectional dimensions of structural members, amount of rebars, *etc*.

② Concrete conditions: Materials used, mix, slump, slump retention time, settling time, concrete temperature, *etc*.

③ Construction work conditions: Placement speed, placement height, compaction method, whether re-vibration is to be performed, *etc*.

When factors that increase lateral pressure are included, the value of lateral pressure used in calculations must be increased appropriately.

The lateral pressure used in formwork design must be determined appropriately under the responsibility of the builder, taking into consideration the speed of construction work, economy, safety, and other factors required by the builder.

When performing design with a high margin of safety, when a method exists of deriving the lateral pressure acting on formwork by treating it as a liquid pressure, the concrete can be regarded as a liquid, and the liquid pressure can be derived by multiplying the unit volume mass of the concrete by the gravitational acceleration and placement height, as in the following equation:

$$p_w = W_C H \qquad \text{(C11.2.1)}$$

where, p_w: liquid pressure (kN/m^2)

W_C: unit weight of concrete (23.5 kN/m^3)

H: placement height of fresh concrete (m)

The lateral pressure of high-fluidity concrete or of high-strength concrete with high fluidity often displays a lateral pressure distribution close to a liquid pressure. Therefore, in principle, **"Chapter 2 High-Fluidity Concrete"** of **"Materials and Constructions: Special Concrete"** and Chapter 4 specify design on the assumption that there is acting liquid pressure. When the unit weight of the concrete has been confirmed, its value may be substituted into the equation after taking variations and other factors into consideration.

The following equation has long been used for the practical calculation of guidelines for lateral pressure when concrete that is made with ordinary Portland cement and that has a slump of no more than approximately 10 cm is poured into a formwork and is compacted using a rod-shaped vibrator. **Figure C11.2.1** shows the relationship between lateral pressure, placement speed, and concrete temperature derived using this equation.

(a) For columns

$$p = \frac{W_c}{3}\left(1 + \frac{100R}{T+20}\right) \le 150 \qquad \text{(C11.2.2)}$$

(b) For walls

(I) $R \ge 2$m/h

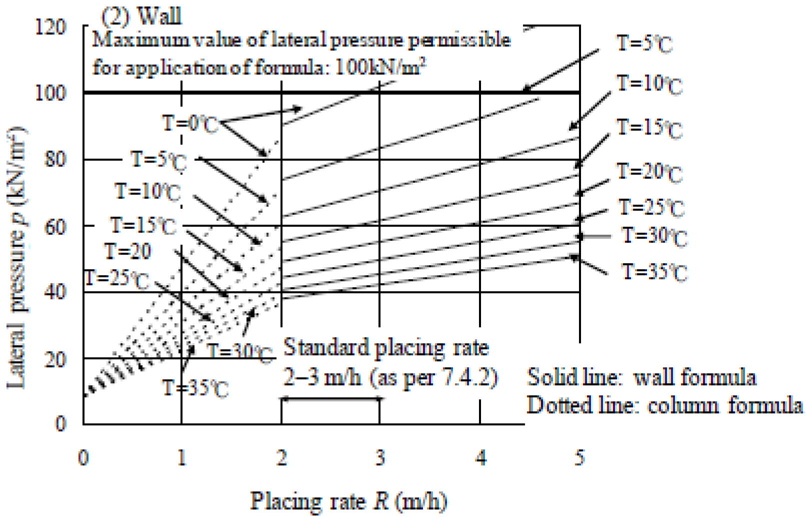
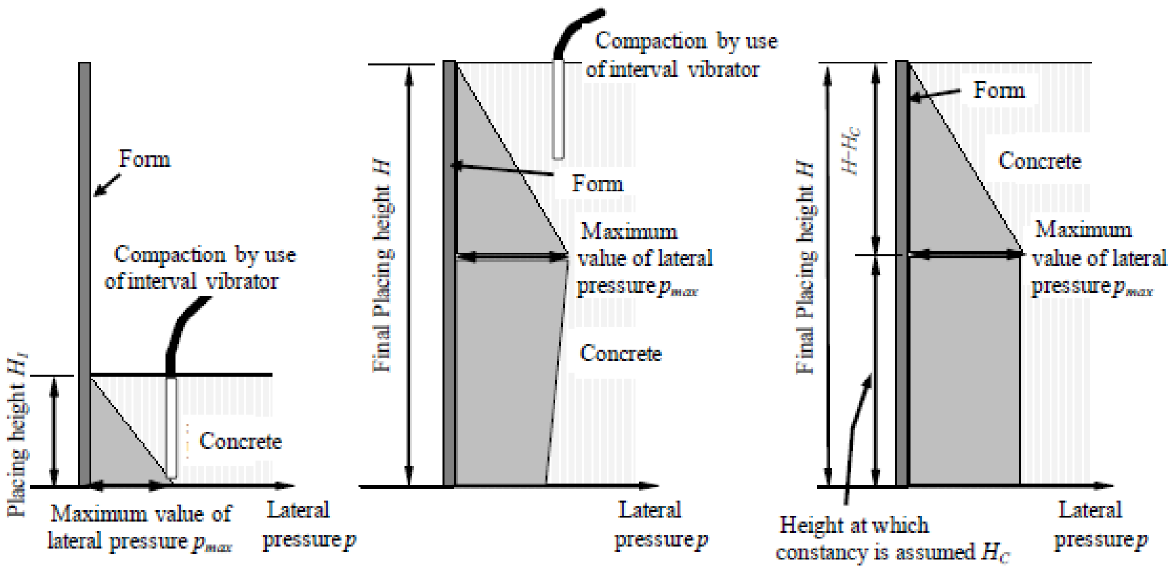


Figure C11.2.1 Lateral pressure of concrete with a slump of about 10 cm or less (wall)



(a) (b) (c)

Figure C11.2.1 Distribution of lateral pressure of concrete with a slump of about 10 cm or less

(a) At initial stage of placement (b) Completion of placement (c) Calculated at design stage

$$p = \frac{W_C}{3}\left(1 + \frac{150 + 30R}{T + 20}\right) \leq 100 \qquad \text{(C11.2.3)}$$

(ii) When R < 2m/h, the calculation for columns may be used.

Where, p: lateral pressure (kN/m²); however, $p > p_w$ is to be used when $p = p_w$ has been calculated

p_w: liquid pressure (kN/m²)

W_C: unit weight of concrete (kN/m³)

R: placement rate (m/h)

T: concrete temperature inside formworks (°C)

Taking into account the experimental conditions under which the calculation equation was derived, the upper limit of the equation is 150 kN/m² for columns and 100 kN/m² for walls. This equation does not include the final placement height. Therefore, the lateral pressure derived from the equation may take a value greater than that of liquid pressure, depending on placement speed, concrete temperature, and final placement height. However, the maximum value of lateral pressure is generally less than or equal to that of liquid pressure; therefore, when a value greater than liquid pressure is calculated, liquid pressure is to be used. **Figure C11.2.2** shows change over time in lateral pressure distribution in concrete and assumptions used in the design calculation. As shown in Equations (C11.2.2) and (C11.2.3), lateral pressure increases as placement height increases. However, when concrete with a low slump is poured in layers with the height of one layer set to 0.4 to 0.5 m, lateral pressure often ceases to increase at a level lower than a given height because of the progress of settling, adhesion to rebars, or other reasons. Taking these lateral pressure changes and safety into account in formwork design, a distribution by which lateral pressure becomes constant below a certain height is assumed, and Equation (C11.2.2) or (C11.2.3) is used to derive the maximum value of lateral pressure.

11.2.5 Special loads

When the effects of special loads expected in construction work are not negligible, these must be considered in the design of formwork and shoring.

Commentary: "Special load" here refers to a load other than normal concrete construction work that is expected to act on formwork and shoring during construction. When the effects of such a special load are not negligible, they must be given due consideration in the design of formwork and shoring.

11.3 Materials

(1) Cement must be selected appropriately with consideration of strength development properties.
(2) Frozen aggregates or aggregates mixed with ice and snow must not be used while in that state.
(3) Chemical admixtures must be chosen to ensure stable concrete quality even when used under low temperatures.
(4) When heating materials, the water or aggregates should be heated. However, the cement itself should not be directly heated under any circumstance. The heating of aggregates must employ methods by which temperature is uniform and drying does not occur.

Regarding (1): The materials used for formwork and shoring are generally reused. Therefore, they are prone to damage, deformation, and corrosion, while also being subjected to relatively large loads. Selection conditions for the materials differ by type of structural object, the number of times the materials are used, and the importance of the formwork or shoring based on their location of use. Therefore, the most suitable materials should be selected, with full consideration of the factors indicated here. **Table C11.3.1** can be used as a guideline for the number of times formwork materials are used.

Table C11.3.1 Standard number of times formwork materials are used

Type of formwork	Number of times
Wood panel	3～4
Plywood	4～8
Steel panel	About 30
Aluminum form	About 50
Plastic form	About 20

In cold weather concrete or mass concrete construction work, it is advisable in some cases to use formwork that exhibit good heat retention with low thermal conductivity.

Regarding (2): Fastening fittings are for the purpose of keeping end-plates from opening beyond a specified interval under lateral pressure from concrete. Annealed steel wire may stretch or break if used as a fastening fitting, and thus must not be used in particularly important structural objects.

Tie rod-type fastening fittings with separators and form ties are often used.

Bleed water accumulates on the underside of plastic cones, separators used as formwork fastening fittings, *etc*., potentially creating bleeding channels after the concrete has hardened and causing water leakage. Therefore, water sealing plates for separator use or water sealing rings should be used in structural objects for which water-tightness or measures against salt damage are required.

11.4 Design of formwork

(1) Appropriate fastening fittings must be selected to ensure that formwork accurately maintains its form and position under acting loads.

(2) Formwork must have a structure that enables easy assembly and removal work and that does not subject concrete or other items to vibration, shock, *etc*. during removal. The structure must be such that the joints of end-plates or panels run perpendicular or parallel to the axes of structural members wherever possible, without leakage of mortar.

(3) Even when not specified, a structure that enables chamfering of the edges of the concrete is set as the standard.

(4) When necessary, temporary openings must be provided at appropriate positions, with consideration of cleaning and inspection of formwork and placement of concrete.

Commentary: Regarding (3): The application of appropriate chamfering materials to the edges of formwork and the chamfering of concrete edges are effective in preventing damage to concrete edges caused by impacts during the removal of formwork or after the completion of construction. This chamfering is also effective in reducing damage caused by weather action, physical action, *etc*. Chamfering may also be performed at the locations of construction joints for reasons of aesthetics and workability.

Regarding (4): When blockages inside formwork present obstacles to cleaning and inspection before the placement of concrete, when the height of the formwork is high and work cannot be performed at a height below the specified placement height or compaction work height, when obstacles are present in the formwork and an internal vibrator cannot be inserted, or in similar cases, temporary openings must be provided at appropriate positions. The stage for closing these openings is also to be considered in advance. The structure is to be such that leaks of paste from gaps or damage caused by lateral pressure from the concrete do not occur, and the closing of the openings in a short time should be enabled so that the work of placement and compacting can be performed smoothly when such openings are used.

11.5 Design of shoring

(1) An appropriate form of shoring must be selected so that the supported load will be securely transmitted to the foundation by an appropriate method.

(2) The shoring must be of a structure that makes assembly and removal convenient, and its joints and connection parts must securely transmit load.

(3) The foundation of the shoring must not cause excessive or uneven settlement.

(4) Appropriate sinking allowance should be worked into the design of the shoring, taking into consideration settlement and deformation during and after construction work caused by the concrete's self-weight.

Commentary: Regarding (1): For shoring to possess sufficient strength against loading in the vertical direction and safety against buckling, measures such as fixing of support columns must be taken, using sufficient sway braces, diagonal braces, *etc*. as necessary. To deal with changes in the load distribution of support pillars caused by unequal subsidence of the foundation, *etc*., measures such as the use of beams, *etc*. to distribute the load among support columns should generally be taken.

For loads in the horizontal direction, measures such as fixing both ends of beams at the upper part of shoring to existing structural objects or other supporting structures, or providing resistance using sway braces, diagonal braces, *etc*. should be taken. When formwork is not horizontal, care must be taken to prevent harmful deformation of shoring caused by pressure from the concrete.

Regarding (2): Jacks, wedges, *etc*. should be used to enable easy and safe removal without causing shock to the structural object.

Measures such as binding with butt joints or bayonet joints at the joints of support columns, or with bolts, clamps, and other metal fittings at the connection parts and intersection parts of steel materials, are effective in transmitting the load of joints and connection parts.

When beams are of large height, connections should be installed between beams to prevent them from falling over.

Regarding (3): To avoid subsidence of the foundation, measures such as distributing the load over the ground in the case of soft ground or appropriately reinforcing the foundation should be taken.

Regarding (4): In some cases, it is also necessary to consider creep and other deflection caused by the weight of the concrete after completion of the structural object. In general, the amount of sinking allowance must be indicated in design drawing documents. Possible causes of subsidence and deformation of shoring include subsidence of the foundation, compression deformation and deflection of shoring members, the fitting of gaps between joints and connection parts in the shoring, and encroachment of contact surfaces. The amount of such subsidence and deformation are to be estimated using calculation, past results, simple on-site measurements performed in advance, or other appropriate methods, and are to be taken into account during construction as necessary.

In general, the fitting of gaps between joints and connection parts of shoring, the encroachment of contact surfaces, *etc*. are approximately 1 to 2 mm at each point.

11.6 Construction of formwork

(1) In the construction of formwork, fabrication and assembly must be performed using specified formwork materials and achieving a specified range of precision. Also, fastening fittings must not be left on concrete surfaces after formwork has been removed..

(2) In principle, the inner surfaces of end-plates are to be coated with a peeling agent.

(3) Before and during the placement of concrete, formwork dimensions and the presence or absence of defects must be confirmed

Commentary: Regarding (1): Separators used as fastening fittings carry the risk of becoming permeation channels for water, corroding and causing stains on the concrete surface, or causing cracks in the concrete. Therefore, the holes remaining after removing the plastic cones must be filled with high-quality mortar or other material. Particularly in structural objects that require water-tightness, this work must be performed with care so as not to create weak points.

Regarding (2): Many types of release agents are available on the market, varying by application (wooden formworks, steel formworks, or both) and by the main components of the agents. Method of use, amount applied, number of times used, and other aspects vary greatly by type of peeling agent. Depending on the type and the method of use, the peeling agent may wash away and lose efficacy because of rain or rinsing of formworks, joints and other parts may become contaminated, or the peeling agent may become mixed into concrete during placement. Therefore, it is important to check the properties and the method of use of a peeling agent prior to use.

Regarding (3): It is important to deal with any discovered gaps through measures such as the use of gap tape.

11.7 Construction of shoring

(1) Shoring must be constructed so as to possess sufficient strength and stability.

(2) During the placement of concrete, the lack of looseness, deformation, or other abnormalities in the shoring must be confirmed.

Commentary: Regarding (1): The ground at the foundation must be leveled before assembly of the shoring to ensure that the shoring has sufficient strength and stability. Appropriate reinforcement must be performed as necessary to prevent uneven subsidence, *etc.* to achieve the required load bearing capacity. When using backfill soil for support, ample rolling compaction must be performed in advance. When there is a possibility that the base of shoring will be rinsed with water, particular care must be taken with the treatment of the water. It is important that assembly of the shoring be performed with consideration of inclination, height, alignment, and other aspects to ensure that the shoring possesses adequate strength and stability.

Gaps and looseness must be prevented from occurring at joints and at the connecting parts and intersecting parts of structural members. Longitudinal axes must be aligned, particularly for joints.

Regarding (2): Even if the allowable margins of error for individual parts are generally acceptable values, in locations that are subjected to considerable effects from the accumulation of these errors, the cumulative allowable margin of error must be set with consideration of said effects.

11.8 Removal of formwork and shoring

(1) Formwork and shoring must not be removed until the concrete has achieved the strength required to support its own weight and to support loads that will be added during construction work.

(2) The time and the order of removal of formwork and shoring must be appropriately determined with consideration of the strength of concrete, the type and importance of the structural object, the type and size of structural members, the load supported by structural members, temperature, weather, ventilation, and other factors.

(3) When subjecting a structural object to loads immediately after the removal of formwork and shoring, the structural object must be prevented from harmful cracking or other damage, taking into consideration the strength of the concrete, the type of structural object, the type and size of the acting load, and other factors.

Commentary: Regarding (1): Removal of formwork and shoring should be performed as gently as possible so as not to damage the structural object.

To judge the time at which concrete achieves its required strength, the compressive strength of concrete specimens cured under the same conditions as the concrete poured into the structural object should be used. However, because a concrete specimen is more susceptible to the effects of atmospheric temperature and drying than the concrete in structural objects, it is advisable to make the judgment based on curing methods with these factors taken into account.

Regarding (2): There are many cases of accidents caused by removal of formwork and shoring at an incorrect time. Therefore, the time for removal of formwork and shoring must be given proper consideration.

Table C11.8.1 shows reference values for the compressive strength of concrete that is required for the removal of formwork and shoring. When removing formwork and shoring that mainly provide support for the weight of concrete, as at the bottom of slabs and beams, flexural stress immediately occurs in the concrete and creep deformation occurs at a young material age. For this reason, the concrete must have sufficient strength. In particular, in structural objects for which dead load composes a large proportion of total design load, care must be taken as nearly all of the load considered in the design will be applied when formwork and shoring are removed. Even after the compressive strength indicated in **Table C11.8.1** has been attained and the formwork has

Table C11.8.1 Reference values of compressive strength of concrete required for the removal of formwork and shoring

Type and position of surface	Example	Compressive strength (N/mm^2)
Vertical or almost vertical surfaces of thick member Upper surfaces of inclined members Outside surfaces of small arch structures	Side of footings	3.5
Vertical or almost vertical surfaces of thin member Lower surfaces of members inclined at 45° or more Inside surfaces of small arch structures	Side of columns wall and beams	5.0
Slabs and beams of bridges and buildings Lower surfaces of members inclined at 45° or less	Bottoms of slabs and beams Inside surfaces of arch structures	14.0

been removed, moist curing must be continued during the period shown in **Table 8.2.1** of **"8.2 Moist curing"** in **"Chapter 8 Curing."**

Regarding the order of removal of formwork and shoring, relatively unloaded parts are generally removed first, followed by remaining important parts.

11.9 Special formwork and shoring

11.9.1 General

When using slip forms, buried forms, permeable forms, water-absorbing formwork, and other special formwork, or when using mobile shoring, mobile work vehicles, or other special shoring, special precautions specific to these must be observed.

Commentary: Special formwork includes slip forms that are slid vertically for high bridge piers, water tanks, *etc*.; formwork that is slid horizontally or diagonally for water channels, *etc*.; and tunnel formwork used in the lining of tunnels. Special shoring includes mobile shoring used in elevated bridges, shoring using trusses, and mobile work vehicles for cantilevered erection of arch bridges. In recent years, in particular in environments that demand durability, buried formwork that is manufactured from polymer cement mortar, polymer impregnated concrete, fiber reinforced concrete, and other materials have been used without modification as a part of structural members, as well as water-absorbing formwork and permeable formwork that reduce the water-to-cement ratio of surface-layer concrete and reduce surface bubbles and pockmarks to enhance the integrity of the surface portion of the structural object.

Chapter 12 Cold Weather Concreting

12.1 General

(1) When the average daily air temperature is expected to be 4 °C or lower, cold weather concreting is necessary.

(2) In cold weather concreting, appropriate measures must be taken with respect to materials, concrete mix, mixing, transport, placement, curing, formworks, shoring, and other considerations to prevent concrete from freezing and to obtain the required quality even under cold conditions.

Commentary: Regarding (1): Under weather conditions in which the average daily air temperature is 4 °C or lower, setting and hardening reactions are significantly slowed, creating a risk of concrete freezing not only at night or early in the morning but even during daytime, which necessitates the consideration of cold weather concreting. Even when concrete does not freeze, exposure to low temperatures of approximately 5 °C or lower considerably delays setting and hardening reactions, meaning structural objects that are subjected to construction loads at an early stage will be prone to problems such as cracks and residual deformation. The approximate freezing temperature of concrete is considered to fall between −2.0 and −0.5 °C, varying slightly with the water-to-cement ratio and the type and amount of admixture.

Regarding (2): A key consideration in cold weather concreting is measures to prevent the concrete from freezing and to ensure that the required quality is not impaired even in cold weather. The following matters are of particular importance:

① The concrete is not allowed to freeze in the initial stage of setting and hardening.

② The concrete is provided with sufficient resistance to expected freezing and thawing actions after curing.

③ The concrete is provided with sufficient strength with respect to expected loads at each stage of construction.

12.2 Materials

(1) Cement must be selected appropriately with consideration of strength development properties.

(2) Frozen aggregates or aggregates mixed with ice and snow must not be used while in that state.

(3) Chemical admixtures must be chosen to ensure stable concrete quality even when used under low temperatures.

(4) When heating materials, the water or aggregates should be heated. However, the cement itself should not be directly heated under any circumstance. The heating of aggregates must employ methods by which temperature is uniform and drying does not occur.

Commentary: Regarding (1): Cold weather concrete differs from ordinary concrete in the compressive strength that is required after curing to prevent freezing damage, *etc.* when the concrete is subjected to initial freezing damage or frequent freezing and thawing actions. The use of high-early-strength Portland cement or ordinary

Portland cement, which can quickly yield required compressive strength, is advisable for cold weather concrete. When using Class-B mixed cement or other cements for reasons such as environmental considerations or inhibition of thermal cracking, it is necessary to implement curing or other measures while paying heed to effects on factors including resistance to deterioration and slowed development of strength at the initial material age of concrete under low temperatures.

Regarding (2): Using frozen aggregates or aggregates mixed with ice and snow may lead to a decline in the temperature of the mixed concrete or fluctuations in unit water content. Therefore, it is advisable to store aggregates in facilities that will prevent freezing or contamination with ice and snow.

Regarding (3): Chemical admixtures used for cold weather concrete should be selected from those that satisfy JIS A 6204 "Chemical admixtures for concrete" and which yield concrete of stable quality even at low temperatures. Reducing the water-to-cement ratio using high-performance water-reducing AE agents is effective in enhancing resistance to freezing. However, in concrete made with high-performance water-reducing AE agents, slump during mixing is lower when the concrete temperature is low than when it is high. Because slump can increase with the passage of time, in using this concrete it is important to pay attention to changes in reduced-water performance and workability at low temperatures.

Regarding (4): In general, no such concern exists if the temperature of the mixture of water and aggregate is kept below 40 °C. As a method for heating aggregates, the use of steam is relatively easy in terms of management.

12.3 Mix proportions

To prevent initial freezing damage, the unit water content must be kept as small as possible within the range that maintains required workability.

Commentary: Reducing the unit water content reduces not only the amount of water available to be frozen but also the amount of bleeding, which tends to be particularly excessive at low temperatures. It is also effective in preventing a decline in concrete temperature.

12.4 Mixing

(1) The temperature at which concrete is mixed must be determined with consideration of factors including weather conditions and transport time, to enable the specified concrete temperature during placement.

(2) When using heated materials, the order of addition must be determined to avoid causing rapid hardening of the cement.

(3) The mixing temperature of the concrete must be controlled to minimize fluctuations between batches.

Commentary: Regarding (1): It is generally thought that, for each hour of concrete transport and placement time, the temperature of the concrete at the end of placement will have declined by approximately 15% of the difference between the concrete temperature and the ambient air temperature. In other words, the concrete temperature at the end of placement can be roughly calculated using:

$$T_2 = T_1 - 0.15(T_1 - T_0)t, \qquad \text{(C12.4.1)}$$

where T_0 : ambient air temperature (°C);

T_1 : temperature of concrete when kneaded (°C);

T_2 : temperature of concrete at the end of placement (°C); and

t : time from kneading to end of placement (h).

Regarding (2): When high-temperature water and cement come into contact with each other, the cement may harden rapidly. Therefore, it is advisable to first add warm water and coarse aggregate, then add fine aggregate, and finally add cement after the temperature of the materials in the mixer reaches 40 °C or lower.

Regarding (3): If the amount of concrete supplied exceeds the capacity of the heating equipment, then the specified mixing temperature cannot be ensured. Therefore, the concrete placement plan should be prepared with sufficient consideration given to capacity for heating materials.

12.5 Transport and Placement

(1) In the transport and placement of concrete, the time from the start of mixing to the start of placement must be kept as short as possible and the temperature of the concrete must be kept from declining by means such as keeping the transport pipes warm.

(2) The temperature of concrete during placement must be kept within the range of 5 °C to 20 °C, taking into consideration factors including the cross-sectional dimensions of the structural object and weather conditions.

(3) Reinforcing bars, formworks, *etc*. must be free of ice and snow when the concrete is poured.

(4) If the concrete in construction joints is frozen, then it must be thawed by appropriate means before the concrete is jointed.

(5) Exposed surfaces of poured concrete must not be left open to outside air for long periods of time.

Commentary: Regarding (1): When using a concrete pump, mortar may freeze and adhere to the inner walls of the transport pipe when the pipe's temperature is too low, resulting in unexpected failures. Prevention of this requires actions including maintaining heat in pipelines and preheating with warm water before placement, and cleaning pipelines after completion.

Regarding (2): In cold weather construction work, hardening slows significantly and there is a risk of concrete freezing if the air temperature drops rapidly. Therefore, a concrete temperature appropriate to the type and size of the structural object and to the weather, air temperature, and curing method must be ensured when placement. However, when the concrete temperature is too high during placement, the unit water content required to obtain the required slump will increase, the concrete will set more quickly, and harmful effects such as decline in long-term strength may occur. Cracking caused by dryness may also appear on the concrete surface, and, when the cross section is thick, cracking caused by temperature due to heat of hydration may occur. Therefore, the concrete temperature for placement was set to the range of 5 °C to 20 °C, taking into consideration factors including weather conditions and the dimensions of the structural members in the structural object, to prevent initial freezing damage to freshly poured concrete. When the concrete temperature is low during placement, lateral pressure and the amount of bleeding increase. Therefore, it is necessary to pay attention to matters including placement speed and placement height.

Regarding (3): Melted water and water used for melting should be removed before placement concrete.

When placement concrete onto ground, the temperature of the concrete will fall rapidly if the ground is frozen, and the concrete will sink when the frozen ground thaws. Therefore, the ground where the concrete will be poured must be covered with sheets or other material to prevent freezing until the concrete is poured, and, if necessary, the ground must be kept warm with floodlights, heaters, *etc*. and water must be prevented from entering. Ground that is already frozen must be thawed by an appropriate method.

Regarding (4): Once the concrete has achieved sufficient strength, the concrete itself will satisfy required performance even when frozen. However, as performing jointing in a frozen state would adversely affect the integrity of joints, it is important to first thaw frozen areas. Concrete in which initial freezing damage has occurred should be removed.

Regarding (5): Because the concrete surface may be rapidly cooled by snowfall or wind between the time the concrete is poured and the start of curing, the surface should be covered with sheets or other appropriate material immediately after placement.

12.6 Curing

(1) The curing method and curing period must be determined with consideration of the outside air temperature, concrete mix, type of structural object, form and size of structural members, *etc*.

(2) Concrete must be sufficiently protected from freezing immediately after placement. In particular, protection against wind must be provided. It is also necessary to measure the concrete temperature and atmospheric temperature, and, when the possibility of adverse effects on concrete quality exists, to make changes to construction work plans and implement appropriate remedial measures.

(3) The maintenance of a temperature of 5 °C or higher until the compressive strength indicated in **Table 12.6.1** is obtained is set as a standard. **Table 12.6.1** "(1) When subjected to frequent freezing and thawing" sets the maintenance of a curing temperature at 0 °C or higher for an additional two days as a standard.

Table 12.6.1 Standard of required compressive strength at the end of keeping curing temperature 5°C or higher (N/mm^2)

Frequency of freezing-thawing until next spring at the end of controlling temperature of 5°C or higher	Size of sections		
	Thin	Ordinary	Thick
(1) Frequent	15	12	10
(2) Infrequent	5	5	5

(4) When supplying heat to concrete, rapid drying or localized heating of the concrete must be prevented.

(5) Concrete must be cured until sufficient strength is developed to withstand the loads expected during construction work.

(6) When finishing insulated curing or heat curing, the temperature of the concrete must not be allowed to fall rapidly.

Commentary: Regarding (1): Cold weather curing methods are classified into insulated curing and heat curing. In insulated curing, the concrete is covered with a highly insulating material and the cement's heat of hydration is used to keep it warm until it reaches the specified strength. In heat curing, heat is supplied from the outside when the air temperature is low or the cross section is thin, and when maintaining the temperature above the freezing point through insulation alone would be difficult. The supply of heat should be performed in combination with insulated curing using sheets or other materials to prevent the supplied heat from dissipating.

Because a higher curing temperature accelerates strength development, the temperature-controlled curing period can be shortened. However, the concrete will become prone to cracking when cooled after the completion of curing. Conversely, when the curing temperature is lowered, the temperature-controlled curing period until the specified strength is obtained will be extended.

Regarding (2): When concrete is subjected to initial freezing damage, strength development will be low even when curing is continued thereafter. Therefore, it is important to protect all areas of poured concrete from freezing until the specified strength is obtained. When constructing concrete in the cold, it is necessary to measure the temperature of the concrete and the atmosphere so as not to interfere with the strength development of the concrete. When it is not possible to follow the construction work plans and a risk of adverse effects on the concrete has been determined to exist, changes should be made to the construction plans and remedial measures such as adjustment of material temperatures, insulated curing, and heat curing should be implemented.

Wind accelerates the evaporation of water from the surface of concrete and lowers the temperature of concrete near the surface. Therefore, windbreaks or other measures must be employed to prevent wind from reaching the concrete surface immediately after placement.

Regarding (3): The curing temperature should be set with consideration of factors including the concrete mix, the outside air temperature, the cross-sectional dimensions of the structural object, the curing method and period, and the temperature-control method. Here, the curing temperature was set to a minimum of 5 °C to prevent initial freezing damage. When the cold is extreme or the cross section is thin, it is advisable to set the temperature to approximately 10 °C. When the cross section is thick, rapid cooling after curing should be avoided as the temperature may have reached 20 °C or higher due to the heat of hydration. In general, it is advisable to perform curing so that the surface temperature of the concrete does not exceed 20 °C. Because resistance to initial freezing damage varies with factors including concrete strength, moisture content, and the amount, size, and distribution of entrained air, an expression in terms of strength alone is not always appropriate. However, if compressive strength is 3 to 5 N/mm^2 or greater, then freezing damage is considered relatively uncommon even after several instances of freezing. Because structural members with thinner cross sections are more susceptible to initial freezing damage and to greater effects of damage, the initial strength of the concrete must be increased. Taking these considerations into account, **Table 12.6.1** shows standards for the compressive strength required until completion of the stage at which the curing temperature is maintained at 5 °C or higher.

The curing period shown here indicates the temperature-controlled curing period for obtaining the required compressive strength. Even after the compressive strength shown in **Table 12.6.1** has been obtained and temperature-controlled curing has been completed, moist curing is then performed within the period indicated in **Table 8.2.1** of **"Chapter 8 Curing."**

The temperature-controlled curing period for obtaining the required compressive strength may be determined through strength testing on specimens which, to the extent possible, have been cured under the same conditions as the concrete at the construction site, or by estimating strength from records of concrete temperature when the relationship between concrete temperature and the compressive strength at specific material ages has been obtained in advance through testing. When specimens are cured on-site, the temperature of said specimens should be measured and compared against the temperature of the structural object.

By measuring the concrete temperature of the structural object, its strength can be estimated from the integrated temperature. Integrated temperature is generally expressed by:

$$M = \Sigma(\theta + A)\Delta t, \qquad \text{(C12.6.1)}$$

where M : integrated temperature (°C/day or °C/hour);

θ : concrete temperature during Δt hours (°C);

A : 10 °C is generally used as a constant; and

Δt : time (days or hours).

Because the relationship between integrated temperature M and the strength of concrete varies non-uniformly with factors such as the materials used, the concrete mix, and the degree of drying and wetting, it is advisable that this be confirmed in advance through testing.

Regarding (4): Warming concrete accelerates the evaporation of water from the concrete. Therefore, a method should be used that provides sufficient moisture and prevents drying of the concrete.

When supplying heat, cracking is likely to occur if temperature differences between areas of the concrete are great due to localized heating. The method used should avoid temperature differences as much as possible.

Regarding (5): Because concrete is exposed to low temperatures following the end of insulating or heat curing, subsequent development of strength is slow. Therefore, unless the time until the concrete is subjected to loading is fairly late, curing must be continued even after the strength required to resist initial freezing damage has been obtained, up until the strength required to resist expected loads is obtained.

Regarding (6): When concrete at a high temperature is suddenly exposed to cold after the completion of insulating curing or heat curing, cracking may occur on the surface of the concrete. An appropriate method should be used to prevent rapid cooling of the surface. When exposure to cold air and freezing after curing are expected, spraying water just before the completion of curing should be avoided.

12.7 Formworks and Shoring

(1) In principle, formworks with good insulating properties should be used.

(2) The lateral pressure that acts on formworks must be set with reference to **11.2.4**, taking into account the temperature of the concrete.

(3) Displacement of the shoring foundation by ground frost heaving or by thawing of frozen ground must be prevented.

(4) Removal of formworks must be conducted in a way that prevents the temperature of the concrete from dropping rapidly.

(5) The timing of the removal of formworks and shoring must be determined by an appropriate method.

Commentary: Regarding (1): When the concrete cross section is thick, the effect of temperature rise due to heat of hydration should be used. Even when the cross section is thin, it is possible to insulate the concrete without the supply of heat through the combined use of formworks and foamed resin, *etc*. Because concrete is susceptible to sudden changes in the outside air temperature when steel formworks are used, attention must be paid to insulation.

Regarding (2): A lower concrete temperature tends to result in greater lateral pressure acting on formworks.

This must be taken into consideration when setting the value of lateral pressure to be used in formwork design, with reference to **"11.2.4 Lateral pressure in concrete"** in **"Chapter 11 Formworks and Shoring."**

Regarding (3): When erecting shoring directly on the ground, ground displacement may occur as a result of frost heaving or thawing, in turn preventing accurate position, form, or dimensions of the structural object or even leading to collapse of the shoring. To avoid adverse effects when displacement due to frost heaving or thawing is expected, remedial measures such as piled foundations or prevention of ground freezing should be taken.

Regarding (4): When the cross section is thick, the temperature inside the concrete will be considerably high due to the heat of hydration. When rapid cooling occurs as a result of removal of formworks, a large temperature difference may occur, resulting in thermal cracking. Therefore, it is necessary to reduce the temperature difference before removing the formworks by means such as adjusting the temperature inside the curing sheets or leaving the formworks in place until the concrete surface will no longer undergo rapid cooling even after sufficient strength to allow the removal of formworks has been obtained.

Regarding (5): The appropriateness of the timing for removing formworks and shoring should also be judged from the strength of specimens cured on site, or from estimated strength based on the concrete temperature and integrated temperature.

Chapter 13 Hot Weather Concreting

13.1 General

(1) When construction work is assumed to take place at a time when the average daily air temperature exceeds 25 °C, hot weather concreting is standard practice.

(2) In hot weather concreting, appropriate measures must be taken with respect to matters including materials, concrete mix, mixing, transport, placement, and curing to prevent the quality of concrete from declining as a result of the high temperatures.

Commentary: Regarding (1): When the air temperature exceeds 30 °C during placement, changes in the properties of the concrete will be significant. When work is performed under an average daily air temperature in excess of 25 °C, it is generally advisable to perform construction work under a hot weather concreting plan.

Regarding (2): When the air temperature is high, the temperature of concrete rises accordingly, increasing the risks of slump reduction during transport, reduction of entrained air volume, occurrence of cold joints, cracking caused by rapid evaporation of surface moisture, thermal cracking, *etc*. Special consideration is required in material handling, concrete mix, mixing, transport, placement, curing, and other steps to ensure that the temperature of the concrete is as low as possible during and immediately after placement.

13.2 Materials

(1) When the specified concrete temperature cannot be obtained, methods of lowering the temperature of materials must be investigated and their efficacy must be confirmed in advance.

(2) Water-reducing agents, AE water-reducing agents, and plasticizers must be retardant types that conform to JIS A 6204 as standard practice. High-performance AE water-reducing agents must comply with JIS A 6204 as a standard practice.

Commentary: Regarding (2): Using a high-performance AE water-reducing agent can significantly reduce unit water content and the unit cement content, even in hot weather concreting. Depending on the type, however, the decrease in slump over time can be significant under high temperatures. Therefore, when any chemical admixtures are used, it is important to select admixtures that can ensure the specified minimum slump until the planned completion time for placement under high temperatures.

13.3 Mix proportions

To prevent initial freezing damage, the unit water content and unit cement content must be kept as small as possible within the range that maintains required strength and workability.

Commentary: A certain relationship must exist between mixing temperature and unit water content in order to achieve the required slump. In general, unit water content tends to increase by 2% to 5% with every 10 °C rise in temperature. The change over time in slump and air content also tends to increase as the temperature of concrete rises. Therefore, if slump is adjusted inappropriately in the setting of the surface moisture ratio of the aggregate without taking remedial measures for hot weather concreting, the actual unit water content will increase and the water-to-cement ratio will increase. For these reasons, concrete characterized by low compressive strength and large drying shrinkage may be poured. Conversely, when the water-to-cement ratio is kept constant to ensure the required compressive strength and the unit cement content is increased in line with the increase in unit water content, conditions conducive to cracking result from hydration of the cement.

When concern exists over a major increase in unit water content needed to achieve the required slump, it is necessary to take actions such as increasing the amount of chemical admixture used, changing from an AE water-reducing agent to a high-performance AE water-reducing agent or other material, and adding plasticizer.

13.4 Mixing

Concrete must be mixed at a temperature that enables the specified concrete temperature during placement, while taking into consideration the effects of factors including weather conditions and transport time.

Commentary: To lower the temperature of concrete after mixing, the temperature of the materials may be lowered. However, when ice is used as a portion of the mixing water, it is necessary to confirm in advance that the ice will melt completely during mixing. Blowing liquid nitrogen into the mixer to directly cool the concrete is another available method.

13.5 Transport

Concrete should be transported using equipment and methods that reduce temperature rise and drying in the concrete.

Commentary: In hot weather concreting, temperature rise and drying during transport are greater than usual. Equipment and methods that lessen these effects should be used, and the concrete should be transported and placed as soon as possible.

13.6 Placement

(1) When placement concrete, areas that could potentially absorb water from the concrete must be kept wet. Appropriate measures such as water spraying and placement of covering must be taken for areas that could be warmed by direct sunlight.

(2) Concrete should be poured as soon as possible after mixing. In principle, the time from the start of mixing to the end of placement should be within 1.5 hours.

(3) As the upper limit for concrete temperature during placement, 35 °C is set in standard practice. If the temperature of concrete exceeds this limit, then it is necessary to confirm that the required concrete quality can be ensured.

Commentary: Regarding (1): When the air temperature is high or when heating occurs due to direct sunlight, the ground, wooden formworks, the surfaces of freshly poured concrete, and the surfaces of hardened concrete (joint surfaces) are susceptible to drying. Because this may lead to a decline in the fluidity (filling ability) of the poured concrete and in the integrity of stacked parts, the concrete must be kept wet by placement of coverings, water sprinkling, or other means.

Regarding (2): In general, the amount of decrease in slump will be small and placement can proceed without problem if it is performed within 1.5 hours of mixing. However, because such changes in quality tend to increase as the air temperature rises, it is advisable to pour concrete continuously and as soon as possible after mixing to prevent the occurrence of cold joints, *etc*.

Regarding (3): A high concrete placement temperature can affect the quality of the concrete in a number of ways. For this reason, it is advisable to lower the placement temperature of the concrete as much as possible.

Results to date suggest that, under general conditions, a placement temperature of 35 °C or lower has little effect on the quality of concrete. Therefore, 35 °C was set as the upper limit for concrete temperature during placement in standard practice.

13.7 Curing

Upon the completion of concrete placement, curing must be started immediately to protect the concrete surface from drying. In particular, when the air temperature is high and humidity is low, cracking may occur as a result of rapid drying immediately after placement. Measures must be taken to protect against direct sunlight and wind.

Commentary: In hot weather concreting, the surface of poured concrete dries rapidly when exposed to direct sunlight or wind. This not only complicates finishing but also increases the possibility of cracking. Therefore, it is important to cure concrete immediately after placement to prevent exposed surfaces from drying. Formworks that are prone to drying should also be kept wet. After the removal of formworks, the exposed surfaces must also be kept wet during the curing period. As standard practice, the moist curing period should satisfy the number of days shown in **Table 8.2.1** of **"8.2 Moist curing"** in **"Chapter 8 Curing,"** and should be set appropriately with construction work conditions and other factors taken into consideration. In particular, when the air temperature is high and humidity is low, the surface is susceptible to rapid drying and cracking. It is important to limit drying of the surface by taking appropriate measures such as placement of coverings or water sprinkling

Chapter 14 Mass Concrete

14.1 General

(1) When thermal stress caused by the heat of hydration of cement is a problem, the concrete must be treated as mass concrete and adequate remedial measures must be considered.

(2) In construction work on mass concrete, it must be confirmed in the construction work plans whether the conditions for verification of thermal cracks described in **"Design"** match the actual construction work conditions. If the verification conditions at the design stage do not match actual construction work conditions, then it is necessary to refer to **"Design"** and perform verification of thermal cracking again in construction work plans, taking into account the actual construction work conditions.

(3) In the production of concrete used in mass concrete and in construction work, it is necessary to appropriately plan and implement temperature control, transport, placement, and curing of the concrete to ensure that the effects of measures inhibiting thermal cracking are sufficiently obtained.

Commentary: Regarding (1): The structural members that should be treated as mass concrete differ in their dimensions, structural form, concrete materials, concrete mix, and construction work conditions. A thickness of around 80 cm to 100 cm for a wide slab or 50 cm or more for a wall with a constrained lower end can be used as a guideline. However, when rich-mixed concrete is used, as in the case of prestressed concrete structural objects, depending on constraint conditions, it may be necessary to handle the concrete as mass concrete even when structural members are thinner.

In construction work on mass concrete, particular attention must be paid to the prevention of thermal cracking or the inhibition of crack width. To satisfy the uses, required functionality, and required performance of a structural object, it is necessary to carry out prevention of cracking, inhibition of crack width, and control of the spacing and position of cracks.

Regarding (2): Methods for the prevention of thermal cracking in mass concrete, the inhibition of crack width, and the control of crack spacing and positions include remedial measures taken at various stages, including design, selection of materials and mix, and construction work. In the **"Design"** volume of the Standard Specifications, verification of thermal cracking is carried out at the design stage based on past construction performance or thermal stress analysis, and materials, concrete mix, and construction work methods are selected to prevent cracking or inhibit crack width.

Verification at the design stage is performed based on assumed construction work conditions, but said conditions may vary in numerous ways during the actual construction work stage. As an example, when the air temperature and other conditions used in the verification of thermal cracking have changed significantly because of a change in the construction work period, it is necessary to reconsider the environmental conditions and other conditions assumed for the construction work stage. When construction work methods in construction work plans, such as the size of placement sections and lift heights, change significantly from the design stage, assumptions for the verification of thermal cracking can no longer be ensured. Therefore, in construction work plans, it is necessary to confirm that preconditions assumed for the design stage conform to actual construction work conditions. When this reveals that preconditions assumed at the design stage do not match actual construction work conditions and it is necessary to revise details of considerations presented in **"Design,"** verification must be conducted again for construction work plans based on the actual construction work conditions, using methods presented in **"Design: General Requirements" (Chapter 12 Verification of Initial Cracking)**. When thermal stress analysis has been performed, **"Design: Standards" Part 6 (Thermal Cracking Verification)** should also be used as a reference. When correction of concrete materials, concrete mix, *etc.* becomes necessary in the repeated verification, the satisfaction of other aspects of required performance (workability, design specified strength, resistance to deterioration, *etc.*) should also be reconfirmed.

Regarding (3): Inhibition of thermal cracking requires deep consideration of not only thermal management during concrete production and the appropriate selection of cement type, admixtures, other materials, and concrete mix, but also production and construction work overall, including the appropriate selection of the size of placement sections, lift height, positions of joints, construction joint time intervals, formwork materials and structure, and curing methods. Depending on the type of the structural object, controlling the location of cracking through joints to control cracking may be effective. These remedial measures in production and construction work can in principle be implemented based on the conditions stipulated in **"Design,"** which describes details including concrete materials, concrete mix, production, and construction work methods for conducting verification of thermal cracking in advance.

14.2 Materials

The cement and admixtures used must be those assumed at the time of verification of thermal cracking based on actual construction work conditions.

Commentary: In mass concrete, the type of cement greatly affects strength, temperature rise, and other aspects of the concrete. In the selection of cement, its properties should be thoroughly investigated to confirm that it can yield concrete of the required quality. In general, it is advisable to use low-heat cements such as moderate-heat Portland cement, low-heat Portland cement, blast-furnace slag cement, and fly ash cement.

Depending on the form, dimensions, and reinforcing-bar arrangement of the structural object, inhibition of cracking using expansive materials is effective in some cases, such as in wall structural objects in which verification of the inhibition of thermal cracking in mass concrete must be performed.

The appropriate use of AE agents, water-reducing agents, AE water-reducing agents, or high-performance AE water-reducing agents improves the workability of concrete, which in turn allows reduction of unit water content and accompanying reduction in unit cement content. Therefore, the use of these chemical admixtures allows the temperature rise in concrete to be lessened.

14.3 Mix proportions

The unit cement content of the concrete must be that assumed at the time of verification of thermal cracking based on actual construction work conditions.

Commentary: The calorific value of concrete is nearly proportional to the unit cement content. In general, the temperature rise in concrete varies at a rate of approximately 1 °C per 10 kg/m^3 of unit cement content. Therefore, to prevent thermal cracking or to inhibit crack width, unit cement content should be kept as low as possible while ensuring the required workability and crack resistance.

14.4 Production

(1) The temperature during production of concrete must be set and managed to prevent a high placement temperature, with transport distance, transport method, placement method, weather conditions, and other conditions taken into account.

(2) When placement concrete from multiple ready-mixed concrete plants at a single location, the required workability of the mixed concrete must be confirmed.

Commentary: Regarding (1): Lowering the concrete placement temperature reduces the temperature difference between the interior and exterior of structural members and the maximum temperature inside structural members, which reduces thermal stress and is effective in the prevention of thermal cracking and inhibition of crack width. Methods of lowering the placement temperature include pre-cooling, by which water, aggregate, and other materials are cooled during the production of concrete. The effect of the temperature of materials on the mixing temperature of concrete is approximately ±1 °C per ±2 °C for aggregate, ±1 °C per ±4 °C for water, and ±1 °C per ±8 °C for cement.

Even if the temperature is lowered during concrete production, concrete temperature rises during transport and the placement temperature could exceed the temperature in construction work plans. Therefore, it is important to set the temperature of the concrete during its production, while anticipating the temperature rise in the concrete caused by transport distance, transport method, weather conditions, and other factors.

Regarding (2): When concrete is supplied from several factories, it is advisable that the same type of cement and chemical admixture be used, and, when possible, that the fine aggregate and coarse aggregate come from the same production areas.

14.5 Placement

(1) The size of placement sections, lift heights, positions and structure of joints, and construction jointing time intervals must be those assumed at the time of verification of thermal cracking based on actual construction work conditions.

(2) Remedial measures must be implemented to keep the concrete placement temperature as low as possible, within a range that does not adversely affect workability or strength development.

Commentary: Regarding (1): In mass concrete, large volumes of concrete are generally divided into sections and poured, which necessitates the provision of joints. In setting the size of the sections (block divisions), the size of the vertical lift divisions, and the positions and structure of joints, it is necessary to comprehensively take into account construction work conditions including constraint conditions, heat dissipation conditions for inhibiting thermal cracking, and the amount of concrete that can be poured at a time.

When placement mass concrete divided into several flat blocks or multiple lifts, the newly poured concrete is constrained by old concrete, resulting in the occurrence of stress in accordance with changes in temperature. This stress increases in line with increases in the difference in temperature and the difference in Young's modulus between the old concrete and the new concrete. Therefore, excessively long time intervals for jointing between the new and old concrete should be avoided. Conversely, when jointing concrete over several layers on top of a surface with a high degree of constraint such as bedrock, if the jointing time interval is made too short, then, depending on the conditions such as the lift thickness, the temperature of the concrete overall may rise and the possibility of cracking may increase.

Regarding (2): Concrete placement temperature is an important condition in the verification of thermal cracking. In actual construction work, it is advisable to keep the temperature as low as possible. At the stages of design and construction work planning, the thermal properties of the materials, the environmental conditions, and other factors are assumed and examined. However, if the placement temperature of concrete greatly exceeds the temperature that was assumed in advance, then it may be difficult to prevent thermal cracking or control the width of cracks. Particularly when placement concrete during hot weather, consideration must be given to the handling of materials and the production, transport, and placement of concrete to keep the placement temperature from becoming high.

It is also advisable to measure the temperature history of poured concrete, along with the temperature of the concrete before placement, to demonstrate the validity of preliminary examinations and to make appropriate changes in construction work plans. When the measured temperature rises differ greatly from the conditions planned in advance, changes should be made to construction work plans, such as the timing of removal of formworks and the curing method.

14.6 Curing

To enable the prevention of thermal cracking and the control of crack width in accordance with plans, curing must be properly performed in line with the assumptions made during verification of thermal cracking based on actual construction work conditions.

Commentary: In addition to the conditions required for normal concrete curing, the curing method determined in line with the findings of examination of temperature cracking and from construction work plans must be reliably implemented in the curing of mass concrete. In particular, consideration must be given to bringing the temperature of the concrete closer to that of outside air as gradually as possible, to prevent the temperature difference between the exterior and interior of concrete structural members, as well as the rate of temperature drop in the structural members as a whole, from becoming large. Therefore, it is advisable to use measures such as covering the concrete surface with a highly insulating material as necessary. However, if the rise in concrete temperature is significant as a result of excessive insulation, then thermal cracking will become likely.

Moreover, it must be noted that lowering the temperature of the concrete surface through excessive spraying of water may promote cracking.
Because mass concrete presents a large surface area for construction work, it may be susceptible to plastic shrinkage cracking due to drying after finishing. Particular consideration must be given to the curing performed immediately after finishing.

14.7 Joints to control cracking

Joints to control cracking for controlling thermal cracking must have the structure assumed during verification of thermal cracking based on actual construction work conditions, and must be installed at set positions.

Commentary: In general, it is difficult to control thermal cracking in massive wall-like structural objects, *etc*. through material- and mix-related measures alone. Depending on the type of the structural object, controlling the location of cracking through joints to control cracking may be effective. One method available in such cases is providing areas of reduced cross section at regular intervals in the longitudinal direction of the structural object in order to induce cracks in those areas and prevent the induction of cracking in other areas, while also facilitating aftercare measures at the locations of cracks. See **"9.3.3 joints to control cracking"** in **"Chapter 9 Joints"** for details.

14.8 Reinforcement work

Reinforcing bars for controlling the width of cracks must be arranged at set positions that were assumed during verification of thermal cracking based on actual construction work conditions.

Commentary: Arranging reinforcing bars near the surface of concrete is an effective method for controlling the width of cracks. Reinforcing bars to control crack width are a measure to be used only in areas where cracking is a concern, and are less costly than measures that address the concrete structural object as a whole, such as changing the concrete mix or cement type. The efficacy of crack width control is greatly affected by factors including the amount, diameter, and arrangement of reinforcing bars, which must be properly arranged to control the width of cracks. Because an increase in the amount of reinforcing bars can cause concrete filling to be difficult in some cases, considerations such as the reinforcing bar spacing required for compaction should also be confirmed. Moreover, because reinforcing bars are arranged near the surface of concrete, care should be taken to ensure the required cover.

14.9 Formwork

For proper prevention of thermal cracking and control of crack width, concrete formwork materials and structures must be selected in line with the assumptions made during verification of thermal cracking based on actual construction work conditions, and the formworks must be left in place for the proper period of time.

Commentary: The use of highly heat-dissipating formworks is a potential way to reduce temperature rise.

In mass concrete, however, the scope of temperature rise control offered by heat dissipation is limited. Moreover, during the winter or when the air temperature is expected to drop significantly after concrete placement, the use of highly heat-dissipating steel formworks may result in a large temperature difference between the interior and the surface, along with subsequent susceptibility to thermal cracking. When formworks are removed, too, a large difference between the temperature of the concrete and of the outside air will cause the concrete to cool rapidly after formwork removal, with subsequent susceptibility to surface cracking. Reducing the temperature difference between the interior and the surface of structural members is effective as a means of controlling thermal cracking as much as possible in such cases, with the difference kept to 15 °C to 20 °C or lower as a general guideline.

Chapter 15 Quality Control

15.1 General

(1) Constructing concrete structural objects economically and with the required quality requires proper quality control at every stage of construction work.

(2) Quality control is a self-directed activity performed by the builder, who must plan and properly implement methods that will yield anticipated efficacy.

(3) Quality control records should be properly stored to enable their use in the quality assurance of constructed structural objects and the quality control of future construction.

Commentary: Regarding (2): Quality control essentially consists of actions aimed at the stability of quality. Therefore, it is important to discover anomalies as early as possible, elucidate their causes, and take appropriate remedial measures to constrain fluctuations in quality. To do so, it is important to create a structure for frictionlessly operating the PDCA (Plan, Do, Check, and Action) cycle. First, it is important to draft an effective and rational quality control plan. In planning, the parties responsible for quality control for all tasks in production and construction work must be determined, and the items subject to control, the methods of control, and the remedial measures to be taken in the event of anomalies must be made clear.

15.2 Quality control of concrete materials and reinforcing materials

Concrete materials and reinforcing materials should be managed so as to satisfy required quality and remain in as stable a state as possible.

Commentary: By presenting the required quality for concrete materials in advance and confirming their quality through means such as testing results charts, producers of ready-mixed concrete ensure that variation in material quality falls within an acceptable range for producing stable ready-mixed concrete.

When purchasing ready-mixed concrete, builders request that producers present quality control reports as required before and during concrete construction, and confirm that the materials used meet the specified quality. It is also advisable that builders investigate producers' material storage facilities in advance to confirm the storage condition of materials. Materials used on-site, such as reinforcing materials and plasticizers, quick-setting agents, and other chemical admixtures, are checked for quality on the basis of testing reports and other sources at the time of acceptance. Because these materials are susceptible to environmental conditions at the site, on-site quality control is important.

When using concrete mixed on-site (*i.e.*, concrete produced on-site under the responsibility of the builder), the builder conducts an acceptance inspection of the materials and performs appropriate quality control.

15.3 Quality control in concrete production

Concrete production plant and production processes are appropriately controlled in order to produce concrete stably and smoothly with the required quality.

Commentary: Storage equipment, weighing equipment, and mixers are appropriately managed during the concrete production period so as to satisfy **"5.2 Production plant"** in **"Chapter 5 Production."** To continuously produce concrete of stable quality, it is important to stabilize the surface moisture ratio of the aggregate. It is also necessary to periodically measure the surface moisture ratio with sufficient accuracy and to correct the target values for material weighing accordingly.

In the management of mixing, the quality of the concrete can be confirmed by such methods as observing the status of concrete mixing via a monitor and estimating the slump from load current values in the mixer. The state of quality control during production can often be confirmed visually or even from machine noise or other means, by noting factors including the properties of the concrete during or after mixing and production plant operating status, cleanliness, and deterioration/damage.

Using appropriate control charts to organize test values for concrete compressive strength, slump, air content, *etc*. enables the early detection of anomalies in production processes. JIS Z 9020-2 should be used as a reference in the creation and assessment of control charts.

15.4 Quality control in the acceptance of ready-mixed concrete

During acceptance of ready-mixed concrete, confirmation of quality through acceptance inspection is standard practice.

Commentary: Builders should select the necessary items to check during acceptance inspections and should conduct the inspections with appropriate timing and frequency.

The points of note below assume acceptance inspections for ready-mixed concrete, but are also applicable to quality control for concrete mixed on site.

(a) Regarding confirmation of concrete mix: Production records should be read and the concrete mix actually produced should be checked.

(b) Regarding quality control for fresh concrete: The condition of fresh concrete should be observed routinely. When any anomalies are observed, the following items should be checked immediately:

(i) Slump

(ii) Air content

(iii) Temperature of concrete

(c) Regarding quality control in hardened concrete: Strength testing using concrete specimens is necessary to confirm the quality of hardened concrete. However, a period of 28 days is generally required to obtain results. Therefore, the performance of strict quality control for materials and production is normally a reasonable alternative.

15.5 Quality control in construction work

Every process of concrete work, reinforcement work, formwork and shoring construction, *etc*. is properly managed to build concrete structural objects that possess the required quality.

Commentary: Builders should carefully check the status of construction work. When improvements are deemed necessary despite work conforming to construction work plans, it is advisable to make improvements to the methods.

(a) Regarding quality control at the preparation stage: Measurements (benchmarks, *etc*.), conditions for the design and construction work of the foundation ground, and other conditions should be checked in advance before carrying out concrete work, reinforcement work, and formwork and shoring construction.

Before starting any task, the quality control system must be checked and the person in charge of quality control must accurately convey necessary matters to workers and incorporate those matters into construction work. It is also important to check weather forecasts, review daily and weekly schedules in line with weather, and make preparations such as protection from rain.

(b) Regarding quality control in concrete work: A fundamental part of the construction work stage is confirming that the equipment, machinery, personnel allocation, and construction work methods for each task conform to design drawing documents and construction work plans. Before placement concrete, it should be confirmed that reinforcing materials, formworks, and shorings have been installed according to plans.

(i) Transport to the construction site: Continuous placement is facilitated by preparing operational plans and control tables that record matters including planned and actual times (shipments, arrivals, the completion of efflux, testing, *etc*.), assumed time intervals, vehicle numbers of agitator trucks, load capacities and total loads, and whether quality control or acceptance inspections are performed. On the day of construction, it is also facilitated by managing production and transport status in close contact with persons in charge of quality control at the production plant and placement sites.

(ii) Transport within the site: It is important for persons in charge of quality control to comprehensively judge the quality of concrete, transport to the site, placement, compaction status, and loads acting on formworks and shoring, and to provide appropriate instruction concerning the pumping method.

(iii) Placement and compaction: A check sheet that describes the construction work methods, placement locations, placement heights, vibrator insertion positions and depths, vibration time, and other matters specified in construction work plans is an effective means of checking status during construction work. However, because the compaction efficacy of vibrators varies with factors such as the state of the concrete and reinforcing-bar arrangement, it is also important to make changes to vibrator insertion intervals and vibration times as appropriate to properly perform concrete filling. When anomalies in the properties of fresh concrete are found during construction work, concrete placement should be halted as necessary to investigate the cause and to take appropriate measures.

(iv) Finishing: The appropriate time for finishing varies with the weather. Because the quality of finished surfaces is also affected by the skill level of workers, it is important to provide instruction on timing and methods that are appropriate for the conditions. If finishing is completed while bleeding is occurring, then subduction cracking and peeling may occur afterward. Therefore, it is important to carefully manage the time and the content of work up to final finishing.

(v) Curing: When moist curing is inappropriate, the concrete will differ significantly in quality from specimens cured in water. Therefore, quality control in curing is important in ensuring the strength and durability of structural objects.

(c) Regarding quality control in reinforcement work: Quality control should be carried out with the following points in mind. This also applies to reinforcing materials other than reinforcing bars.

(i) Confirmation of the reinforcing bar arrangement diagram

(ii) Reinforcement bar work plan

(iii) Acceptance and storage of reinforcing bars

(iv) Processing of reinforcing bars

(v) Assembly of reinforcing bars

(d) Regarding quality control for formworks and shoring: Quality control should be carried out with the following matters concerning formworks and shoring in mind:

(i) Calculation and confirmation of formworks and shoring

(ii) Confirmation during assembly of formworks and shoring

(iii) Management of formworks and shoring during concrete placement

(iv) Confirmation of the timing and method of removal

15.6 Management of concrete structural objects

To deliver concrete structural objects with the required quality to clients, the structural objects must be managed until the completion of delivery.

Commentary: The concrete surface should be observed after the removal of formworks. If cracking, flaking, honeycombing, cold joints, exposure of reinforcing bars due to insufficient cover, or other defects are found, then these should be promptly reported to the ordering party. Following confirmation with the ordering party, causes should be investigated, remedial measures discussed, and appropriate measures taken as necessary. With respect to concrete structural objects under construction, confirmation should be carried out for the same items addressed in the inspection of the surface condition of the concrete; the positions, forms, and dimensions of concrete structural members; the quality and cover of concrete in the structural object; *etc*. During and after construction and until delivery to the ordering party has been completed, the condition of not only the concrete structural object but also the protection of its surroundings and other matters must be periodically observed, and the absence of anomalies must be confirmed.

Chapter 16 Construction Work Records

16.1 General

Builders compile records of processes in concrete construction, production methods, construction work methods, weather, temperature, quality control, and other matters.

Commentary: Regarding (1): The structural members that should be treated as mass concrete differ in their dimensions, structural form, concrete materials, concrete mix, and construction work conditions. A thickness of around 80 cm to 100 cm for a wide slab or 50 cm or more for a wall with a constrained lower end can be used as a guideline. However, when rich-mixed concrete is used, as in the case of prestressed concrete structural objects, depending on constraint conditions, it may be necessary to handle the concrete as mass concrete even when structural members are thinner.

In construction work on mass concrete, particular attention must be paid to the prevention of thermal cracking or the inhibition of crack width. To satisfy the uses, required functionality, and required performance of a structural object, it is necessary to carry out prevention of cracking, inhibition of crack width, and control of the spacing and position of cracks.

Regarding (2): Methods for the prevention of thermal cracking in mass concrete, the inhibition of crack width, and the control of crack spacing and positions include remedial measures taken at various stages, including design, selection of materials and mix, and construction work. In the **"Design"** volume of the Standard Specifications, verification of thermal cracking is carried out at the design stage based on past construction performance or thermal stress analysis, and materials, concrete mix, and construction work methods are selected to prevent cracking or inhibit crack width.

Verification at the design stage is performed based on assumed construction work conditions, but said conditions may vary in numerous ways during the actual construction work stage. As an example, when the air temperature and other conditions used in the verification of thermal cracking have changed significantly because of a change in the construction work period, it is necessary to reconsider the environmental conditions and other conditions assumed for the construction work stage. When construction work methods in construction work plans, such as the size of placement sections and lift heights, change significantly from the design stage, assumptions for the verification of thermal cracking can no longer be ensured. Therefore, in construction work plans, it is necessary to confirm that preconditions assumed for the design stage conform to actual construction work conditions. When this reveals that preconditions assumed at the design stage do not match actual construction work conditions and it is necessary to revise details of considerations presented in **"Design,"** verification must be conducted again for construction work plans based on the actual construction work conditions, using methods presented in **"Design: General Requirements" (Chapter 12 Verification of Initial Cracking)**. When thermal stress analysis has been performed, **"Design: Standards" Part 6 (Thermal Cracking Verification)** should also be used as a reference.

When correction of concrete materials, concrete mix, *etc*. becomes necessary in the repeated verification, the satisfaction of other aspects of required performance (workability, design specified strength, resistance to deterioration, *etc*.) should also be reconfirmed.

Regarding (3): Inhibition of thermal cracking requires deep consideration of not only thermal management during concrete production and the appropriate selection of cement type, admixtures, other materials, and concrete mix, but also production and construction work overall, including the appropriate selection of the size of placement sections, lift height, positions of joints, construction joint time intervals, formwork materials and structure, and curing methods. Depending on the type of the structural object, controlling the location of cracking through Joints to control cracking may be effective. These remedial measures in production and construction work can in principle be implemented based on the conditions stipulated in **"Design,"** which describes details including concrete materials, concrete mix, production, and construction work methods for conducting verification of thermal cracking in advance.

Chapter 17 Other Points of Note in Construction Work

17.1 General

(1) When constructing attachments, the attachments shown in the design drawing documents must be installed at the specified positions and in the specified quantity.

(2) When changing the type, position, quantity, etc. of attachments shown in design drawing documents, or when performing construction work on attachments not shown in design drawing documents, approval must be obtained from the commissioning party or builder at the construction work planning stage.

Commentary: Regarding (1): In the design and construction of attachments, it is necessary to coordinate civil engineering planning and facility planning at the detailed design stage of civil engineering planning, and to draft a plan that adequately reflects each of them. Attachments are typically constructed after the completion of the concrete construction and are often constructed by a different builder than the original concrete construction builder. Therefore, at the construction work planning stage, it is important for the builder of the attachment to check the positions of the rebars, the cover, strength of concrete, and other items using the design drawing documents for the concrete frame, and to consider the appropriate construction work period and construction work methods. The types, positions, and quantity of the attachments indicated in the design drawing documents must also be observed.

Inspection Standards

Chapter 1 General Rules

1.1 General

(1) "Materials and Construction: Inspection Standards" presents standards for inspections conducted under the responsibility of the commissioning party at each stage of concrete construction work for general new civil engineering structural objects and for completed structural objects.

(2) Inspections must be conducted using a method that enables objective judgments based on predetermined criteria. Methods specified in Japanese Industrial Standards (JIS) or in Japan Society of Civil Engineers (JSCE) standards are set as standard methods.

(3) When the findings of inspections are not judged to be acceptable, appropriate measures must be taken so that structural members and the structural object satisfy the required performance.

Commentary: Regarding (1): The inspections presented in these inspection standards consist of actions by which the commissioning party of the construction judges whether the manufactured and constructed concrete, structural members, structural objects, and so on satisfy the initially set required performance and whether the completed structural object can be accepted.

Ideally, the performance of the structural object is subject to direct inspection. At present, however, the items that can be inspected in a completed structural object are limited to a small number, including the surface condition of the concrete and the positions, forms, and dimensions of the structural members. Therefore, it is important to formulate a rational, economical, and systematic inspection plan based on the design drawing documents and construction work plans, and to conduct appropriate inspections at every stage of construction work, to prevent defects upon completion.

Inspections include inspections of structural members and structural objects conducted upon completion, and inspections of specific processes conducted during construction. Inspections are conducted under the responsibility of the commissioning party, including inspections of specific processes. However, it is not always practical for the builder and the commissioning party to perform duplicate inspections of items for which the builder performs acceptance inspections as a component of quality control, as in acceptance inspections of ready-mixed concrete. Therefore, inspections by the commissioning party can be replaced by confirmation by the commissioning party of the findings of inspections performed by the builder. “**Chapter 15 Quality Control”** in **"Construction Standards"** presents acceptance inspections conducted by builders as a component of quality control.

Regarding (2): Objective testing methods and pass/fail criteria are required in inspections. For that purpose, the use of methods stipulated in the Japanese Industrial Standards (JIS) and the Japan Society of Civil Engineers Standards for various types of testing was set as standard practice.

Regarding (3): Inspections are fundamentally for the purpose of judging whether a structural object can be accepted. In principle, when it has been determined from inspection findings that a structural object does not satisfy the required performance, acceptance is to be refused. In the case of civil engineering structural objects, however, when factors such as the scale of the structural objects, the length of the construction work period, the social impacts of postponing completion, and the impacts caused by dismantling and disposal are considered, dismantling and reconstruction is not an optimal measure in all cases. When individual processes during construction are deemed to have failed inspection, various responses must be considered. Such cases may be dealt with through partial measures during construction, but measures such as reinforcement may be required at the final stage. These inspection standards are structured to allow consideration of such measures.

Chapter 2 Inspection Planning

2.1 General

(1) The commissioning party must present an inspection plan to the builder during commissioning.

(2) Items to be inspected are selected in line with the design drawing documents, and the inspection plan is created in advance, covering inspection methods, the period and frequency of inspections, pass/fail criteria, and other matters.

(3) The required performance of the structural object, its efficiency, any particularities of its construction, environmental conditions, and other factors must be taken into consideration when formulating the inspection plan.

Commentary: Regarding (1): At the time of commissioning, the commissioning party must present an inspection plan for every stage of concrete production, construction work, and completion of the concrete structural object. If changes are made to construction work plans, the inspection plan must be reviewed accordingly.

Regarding (2) and (3): The ultimate purpose of inspection is to confirm that a structural object has been constructed in accordance with its design drawings and that the required performance is ensured. Inspection planning is the act of determining inspection items, inspection methods, inspection period and frequency, inspection pass/fail criteria, and other matters. However, multiple inspection methods may be used to achieve these purposes. In such cases, the required performance of the structural object, its efficiency, any particularities of its construction, environmental conditions, and other factors must be considered.

It is important to make changes to inspections in line with factors including the reliability of the construction method. The frequency of inspections may be increased for construction methods that are more prone to human error and may be decreased for construction methods in which errors rarely occur in construction work processes.

The construction work capabilities of engineers and technicians are also important in considering the frequency of inspections. For example, persons holding ISO 9000 series qualifications, other certification holders, and persons who have demonstrated an outstanding record of achievements may serve as benchmarks for such judgments.

If quality control is deemed to have not been sufficiently implemented at a site, unannounced inspections may be worth considering, taking the efficiency of inspections into account. Depending on how they are used, unannounced inspections may lead to increased awareness of quality assurance on the part of the builder.

Special Concrete

Chapter 1 General Rules

1.1 General

"Materials and Construction: Special Concrete" deals with concrete and concrete structural objects for which materials, functions, construction work methods, construction work environment, structural forms, production methods, or other aspects are specialized, and presents standards concerning matters of particular necessity when performing such production or construction work.

Commentary: For various concrete structural objects to satisfy their required performance, concrete involving sp ecial materials, functions, construction work methods, structural forms, production methods, and other aspects may be used, or concrete construction work may be performed under special construction work environments.

Concretes that are made with special materials or that have special functions include expansive concrete, lightweight aggregate concrete, short fiber reinforced concrete, and high-strength concrete. Concretes that involve special construction work methods include flowing concrete, high-fluidity concrete, and sprayed concrete. Concretes that involve special construction work environments include marine concrete and underwater concrete. Concretes that involve special structural forms or production methods include prestressed concrete, precast concrete, and factory products. Of these concretes, the summary version of these Standard Specifications presents standards for particularly necessary matters concerning the production of high-fluidity concrete and construction work using the concrete.

Chapter 2 High-Fluidity Concrete

2.1 General

(1) This chapter presents standards concerning particularly necessary matters in the mix design, production, and construction work involving self-consolidating high-fluidity concrete.
(2) In construction work involving high-fluidity concrete, the materials and mix must be selected and the production, construction work, and quality control methods must be appropriately determined under the guidance of engineers who have sufficient knowledge of and experience with high-fluidity concrete to ensure that the required self-compactability is achieved.

Commentary: Regarding (1): High-fluidity concrete is concrete in which fluidity has been enhanced without impairment of the material separation resistance of the fresh concrete. **Figure C2.1.1** shows the relationship between the fluidity of fresh concrete and the degree of compaction required for filling, comparing normal concrete and high-fluidity concrete. The fluidity of high-fluidity concrete is generally controlled by slump flow. Depending on the presence or absence of compaction, however, it is categorized as either high-fluidity concrete that exhibits self-compactability (i.e., the ability to uniformly fill the entire volume inside formwork under its own weight without compaction), or high-fluidity concrete that requires light compaction in accordance with the construction work conditions of the structural object. This chapter deals with high-fluidity concrete that exhibits self-compactability (hereinafter referred to as "high-fluidity concrete"). The properties of high-fluidity concrete differ significantly from those of ordinary concrete, particularly in the case of fresh concrete. The production of the concrete is of particular importance in fully obtaining said properties, with many points to be noted. Accordingly, this chapter presents standards for particularly important matters concerning the selection of materials, mix design, production, construction work, quality control, and inspection when using high-fluidity concrete.

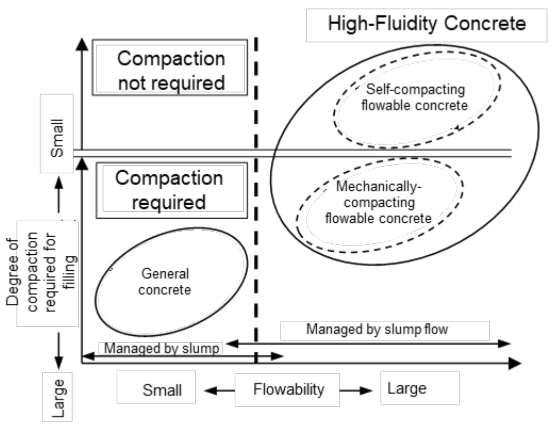


Figure C2.1.1 Relationship between flowability and degree of compaction required for filling

2.2 Quality of high-fluidity concrete

2.2.1 General

High-fluidity concrete must exhibit the required self-compactability as well as the required post-hardening quality, with little unevenness in quality.

2.2.2 Self-compactability

(1) Self-compactability in high-fluidity concrete must be set appropriately with consideration of the form and dimensions of the structural object in which placement will occur, as well as the arrangement of rebars within the structural object.

(2) In principle, the level of self-compactability should be selected from the following three ranks.

Rank 1: The level at which uniform filling is possible under the concrete's own weight alone in structural members or in locations with complex cross-sectional forms and small cross-sectional dimensions, with minimum gaps of about 35 to 60 mm in steel material.

Rank 2: The level at which uniform filling is possible under the concrete's own weight in reinforced concrete structures or structural members, with minimum gaps of about 60 to 200 mm in steel material.

Rank 3: The level at which uniform filling is possible under the concrete's own weight alone in structural members, locations, or non-reinforced concrete structures with a large cross-sectional dimension and a small amount of rebars, with minimum gaps of no less than about 200 mm in steel materials.

(3) When high-fluidity concrete is used for ordinary reinforced concrete structural objects or structural members, Rank 2 is set as the standard self-compactability level.

(4) In principle, evaluation methods for self-compactability are to follow JSCE-F 511 "Test methods for compactability of self-compacting concrete."

(5) Characteristic values that satisfy each rank of self-compactability are to be in accordance with **Table 2.2.1**.

Table 2.2.1 Characteristic values that satisfy each rank of self-compactability

Evaluation method		Self-compactability rank		
		1	2	3
JSCE F-511	Obstacles condition	R1	R2	No obstacles
	Filling height	over 300	over 300	over 300

Commentary: Regarding (2) and (3): Ranks 1 to 3 were set as levels of self-compactability in high-fluidity concrete, based on the cross-sectional form, dimensions, and rebar arrangement conditions of the structural objects or structural members.

In general, for reinforced concrete structural objects and structural members, Rank 2 should be adopted as the standard level of self-compactability for high-fluidity concrete used in structural objects and structural members, with minimum gaps of about 60 to 200 mm in steel material and with an amount of steel material of about 100 to 350 kg/m^3. In addition, when high-fluidity concrete is applied to the filling of steel shells in steel-concrete composite structures or the filling of narrow spaces as in the case of reverse-cast concrete, the level of self-compactability should be set to Rank 2 or Rank 1, regardless of rebar arrangement conditions.

Regarding (4) and (5): The evaluation of self-compactability in high-fluidity concrete should be in accordance with "Test methods for compactability of self-compacting concrete," which sets flow obstruction conditions for each rank. **Figure C2.2.1** shows the shape of the filling apparatus and flow obstruction. In this case, the characteristic values that satisfy levels of self-compactability for each rank are shown in **Table 2.2.1**. In the case of Rank 1 or Rank 2, when evaluation of self-compactability through testing is deemed difficult, consolidation experiments using model specimens should be performed as necessary.

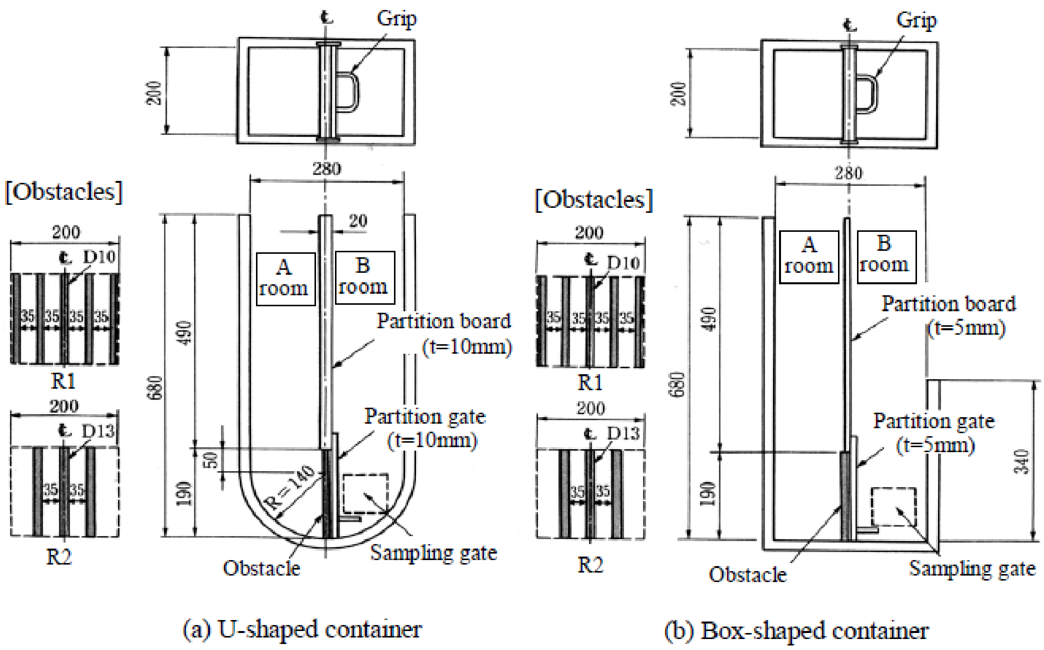


Figure C2.2.1 Shapes and dimensions of apparatuses for self-compactability test

2.3 Materials

(1) Materials used in high-fluidity concrete are to conform to JIS or to the standards of the Japan Society of Civil Engineers.

(2) High-performance AE water-reducing agents or high-performance water-reducing agents that conform to JIS A 6204 "Chemical admixtures for concrete" are to be used.

(3) When materials other than (1) or (2) are used, reliable documents and testing must be used to confirm that concrete made with those materials satisfies the required self-compactability and post-hardening concrete quality.

Commentary: Regarding (3): Fine limestone powder and thickening agents are examples of materials for which quality standards are not stipulated by the JSCE standards but which have a record of use in high-fluidity concrete.

2.4 Mix proportions

2.4.1 General

An appropriate mix of high-fluidity concrete is to be selected from powder-based high-fluidity concrete, combination high-fluidity concrete, and thickened high-fluidity concrete, with consideration to the type of structural object, structural conditions, construction conditions, types of available materials, constraints on the concrete production plant, *etc*.

Commentary: Powder-based high-fluidity concrete is concrete in which the powder content has been increased as necessary without the use of thickening agents to ensure material segregation resistance suitable for the high fluidity imparted by the addition of a high-performance AE water-reducing agent. Combination high-fluidity concrete is concrete in which a thickening agent has been added to powder-based high-fluidity concrete to reduce quality inconsistency in fresh concrete caused by factors including measurement errors and quality fluctuations in materials. This enables the relatively easy manufacture and construction of high-fluidity concrete with stable self-compactability. Thickener-based high-fluidity concrete is concrete in which material segregation resistance has been enhanced by a thickening agent.

2.4.2 Mix design

The mix design of high-fluidity concrete must exhibit the required fluidity, material segregation resistance, and self-compactability for the structural conditions, construction work conditions, and environmental conditions of the structural object, so as to enable strength and other aspects of quality in the hardened concrete.

Commentary: In general, the mix design of high-fluidity concrete is to follow the procedures below. The type of high-fluidity concrete is first selected. **Tables C2.4.1 to C2.4.3** show target ranges for the evaluation metrics for self-compactability ranks of powder-type, combination, and thickened high-fluidity concrete. Among the ranges of target values for properties shown in the tables, the range for slump flow is a range of set values for use in selecting mixes and differs from the allowable range of variation in slump flow for use in quality control and inspection during construction work. Time to reach 500 mm flow and funnel efflux time should be set to satisfy any of the evaluation metrics.

Next, to ensure the required self-compactability, the maximum size of the coarse aggregate and the absolute volume of unit coarse aggregate (i.e., the absolute volume of coarse aggregate used to make $1m^3$ of concrete) are set. Although the maximum size of coarse aggregate is typically 20 mm or 25 mm, when stricter pore permeability is required, coarse aggregate with a smaller maximum size may be used. The absolute volume of unit coarse aggregate must achieve the required self-compactability to an extent that does not impair the quality of the hardened concrete. **Table C2.4.4** shows the standard range of volume of coarse aggregate for self-compactability ranks of powder-type, combination, and thickened high-fluidity concrete.

Following this, the unit water content, water-to-powder ratio, unit powdery material content, and added amount of high-performance AE water-reducing agent are set. The water-to-binder ratio, unit binder content, unit fine aggregate content, and amount of admixture to be used are further determined, and the fresh properties of the concrete mix determined from these is checked for the required fluidity, material separation resistance, and self-compactability.

Tables C2.4.1 Target ranges for the evaluation metrics for self-compactability ranks（Powder type）

Rank of self-compactability		1	2	3
Filling height by the box-shaped container or U-shaped container		over 300 （R1）	over 300 （R2）	over 300 （No obstacles）
Target values	Slump flow[1]（mm）	700	650	600
	500 mm flow time[2] (s)	5～20	3～15	3～15
	Funnel flow time of V_{75} funnel or O funnel[2] (s)	9～20	7～13	4～11

1) General range of targeted slump flow by past record is as follows; Rank 1: 650~750mm, Rank 2: 600~700mm, Rank 3: 550~650mm.
2) It shows general range of targeted values by past record of evaluation methods.

Tables C2.4.2 Target ranges for the evaluation metrics for self-compactability ranks（Combination type）

Rank of self-compactability		1	2	3
Filling height by the box-shaped container or U-shaped container		over 300 （R1）	over 300 （R2）	over 300 （No obstacles）
Target values	Slump flow (mm)	700	650	600
	500 mm flow time[2] (s)	5～20	3～15	3～15
	Funnel flow time of V_{75} funnel or O funnel[2] (s)	10～25	7～20	7～20

1) General range of targeted slump flow by past record is as follows; Rank 1: 650~750mm, Rank 2: 600~700mm, Rank 3: 550~650mm.
2) It shows general range of targeted values by past record of evaluation methods.

Tables C2.4.3 Target ranges for the evaluation metrics for self-compactability ranks（Viscosity agent type）

Rank of self-compactability		2	3
Filling height by the box-shaped container or U-shaped container		over 300 （R2）	over 300 （No obstacles）
Target values	Slump flow (mm)	650	600
	500 mm flow time[2] (s)	3～15	3～15
	Funnel flow time of V_{75} funnel or O funnel[2] (s)	7～20	7～20

1) General range of targeted slump flow by past record is as follows; Rank 2: 600~700mm, Rank 3: 550~650mm.
2) It shows general range of targeted values by past record of evaluation methods.

Tables C2.4.4 Standard range of the absolute volume of unit coarse aggregate

Rank of self-compactability	the absolute volume of unit coarse aggregate（m^3/m^3）		
	Powder type	Combination type	Viscosity agent type
1	0.28 ～ 0.30	0.28 ～ 0.30	—
2	0.30 ～ 0.33	0.30 ～ 0.33	0.30 ～ 0.33
3	0.33 ～ 0.35	0.33 ～ 0.35	0.33 ～ 0.35

2.4.3 Trial mixing

Trial mixing must be performed to confirm that the high-fluidity concrete satisfies the specified required quality.

Commentary: Trial mixing must be used to confirm that high-fluidity concrete for which mix design has been determined satisfies the specified self-compactability. High-fluidity concrete that satisfies self-compactability should also be confirmed to fall within the ranges for slump flow, time to reach 500mm flow, and funnel efflux time that were set in the mix design. If this confirmation reveals that the required performance is satisfied, the confirmed values are to be used as target values for quality control and inspection during construction work. If the required performance is not satisfied, the selection of materials and the mix design must be performed again and the trial mixing must be repeated. When doing so reveals that the concrete mix design cannot be practically achieved, the construction work conditions must also be reviewed.

2.5 Production

2.5.1 Selection of a plant

Plants that manufacture high-fluidity concrete must be plants approved to use the ㊜ symbol or that have equivalent or better production facilities and management systems. Consideration must also be given to factors including transport time to the construction site, concrete shipping capacity, and quality control capabilities.

Commentary: Self-compactability of high-fluidity concrete is susceptible to fluctuations in material quality and the environment. Therefore, the production of high-fluidity concrete of stable quality requires stricter quality control than is required for ordinary concrete.

The quality of high-fluidity concrete is particularly susceptible to fluctuations in the percentage of surface moisture and in the distribution of particle size. Therefore, it is advisable to select a plant that implements storage and management methods that minimize these fluctuations.

2.5.2 Storage of aggregates

Aggregates must be stored in a manner that minimizes fluctuations in percentage of surface moisture.

Commentary: To produce high-fluidity concrete of a specified quality, it is important to secure adequate storage capacity to prevent switching of aggregate lots while production is in progress. It is also important to reduce fluctuations in the percentage of surface moisture of aggregate, particularly fine aggregate.

When the percentage of surface moisture of aggregate is large, it is prone to non-uniformity. Therefore, it is advisable to control the percentage of surface moisture to 5% or less in fine aggregate and to 1% or less in coarse aggregate. To minimize the percentage of surface moisture of aggregate and to make the percentage uniform, it is advisable to store aggregate in a storage facility for several days prior to production.

2.5.3 Mixing

(1) A batch cycle forced action mixer is set as the standard for mixing high-fluidity concrete.

(2) The mixing method for high-fluidity concrete must be determined on the basis of testing or past results.

(3) The amount of high-fluidity concrete to be mixed per batch is to be determined with consideration of factors including the type of high-fluidity concrete and the mixing performance of the mixer, with at most 80% to 90% of the mixer's maximum capacity set as the standard.

(4) The mixing time per batch of high-fluidity concrete is to be determined in consideration of factors including the type of high-fluidity concrete and the mixing performance of the mixer, with a time of no less than 90 seconds set as the standard in the case of a forced action mixer.

Commentary: Regarding (1): High-fluidity concrete tends to exhibit greater viscosity than ordinary concrete. Therefore, the use of a batch cycle forced action mixer with high mixing performance was set as the standard. In general, a horizontal twin-screw forced action mixer, which enables shortened mixing time and also has a short discharge time, should be used.

Regarding (2): High-fluidity concrete has a higher mixing load than normal concrete. In particular, when coarse aggregate is added, excessive load is applied, potentially causing the mixer to stop. Therefore, care must be taken in the order in which materials are added. In addition, the powder material dispersion effect caused by high-performance AE water-reducing agents is affected by factors including the type of powder material used, the quality of the aggregate, the order in which materials are added, and the mixing performance of the mixer. Therefore, the mixing method must be determined appropriately through testing or with reference to past results.

2.6 Construction work

2.6.1 General

Construction work must be performed on the basis of construction work plans with consideration of the properties of the high-fluidity concrete, in order to ensure that placement is completed within the specified time.

Commentary: The standard construction work conditions assumed for high-fluidity concrete are a maximum placement free fall height of no more than 5 m, a maximum horizontal flow distance of no more than 8 to 15 m, a flow gradient of 1/10 to 1/30, and, as a transfer condition, a horizontal conversion distance of no more than 300 m in a 4- or 5-inch pipe. Properties of high-fluidity concrete include a higher viscosity than ordinary concrete, which causes higher resistance to pumping; high fluidity, which increases lateral pressure on formwork; and low bleeding, which facilitates drying of the concrete surface after placement. Construction work plans must be formulated to allow the completion of placement within the specified time while maintaining self-compactability, with an understanding that material separation tends to occur more readily when the flow distance is great.

2.6.2 Transport and placement

(1) To complete placement within the time that enables the maintenance of the quality of high-fluidity concrete, the concrete must be transported to the site within an appropriate amount of time, taking the time required for placement into consideration.

(2) When concrete pumps are used for on-site transport, a pumping plan that covers the selection of models and numbers of concrete pumps, *etc*. and a piping plan that covers the diameter of transport pipes, piping routes, piping distances, *etc*. must be proposed, with consideration of the quality of the high-fluidity concrete, pumping conditions, workability, and safety.

(3) The placement speed of the high-fluidity concrete must be determined with consideration of factors including the mix of the high-fluidity concrete, the forms of the structural members, and the arrangement of rebars.

(4) In principle, the free fall height of high-fluidity concrete should be no more than 5 m.

(5) The horizontal flow distance of high-fluidity concrete should be no more than 8 m from the placement location in the case of placement over a wide level area, or no more than 15 m from the placement location in the case of placement from one side into a structural member that is long in one direction with a small cross-section.

Commentary: Regarding (1): Even when the required self-compactability has been secured from mixing until the end of placement, the properties of high-fluidity concrete may vary due to differences in the type and amount of high-performance AE water-reducing agents, the mix, atmospheric temperature, and concrete temperature. In other words, as there are cases in which the time over which required quality can be maintained is shortened, or in which slump flow gradually increases after mixing and additional time is required to achieve the required self-compactability, the properties of the selected mix must be thoroughly confirmed.

Regarding (2): High-fluidity concrete has a larger unit weight of powder than concrete, and therefore tends to result in a greater pressure loss in the pipe as the pump discharge rate increases. Therefore, it is advisable to make the transport pipe diameter larger than usual and to select a concrete pump that has a greater theoretical discharge pressure. Regarding the effects of pumping on quality, pumping often results in decreased fluidity and slump flow, as in ordinary concrete. However, depending on the mix and on environmental conditions, slump flow can conversely increase in some cases. Therefore, it is necessary to understand the tendencies of post-pumping quality fluctuations in advance and to take necessary countermeasures.

Regarding (3): Placement speed is generally slower than that of ordinary concrete. Based on examples from past construction, placement speed is often set to 10 to 40 m^3/h. For complicated structural members such as junctions, an appropriate placement speed must be set based on testing and past results, with attention paid to the state of filling.

Regarding (4): The free fall height for concrete was set to no more than 5 m in principle. However, it is advisable to consider construction work methods that minimize the free fall height.

Regarding (5): In high flowability concrete, material separation at the leading point tends to occur more readily as the flow distance increases. The maximum value of the flow distance before which significant material separation occurs varies with the form of the structural member. Flow distance was set in principle to no more than 8 m from the placement location in the case of placement over a wide level area, such as a slab, or no more than 15 m from the placement location in the case of placement from one side into a structural member that is long in one direction with a small cross section, such as a beam.

2.6.3 Finishing and curing

(1) Finishing must be performed at an appropriate time after the concrete has been leveled to the specified form and dimensions but before the surface stiffens.

(2) Measures must be taken to prevent surface drying so that plastic shrinkage cracking does not occur.

Commentary: Regarding (2): High-fluidity concrete bleeds less than ordinary concrete. Therefore, it tends to be susceptible to plastic shrinkage cracking associated with rapid surface drying.

2.6.4 Construction joints

Treatment of construction joint surfaces may be simplified when using high-fluidity concrete if the construction joints can be confirmed to possess the required quality.

2.6.5 Formworks

(1) In principle, the lateral pressure of high-fluidity concrete acting on formwork is to be treated as liquid pressure in design.

(2) In principle, when placement high-fluidity concrete into a closed space, air holes should be provided at appropriate positions in the formwork.

(3) When many air bubbles remain in the concrete surface, careful consideration must be given to the material of the end-plates and to the type of peeling agent.

Commentary: Regarding (1): High-fluidity concrete exhibits excellent fluidity, and therefore acts in a manner nearly identical to liquid pressure. In addition, it settles slowly, which causes its lateral pressure to act for a longer time than that of ordinary concrete after the completion of placement. Force acts evenly on separators by design, which means that unevenness in separator clamping force is unsafe. Therefore, it is advisable to properly control the precision of the formwork assembly and the clamping force of the separators, while also increasing the safety factor in the design beyond the usual as necessary.

2.7 Quality control

To achieve the required self-compactability, satisfaction of required quality by the concrete must be controlled by appropriate means at every stage from mixing to placement.

Commentary: Quality control during production includes visual checks, as well as measurement of slump flow, time to reach 500 mm flow or funnel efflux time, air volume, and concrete temperature. The power consumption value of the mixer during mixing is also a metric to be managed. During unloading, the self-compactability should be checked, visual checks are to be performed, and slump flow, time to reach 500 mm flow or funnel efflux time, air volume, concrete temperature, *etc*. are to be measured.

2.8 Inspection

Acceptance inspections of high-fluidity concrete are to be conducted in accordance with **Table 2.8.1**.

Table 2.8.1 Inspection of high-fluidity concrete

Item	Testing method	Period・Frequency	Criteria for judgement
Self-compactability	In accordance with JSCE-F 511 Applying the flowing obstacles corresponding to each rank	・At unloading location ・At least once for each 50~150m^3 corresponding to construction scale	Filling height is not less than 300 mm

Commentary: Concrete for which inspection has failed and in which material segregation has occurred should be discarded and not poured. However, when decreased fluidity of fresh concrete is the reason for failure to pass self-compactability inspection, high-performance AE water-reducing agents, *etc*. may be added within a predetermined range, with inspections then performed again. In either case, the causes of the failure must be identified and appropriate measures must be taken to effect quick improvements in subsequent production.

Maintenance

Standard Specifications for Concrete Structures -2018

“Maintenance”

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Main Volume

Chapter 1 General Rules

1.1 Scope of application

(1) "Maintenance" within Standard Specifications for Concrete Structures is to be applied to the maintenance of concrete structures.
(2) "Maintenance: Main Volume" describes principles for the maintenance of concrete structural objects.
(3) "Maintenance: Standards" describes standard methods for the maintenance of concrete structures.
(4) "Maintenance: Standards Appendix" describes particulars for specific types of deterioration mechanisms, structures, and structural members, and particulars to be used as reference when making changes to the required performance level.

Commentary: Regarding (1): "Maintenance" in Standard Specifications for Concrete Structures presents principles and general techniques related to maintenance actions conducted to maintain the performance of concrete structures (hereafter "structures") at or above a required level throughout their period of service. To maintain the performance of a structure at or above the required level, it is necessary that not maintenance alone, but design, construction work, and maintenance be performed in a coordinated manner. Therefore, it is advisable to comprehensively consider the coordination of design, construction, and maintenance in accordance with "General Principles."

"Maintenance" addresses unreinforced concrete, reinforced concrete, and prestressed concrete, without focusing on structures constructed according to specific structural forms, design methods, or construction methods. In other words, "Maintenance" is also able to address structures that have not been designed and constructed in conformance with "Design" and "Construction".

"Maintenance" addresses both new and existing structures. For new structures designed and constructed in accordance with "Design" and "Construction", it is unlikely that significant deterioration will occur and significantly affect the performance of the structure during the design service period. However, it cannot be denied that new and existing structures that were not designed and constructed in accordance with "Design" and "Construction" of the Standard Specifications may include those in which inappropriate materials were used or for which inadequate consideration was given to durability. Moreover, initial defects may be overlooked in post-completion inspections or may not have been adequately repaired when the structure enters use. Even if these situations do not occur, there are cases in which the performance is not quantitatively verified with respect to all envisioned actions that will affect the structure or in which environmental conditions are more severe than expected owing to factors such as climate change, with deterioration over time manifesting more quickly than expected and performance of the structure undergoing a decline.

For these reasons, it is necessary in maintenance to accurately assess the anomaly of the structure through means such as inspections to clearly determine whether such an anomaly is primarily due to initial defects, damage, or deterioration, and to represent appropriate remedial measures. When deterioration that affects changes in the performance of the structure over time becomes apparent, it is necessary to clarify the mechanisms of deterioration to the extent possible, predict the progress of deterioration, evaluate performance, determine the necessity of remedial measures, and, when necessary, implement appropriate remedial measures.

"Maintenance" addresses unitary structures but not directly the maintenance of groups of structures. The maintenance of actual structures commonly involves management of multiple structures, including the one targeted for maintenance. Optimizing such individual cases of maintenance can be difficult due to institutional or budgetary constraints. In such cases, the assessments presented in Chapter 3 are to be first carried out for each structure. From the findings, the necessity of remedial measures can be determined. When remedial measures targeting numerous structures overlap nearly simultaneously, the order of priority must be considered in implementing the remedial measures. In doing so, not only anomalies but also the importance of the structure, the remaining planned service period, economic performance, and other factors are taken into consideration, and the situation is handled by advancing or delaying the implementation of remedial measures for structures or structural members. To address concerns over performance progressively deteriorating and falling below the required level, sufficient care must be taken when delaying the implementation of remedial measures.

Regarding (2)–(4): "Maintenance" volume of the Standard Specification consists of three volumes. "Maintenance: Basics" describes general basic matters for maintenance, clarifies the structure of "Maintenance" volume of the Standard Specification, and indicates the method of its use. "Maintenance: Standards" describes standard approaches, methods, and procedures for comprehensively considering initial defects, damage, and deterioration and for implementing rational maintenance. "Maintenance: Standards Annexes" describes particulars to be used as reference when making changes to the required performance level, or with respect to each deterioration mechanism, structures, and structural members in the implementation of maintenance, based on the content of "Maintenance: Standards." While not included in this English version of "Maintenance" volume of the Standard Specification, examples of performance evaluation drafts and maintenance based on "Maintenance" are included as an appendix at the end of the original "Maintenance" volume of the Standard Specification.

1.2 Principles of maintenance

(1) The maintenance manager of structures shall formulate maintenance plans to maintain the performance of the structure at or above the required level throughout the planned service period, and shall appropriately perform maintenance following the construction of a specified system for maintenance.

(2) In the maintenance of structures, the required performance of structures must be made clear.

Commentary: Regarding (1): Taking into consideration the circumstances of a structure, the maintenance manager of the structure must appropriately predict changes over time in the performance that was set during design and must formulate a maintenance plan to maintain the performance of the structure at or above the required level during the planned service period. In accordance with this maintenance plan, the manager must then evaluate the performance of the structure during service, and, while implementing remedial measures as necessary, must maintain the performance of the structure at the ideal level in accordance with scenarios.

Maintenance of the structure will be performed continuously over a long period. During that process, situations in which greater-than-anticipated changes occur in social circumstances and the environment resulting in the utility value and importance of the structure diverging from initial assumptions, or situations in which the initial maintenance plan requires review in terms of securing the maintenance budget and personnel, are to be predicted. The maintenance manager must therefore build a maintenance system that is able to respond appropriately and flexibly to the above situations.

Figure C1.2.1 presents an example of a maintenance system for civil engineering structures. To properly carry out maintenance of a structure, whether new or existing, it is fundamental that an engineer who possesses the required abilities perform the work, following the preparation of structures for implementation and accountability.

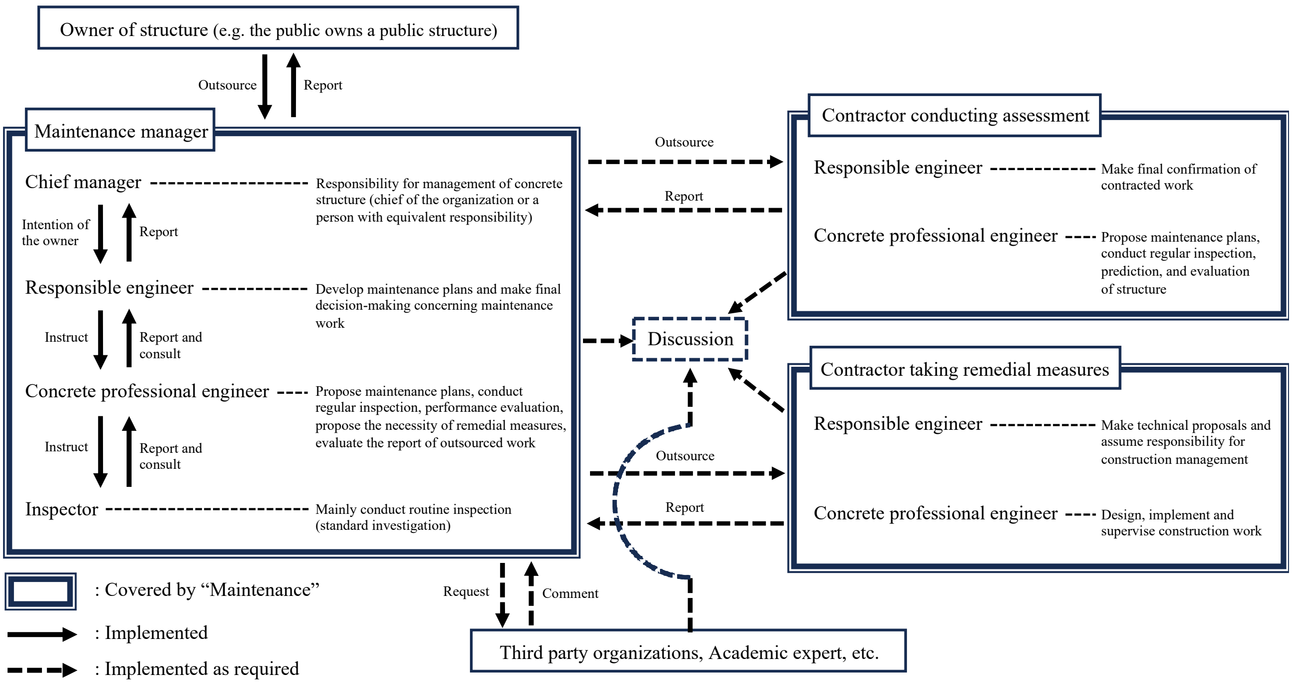


Figure C1.2.1 Example of maintenance system for a structure and the roles of engineers

At the same time, when it is necessary to outsource portions of the work to business operators with more specialized technical capabilities, a system must be adopted that allows the maintenance manager to confirm that responsible engineers who can responsibly perform the outsourced work are assigned within the outsourced business operator, and that also allows the maintenance manager to personally conduct final evaluations or render decisions on outcomes of the work.

Regarding (2): If the required performance of the structure is left ambiguous, maintenance will be inconsistent and symptomatic, making the realization of rational maintenance impossible. Therefore, clarifying the required performance of the structure is an essential requirement in carrying out maintenance. In maintenance, it must be confirmed that the performance of the structure is maintained at or above the required level set on the basis of required performance throughout the planned service period; if evaluation finds that this level is not maintained, remedial measures must be taken as necessary. As the required level of performance in this case is first set on the basis of the required performance considered in design and the required performance level set in verification, these must be made clear. However, as the required performance or the required performance level of the structure may diverge during service from what was set at the time of initial design, the required level of performance must be adjusted appropriately as need arises.

For many existing structures, design drawings and other records cannot be used and design specifications are not clear. Even for existing structures and new structures for which design drawings and other records can be used, depending on the design standards that were applied, the required performance level may not be clearly indicated. Therefore, in the maintenance of these structures, it is necessary to reconfirm and clarify required performance in accordance with current thinking.

For civil engineering structures that have an in-service period of 50 to 100 years or longer, there are cases in which remedial measures will have to be taken because of changes in the magnitude of actions that should be considered in design or changes in the limit values of performance, in accordance with reviews of seismic actions, increased wheel load on bridge due to the traffic larger vehicles, or other changes in societal circumstances. These mainly surface as problems by which structures and structural members designed and constructed according to previous standards fail to conform to current design standards following the revision of design standards or the establishment of new standards. Here, the revised standards or newly established standards address only structures designed and constructed after the revision or establishment; structures designed and constructed prior to that are not required to satisfy those standards. However, revision and establishment of standards are performed for the purpose of improving matters for consideration in the design and construction of structures to a state in line with current conditions. Therefore, given the highly public nature of civil engineering structures, serious damage, decay, collapse, or lack of functionality stemming from noncompliance with current standards should be avoided. It is advisable to appropriately address revisions or establishment of standards so that structures in service are as compliant as possible.

1.3 Definitions of terms

In "Maintenance," terms are defined as follows.

Maintenance：All actions for maintaining the performance of the structure above the required level during its service period.

Required performance：The performance required of the structure according to its purpose and function.

Maintenance category : The level of maintenance to be set based on the basic concept in the maintenance of structures.

Maintenance limit : Performance or degree of deterioration (appearance grade), etc. to be set as a limit in maintenance.

Planned service period：The period during which the structure is scheduled to be in service.

Design service period：The period during which the structure or member should fully achieve its objective functions as specified in the design phase.

Remaining planned service period：The period between the time of inspection or review and the end of planned service period.

Remaining design service period：The period between the time of inspection or review and the end of design service period.

Durability：The ability of a structure to remain safe, serviceable, and restorable over its planned service period.

Safety：The ability of a structure for keeping the life or property of the user or of a person in the vicinity free from any threat under all possible actions.

Serviceability：The ability of a structure to be used normally under the assumed actions during normal use.

Restorability：The ability to restore the performance of a structure that has been degraded by seismic effects or other accidental actions and to enable its continued use.

Hazards for third party：Impacts (including damage) on people or property caused by concrete pieces falling off the structure. Included in safety.

Appearance：The ability to ensure that deformations, discolorations, etc. on the surface of a structure do not cause uneasiness or discomfort to the surrounding environment and do not interfere with the use of the structure. Included in serviceability.

Action：All actions that cause an increase or decrease in the stress or deformation of a structure or member or a change in material properties over time.

Anomaly：An abnormal condition occurring in concrete or concrete structures due to various reasons, deviating from their intended state. A generic term for initial defects, damage, deterioration, etc.

Initial defect : Anomalies that occurred during construction, such as cracks, honeycombs, cold joints, and sand streaks, that could be harmful. Insufficient cover, insufficient PC grouting, etc. are also included.

Damage：Anomalies, such as cracking or delamination caused by an earthquake or impact, that occur within a short period of time and do not subsequently progress over time.

Deterioration：Anomalies that progress over time.

Assessment：A generic term for a series of actions taken in maintenance to determine the presence or absence of anomalies in a structure or member and to determine its condition, including inspection, deterioration prediction, evaluation, and judgment.

Inspection：A generic term for the action of investigating the presence or absence of factors that cause anomaly or anomalies in a structure or member, or the degree of such anomalies, in the assessment.

Initial inspection：Inspection that is made at the start of maintenance mainly to identify the initial conditions of the structure.

Routine inspection：Inspection made periodically once every few days to once a week, and is composed mainly of simple investigations including visual observation.

Regular inspection：Inspection made periodically once every few years to identify the conditions of the structure or member that cannot be confirmed by investigations in routine inspection.

Extraordinary inspection：Inspection made to identify the damage conditions of the structure caused by accidental actions immediately after encountering a great earthquake or typhoon, or by artificial accidental actions at the time of a crash of vehicles or ships or fire.

Emergency inspection：Inspection made urgently after an accident or damage greatly affecting a structure to verify whether or not similar structures or structures under similar conditions are subject to a similar accident or damage.

Standard investigation：A specified package of investigation items in the maintenance plan.

Detailed investigation：Any investigation to collect detailed data not obtained in standard investigation.

Monitoring：Identifying the conditions of the structure or member by installing sensors in the structure or member.

Remedial measure：These are implemented when deterioration progresses or performance declines in a structure, and include inspection strengthening, repair, retrofit, limitation of service, and demolition/removal.

Repair：A remedial measure to remove the hazards for third party or restore or improve appearance or durability. Included are the remedial measures to restore mechanical performance in terms of safety and serviceability to the level available to the structure at the start time of service.

Retrofit：A remedial measure to increase the mechanical performance in terms of safety and serviceability above the level available to the structure at the start time of service.

Chapter 2 Required Performance

2.1 General rules

(1) The required performances for the maintenance of a structure are generally safety, serviceability, restorability, hazards for third party, appearance, and durability.
(2) "Safety" addresses performance as determined by the dynamics of a structure, such as failure or collapse due to variable actions or accidental actions such as earthquakes.
(3) "Serviceability" addresses performance that allows normal use of a structure under actions expected during ordinary use.
(4) "Restorability" addresses performance that enables restoration of the performance of a structure that has declined due to accidental actions such as earthquakes, enabling continuous use.
(5) "Hazards for third party" addresses resistance to harm to users of a structure originating in the object, such as spalling of cover concrete lumps or the noise that is caused while the structure is in service, or public damage to third party.
(6) "Appearance" addresses performance by which anomaly, dirt, etc. arising on the surface of a structure do not cause uneasiness or discomfort to surrounding persons and do not interfere with the serviceability of the structure.
(7) "Durability" addresses performance by which a structure maintains safety, serviceability, restorability, hazards for third party, and appearance over the planned service period.

Commentary: Regarding (1): In "Maintenance," performance required of general structures are broken down into safety, serviceability, restorability, hazards for third party, appearance, and durability, in accordance with "General Principles" and "Design." In the structural planning and design stage, the required performances are selected from among these according to the purpose and function of the structure. Therefore, the performance that was verified in design is commonly set as the target of maintenance as well; however, the priority of performance required of a structure may differ between design and maintenance. Moreover, environmental performance must be considered in the selection of construction methods, with details as presented in "General Principles."

Hazards for third party is included within safety in "Design," representing safety in aspects other than physical properties of the structure. In "Maintenance," however, maintenance involving the hazards for third party is described separately from safety at times as necessary because it is often emphasized in the maintenance of structures. Similarly, in "Design," appearance is included in serviceability, but, in "Maintenance," maintenance with respect to appearance is described separately from serviceability at times as necessary.

Regarding (2): Safety was set in the same manner as that in "Design." Safety based on physical properties of a structure includes safety related to sectional fracture, safety related to fatigue failure, and safety related to the stability of the structure. In each case, evaluation is performed with respect to all loads, environmental effects, etc., that may act during service.

Regarding (3): Serviceability is set in the same manner as "Design," addressing performance with respect to comfort and various functions in the operation of the structure. Comfort in use includes ride comfort, pedestrian comfort, appearance, noise, vibration, and other performances. When aspects such as noise and vibration are considered particularly important, they may be described as degrees of effect on third parties, separate from serviceability. The same applies to appearance.

Performance with regards to various functions generally addresses performance of physical shielding and permeability aspects such as watertightness, permeability, sound proofing, moisture proofing, cold or heat resistance, and fire resistance, as evaluated in maintenance. However, many of these performances have properties that change from their initial levels over time. Among performance with respect to various functions, the number of road bridge traffic lanes able to handle the actual traffic volume is an example of a possible target of maintenance. These typically change in response to social circumstances.

Regarding (4): As set out in "General Principles," for structures it is considered practical and rational to replace restorability with safety, serviceability, or repairability—that is, the ease of repairing damage to the structure—in evaluations.

Regarding (5): Hazards for third party refers to the user-facing safety or resistance of a structure to public damage to third parties due to actions such as the spalling of cover concrete. Hazards for third party also includes problems such as noise that affect the surrounding people associated with operation of the structure.

Regarding (6): Anomalies such as dirt, rust, and cracks may occur in a structure, and the state of the appearance of structure may sometimes be important in the determination of remedial measures in maintenance. Therefore, it was decided to address this separately.

Regarding (7): Durability is a performance metric for comprehensively determining whether safety, serviceability, restorability, hazards for third party, and appearance can be maintained at or above the required levels during the planned service period. Therefore, if changes over time in these performances can be verified with sufficient accuracy, it is not necessary to consider durability as a performance indicator. However, technology for prediction of deterioration and evaluation of performance cannot be considered to have sufficiently matured to this level. Therefore, "Maintenance" uses durability because it is widely and generally used as a blanket representation of the state of these performances during the planned service period.

Considering durability as required performance involves the matter of how to set the planned service period, or how to set the levels that should be satisfied by safety, serviceability, restorability, hazards for third party, and appearance during that period. Durability can be verified by assessing the performance of the structure during inspection and predicting the degree of future decline of these performances.

Chapter 3 Maintenance Methods

3.1 General rules

The maintenance manager for a structure shall formulate a maintenance plan such that the performance of the structure is maintained at or above the required level throughout its planned service period; based on the plan, the maintenance manager shall conduct assessment of the structure and implement remedial measures according to the results, while properly recording the results of this series of tasks.

Commentary: Maintenance of a structure is a collective term for actions taken for the purpose of maintaining the performance of the structure at or above the required level during the planned service period. Maintenance consists of inspection, estimation of mechanisms of deterioration, prediction of the progress of deterioration or decline in performance, assessments that consist of evaluation of the performance of the structure and determination of the need for remedial measures, remedial measures implemented as necessary based on the results of assessments, and keeping of records of these matters. To properly perform this series of tasks, the structure’s maintenance manager must formulate a maintenance plan that sets out the content of the tasks in advance. It is also necessary to review the maintenance plan as necessary to confirm that it reflects the actual circumstances.

3.2 Maintenance plan

(1) The foundation of a maintenance plan is the indication of assessment methods consisting of inspection, prediction, performance evaluation, judgment of the need for remedial measures, and other matters for each targeted structure or structural member in accordance with the maintenance category of the structure and the estimated mechanisms of deterioration, along with the indication of methods for selecting remedial measures, methods of recording, etc., as well as the setting of the maintenance limits that serve as the standards for determining the necessity of remedial measures.

(2) The maintenance plan is to be reviewed as necessary.

Commentary: <u>Regarding (1)</u>: A maintenance plan presents the outcomes of comprehensively planning the time, frequency, methods, and systems (organizations, personnel, budget, etc.) for the implementation of assessments, remedial measures, recording, etc., taking into account the circumstances of the structure.

In formulating a maintenance plan, it is important to first set the maintenance category that forms the basic policy for maintenance in terms of how maintenance of the structure will be implemented. The maintenance category should be set at the design or planning stage from a standpoint of ensuring the performance of the structure through cooperation among design, construction, and maintenance. Approaches at this time are presented in "General Principles." If the maintenance category for an existing structure is not clear, it must be set prior to the initial assessment that will be implemented based on "Maintenance."

In the maintenance plan, details of investigations to be performed in inspection (their method, frequency, etc.), methods of prediction and performance evaluation for a deteriorated structure, maintenance limits (criteria for determining the need for remedial measures), methods of recording, and so on are concretely set. The methods, scale, time, and order of implementation for remedial measures to address expected future deterioration must also be indicated. Studying the assessment methods and the effects of remedial measures assumed in advance can enable the formulation of a maintenance plan that reduces life cycle cost and achieve more efficient maintenance. When formulating a maintenance plan in the planning and design stages of a new structure, it is effective to create a plan linked to the design of the structure to enable the implementation of more rational maintenance.

Regarding (2): For a new structure, it is advisable to formulate the maintenance plan in the structural planning and design stages, collect information on the structure in the initial assessment following completion of construction to confirm the validity of the plan, and, after reviewing the content of the information as appropriate for conditions, make final decisions concerning the maintenance plan. Even when implementing maintenance for an existing structure based on "Maintenance," the initial assessment is to be incorporated into the maintenance plan prior to said implementation, and review and final decision concerning the maintenance plan are to be carried out based on information on the structure obtained through actual initial inspection. After the implementation of major remedial measures or when the results of prediction of the progress of deterioration based on inspection findings differ from initial predictions, it is important to review the executed maintenance plan as necessary. As the service period of a structure extends over a long time, the required performances of the structure are likely to change during that period, possibly necessitating an accompanying review of the maintenance plan.

3.3 Assessment

3.3.1 General

(1) In the assessment of a structure, inspection shall be carried out according to the maintenance plan. Based on the results, confirmation of the state of deterioration, identification of deterioration mechanisms, prediction of the progress of deterioration, evaluation of the performance of the structure and judgment of the need for remedial measures shall be done appropriately.

(2) Assessments include initial, periodic, and extraordinary assessments. Assessments in line with the objectives of each of these shall be carried out by engineers who possess sufficient knowledge and experience based on assessment plans that are defined during the formulation of the maintenance plan.

Commentary: Regarding (1): Assessment includes inspection, identification of deterioration mechanisms, prediction of the progress of deterioration, evaluation of performance, and determination of the necessity of remedial measures. It is a collective term for a series of actions for the purpose of investigating the presence or absence of anomalies in structures and structural members during maintenance and determining their condition. To implement appropriate and systematic maintenance, inspection must first be performed to discern the condition of the structure at that point in time. Based on the results, it is necessary to estimate the deterioration mechanisms that are occurring or could occur and to predict the progress of deterioration in the future. Following this, based on the inspection findings and the results of prediction, the necessity of remedial measures is determined. In some cases, the performance of the structure can be predicted from the state of deterioration. If the relationship between the state of deterioration and the performance of the structure is clear, the performance of the structure may be predicted from the results of prediction of the progress of deterioration.

"Maintenance" considers deterioration to be an anomaly that progresses over time and distinguishes it from potentially harmful initial defects occurring during construction or damage that occurs over a short period and does not change significantly afterward, such as cracking or peeling due to earthquake or collision. Therefore, "Maintenance" shows maintenance methods that primarily target decline in the performance of structures due to deterioration. However, if some initial defects remain untreated in the structure, when damage occurs due to load action, etc., it will be necessary to evaluate how these factors affect the performance of the structure by assessment.

In cases where changes occur in the type and magnitude of expected actions or in the required performance of the structure during the service period due to revision of design standards, it may be necessary to perform assessments to evaluate the performance of the structure, review the maintenance plan so that maintenance can satisfy the new standards, and enact remedial measures, such as retrofitting, as required by the situation.

Regarding (2): In "Maintenance," assessments are broadly divided into the initial assessment first performed when carrying out maintenance, periodic assessments performed on a routine or regular basis during service, and extraordinary assessments performed when the structure has been subjected to accidental actions, etc.

3.3.2 Initial assessment

(1) The initial assessment shall be performed to assess the initial state of the structure.
(2) In the initial assessment, it is necessary to estimate the mechanisms of deterioration based on the findings of the initial inspection, predict the progress of deterioration, evaluate the performance of the structure, determine whether remedial measures are necessary, and confirm the validity of the maintenance plan.
(3) If emergency or other remedial measures are found to be necessary in the initial assessment, appropriate actions shall be taken.

Commentary: Regarding (1): The initial assessment corresponds to assessment performed immediately after a new structure enters service, or, for an existing structure, the first assessment performed after formulating and finalizing a new maintenance plan, performing major repairs or reinforcement, or determining that review of the management plan is necessary for some other reason. In the case of a new structure, the findings of the completion inspection can be utilized in the initial assessment.

Regarding (2) and (3): The objectives of the initial assessment are broadly divided into the following three:

(i) To confirm the validity of the maintenance plan formulated prior to the initial assessment and obtain data for finalizing the plan.

(ii) To obtain initial values for use as basic data for beginning maintenance.

(iii) To identify initial defects, damage, deterioration, or other points likely to cause issues for future maintenance, and undertake remedial measures at the initial stage.

Inspection performed in the initial assessment is the initial inspection. The primary objective of the initial inspection is to investigate whether a new structure has been properly constructed or whether major repairs and retrofit have been properly performed for a structure, as well as to collect data necessary to begin maintenance.

To perform proper and systematic maintenance of the target structure, it is also important to understand how the performance of the structure and its structural members will change over the planned service period. Therefore, in the initial assessment, the mechanisms of deterioration that may occur in the structure should be estimated from the findings of initial inspection, and the progress of deterioration should be predicted using deterioration models appropriate to the mechanisms of deterioration.

If no deterioration, damage, initial defects, etc. in the structure or structural members are found in the initial inspection, the initial state of maintenance can be considered as satisfying the required performance. At the same time, by evaluating the performance of the structure during the planned service period based on the results of the above prediction of progress of deterioration, it is possible to evaluate the soundness of the structure at the end of the planned service period. Taking these results into account, the validity of the maintenance plan that has been formulated is to be examined, and review of the maintenance plan is to be performed in line with conditions. However, if significant deterioration is apparent in the initial assessment and if a high probability of decline in the performance of the structure is determined from the results of prediction of the progress of deterioration, then, after the implementation of appropriate remedial measures, decisions will be necessary on matters including a repeat of the initial inspection.

3.3.3 Periodic assessments

(1) Periodic assessments are performed to evaluate the performance of the structure during service and to determine the necessity of remedial measures.

(2) During periodic assessment, the presence or absence of damage and deterioration, as well as changes in these over time, are assessed through routine inspections or regular inspections. It is also necessary to estimate the mechanisms of deterioration and predict the progress of deterioration, and, followed by evaluating the performance of the structure, the necessity of remedial measures shall be judged.

(3) The validity of the maintenance plan must be confirmed from the results of periodic assessments, and the maintenance plan shall be reviewed as necessary.

Commentary: Regarding (1): Performing assessment periodically enables the assessment of changes in the state of the structure during service. This can lead to early detection of anomalies in the structure and prevention of a decline in performance. It also enables systematic preparation of repair and other remedial measures, thereby aiding in carrying out efficient and rational maintenance. As such, periodic assessments are the most fundamental and important actions in maintenance.

Regarding (2): Two types of inspection are performed in periodic assessments: routine inspection performed at intervals of several days to a week, and regular inspection performed every few years at relatively spaced intervals. In routine inspection, it is important to qualitatively grasp any anomalies of the structure at an early stage. By contrast, in regular inspection, it is important to quantitatively assess any anomalies of the structure. When direct inspections are difficult, indirect inspections can be performed as necessary on the basis of factors including the condition of surrounding structures.

If no deterioration, damage, initial defects, etc. in the structure or structural members are confirmed through the inspections performed in periodic assessment, the structure can be considered to satisfy the required performance at the time of inspection. If the performance of the structure at the end of the planned service period is predicted from the inspection findings, and if the retained performance of the structure is evaluated as satisfying the required performance, then there is no need to change the maintenance plan. In reality, however, it is not an easy task to predict the future condition of a structure from limited inspection findings. Therefore, if no anomalies are found through routine inspections carried out at a high frequency, the structure may be deemed sound without making predictions. For cases in which deterioration, damage, or initial defects such as delamination of surface concrete, which could create a problem with the degree of effects on third parties due to spalling of concrete, are confirmed in inspections, urgent remedial measures are required.

Conversely, if an anomaly is found in a structure through routine or regular inspection, it is necessary to carry out detailed investigations in addition to the standard inspections to ascertain whether the anomaly is deterioration, damage, or initial defect. If the anomaly is due to deterioration, it is necessary to estimate the mechanism of deterioration and make predictions that appropriately incorporate the inspection findings. Following the prediction, the performance of the structure during the planned service period is evaluated from the results, and the necessity of remedial measures is determined. In principle, this determination is made using the maintenance limits predetermined in the maintenance plan.

3.3.4 Extraordinary assessments

(1) Extraordinary assessments are performed when a situation that requires emergency assessment has occurred, such as when an accidental loads act on a structure.

(2) Extraordinary assessments are performed for the purpose of evaluating the performance of the structure by assessing the degree of anomalies due to damage or deterioration through extraordinary inspection or emergency inspection, and then determining the necessity of remedial measures.

(3) In extraordinary assessments, if bodily injury or serious social and economic effects are expected due to reasons such as collapse of a structure, appropriate remedial measures shall be enacted as quickly as possible.

Commentary: <u>Regarding (1)</u>: Situations that require extraordinary assessments are summarized below.

<u>Extraordinary assessment following a natural disaster</u>: For example, a structure that has been subjected to major seismic forces exceeding the seismic force set in the design may suffer serious damage or collapse. Therefore, in the event of an earthquake of a scale that is expected to affect the structure, it is necessary to confirm the presence or absence of damage to the structure, parts, and structural members through extraordinary assessment.

When a typhoon passes through the area surrounding a structure, large wind load or wave force may act on and damage the structure. In addition, damage to structures from collisions with trees carried by flooded rivers, subsidence of bridge piers by scouring, and collapse of earth retaining walls due to rising groundwater levels during heavy rainfall could occur. Therefore, after the passing of a typhoon of a scale that is deemed to have caused some impact to structures, it is necessary to confirm the presence or absence of damage to the structure, parts, and structural members through extraordinary assessments. In addition, for structures similar to the damaged structure, it is necessary to check for the possibility of similar deterioration or damage in the future as well as to consider preventive remedial measures, reviews of maintenance plans, etc. as necessary. It is necessary to perform extraordinary assessments based on the same approach for natural disasters other than earthquakes and typhoons as well following a disaster of a scale that may have affected structures.

<u>Extraordinary assessment following a fire</u>: When a structure, part, or structural member has been exposed to high temperature due to a fire, the properties of concrete or reinforcing materials may be changed. Therefore, after a fire has occurred in the vicinity of a structure, it is necessary to perform extraordinary assessment of parts, structural members, and the surrounding areas that were subjected to fire.

<u>Extraordinary assessment following collision by vehicle, ship, etc.</u>: Extraordinary assessment is required following collision by a vehicle, ship, etc., as the position of the collision, parts nearby, and structural members may have been damaged.

<u>Extraordinary assessment associated with revision of standards, etc.</u>: In cases where levels for design load or required performance have changed due to revision of standards, etc. during the service period, it is necessary to confirm whether the performance of a structure constructed according to the old standard satisfies the performance demanded by the new standard. The assessment performed at this time is included within extraordinary assessments.

<u>Extraordinary assessment requiring urgency</u>: In the event of an accident occurring due to initial defect, damage, or change in the state such as deterioration in the structure, the potential for a similar accident to occur in a structure similar to or in an environment similar to that in which the accident occurred cannot be denied. Therefore, assessments of these structures performed on an emergency basis to prevent the occurrence of similar accidents are also regarded as extraordinary assessments. In addition, for structures constructed at the same time or in the same format as structures in which significant change in form was confirmed through periodic assessments, extraordinary assessments should be carried out to check for the possibility of similar deterioration or damage.

<u>Regarding (2)</u>: An extraordinary inspection is an inspection conducted to assess the status and degree of damage to a structure or structural members following a natural disaster, accident, etc. and to obtain information for conducting evaluations and making decisions concerning the structure.

Following a natural disaster or accident, a structure may suffer greatly impaired safety, or even undergo collapse or failure. Therefore, it is necessary to conduct extraordinary inspections as soon as possible following such occurrences. Even if the appearance of a structure is maintained, appropriate remedial measures such as prohibiting entry into the area around the structure or restricting its service must also be taken.

<u>Regarding (3)</u>: In a worst-case scenario, a natural disaster or accident could result in the collapse of a structure. Therefore, it is important to first prevent major social and economic effects such as the obstruction of lifeline functions or secondary disasters in the form of physical harm to people. If extraordinary assessment uncovers conditions that could affect people or properties,

such as the spalling of concrete due to the delamination of cover concrete, it is important to immediately enact appropriate remedial measures.

3.4 Remedial measures

When assessment has determined a need for remedial measures to address the progress of deterioration of the structure or a decline in performance based on maintenance limits, it is necessary to select appropriate and implement remedial measures that will subsequently enable maintenance of the performance of the structure for the required period while taking into consideration factors including the importance of the structure, maintenance category, remaining planned service period, ease of maintenance, life cycle cost, and environmental performance.

Commentary: Remedial measures are categorized into strengthening of inspections, repair, reinforcement, restriction of service, and dismantling/removal. When implementing remedial measures, it is necessary to conduct investigations to determine the specific methods and to prepare an appropriate plan that covers matters including budget, organization, and personnel so that remedial measures will be carried out rationally. When repair or retrofit has been selected, appropriate methods must be selected in consideration of the above and of the characteristics of the construction methods.

3.5 Recording

Inspection of a structure, prediction of the progress of deterioration, evaluation of performance, determination of the necessity of remedial measures, and other results of assessments, along with the content of remedial measures, should be recorded using appropriate means and should in principle be kept throughout the service period of the structure.

Commentary: Recording is an essential action in rational maintenance. It serves not only as data for maintenance of the structure but also as reference information for the maintenance of similar structures and determination of the validity of design. Therefore, results obtained through assessment, the content of remedial measures, and other contents necessary for maintenance are to be recorded in a format that facilitates reference, and this record is to be in principle throughout the period during which the structure is in service.

It is necessary to fully consider the format and storage method of the records to enable easy search, referencing, and understanding of the content, envisioning cases in which the maintenance manager is replaced during the service of the structure or in which multiple managers perform maintenance for a single structure.

Standards

Chapter 1 General Rules

1.1 General

(1) "Maintenance: Standards" (hereafter, "this Volume") presents standard methods for the maintenance of concrete structures.

(2) The principle is to estimate the deterioration mechanisms that cause an anomaly or a decline in the performance of structures and structural members, as well as perform maintenance based on those deterioration mechanisms.

(3) Maintenance of a structure is to be carried out with full attention given to the actions that affect the structure and members, and to the deterioration phenomena occurring in the structure and members.

Commentary: Regarding (1) and (2): This Volume presents standard approaches, methods, and procedures for the rational maintenance of structures, with comprehensive consideration of initial defects, damage, and deterioration.

Chapters 2 to 8 present standard methods for each action, following the maintenance steps shown in **Figure C1.1.1**. As conditions differ by structures, it is necessary to first fully examine whether these standard methods are applicable, and, if necessary, to revise the standard methods in line with "Maintenance: Main Volume."

Maintenance of structures begins with the formulation of the basic policy for maintenance. The basic policy for maintenance is set at the time of the formulation of the structural plan or design as a maintenance category. If a maintenance category has not been set for an existing structure, it must be set when formulating the maintenance plan. Maintenance limits are set according to the maintenance category. Inspections are then performed on the basis of the maintenance plan, and data for evaluating the state and the retained performance of the structure are obtained.

Estimation of the deterioration mechanisms is important for evaluating the performance of the structure with higher accuracy during inspection and for predicting the future progress of deterioration and decline of performance. Therefore, in principle, deterioration mechanisms are first estimated. The possibility of actions combined by other deterioration must also be considered. In the case of combined deterioration, particular caution is required as evaluation and prediction based on a single deterioration mechanism could yield results that fall on the hazardous side.

Subsequently, the performance of the structure is evaluated, and future maintenance methods are examined. Therefore, it is necessary to not only perform evaluation during inspection but also predict performance at the end of the planned service period. When deterioration mechanisms can be estimated, predicting the degree of progress of deterioration in accordance with the methods for verification of durability in "Design" is possible, as is performing probabilistic prediction by statistically analyzing the degree of deterioration. The present performance and the future performance of the structure are evaluated by methods such as the appearance grade of the structure, the performance evaluation formula in design, or nonlinear finite element analysis. The necessity of remedial measures is determined by noting whether the performance of the structure achieves or falls below the maintenance limits, or whether it may do so in the future.

If a need for remedial measures is determined, appropriate types of remedial measures are selected and implemented. These results must be recorded and stored such that they can be utilized in the future.

Even if nearly no deterioration in performance occurs during the service period, it may become impossible to ensure the required performance because the use of the structure may undergo change, or there may be changes in the values of actions (load) or in verification limit values, etc., due to changes in the design standards. Remedial measures to address these are to be examined as necessary in accordance with "Maintenance: Standards Appendix" Volume 3.

Regarding (3): Anomalies occur in structures and members due to both internal and external factors. However, even when the causes are internal factors, external actions may promote the occurrence and progression of the anomalies. Therefore, it is of utmost important when performing maintenance to fully understand whether the actions on structures and structural members are internal or external factors. In addition, it is important to go beyond estimation at the time of design to continually grasp during maintenance whether changes take place in the actions that affect a structure or structural members to prevent anomalies from occurring and to curb the progress of deterioration. Even if it is not possible to estimate the mechanisms of deterioration initially, during the ongoing maintenance process it may become possible to estimate the mechanisms of deterioration more reliably. In some cases, it may also be possible to clarify the involvement of actions that had been overlooked by estimating from deterioration phenomena.

In this way, constantly striving to understand the actions working on and the deterioration phenomena occurring in structures and structural members during maintenance helps prevent the occurrence of anomalies and leads to curbing the progress of deterioration, as well as yields more reliable estimations of the mechanisms of deterioration.

"Maintenance" adopts the principle of estimating the mechanisms of deterioration and performing maintenance based on those mechanisms. However, when detailed surveys are technically not feasible, or when the mechanism of deterioration is unknown but the remaining planned service period is short and issues can be addressed by simple remedial measures, it is possible that, in reality, the mechanisms of deterioration cannot be or will not be estimated. In such cases, it is particularly important to pay full attention to actions that affect the structure or members and the deterioration phenomena occurring in them.

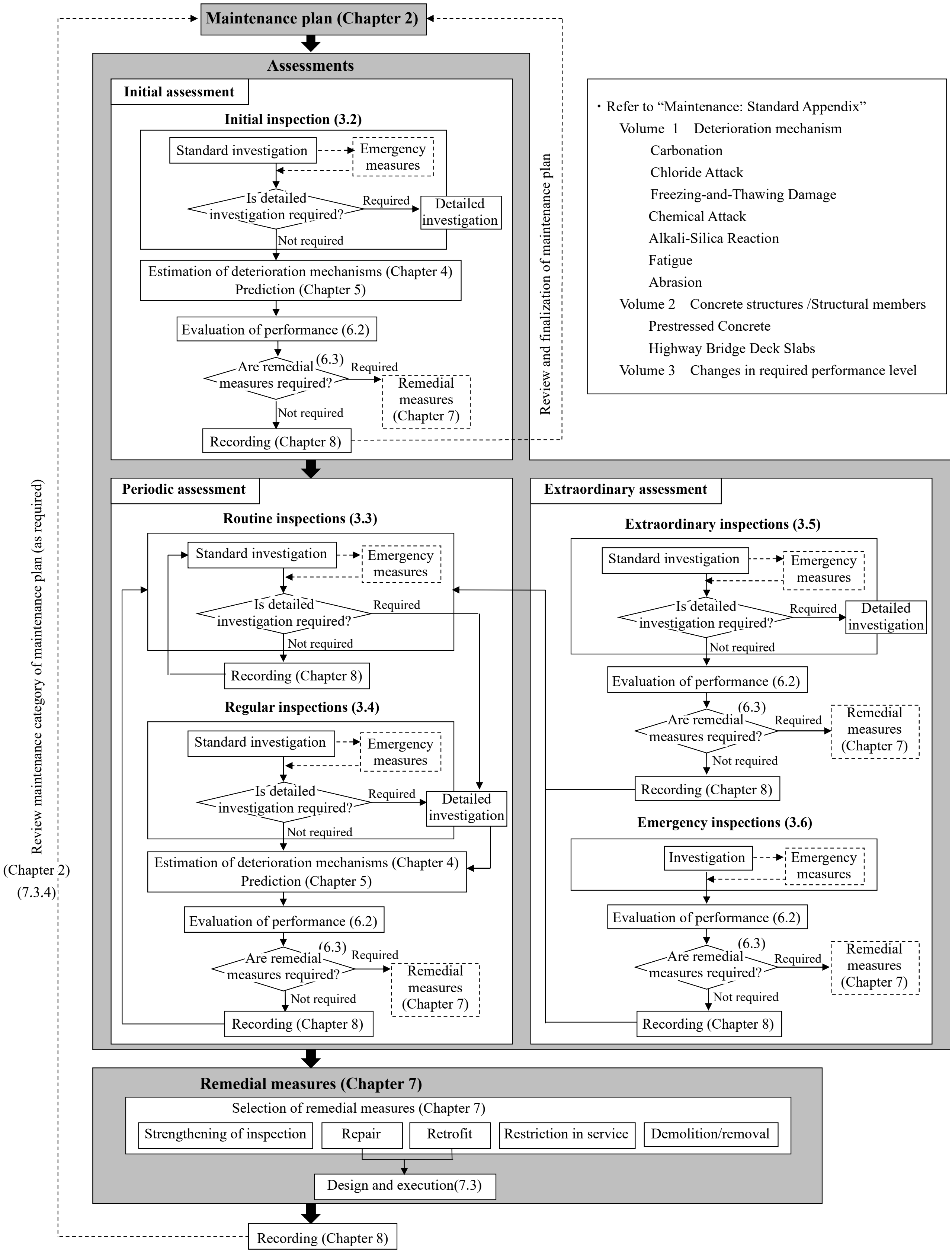


Figure C1.1.1 Flow of standard maintenance

Chapter 2 Maintenance Plan

2.1 Formulation of the maintenance plan

(1) The maintenance plan is based on the setting of assessment methods consisting of inspection, prediction, performance evaluation, determination of the necessity of remedial measures, and other matters for each targeted structure or structural member, along with methods for selecting remedial measures, methods of recording, etc., in accordance with the maintenance category of the structure and the estimated mechanisms of deterioration.

(2) In principle, the maintenance plan is to be formulated prior to the implementation of maintenance and determined after the addition of corrections as necessary, based on the results of the initial assessment that is subsequently carried out. The maintenance plan is to be reviewed as necessary during the service period of the structure.

(3) After formulation of the maintenance plan, specific procedures that enable the performance of maintenance based on the plan should be clarified.

Commentary: Regarding (1): When formulating the maintenance plan, it is necessary to first determine the period for carrying out maintenance on the basis of the service period of the structure that was considered in the design stage. When formulating a maintenance plan for an existing structure, the remaining planned service period, which is obtained by subtracting the period of service to the present from the service period, is normally set as the maintenance period.

As the service period, environmental conditions, importance, and other factors differ for every structure, it is not rational to conduct maintenance under identical conditions for structures that are subjected to varying conditions. Therefore, when formulating a maintenance plan, it is important to determine an appropriate maintenance category in line with the circumstances of the structure.

Decline in the performance of a structure is caused by an anomaly of the structure. Factors that cause anomalies of a structure can be broadly classified into external factors related to the environmental conditions of the structure and internal factors related to design and construction. Even for structures that face identical environmental conditions, the mechanisms underlying the anomaly may differ because of varying internal factors in the structures. Therefore, it is necessary to formulate a maintenance plan based on the assumed mechanisms of deterioration. When it is difficult to estimate the mechanisms of deterioration, the maintenance plan should be reviewed as necessary after its formulation when mechanisms of deterioration can be estimated in the maintenance stage.

Regarding (2): The maintenance plan is formulated in the structural planning and design stages. Final decisions on the maintenance plan are made after information on the structure has been collected and its validity has been confirmed in the initial assessment performed immediately after the structure enters service and after the plan’s content has been reviewed as circumstances require. Even when implementing maintenance for an existing structure based on "Maintenance," the initial assessment is incorporated into the maintenance plan created prior to said implementation, and review and final decisions concerning the maintenance plan are carried out on the basis of information on the structure obtained through the actual initial inspection.

Regarding (3): Maintenance work on a structure in service will not necessarily be performed by the same persons who formulated the maintenance plan. Moreover, routine inspection work may not always be performed by a professional concrete engineer. Therefore, after the maintenance plan has been formulated, it is effective to prepare manuals, etc., that indicate specific procedures and other matters to enable the systematic performance of planned maintenance work.

2.2 Maintenance categories

(1) The maintenance category is to be determined on the basis of factors including the importance of the structure or structural members, required performance, service period, ease of inspection, environmental conditions, and economic performance.

(2) In principle, the maintenance category is to be selected from the following three categories:

(i) Maintenance Category A (Preventive maintenance)

(ii) Maintenance Category B (Corrective maintenance)

(iii) Maintenance Category C (Observational maintenance)

Commentary: Regarding (1): It is advisable that the maintenance category be set at the stage of structural planning. When carrying out maintenance according to "Maintenance" for an existing in-service structure, it is necessary to set the maintenance category prior to starting maintenance.

Regarding (2): Three maintenance categories are set in "Maintenance." Maintenance Category A and Maintenance Category B are treated as standard maintenance categories, taking into consideration the basic procedures for maintenance presented in "Maintenance: Main Volume" Chapter 3.

The characteristics of structures classified into these three categories are generally as follows:

(i) Maintenance Category A

These are structures or members for which preventive maintenance is performed and for which remedial measures are implemented before deterioration becomes apparent.

(a) Structures or members for which deterioration is not allowed to occur because of the difficulty of enacting remedial measures such as repair after deterioration becomes apparent or, even if such remedial measures can be taken, due to the magnitude of impacts caused by suspension of service, etc., associated with implementation of the remedial measures.

(b) Structures or members for which performance declines immediately following the appearance of deterioration in the concrete surface, resulting in failure.

(c) Structures or members for which the degree of effects on third parties is particularly significant.

(d) Structures or members for which implementation of preventive maintenance is able to reduce the life cycle cost of the structure.

(ii) Maintenance Category B

These are structures or members for which corrective maintenance is performed and for which remedial measures are implemented in accordance with the degree of decline in performance.

(a) Structures or members for which remedial measures can be easily implemented even after deterioration has become apparent.

(b) Structures or members for which a prolonged timeframe for implementing remedial measures does not present problems.

(iii) Maintenance Category C

These are structures or members for which observational maintenance is performed and for which repair, reinforcement, or other remedial measures are not implemented even when an anomaly has occurred.

(a) Structures or members with a short design service period, such as a temporary structure.

(b) Structures or members for which implementation of remedial measures is difficult.

2.3 Maintenance limits

(1) In maintenance plans, the maintenance limits are to be set according to the maintenance category as the standards for determining the necessity of remedial measures.

(2) Maintenance limits are to be corrected as necessary based on the results of assessment.

Commentary: Regarding (1) and (2): Maintenance limits are limit values for management indexes set as effective management targets in maintenance for in-service structures to satisfy the required performance level. Therefore, in principle, maintenance limits should be set to equal the performance level, taking into account the degree of margin in the required performance level of the structure.

If the performance of the structure reaches or falls below maintenance limits, major remedial measures can be expected to be required in the future, even if the required performance level is satisfied. Therefore, maintenance limits are to be set so as to fully incorporate the following points:

(i) Required performance of the structure

(ii) (Remaining) design service period or (remaining) planned service period

(iii) Maintenance category

(iv) Method of assessment

(v) Estimated mechanisms of deterioration, methods of predicting the progress of deterioration, and expected rate of deterioration progress

(vi) Technical level of maintenance

(vii) Available remedial measures

(viii) Life cycle cost

(ix) Margin of time that can be expected before implementation of remedial measures

(x) Others

When setting maintenance limits, one of the following methods is used:

(a) A performance level (limit value) set based on the required performance is used as an index

(b) The state of deterioration is used as an index

The relationship between the deteriorated state and performance varies with the structure, its surrounding environment, and the mechanisms of deterioration. Therefore, when the appearance grade, etc., of the structure are used as an index in maintenance limits, it is necessary to set criteria on the side of safety.

When multiple aspects of performance are required for a structure, maintenance limits must be set that fall on the safest side. Even within a given structure, maintenance limits may differ by part or structural member.

Chapter 3 Inspection

3.1 General rules

(1) Inspection performed in assessment of a structure shall consist of appropriate investigations according to the purpose of the assessment and performed using appropriate methods.
(2) Initial inspection is to be performed to assess the initial state of maintenance of the structure.
(3) Routine inspection and regular inspection is to be performed to assess anomalies of the structure over time.
(4) Extraordinary inspection or emergency inspection is to be conducted as necessary, according to the purpose.
(5) For inspection, standard investigations are to be conducted in accordance with the frequency, items, and methods specified in the maintenance plan.
(6) During standard investigations for inspection, if anomalies of a structure are confirmed and more detailed information is deemed necessary for estimation, prediction, evaluation, and determination of the mechanisms of deterioration, detailed investigations are to be performed.
(7) When inspection has revealed anomalies for which urgent remedial measures are deemed necessary, these shall be performed promptly.
(8) When repairs or retrofits have been performed on a structure, the efficacy of these must be continuously confirmed by inspection.

Commentary: Regarding (1)–(4): Assessment consists of inspection, estimation of mechanisms of deterioration, prediction, evaluation of performance, and judgment of the necessity of countermeasures. In performing appropriate assessments, the quality of inspection is of extremely high importance.

Inspection can be variously classified according to its purpose, frequency, the content of investigations to be conducted, etc. In "Maintenance," inspection is divided broadly into three types according to the purpose of assessment: that performed at the time of initial assessment, that performed at the time of periodic assessments, and that performed at the time of extraordinary assessments. Inspection performed at the time of periodic assessments consists of routine inspection and regular inspection, depending on the frequency and method. For extraordinary assessment, extraordinary inspection is distinguished from emergency inspection, depending on the purpose of assessment and differences in the structures subject to the assessment. Conducting these inspections and comparing and examining the information obtained from each allows for the prediction of changes in the state of the structure from the start of service onward, as well as identifies the anomalies that have occurred since inspection was performed, thereby enabling rational maintenance.

The purpose of inspecting a structure is to assess the state of the in-service structure and actions affecting it to the highest degree possible. Therefore, inspection must be implemented using the methods that are most rational for the state of the structure. Therefore, when conducting inspection, it is necessary to appropriately determine the necessary investigation items, targeted parts, frequency, and investigation methods according to the purpose of each inspection. In "Maintenance," the individual measurements, tests, and so on performed to obtain specific information on the state of a structure and its structural members are called investigations, with an inspection consisting of one or more investigations.

In general, investigations performed during inspection are focused on the collection of information and do not explicitly include actions related to evaluation and decisions. However, it is conceivable that information normally related to evaluation and decisions, such as appearance grade indicated in Chapter 6, may be obtained during inspection. The information that should be obtained in inspection differs by evaluation method. The selection of parts of the structure to be inspected can be viewed as partially incorporating the aims of evaluation and judgment. Moreover, structures undergo anomaly because of mechanical actions, such as loading, and environment actions, with this change manifesting as the deterioration phenomenon. These must be taken into consideration in setting necessary investigation items for inspection.

<u>Regarding (5) and (6)</u>: Standard investigations for individual inspection in initial assessment, periodic assessments, and extraordinary assessments must be performed with full understanding of the purpose of the inspection and must appropriately set out the items, methods, scope, and other matters for efficiently obtaining the information needed to assess the state of the structure.

Detailed investigations adopt reliability over efficiency or speed as their condition for obtaining necessary information, with the content of investigations to be conducted also dependent on the results of standard investigations. However, if the mechanisms of deterioration of a structure can be estimated to a degree from the stage of formulating the maintenance plan, the maintenance plan should be determined with the content and methods to be implemented in detailed investigations assumed in advance on the basis of past cases and other information.

<u>Regarding (8)</u>: The purpose of inspection for confirming the efficacy of repairs or retrofit includes the provision of information that will aid the selection of later repair and retrofit work. Re-deterioration often occurs several years after repair or retrofit, necessitating re-repair. When selecting repair or retrofit work, obtaining information on the efficacy of repairs or retrofit and confirming the sustainability of this efficacy through inspection is important.

As the period over which the efficacy of repair and retrofit is confirmed through inspection extends to several decades, it is necessary to devise methods of investigation and recording that enable the continuation of inspection even if the persons in charge of inspections are replaced.

3.2 Initial inspection

(1) Initial inspection is performed to assess initial states concerning the performance of the structure at the start of maintenance.
(2) In principle, initial inspection must be performed for the structure as a whole.
(3) In the initial inspection, visual inspection, tapping, simple measurement, etc., of the structure as a whole as well as investigations of design- and construction-related documents are to be the standard.
(4) When standard investigations in the initial inspection reveal that more detailed information is required, detailed investigations must be conducted in the initial inspection.
(5) When inspection reveals anomalies for which urgent remedial measures are deemed necessary, they must be performed promptly.

Commentary: Regarding (1): "At the start of maintenance" here refers to the time that a new structure is put into service as well as to immediately after the application of major repairs or retrofits to an existing structure. It also refers to the time at which maintenance is started for existing structures that have never been subjected to maintenance and the time at which maintenance plans are reviewed and maintenance has begun under a new method.

Regarding (2): The locations at which deterioration or damage will become apparent in an in-service structure cannot always be identified in advance. As the maintenance plan of a structure is ultimately determined on the basis of information on the structure obtained during the initial inspection, it is important to assess the initial state of the structure as a whole as accurately as possible in the initial inspection. With regard to parts for which investigations are difficult after the start of service, the initial state of said parts is to be estimated based on investigations such as design records and construction records. In the case of new structures, the findings of completion inspections can also be used as investigation results in the initial inspection.

Regarding (3): A structure targeted for initial inspection will have the characteristics described below, depending on whether the structure is new, existing, or has been subjected to major repairs or retrofit. The initial inspection should be performed accordingly.

New structures and those that have just been subjected to major repairs or retrofit rarely exhibit deterioration with the passage of time. Investigations of documents and visual confirmation of the presence or absence of initial defects or damage form the main content of standard investigations.

For existing structures that have never undergone maintenance, investigations of design records, investigations of inspection records created during construction, visual inspections, and tapping investigations should be supplemented by investigations performed using nondestructive testing equipment as necessary in order to obtain the information required for determining the initial values. It should be noted that design records and inspection records created during construction may not have been saved in some cases, necessitating additional investigations to collect the information that should have been obtained from these.

Regarding (4): When the results of standard investigations have confirmed any of the following cases and more detailed information that was not obtainable through standard investigations has been deemed necessary to perform estimation of the mechanisms of deterioration, prediction, evaluation, and judgment, detailed investigations must be conducted.

i) Deterioration has been confirmed.

ii) Damage or initial defects have been confirmed.

iii) Anomalies have not been confirmed but investigations of documents, etc. have revealed concern over the occurrence of deterioration, and a need to strengthen inspections and observe progress has become apparent.

Each of these cases implies that the structure is not in the condition envisioned at the time of formulation of the maintenance plan, and it is necessary to carefully determine whether the maintenance plan to be applied during the subsequent service period is appropriate and to finalize the maintenance plan after revising its content as necessary.

See 3.7 for items in detailed investigations and examples of applicable methods.

Regarding (5): When inspections have revealed anomalies that are deemed to require urgent remedial measures, such as concrete peeling that could result in pieces of falling concrete, said remedial measures must be implemented promptly.

3.3 Routine inspections

(1) Routine inspections are conducted for the purpose of assessing the presence or absence and the degree of deterioration and damage within a scope that allows inspection through patrols.

(2) The frequency of routine inspections, investigation items, and methods shall be appropriately determined by considering the purpose of the routine inspections, maintenance category, maintenance limits, importance of the structure, prediction of the progress of deterioration, etc.

(3) During routine inspections, visual inspections shall be the primary focus, and investigations of documents, tapping, and other methods as required for the situation shall be the standard.

(4) When standard investigations during routine inspections set in advance in the management plan find that more detailed information is required, detailed investigations must be conducted.

(5) When routine inspections have confirmed anomalies, such as the spalling of concrete that could have an effect on third parties caused by pieces of concrete falling, urgent remedial measures must be promptly taken.

Commentary: Regarding (1): Routine inspections are primarily conducted visually at a distance and in some cases through a combination of simple methods such as tapping to check for the occurrence of anomalies that could impede the functions of the structure. When conducting routine inspections, inspection manuals, etc,. should be prepared in line with the characteristics of the structure, with investigations implemented following these.

Regarding (2) and (3): i) Investigation items: Examples of the main investigation items in routine inspections are as follows:

・ Degree of anomalies
・ State of usage of the structure
・ State of concrete and steel
・ State of incidental facilitates, etc.
・ State of environmental actions
・ State of past remedial measures

Investigation methods in routine inspections include visual investigations, such as by the naked eye, photographs, or binoculars, and investigations by means of tapping. In some cases, it is advisable to conduct an investigation of documents and to create and carry a list of investigation items and the initial states of the structure for use in necessary inspections.

ii) Locations of inspection: Routine inspections must be conducted within a scope that allows inspection through patrols. However, it is advisable to cover as wide a scope as possible. Locations where anomalies were found in earlier inspections should be paid particular attention. Even when inspections through patrols are difficult, for important structures, parts, and structural members, or for parts, structural members, etc. thought to be susceptible to deterioration or damage, it is advisable from the design stage to consider the installation of incidental facilities such as scaffolding for use in inspections.

iii) Frequency of inspections: The frequency of inspections must be set appropriately taking into account the scale of the personnel and the budget for actual investigations and in consideration of the importance of the structure, the degree of effects on third parties, the results of prediction of the progress of deterioration, maintenance limits, etc.

Regarding (4): When standard investigations set out in advance in the maintenance plan have confirmed anomalies and i) said anomalies are significant, ii) the causes of the anomalies are unknown, or iii) deterioration differs significantly from predictions of the progress of deterioration, and when the results of the standard investigations alone do not yield sufficient information, then following confirmation of this state by a concrete specialist engineer, detailed investigations must be conducted within routine inspections. In some cases, the standard investigations during regular inspections may be performed ahead of schedule as detailed investigations in routine inspections.

Regarding (5): See the Commentary in 3.2 (5).

3.4 Regular inspections

(1) Regular inspections are conducted for the purpose of assessing in greater detail the presence or absence and the degree of deterioration and damage affecting the structure as a whole to a degree not possible through routine inspections.

(2) The frequency, target parts, investigation items, and methods of regular inspections shall be appropriately determined in consideration of the purpose of the regular inspections, the maintenance category, maintenance limits, importance of the structure and its parts and structural members, existing maintenance records, predictions of the progress of deterioration, etc.

(3) Investigations of documents, visual- or tapping-based investigations, etc. shall be the basic methods for regular inspections, and combining the use of nondestructive testing equipment, the testing of collected cores, and other methods as necessary shall be the standard.

(4) When standard investigations in regular inspections set in advance in the management plan have found that more detailed information is required, detailed investigations shall be conducted.

(5) When regular inspections have confirmed anomalies, such as the spalling of concrete that could have an effect on third parties caused by pieces of concrete falling, urgent remedial measures must be promptly taken.

Commentary: Regarding (1): Regular inspections are performed under the guidance of a concrete specialist engineer, responsible engineer, or an engineer possessing knowledge concerning the design, construction, and maintenance of equivalent structures following manuals created on the basis of specialized knowledge of the design, construction, and maintenance of the structure.

Regarding (2) and (3): i) Investigation items: In investigations of anomalies, etc., the necessary investigation items, the initial state of the structure, and the state at the time of the previous investigation must be confirmed in advance, and required information must be carried on person. When investigations can be conducted close to each other, it is possible to combine some items, such as methods using nondestructive testing equipment and collection of cores, as necessary. It is advisable to quantitatively assess anomalies of the concrete surface by measuring the width and length of cracks, the scope of delamination, etc., using scales or other tools. Moreover, appropriately combining methods using nondestructive test equipment enables estimating mechanisms of deterioration and confirming the status of deterioration at an early stage. Considering budgetary constraints, implementation of effective maintenance, and other factors, other methods that can be considered include limiting investigation items and the scope of investigations to regular inspections every other year to keep regular inspections comparatively simple, or increasing investigation items approximately every 10 years to conduct wider-ranging investigations.

ii) Locations for inspection: In principle, locations for inspection must cover the structure as a whole. For locations that are difficult to check through routine inspections or parts and structural members that are assumed to be prone to deterioration or damage, it is important to perform inspections with care. For large structures such as viaducts, inspecting the entirety of the object at once may be difficult. In such cases, dividing the scope of inspection and performing inspections sequentially at appropriate intervals can be considered. In doing so, it is advisable to determine the scope, sequence, etc. of inspections to be performed in consideration of past maintenance records, the importance of parts and structural members, economic performance, etc.

iii) Frequency of inspections: The frequency of regular inspections must be set appropriately by considering the maintenance category, importance of the structure and its parts/structural members, form, remaining design service period, remaining planned service period, environmental conditions, past maintenance records, predictions of progress of deterioration, maintenance limits, economic performance, etc., with regular inspections generally performed once every few years. Flexible actions such as increasing the interval of the initial stage of service when deterioration is unlikely to manifest and shortening the interval in the stage when deterioration is expected to manifest based on predictions of progress of deterioration are also important in performing rational maintenance.

Regarding (4): When standard investigations set forth in advance in the maintenance plan confirm anomalies, and when i) said anomalies are caused by deterioration and the mechanisms of deterioration are unclear or differ from those estimated in previous assessments, ii) said anomalies are caused by deterioration, the progress of which differs significantly from predictions, or iii) the causes of said anomalies are unknown or anomalies have not been confirmed but the usage conditions, load conditions, environmental actions, etc. of the structure have changed significantly, and further, when prediction of the progress of deterioration in the structure or evaluation of performance is not possible using the results of the standard investigations alone, detailed investigations are to be conducted within regular inspections. These should be performed with the maintenance categories, importance, maintenance limits, remaining planned service period, degree of anomaly, predictions of the progress of deterioration, etc. of the structure and its parts/structural members taken into account. Even when it is clear that anomalies consist of damage or initial defects, deterioration may be accelerated and quality defects may occur in concrete or reinforcing materials. Therefore, it is advisable to perform detailed investigations to properly evaluate the effects that these have on the performance of the structure.

Conversely, if the anomalies are caused by deterioration, the time of their occurrence is consistent with predictions of the progress of deterioration, and the degree of deterioration is also approximately consistent with what was considered in the initial maintenance plan, or if the progress of deterioration is slower than what was predicted, detailed investigations are in general not necessary, and inspections should be continued in line with the initial maintenance plan.

Regarding (5): See the Commentary in 3.2 (5).

3.5 Extraordinary inspections

(1) Extraordinary inspections of structures, parts, structural members, etc. that have or may have undergone anomalies because of disaster or accident are to be conducted promptly and with the safety of inspectors ensured.

(2) Standard investigations performed in extraordinary inspections shall be determined in advance in the maintenance plan, envisioning the occurrence of earthquakes and other disasters. These are to be performed using investigation methods that can be implemented at that time.

(3) When standard investigations in extraordinary inspections set in advance in the management plan find that more detailed information is required, detailed investigations shall be conducted.

(4) When extraordinary inspections have confirmed anomalies, such as the possibility of secondary disasters or degree of effects on third parties, urgent remedial measures shall be promptly taken.

Commentary: Regarding (1): Extraordinary assessments performed after a disaster or accident often demand urgency. Even if a structure has been spared from collapse or failure, its safety may be greatly impaired following the disaster. Therefore, following the occurrence of a disaster or accident, it is necessary to prevent secondary disasters and promptly conduct extraordinary inspections while ensuring the safety of inspectors. In particular, after a major earthquake there is a high risk of aftershocks that cause new anomalies or damage to structures during inspection work; therefore, it is important to further ensure the safety of inspectors. When addressing a structure after a fire, it is important to perform inspections after confirming that the fire has been extinguished and generation of toxic gas has subsided.

Regarding (2): Extraordinary inspections are first performed after unexpected events such as an earthquake or other disasters occur. However, to enable prompt action, the occurrence of earthquakes or other disasters is to be envisioned in advance, with standard investigation items and content set out in the maintenance plan. In general, this is performed through simple methods such as close-up viewing or tapping after visual confirmation from a distance that there is no risk of collapse. In some cases, it is advisable to conduct an investigation of documents and create and carry a list of required investigation items and the initial states of the structure.

Regarding (3): When it is easily determined from the appearance of concrete in standard investigations that remedial measures are necessary, it is necessary to omit detailed investigations and promptly implement remedial measures such as repairs and retrofit. However, if the damage is minor and it has been evaluated from the appearance that there is no need for emergency remedial measures, detailed investigations may be conducted again at an appropriate time after the effects of accidental external forces have subsided, and data (e.g., depth of cracks, load bearing capacity and stiffness of the structure) of use in evaluating retained performance may be collected. When an investigation method set out in the maintenance plan cannot be performed at the stage immediately after an earthquake or other disasters but is performed later, this is also classified as a detailed investigation.

Regarding (4): Urgent remedial measures that can be considered in extraordinary inspections include i) setting the area around the location as an off-limit area, ii) installing netting to catch falling objects under the location, iii) installing falsework to prevent collapse, and iv) restricting the speed and weight of trains and vehicles. Even after such urgent remedial measures are completed, the remedial measures must be preserved during the required period until the structure is restored to its normal state.

However, when performing urgent remedial measures in extraordinary inspections, it is necessary to fully understand that conditions differ from those of urgent remedial measures in other inspections and to pay special heed to safety aspects of the work to prevent secondary disasters from occurring during implementation of the emergency remedial measures.

3.6 Emergency inspections

(1) When an accident has occurred because of anomalies in a structure, or when an accident has not occurred but significant anomalies have been confirmed in the structure, emergency inspections shall be performed for parts and structural members in which the same sort of anomalies may occur.

(2) Investigations in emergency inspections must be performed using appropriate methods that enable confirmation of anomalies of the same sort as the cause of accident.

(3) When an emergency inspection has confirmed anomalies, such as delamination of concrete that could create a problem with the degree of effects on third parties, urgent remedial measures must be promptly taken.

Commentary: Regarding (1): It is likely that structures built at the same time or with the same form were designed and constructed according to the same standards. Therefore, if an accident has occurred because of anomalies in a structure, or when an accident has not occurred but significant anomalies have been confirmed in the structure through periodic assessments, etc., it is likely that a similar problem will occur in similar structures constructed in the same period.

The parts or structural members subject to emergency inspection should be those parts or structural members in which the same sort of anomalies could occur, with reference to earlier accidents as examples. During inspections, full consideration must be given to ensuring the safety of inspectors.

Regarding (2): Investigations implemented during emergency inspections must combine not only methods such as visual inspection and tapping but also methods using nondestructive testing equipment and chipping investigation, according to the condition of the anomalies. In the case of structures using similar design standards and materials, not only the applied design standards but also past information on the structures has great significance, which makes investigations of documents such as design drawings and construction records important. Moreover, when environmental actions differ, investigations of these are important.

Regarding (3): See the Commentary in 3.2 (5).

3.7 Investigations in inspections

3.7.1 General rules

(1) Inspection shall be performed with appropriate investigation items selected and with appropriate methods used in order to obtain specific information on the state of a structure, parts, and structural members.
(2) Investigation items shall be selected appropriately in consideration of the type and purpose of the inspection, the condition of the target structure, the information required, the causes of deterioration in the structure, etc.
(3) Appropriate investigation methods that enable obtaining information on the selected investigation items shall be selected.

Commentary: Regarding (1): Inspections conducted during the maintenance period of a structure must assess the state of the structure and structural members as specifically and quantitatively as possible. Investigations are conducted to achieve this.

Standard investigations conducted in individual inspections in the initial assessment, periodic assessment, and extraordinary assessment must be performed with full understanding of the purpose of the inspections and must appropriately set out the items, methods, and other matters for efficiently obtaining the information needed to assess the state of the structure, fully considering the usefulness, etc. of the obtained information.

Regarding (2): Investigations can be broadly divided into standard investigations and detailed investigations. In principle, however, investigation items in either case should be selected to enable the required information to be obtained. Items in the standard investigations performed in each type of inspection are set out in advance in the maintenance plan. General investigation items include an overview of the structure, its in-service state, anomalies or deformation of appearance, the condition of concrete and steel, structural details, the condition of incidental facilities, environmental actions, and the state of past remedial measures. It is important to refer to the results of standard investigations to appropriately select investigation items matched to the purpose of detailed investigations.

Regarding (3): "Maintenance" describes basic content for the following general investigation methods: i) methods based on documents, etc. (investigations of documents), ii) visual-based methods, iii) tapping-based methods, iv) methods using nondestructive testing equipment, v) methods involving localized destruction, vi) methods using load testing and vibration testing of actual structures, and vii) investigation methods for evaluating environmental actions, etc.

When the mechanisms of deterioration can be estimated and more detailed information is required, "Maintenance: Standards Appendix" Volume 1 can be used as reference.

3.7.2 Investigations of documents

Investigations of documents are to be selected when it is necessary to investigation information on the design, construction, and maintenance of structures.

3.7.3 Visual- and tapping-based investigations

Visual- and tapping-based methods are to be selected when it is necessary to assess the appearance of a structure and anomalies such as delamination and peeling of surface layer concrete.

3.7.4 Investigations using nondestructive testing equipment

(1) For investigations using nondestructive testing equipment, appropriate methods shall be selected after the purpose, scope of application, application limits, and required measurement accuracy have been made clear.
(2) Methods based on the surface hardness shall be selected when it is necessary to estimate the surface strength of concrete, deterioration in the surface layer, and quality.
(3) Two methods that utilize electromagnetic induction are available: a method that utilizes the conductivity and magnetism of steel materials, and a method that utilizes the dielectric property of concrete. These shall be selected mainly when it is necessary to obtain information on the position of and failure of steel in the concrete and the water content of the concrete.
(4) Methods using elastic waves include the hammering method, vibration testing, the ultrasonic method, the impact elastic wave method, the acoustic emission (AE) method, and elastic wave tomography. These shall be selected mainly when it is necessary to obtain information on the mechanical properties of the concrete, cracking, voids such as delamination, the steel arrangement, and the condition of grout.
(5) Methods using electromagnetic waves include X-ray methods, infrared thermography, and the electromagnetic wave radar method. These are selected mainly when it is necessary to obtain information on the steel arrangement and cracking in the concrete, voids such as delamination, and the condition of grout.
(6) Electrochemical methods include the natural electric potential method, the polarization resistance method, and methods using electrical resistance. These shall be selected mainly when it is necessary to obtain information on corrosion in the steel in concrete.
(7) Methods that use light include FBG methods that use the expansion and contraction of a diffraction grating in optical fiber and the OTDR method that uses the scattering of light. These shall be used mainly when it is necessary to obtain information on strain and its distribution inside the concrete, as well as on temperature.
(8) Methods that use digital imaging include digital photogrammetry, the digital image correlation (DIC) method, the fluorescent agent impregnation method, and the Moire method. These are selected mainly when it is necessary to obtain information on the displacement of the concrete surface, the distribution of stress and its change over time, and the distribution of internal cracks and pores.
(9) In addition to the above, the surface water absorption test, surface air permeability test, etc. shall be selected when it is necessary to obtain information specific to the surface properties of concrete.

3.7.5 Investigations involving localized destruction

Investigations involving localized destruction mainly include methods using core sampling, methods using drilling powder, methods using chipping, and methods that sample steel. These shall be selected when it is necessary to obtain more precise information on the physical properties and state of deterioration of concrete and steel materials.

3.7.6 Investigations based on load testing and vibration testing of structures

Investigations based on load testing and vibration testing of structures shall be selected when it is necessary to obtain information on cross-sectional rigidity and vibration properties of structural members.

3.7.7 Investigations to evaluate load and environmental effects

For investigations to evaluate load and environmental actions, appropriate methods shall be selected to allow the obtaining of information on the environmental conditions, load conditions, etc. under which the structure is placed.

3.7.8 Investigations based on monitoring using sensors

For investigations based on monitoring using sensors, appropriate methods must be selected so that information necessary for the purpose can be obtained stably and continuously over the required period.

3.8 Investigations to confirm the efficacy of repairs and retrofit

(1) When repairs or retrofit have been performed on a structure, investigations shall be conducted to confirm the efficacy of the repairs or retrofit, with investigation items and methods appropriately determined in consideration of the maintenance limits.
(2) Maintenance plans and inspection methods following repair or retrofit shall be according to Chapter 7, while performance evaluation methods are to be according to Chapter 6 and Chapter 7.

Chapter 4 Estimation of Deterioration Mechanisms

4.1 General rules

Estimation of the deterioration mechanisms shall be conducted on the basis of the findings of inspections, taking into consideration the design documents, materials used, construction management, inspection records, and actions affecting the structure.

Commentary: If anomalies in structural members or a structure are found during inspection, it is necessary to clarify as much as possible the main causes of the anomalies, whether deterioration, initial defect, damage, or a combination of these, and to enact appropriate remedial measures. When the anomalies have been determined to be caused by deterioration, it is necessary to estimate the deterioration mechanisms.

When performing the estimation of deterioration mechanisms, if design documents, construction reports, or other documents exist, it is possible to check the deterioration mechanisms assumed at the time of design or the time of formulation of the maintenance plan and to estimate the deterioration mechanisms based on the assumed environmental conditions and usage conditions. By checking the materials used and records from construction management and inspections, it may be possible to estimate the causes of deterioration potentially inherent in the structure, which may aid estimation of the deterioration mechanisms. However, as actions affecting a structure may differ from what was assumed at the time of design or formulation of the maintenance plan, these conditions must be confirmed through inspection. If information on deterioration metrics has been obtained through inspection, or if anomalies have been confirmed and their characteristics have become clear, it is possible to further narrow down the deterioration mechanisms using 4.2 or "Maintenance: Standards Annex" Volume 1 as a reference. When deterioration mechanisms cannot be estimated, as in the case of combined deteriorations, one or more deterioration metrics corresponding to the occurring deterioration are to be selected, with "Maintenance: Standards Appendix" Volume 1 used as reference, and detailed investigations are to be performed with a focus on these metrics.

Conversely, in the case that no changes are observed in the degree of anomalies from past inspections despite the passage of decades since the start of service, it can be predicted that the progress of deterioration is not rapid, and the performance of the structure at present and in the future is evaluated by assuming that it is no different from the one evaluated at the time of design or past inspections. However, this does not mean that prediction, evaluation, and judgments are omitted, and it is necessary to be aware that a series of decisions concerning the present and future state of the structure is being performed on the basis of past performance.

4.2 Methods for the estimation of deterioration mechanisms

(1) Estimation of the deterioration mechanisms when deterioration has not manifested is in principle to be performed based on (i) the results of investigations of actions determined from environmental conditions, usage conditions, etc., and (ii) information such as the condition of design and construction work in the structure. However, for structures to which Maintenance Category A is applied, in addition to (i) and (ii), (iii) information on deterioration metrics inside the structure is to be used in principle.

(2) In principle, estimation of deterioration mechanisms when deterioration has manifested must be performed based on (iv) characteristics of anomalies, in addition to (i)–(iii) above.

Commentary: Regarding (1): For cases in which it has been determined through investigations mainly employing visual or tapping methods that significant deterioration is not occurring in parts and structural members, data are extremely limited, which means that it is often difficult to predict deterioration that could occur in the future from these results alone. However, when actions affecting the structure have been identified and understood in the investigations, or when actions considered at the time of design have been confirmed from the design documents, the deterioration mechanisms can be estimated to a certain degree by referring to **Table C4.2.1**.

Table C4.2.1 Deterioration mechanisms estimated from environment and actions

		Estimated deterioration mechanism
Environment	Coastal area	Chloride attack
	Cold region	Freezing-and-thawing damage, chloride attack
	Hot spring area	Chemical attack
Actions	Repeated wetting and drying	Alkali-silica reaction, chloride attack, Freezing-and-thawing damage
	Use of deicing agent	Chloride attack, alkali-silica reaction
	Repeated loading	Fatigue, abrasion
	Carbon dioxide	Carbonation
	Acidic water	Chemical attack
	Running water, vehicles, etc.	Abrasion

When periodic assessments determine that deterioration has not manifested and there is no difference between this result and the predictions of progress of deterioration in the initial assessment, the deterioration mechanisms can basically be considered the same as those estimated in the initial assessment. However, as actions affecting the structure may come to differ over time from what was initially assumed, it is advisable to confirm the validity of the estimated deterioration mechanisms.

In structures constructed in conformance with the standards of a period when deterioration of structures was not widely recognized, these design- and construction-related factors may greatly affect the occurrence of deterioration. Therefore, in the estimation of deterioration mechanisms, it is advisable to refer to the design of the structure, the state of its construction, etc.

Regarding (2): When anomalies have been found through inspection, it is necessary to examine whether the anomalies are caused by initial defects, damage, or deterioration.

i) Initial defects

Among the anomalies that occur during construction or shortly after its completion, potentially harmful cracks, honeycombs, cold joints, sand streaks, insufficient cover thickness, and grout defects are typical examples of initial defects. With regard to cracks, it may be possible to identify those caused by initial defects by roughly classifying the causes of occurrence into materials, heat of hydration, shrinkage, construction work, and structural factors, based on the regularity, form, etc. of occurrence. When cracks have been estimated to be related to structural factors, the load bearing capacity of the cracked cross-section and the cross-sectional force due to external load must be calculated from the design documents and inspection findings. From this finding, the causes must be made clear through comparison of the magnitudes, types, directions, etc. of forces that generate the cracks with the observed crack patterns, and the effects on performance must be evaluated.

ii) Damage

Damage such as cracks and peeling that occur when an excessive external force, such as an earthquake or collision, acts on a structure can be easily distinguished from anomalies caused by deterioration if the cause of occurrence is understood. When the cause is unknown, structures in the vicinity of the target structure and of the same form can be subjected to a detailed investigation, and the occurrence of anomalies can be compared or damage can be assumed and the validity can be analyzed from the position and magnitude of the anomalies to determine the presence or absence of damage.

iii) Deterioration

When anomalies caused by deterioration are suspected, the envisioned deterioration mechanisms should first be screened using the actions determined by the environmental and usage conditions, followed by consideration of factors centered on materials used and workability. Comparison of the characteristics of anomalies assessed through inspection with the characteristics of deterioration mechanisms shown in "Maintenance: Standards Annex" Volume 1 should then be performed, followed by evaluation of the validity of the estimation results as well as estimation of the appropriate deterioration mechanisms.

If anomalies caused by deterioration are at a stage that may have a significant effect on the safety of the structure, it is necessary to not only perform actions such as harmful substance analysis or physical testing of the concrete using core samples, but to also investigate the arrangement of steel and the state of corrosion. The deterioration mechanisms are to be estimated from the relationship between these results and actions affecting the structure, and the necessity of remedial measures and the timing of their implementation are to be examined. As deterioration mechanisms are often combined at such a stage, it is necessary to select all deterioration mechanisms involved in deterioration and to evaluate their degree of effect on performance decline.

iv) When estimation of deterioration mechanisms is difficult

Even at the stage at which anomalies have manifested, for cases in which the deterioration mechanisms cannot be estimated because they are complicated or because deterioration, initial defects, and damage are compounded, it is important to prevent further deterioration progress by enacting necessary remedial measures to the extent possible, based on cracks, steel material corrosion, or other phenomena that appear in the structure. When continuing such maintenance and accumulating inspection data makes it possible to estimate deterioration mechanisms at a given point, it is possible at that time to implement appropriate maintenance in line with the deterioration mechanisms, with reference to "Maintenance: Standards Appendix" Volume 1. If multiple deterioration mechanisms are compounded, it is advisable to refer to multiple chapters in "Maintenance: Standards Appendix" Volume 1.

Such maintenance constitutes a provisional measure until deterioration mechanisms can be estimated more reliably. When deterioration mechanisms have been estimated or when the effects of cracks and of steel corrosion have been excluded, maintenance of the deterioration mechanisms shown in "Maintenance: Standards Appendix" Volume 1, or maintenance equivalent to that performed before the mechanisms were discovered, must be resumed.

Chapter 5 Prediction

5.1 General rules

(1) To evaluate the future decline in performance of a structure during its remaining planned service period, it is necessary to predict the deterioration progress during that period.
(2) Prediction of the deterioration progress shall be performed by selecting appropriate methods while referring to factors such as the surrounding conditions or in similar structures.

Commentary: Regarding (1): For the maintenance of structures, it is essential to predict and evaluate not only the performance of the structures at the time of assessment but also future changes in their performance caused by the deterioration progress. Changes over time in the performance of a structure or in the structural members that compose it are primarily caused by deterioration. Therefore, in principle, it is necessary to estimate the mechanisms of deterioration and to predict the deterioration progress using appropriate methods in accordance with the status of deterioration.

Regarding (2): It is advisable to quantitatively predict the deterioration progress by selecting an appropriate method based on the past knowledge, further adding to the data that was obtained through inspections. In doing so, it is advisable to use as reference the state of nearby or similar structures that face nearly the same conditions as the structure in question.

5.2 Prediction of deterioration progress based on deterioration mechanisms

(1) When it is possible to estimate the mechanisms of deterioration, the deterioration progress must be predicted after selecting appropriate methods to express the deterioration progress.
(2) When predicting the deterioration progress, variability in the deterioration progress must be appropriately taken into consideration.

Commentary: Regarding (1): When the mechanisms of deterioration can be estimated, it is possible to predict deterioration progress in accordance with these mechanisms using the deterioration prediction models presented in "Design" and "Maintenance: Standards Appendix" Volume 1. By inputting the prediction results into the performance evaluation formula used in design or into nonlinear finite element analysis, changes in the performance of the structure can be predicted. Toward that end, it is necessary to collect required data paying attention to quantity and quality when performing inspections to enhance the accuracy of prediction results in line with the prediction method used.

When predicting the progress of compound deterioration caused by multiple mechanisms of deterioration, attention must be paid to the fact that this progress is not necessarily a simple summation of the deterioration states from cases of singular occurrences of the acting deterioration mechanisms. It must also be noted that the degree of accuracy with which the deterioration progress can be predicted varies with the state of deterioration of the structure, parts, or structural members.

Regarding (2): Even when mechanisms of deterioration can be estimated and methods that enable quantitative prediction can be selected, it is necessary to select the various characteristic values used in prediction and evaluate the results, considering the presence of variability in the actual deterioration progress. As maintenance requires evaluation that is premised on significant variability, a stochastic prediction formula is required in some cases. Therefore, it is necessary to accumulate data for accurately quantifying the variability. It is necessary to keep in mind that, because of differences in the micro-environment such as the shapes of structural members or the presence of water exposure, the deterioration progress is not constant even within a single structure or structural member.

5.3 Prediction of deterioration progress based on statistical data

When prediction of deterioration progress based on mechanisms of deterioration is difficult, the deterioration progress may be predicted by statistically analyzing the degree of deterioration occurring in the structure.

Commentary: When estimating mechanisms of deterioration is difficult and when multiple mechanisms of deterioration are likely but cannot be narrowed down to a single mechanism of deterioration, it may be difficult to make predictions based on the deterioration mechanisms even when said mechanisms have been estimated, for reasons such as the lack of appropriate prediction methods. In such cases, the structure is evaluated through methods such as its appearance grade. If said evaluation reveals an accumulation of many changes over time, the performance of statistical analysis using these results may enable prediction of deterioration progress with relative ease.

Chapter 6 Evaluations and Judgments Concerning Performance

6.1 General rules

(1) Evaluation of the performance of a structure shall be performed using appropriate methods and in consideration of the changes that occur during the planned service period.

(2) Judgments on the necessity of remedial measures shall be made on the basis of the results of evaluations of performance and on maintenance limits.

Commentary: Regarding (1): The performance of a structure during its service period declines due to deterioration or damage. Therefore, the performance of the structure not only at the time of inspection but also in the future must be evaluated as quantitatively as possible, and judgments must be made on the necessity of implementing remedial measures, based on the findings of inspections and on predictions of the progress of deterioration.

When there are no obvious anomalies of the structure or when anomalies are minor and there is room for further progress in performance and deterioration with respect to the set maintenance limits, it is possible to perform evaluation of inspection findings and of anomalies during the remaining planned service period that was predicted on the basis of inspection findings by comparing the categorized appearance grade according to the degree of anomalies. In these cases, however, because detailed examination is omitted, the basic approach is to conduct evaluation to be on the safe side or to set maintenance limits on the safe side in advance. At present, performance evaluation formulas have not been established for the degree of effect on third parties and appearance in structures; however, evaluation based on grading is effective for these as well.

The basic approach is to conduct evaluation of the performance of the structure during inspections and at the end of the planned service period at a minimum. However, in cases such as a long remaining planned service period, points in time during the remaining planned service period and the performance required for the structure up to those points may be set considering maintenance limits, etc., and performance during the remaining planned service period may be evaluated by conducting repeated performance evaluations of the structure at the set times. When it is difficult to estimate the mechanisms of deterioration, care must be taken so that evaluations of the future performance of the structure will be sufficiently on the safe side.

Regarding (2): If the performance of the structure at the time of inspection has already reached or fallen below the maintenance limits then some form of remedial measures are required at an early stage. If, instead, performance has not reached the maintenance limits at the time of inspection but is predicted to reach the maintenance limits by the end of the planned service period, the necessity of remedial measures must be determined from the degree of decline in evaluated performance as well as by comprehensive consideration of the importance and planned remaining planned service period of the structure, as well as higher-level plans such as maintenance plans for the structural group and the area as a whole.

6.2 Evaluation of performance

6.2.1 General

(1) Evaluation of the performance of a structure shall be performed in accordance with the purpose of assessments.
(2) Evaluation of the performance of a structure shall be performed using the findings of inspections.
(3) In the initial assessment, performance at that time is evaluated from the state of the structure as confirmed through inspections, the progress of deterioration is predicted as required, and performance during the remaining planned service period is evaluated from the results.
(4) During periodic assessments, inspection findings are compared with past findings, changes over time in the state of the structure and structural members are assessed, and performance at the time is evaluated. Prediction of the progress of deterioration is also performed, and performance during the remaining planned service period is evaluated from the results.
(5) For extraordinary assessments through extraordinary inspections, the performance of the structure is evaluated appropriately and as quickly as possible by inspection to quickly enable remedial measures aimed at preventing harm to persons as well as serious social or economic impacts caused by the collapse or failure of the structure.
(6) For extraordinary assessments through emergency inspections, the presence or absence of anomalies is confirmed in structures that are similar to the anomalies of the structure with the potential to cause accidents have occurred, and, even if no anomalies are confirmed, performance is evaluated in accordance with conditions and considering the possibility of anomalies occurring by the end of the planned service period.

Commentary: Regarding (3): During the initial inspection of a new structure or an existing structure and members immediately after repairs or reinforcements have been performed, visual observation can be used to confirm that initial defects, damage, deterioration, and other anomalies are absent or that remedial measures to address these have been properly implemented with apparent anticipated effects, while also confirming design drawings, construction records, and other records for the structure through document investigations. By doing so, evaluation of whether the performance of the structure at the start of maintenance is equivalent to the performance envisioned at the design stage or the stage of implementing remedial measures should be conducted. In addition, it is possible to predict and evaluate the performance of a structure until the end of its expected service period to some extent by applying the verification methods for the durability of the structure described in the design drawings and considering information such as the condition of the structure obtained through standard investigations, environmental and usage conditions. These are the same as the methods used for periodic assessments (see (4)).

When beginning new maintenance for an existing structure, anomalies may already be occurring at that time. In particular, when anomalies are caused by deterioration, the performance at the time of inspection and the future performance of the structure or members should be evaluated as quantitatively as possible based on the results of estimation of mechanisms of deterioration and prediction of deterioration progress.

In the case of an existing structure for which verification of the durability was not performed at the time of design, circumstances may arise in which it is difficult to properly evaluate performance at the time of inspection or future performance using general standard investigations. In such cases, it is advisable to perform additional detailed investigations as necessary in the initial inspection to enable quantitative evaluation of performance.

Regarding (4): Because the purpose of periodic assessment is to periodically check the state of the structure or structural members based on the maintenance plan, periodic assessments evaluate whether inspection findings are as envisioned at the time of formulating the maintenance plan and to evaluate the performance based on differences from the plan, etc.

(i) Assessments based on routine inspections: For assessments based on routine inspections, the effects of anomalies on the service of the structure are evaluated by checking for the presence or absence of new anomalies within routine work, or, if anomalies are already occurring, by checking the condition of their progress. As a finding of routine inspection, when anomalies that differ significantly from the trends estimated from inspection findings have been confirmed, it is necessary to estimate the cause and evaluate performance from the degree of the anomalies.

Because standard investigations in routine inspections mainly involve visual investigations of structures and structural members, the degree of deterioration, damage, or initial defects are assessed from anomalies. Therefore, if degrees of respective anomalies are associated with appearance grades and are set in advance when creating manuals for routine inspections, the performance of the structure and structural members can be efficiently evaluated through routine inspections. These evaluation standards can be set by considering the service conditions of the structure and structural members, the maintenance limits, etc. However, the basic approach is to assume that detailed examination will be omitted and to create evaluation standards that fall on the safe side.

(ii) Assessments through regular inspections: When anomalies are confirmed through regular inspections, the causes of anomalies should be estimated according to the above method for routine inspections and should be classified as corresponding to deterioration, damage, or initial defect. The degree of anomalies should be assessed as quantitatively as possible, and the performance of the structure and structural members should be evaluated. If the anomalies are due to deterioration, estimation of the deterioration mechanisms and prediction of the deterioration progress should be performed by comparing the inspection findings with past findings, after which the performance of the structure and structural members at the time of inspection and during the remaining planned service period should be evaluated as quantitatively as possible.

For structures that have a history of repairs and reinforcement, there may be cases where direct observation is not possible, such when the concrete is covered. Therefore, it is necessary to select investigation methods suited to the construction work methods of the implemented remedial measures and to consider the results of these in evaluating performance.

Regarding (5): When extraordinary assessments must be conducted, it is necessary to evaluate the performance of the structure at that time as quickly as possible using methods appropriate to the purpose of the inspection and to check conditions around the structure and clarify the performance for which restoration is required.

For extraordinary inspections performed when design standards, etc. are revised, it is necessary to evaluate conformance of the performance of the structure with standards and to confirm that problems will not arise before the end of the planned service period.

As the content of standard investigations for extraordinary inspections, which were set in advance in the maintenance plan in anticipation of disasters, may be insufficient even when additional detailed investigations are performed, evaluations of load bearing capacity, stiffness, etc. conducted to aid in the selection of remedial measures must be performed as quickly as possible in light of urgency. Conversely, when performing subsequent detailed investigations for safe use of a structure for which urgent remedial measures have been implemented, performance evaluations of the structure based on the investigation results are necessary, envisioning normal conditions of use following restoration.

Regarding (6): When anomalies have been confirmed in emergency inspections, the possibility of an accident occurring even in an inspected structure is high. Therefore, it is necessary to consider the implementation of appropriate remedial measures.

In contrast, even when no anomalies have been confirmed, the design standards applied to the structure, materials used, construction methods, and other information should be collected at the time of inspection according to the circumstances, and the probability of anomalies occurring by the end of the planned service period should be estimated.

6.2.2 Methods of performance evaluation

(1) Evaluation of the performance of a structure is in principle performed by selecting appropriate evaluation methods according to the aspects of performance to be evaluated and by appropriately modeling anomalies to suit the selected methods.

(2) Evaluation of performance is in principle performed not only for the time of inspection but also for predicting the state at the end of the planned service period or at any other arbitrary time.

Commentary: Regarding (1): Methods for evaluation of the performance of the structure can be broadly divided into methods based on (i) appearance grade, etc., (ii) performance evaluation formulas in design, and (iii) nonlinear finite element analysis. Depending on the purpose of evaluation, a judgment is made on which of these methods or combination of them should be adopted. Because the properties of the materials that compose the structure and structural members, the degree of anomalies, and so on normally exhibit significant unevenness within the structure or structural members, to achieve precise performance evaluation it is necessary for inspections to obtain data of the quantity and quality demanded by the performance evaluation methods to be applied.

General methods of performance evaluation in accordance with typical required performance are as follows.

It is advisable to quantitatively evaluate load bearing capacity, the most important metric of safety. In general, this is classified according to the presence or absence of a decline in the performance of the structure and structural members or the degree of anomalies by using methods based on appearance grade. When more detailed evaluation is deemed necessary, it is evaluated quantitatively using methods based on performance evaluation formulas in design or methods based on nonlinear finite element analysis.

The degree of effects on third parties can be evaluated using the state of cracks as a metric. In general, however, because the occurrence of cracks is often set as a maintenance limit, evaluation based on appearance grade as shown in 6.2.3 is commonly adopted.

Serviceability can often be evaluated using metrics obtainable directly through inspections, including deflection, vibration properties, inclination, and the presence or absence of water leaks. Evaluation of the stiffness of structural members may be performed in the same manner as evaluations of the load bearing capacity described above.

Because evaluation of appearance is prone to subjectivity, when physical metrics for evaluating appearance cannot be set, evaluations based on appearance grade, as presented in 6.2.3, are adopted.

Durability is performance by which structures maintain safety, serviceability, restorability, degree of effects on third parties, and appearance over the planned service period. In maintenance, the evaluation of durability can be considered synonymous with the evaluation and prediction of deterioration.

Regarding (2): As the implementation of remedial measures, reviews of the maintenance plan, etc. are considered based on the evaluation results, performance evaluations must predict the state at the point in time set as the target in future period of service.

6.2.3 Performance evaluations based on appearance grade, etc.

When conducting performance evaluations based on the degree of anomalies in the structure, the relationship between appearance grade and the performance to be evaluated shall be set in advance.

Commentary: It is advisable to conduct performance evaluations using quantitative methods for all aspects of performance to be evaluated. However, evaluation formulas may not exist for all aspects of performance to be evaluated, or the precision of an evaluation formula may not be adequate. Moreover, the data needed as input for analysis models or evaluation formulas may not have been obtained. In such cases, the state of the structure, as assessed through inspections, may be expressed using means such as appearance grade, and the performance of the structure or structural members may be evaluated from the results. Appearance grade, etc. can also be used in screening to determine whether it is necessary to conduct performance evaluations through the more detailed methods described later.

When conducting performance evaluations using such methods, for each mechanism of deterioration, the relationship between appearance grade, etc. associated with the progress of deterioration and the deterioration process (i.e., periods of latency, progress, acceleration, and deterioration), as well as the relationship between appearance grade, etc. and the performance of the structure and structural members, are to be made clear. Based on the inspection findings, the degree of anomalies is then evaluated based on appearance grade, and the current performance is evaluated from the relationship between this grade and target performance.

6.2.4 Performance evaluations through evaluation formulas in design

(1) When conducting performance evaluations by applying evaluation formulas in design based on theories of reinforced concrete, the scope of application and validity shall be given full consideration.

(2) In principle, the characteristic values of materials, the conditions and specifications of structures and structural members, and other information used in performance evaluations are to be obtained through inspection.

Commentary: Regarding (1) and (2): If the degree of anomalies is small, such as when no significant initial defects or cracks caused by corrosion of steel materials are present, the assumptions set using performance verification in design can generally be considered to hold. However, when the degree of anomalies is large, such as in cases of cracks and other changes of the state in which corrosion cracks aligned with steel materials in the axial direction occur over the entire area, or other specific locations exhibits localized deterioration or damage such as concrete at lap joints and anchorage zones, the nonlinear finite element analysis shown in 6.2.5 should be used. Although there are also cases in which verification is omitted for reasons such as deformation of structural members being addressed through structural details when new installation is performed, if anomalies are apparent, the potential need for evaluation of the deformation arises. Therefore, it must be noted that for some items, evaluation using performance evaluation formulas in design is difficult.

When the applicability of performance evaluation formulas in design has been confirmed, evaluation of mechanical performance is possible through appropriate consideration of the effects of anomalies in material strength, dimensions, etc. However, the setting of specific numerical values must be performed with care, through means including referring to accelerated deterioration test results at the structural member level.

6.2.5 Performance evaluations through nonlinear finite element analysis

(1) When using nonlinear finite element analysis, analysis methods for which validity and scope of application have been verified shall be selected according to the purpose of the analysis.

(2) Nonlinear finite element analysis may be applied to any or all performance evaluations or calculations of response values and limit values in structures or structural members in which anomalies have occurred.

Commentary: Regarding (1) and (2): Nonlinear finite element analysis enables direct consideration of the effects of cracks and differences in material properties due to differences in position inside the structure. In other words, it can be considered suitable for accurately evaluating the performance of structures and structural members in which anomalies exhibiting considerable variability have occurred. Therefore, it should be applied when the objective is more detailed examination, such as when the effect of anomalies on the performance of the structure cannot be easily estimated, or when quantitatively evaluating cross-sectional failure of safety. When applying nonlinear finite element analysis, it is necessary to appropriately consider the effects of anomalies in modeling the structural members and materials that compose the structure, based on the results of detailed investigations of the structure.

6.3 Determination of the necessity of remedial measures

(1) The necessity of remedial measures is determined by whether performance at the time of inspection or the future performance, as evaluated from inspection findings, reaches or falls below the maintenance limits.

(2) When anomalies that affect safety, including the degree of effects on third parties, are confirmed in inspections, countermeasures shall be quickly considered.

(3) When anomalies have been confirmed in structures that are similar to structures in which anomalies with the potential to cause accidents have already occurred, immediate countermeasures shall be considered. Even when no anomalies have been confirmed, the necessity of remedial measures should be determined in accordance with circumstances.

Commentary: Regarding (1): Cases in which remedial measures are deemed necessary include (i) when performance retained by the structure is declining at present and has reached or fallen below the maintenance limits, (ii) when prediction of the deterioration progress reveals a high probability of the structural performance declining during the remaining planned service period, even if no problems exist at present, and (iii) when design standards for working load and seismic performance have been reviewed and adaptation to meet standards is necessary. When conducting evaluation and making judgments on long-term future performance, it is necessary to pay sufficient attention to the accuracy of predictions. For structures and structural members for which initial defects have been confirmed in inspections, the effects of the initial defects on the structure must be appropriately examined, and, when the initial defects are considered to adversely affect aspects of the structural performance or when they are not themselves progressive but could become factors that promote future deterioration, it is necessary to consider appropriate remedial measures.

Regarding (2): When it has been determined that a problem exists with the degree of effects on third parties due to falling pieces of concrete, as in delamination, peeling, or cracking of cover concrete, it is necessary to promptly take measures such as knocking down the cover concrete. In addition, in cases such as impairment of safety in terms of vehicular traffic due to punching out of deck slabs, urgent remedial measures are required, and, in some cases, it is necessary to determine the necessity of remedial measures with an eye toward enacting temporary remedial measures until permanent remedial measures can be implemented.

Structures that have suffered damage from accidental action often bear important roles in regional restoration. At the same time, because collapse of said structures due to damage could cause harm to large numbers of people nearby, it is highly necessary to enact urgent remedial measures for structures in disaster areas. Therefore, if examination of remedial measures can be performed solely through visual inspection of the concrete, appropriate repairs and reinforcement are to be implemented immediately. If it has been determined that the functions to be satisfied will not be secured even if remedial measures are taken, it is important to quickly enact remedial measures such as restriction of service or demolition/removal and to take action such that alternative functions can be smoothly secured.

For extraordinary assessments, evaluations are conducted and judgments are made with speed as the highest priority. Therefore, when evaluation reveals that there are no notable problems with the performance of a structure at the time of inspection, it can be determined that immediate remedial measures are not necessary. However, depending on the circumstances, a detailed investigation to evaluate performance should be conducted again at an appropriate time after the effects of the accidental action have subsided, and the necessity of remedial measures should be determined considering normal use.

Regarding (3): When an accident has occurred due to anomalies occurring in a structure or if an accident has not occurred but significant anomalies have been confirmed in the structure, it is necessary to conduct emergency inspections on similar structures to confirm whether similar anomalies are occurring. If such anomalies have been confirmed, urgent remedial measures will be deemed necessary even for structures that have been inspected. In particular, when there is a possibility of accidents in which the effects on third parties could be a problem, emergency remedial measures must be taken. Conversely, even if no anomalies have been confirmed, the necessity of remedial measures, including strengthening of inspections based on inspection findings, should be determined.

Chapter 7 Remedial Measures

7.1 General rules

(1) When the implementation of remedial measures has been deemed necessary, the performance to be targeted shall be set by considering the importance of the structure, maintenance category, remaining planned service period, mechanisms of deterioration, degree of decline in the performance of the structure, etc. After post-implementation ease of maintenance, economic performance, and environmental performance have been considered, appropriate types of remedial measures shall be selected and implemented.

(2) In implementing the remedial measures, an implementation plan and a post-implementation maintenance plan shall be formulated. When the mechanisms of deterioration of the structure are clear, implementation with reference to "Maintenance: Standards Appendix" Volume 1 shall be made the standard.

(3) When anomalies indicating immediate problems are observed, such as anomalies that involve a high possibility of effects on third parties, appropriate urgent remedial measures shall be promptly implemented.

Commentary: Regarding (1): Three performance levels to be set as targets: (i) performance between performance at the start of service and maintenance limits, (ii) performance at the start of service, and (iii) performance higher than performance at the start of service. Regarding (i), it also includes cases in which the remaining planned service period is short and resistance to deterioration is not a high requirement in repair and retrofit and cases in which time-limited remedial measures are implemented to cover a span of several years until the implementation of full-scale remedial measures.

Regarding (2): When maintenance plan after implementation of remedial measures has been significantly changed from the plan before implementation, the validity of the maintenance category should also be examined. When performing strengthening of inspections or restriction of service as a remedial measure, the post-implementation maintenance plan itself can serve as an implementation plan for the remedial measures.

Cases in which it is difficult to estimate mechanisms of deterioration are addressed through symptomatic remedial measures. When doing so, however, it is necessary to properly assess the efficacy of the remedial measures through means such as increasing the number of times inspections are performed after implementation of the remedial measures, while also obtaining information that allows performance evaluation of the structure to be conducted more properly.

Regarding (3): In cases such as subsidence in road surfaces or a high probability of problems involving the degree of effects on third parties such as when pieces of concrete are expected to fall, it is necessary to promptly implement appropriate urgent remedial measures for repair of the road surfaces, prevention of falling concrete through means such as anti-spalling net, restrictions on service or entry, etc.

7.2 Types and selection of remedial measures

Remedial measures shall be appropriately selected from the available ones, including strengthening of inspections, repairs, retrofits, restrictions on service, and demolition/removal.

Commentary: Regarding strengthening of inspections: It is necessary to review the maintenance plan based on the remaining planned service period and the results of evaluations and decisions, then determine the appropriate inspection frequency and investigation items and implement the strengthening of inspections. It is also advisable to determine criteria in advance for conducting future repairs, retrofits, service restrictions, or demolition/removal.

Regarding repairs:

Repairs are performed mainly for the following purposes:

(i) Repair of cracks, peeling, and other anomalies occurring in concrete structures
(ii) Removal of concrete that has absorbed deterioration factors due to permeation by chloride ions and carbonation
(iii) Prevention of repeat permeation by harmful substances through surface coating, etc.
(iv) Prevention of falling pieces of concrete, etc.
(v) Recovery of the appearance of the structure
(vi) Prevention of water leaks in structures for which watertightness is demanded, and other recovery of serviceability

From the standpoint of prevention, during the period of latency when deterioration has not manifested, planning repairs for the purpose of blocking deterioration factors or curbing the progress of deterioration is also important as it can reduce the burden of subsequent maintenance.

Regarding retrofit:

Retrofit is implemented for the purpose of enhancing mechanical performance, mainly in cases such as the following:

(i) When an existing structure no longer satisfies the performance required by the latest standards due to changes in design load, raising of required performance level, etc. In such cases, see "Maintenance: Standards Appendix" Volume 3.
(ii) When required performance is currently satisfied or could be satisfied through repairs, but is predicted to not be satisfied in the future because of decline in performance caused by loading and environmental actions.
(iii) When there is otherwise a necessity to enhance the mechanical performance of existing structural members, such as performing retrofit at the same time as increasing the number of traffic lanes or installing sound insulation barriers to cope with increased traffic volume.

Regarding restrictions on service: Restrictions on service must be implemented based on an appropriate implementation plan after determining the degree and methods of restricting working load based on the results of performance evaluation. Then, a maintenance plan must be formulated after the restriction of service.

In implementing service restrictions, as with strengthening of inspections, it is important to review the maintenance plan based on results of evaluation and judgments concerning the structure and on the remaining planned service period as well as to determine the appropriate inspection frequency and investigation items.

Regarding demolition/removal: It is necessary to formulate and implement an appropriate plan for demolition/removal considering impacts on aspects such as the surrounding environment, work safety, methods of disposal after demolition, the construction period, and economic performance. When formulating an implementation plan, construction methods suited to the structure must be selected. When selecting construction methods, full consideration must be given to impacts on the surrounding environment, work safety, recycling/reuse and other methods of disposal after demolition, construction period, economic performance, and so on.

Performing detailed investigations of the relationship between decline in performance of the structure and the state of deterioration of materials is comparatively easy at the time of demolition/removal. Therefore, for the purpose of improving maintenance technology, the degree of deterioration and durability should be investigated at the time of demolition/removal if possible, with the results recorded and used as reference when performing maintenance of similar structures.

7.3 Repairs and retrofit

7.3.1 General

(1) Repairs and retrofit of a structure shall be implemented with the target for required performance made clear, so that the structure after repairs and retrofit satisfies the required performance for a set period.

(2) Repair and retrofit design shall be performed on the basis of assessment results and preliminary investigations. An appropriate construction plan based on the content of design shall be created, and a maintenance plan must be created for use after the repairs and retrofit.

(3) Construction work for repair and retrofit shall be performed on the basis of a construction plan, and appropriate management and examinations shall be implemented during the work and after its completion.

(4) When performing repairs or retrofit of a structure, parts, or members for which mechanisms of deterioration have been estimated, "Maintenance: Standards Appendix" Volume 1 should be used as reference.

Commentary: Regarding (2): The standard investigations and detailed investigations performed in inspections are often insufficient. Therefore, it is advisable to consider conducting repeat investigations for the design of repairs and retrofit. In this case, it is important to refer to design drawings, construction reports, or maintenance records while directly investigating the structure itself to confirm design conditions, the status of construction work, anomalies occurring in the structure, environmental actions, load action, and other actions and to assess constraint and problem points in performing the repairs and retrofit.

7.3.2 Design of repairs and retrofit

(1) In the design of repairs and retrofit, policies for the repairs and retrofit must be established and construction methods and materials shall be appropriately selected so as to satisfy the targeted performance.
(2) In the design of repairs and retrofit, appropriate methods shall be used to verify that the structure, parts, and structural members after repairs and retrofit will satisfy the target performance for the predetermined period.

Commentary: Regarding (1): In the design of repairs and retrofit, the target performance should be set and the composition of policies, scope, construction method, and materials, along with specifications, should be selected, taking into account mechanisms of deterioration and the items shown below.

(i) Current state of the target structure: Form/dimensions, arrangement/diameter of steel, etc.
(ii) Results of evaluations and judgments concerning performance
(iii) Mechanisms of deterioration
(iv) Importance of the structure (such as the presence or absence of detours, traffic volume, etc. for a road bridge)
(v) Environmental conditions: Weather conditions, site location conditions (mountainous areas, coastal areas), etc.
(vi) Load conditions: Load conditions that determine the dimensions of structural members and the arrangement of steel (such as the repetitive action of vehicle load that dominates in road bridge slabs, the seismic load that similarly dominates in bridge piers, etc.)
(vii) Constraint conditions on construction work: Construction time, construction period, available time, construction environment (ease of securing work space on site, noise, vibration, odor, etc.)
(viii) Types and combinations of types of repair and retrofit materials, thickness of surface covering materials, cross-sectional dimensions after repair and retrofit, construction methods, etc.
(ix) Maintenance: Ease of maintenance after repairs and retrofit, etc.
(x) Remaining planned service period
(xi) Economic performance, life cycle cost

Regarding repair of cracks: The main purpose of implementing remedial measures in structural members in which cracks occur is to restore durability, watertightness, and other aspects that have declined because of the occurrence of cracks. However, depending on the cause cracking, the repair of cracks alone may be inadequate, and it may be necessary to remove the causes of deterioration in structural members and to compensate for insufficient load bearing capacity following the descriptions in "Maintenance: Standards Appendix" Volume 1.

Points to note regarding types of cracks are as follows:

(i) Nonprogressive cracks

For initial cracks that are nonprogressive, repairing the cracks alone may enable the recovery of watertightness and resistance to deterioration that declined due to cracks. Because it is generally difficult to repair all cracks, it is necessary to appropriately determine the scope of cracks targeted for repair by incorporating the decisions in Chapter 6 at the stage of formulating plans for repair.

(ii) Progressive cracking

When the root cause of progressive cracks cannot be eliminated, it is difficult to effect sufficient repairs by repairing cracks alone. Therefore, to achieve the objectives of remedial measures, it is necessary to implement repairs in combination with effective remedial measures against the progress of deterioration, following the description in "Maintenance: Standards Appendix" Volume 1.

(iii) Cracks caused by structural factors

If cracks are estimated to have occurred due to the effects of load and other actions, it is necessary to confirm whether these are cracks that were envisioned in design. As a result, when cracks have been determined to be harmful cracks that exceed assumptions in design, it is necessary to implement repairs and retrofit related to mechanical performance as described later. In addition, because there is a danger that cracks caused by structural factors could affect the safety of the structure, it is necessary to remedial measure the width of cracks, stress in steel, and other items, and, depending on the remedial measurements, to implement service restriction or remedial measures to prevent harm to third parties.

Regarding repair of steel corrosion: Principles for the repair of steel corrosion include the following:

(a) Reduction of the supply of oxygen, water, etc. involved in corrosion (surface treatment methods, etc.)

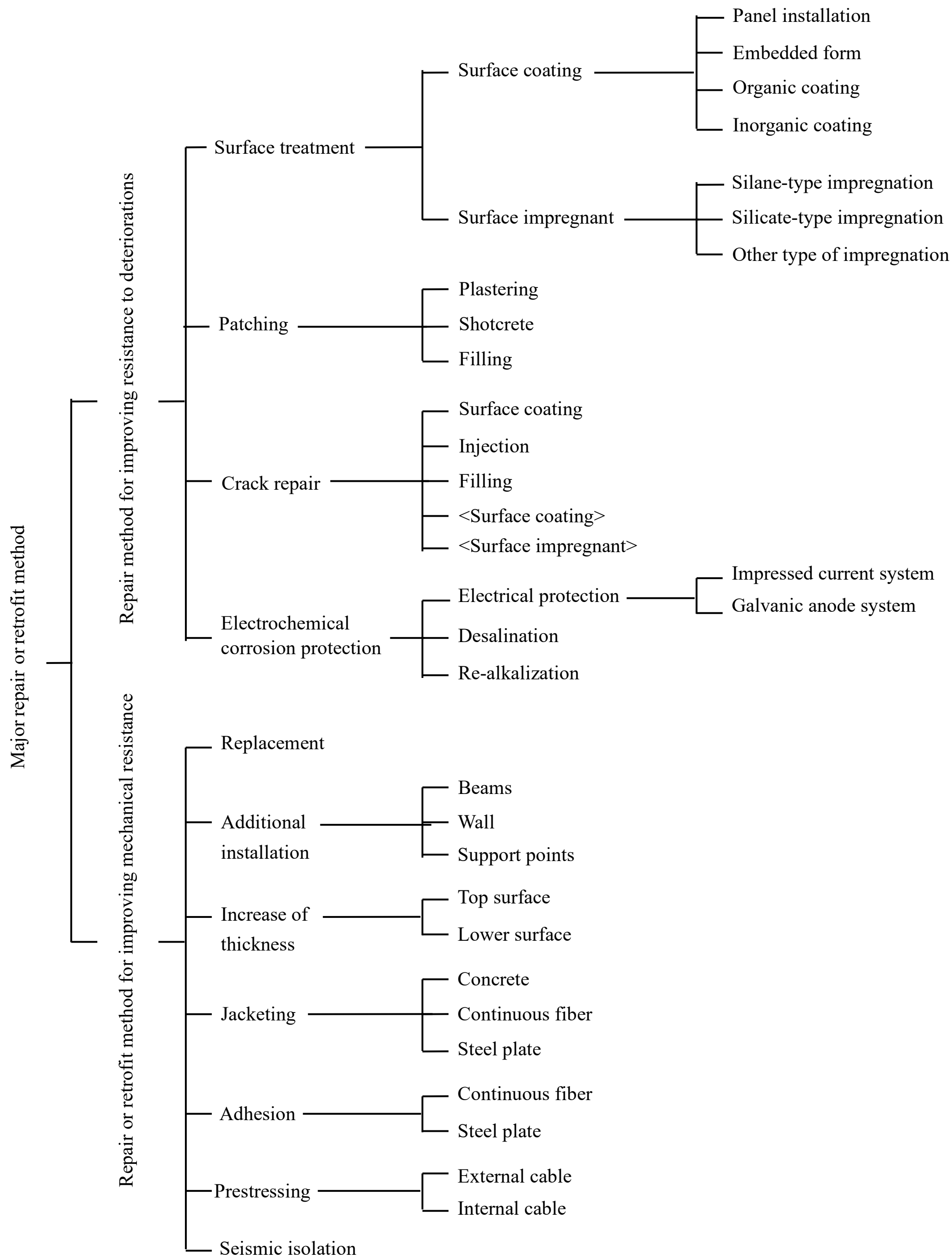
(b) Restoration of concrete cover and/or steel (cross-section restoration methods, etc.)

(c) Use of electrochemical methods

Regarding repair of damage or deterioration of indeterminate cause: Inappropriate repairs implemented to address damage or deterioration for which the cause cannot be identified may conversely accelerate the progress of deterioration. Moreover, the application of methods such as surface coating methods may make inspection difficult after the implementation of remedial measures. Therefore, when performing repairs in cases of indeterminate causes, it is important to examine remedial measures with care and to perform inspections continuously after the implementation of remedial measures.

Regarding repair and retrofit related to mechanical performance: In the design of repairs and retrofit aimed at the recovery or improvement of mechanical performance, it is important to comply with "Design," etc. and to create plans such that the performance required of the structure is satisfied for the predetermined period. It is also vital to clarify in advance any points of note in the design and construction work that are specific to repair and retrofit. It is further important to select construction methods and materials that facilitate investigations, considering maintenance after implementation of the repairs and retrofit. In addition, when selecting construction methods for the repairs and retrofit, it is necessary to consider impacts on the surrounding environment, work safety, disposal methods for the waste generated from construction, and other aspects of environmental performance, as well as the construction period, economic performance, and other matters.

In some cases of repair and retrofit, there may be no technology that fully satisfies the targeted performance. In such cases, the construction method and materials must be selected with repeated repair and retrofit in mind. When selecting construction methods and materials, it is important to assess properties of these in detail, to confirm efficacy through experiments, and to investigate the record of their application. The main repair and retrofit methods currently in use can be classified as shown in **Figure C7.3.1**.



※Surface coating and surface impregnation methods are classified as surface treatment methods, but they may also be used to repair cracks.

Figure C7.3.1 Main repair and retrofit methods applied to concrete structures

Regarding (2): Performance verification after repairs or retrofit must be performed with consideration of the efficacy or effects of their implementation. In doing so, it is necessary to set properties of materials considering the efficacy, etc. of repairs and retrofit. When the mechanisms of deterioration have been estimated, necessary material properties for repairs and retrofit must be set in accordance with the models for prediction of deterioration progress presented in "Maintenance: Standards Appendix" Volume 1. In the evaluation of performance, it is also necessary to set mechanical models by considering the effects that repairs and retrofit will have on mechanical properties. Because the methods for considering the efficacy or effects of these remedial measures with respect to mechanical properties are the same as in performance evaluation of existing structures, Chapter 6 may be applied.

Performance of the structure after repairs and retrofit will manifest through integration of the existing structure with the elements that compose the repairs or retrofit. Therefore, it is also necessary to appropriately evaluate the degree of integration of the two. In addition, when a possibility exists that repairs or retrofit will cause the behavior of the structural system as a whole to differ significantly from its behavior prior to the repairs or retrofit, it is necessary to appropriately verify not only the performance of locations that were repaired or reinforced but also the performance of the structure as a whole.

7.3.3 Repair and retrofit work

(1) Construction plans for repairs and retrofit shall be formulated based on design and fully considering constraints in construction work.

(2) Repair and retrofit work shall be performed with care based on the construction plan by considering specifications, precautions, and characteristics of the construction methods and materials used.

(3) In repair and retrofit work, rational and economical work management, inspection items, and inspection methods shall be set and implemented during and after the repair and retrofit work.

(4) Information concerning repair and retrofit work shall be recorded and managed by appropriate methods.

Commentary: Regarding (1): Failure to perform repairs and retrofit according to the construction plan may result in failure to obtain the targeted performance and in significant effects on subsequent maintenance. Therefore, it is important to formulate an appropriate construction plan with full understanding of the aims of design and the constraints on construction, with reference to 7.3.2.

Regarding (2): It is important that a concrete specialist engineer who possesses adequate construction work skills is assigned and that repair and retrofit work is performed with care based on the construction plan.

Regarding (3): When inspections clearly indicate that repairs and retrofit have not been performed according to design, it must be assumed that target performance has not been achieved and appropriate remedial measures must be enacted.

Regarding (4): When repairs or retrofit have been performed on a structure, the contents and results of the construction work and inspections will serve as key initial values for subsequent maintenance. Therefore, these must be recorded appropriately and stored in a form that allows easy use.

7.3.4 Maintenance after repairs and retrofit

(1) The maintenance plan following repairs and retrofit shall be appropriately formulated so that the structure will maintain the targeted performance throughout the remaining planned service period.
(2) Even when localized repairs and retrofit have been applied, maintenance shall be performed with the structure as a whole as its target.
(3) When repairs or retrofit have been applied to a structure, investigation items and methods shall be appropriately set with understanding of the locations and the content of the repairs or retrofit and considering maintenance limits, and inspections shall be performed.

Commentary: Regarding (1): After repairs and retrofit, the targeted performance of the structure must generally be maintained throughout the remaining planned service period or the design service period specified in the repair and retrofit design; in addition, continuity of the efficacy of repairs and retrofit must be confirmed. Therefore, in addition to reviewing the maintenance plan before implementation of remedial measures as necessary, a new maintenance plan that takes into account the state of the applied repairs and retrofit must be formulated. Assessment of the structure must be performed using appropriate methods based on this maintenance plan, and the results must be recorded. New remedial measures must also be systematically implemented as necessary.

Regarding (2): When localized repairs or retrofit have been performed, it is possible for the application of these to cause partial change in the degree of effects of environmental action or load action on the structure or structural members. This may cause changes in the stress distribution, the state of water content distribution inside the concrete, substance transfer properties, electrochemical balance, and other matters, resulting in anomalies that differ from the case of nonapplication of the repairs or retrofit. Depending on the type of structure and the environmental and loading conditions, new anomalies may occur in locations where repairs or retrofit were not applied, and the progress of these anomalies could become a major factor in performance decline in the structure as a whole. Therefore, even when localized repairs and retrofit have been applied, it is important to properly perform maintenance with the structure as a whole as its target, rather than focusing solely on the parts to which the repairs and retrofit were applied.

Regarding (3): In the initial inspection performed after repairs and retrofit, it must be confirmed that the performance targeted in repair and retrofit design has been achieved and that the scope of construction work, the construction methods, the composition of materials, and specifications are as envisioned at the time of repair and retrofit design and at the time of formulation of the construction plan. Records concerning repair and retrofit design and construction work may stand in for some investigation items in initial inspection after the implementation of remedial measures. In addition, comparing the state immediately after repairs and retrofit, as learned through investigations of documents in subsequent inspection, with conditions at the time of inspection can enable confirmation of whether the efficacy has been sustained. When conducting inspections, it is advisable to select investigation content that enables quantification of the efficacy of repairs and retrofit.

Chapter 8 Recording

8.1 General rules

In the maintenance of structures, the results of assessments and remedial measures shall be recorded and stored by appropriate means based on the maintenance plan.

Commentary: To perform maintenance of a structure efficiently and rationally, it is important to create and store records of the specifications of the structure, its design, and standards applied during construction work, along with construction records and records concerning assessments and remedial measures during service according to the maintenance plan and using methods that allow easy use as reference.

The validity of the maintenance plan and maintenance technology can be confirmed from maintenance records. Furthermore, analysis of records can aid in the progress of technology by clarifying problem points in design and construction work and points of improvement from a maintenance standpoint for incorporation into the design and construction plans, maintenance plans, and other plans for the construction of similar structures in the future.

8.2 Methods of recording

Recording shall be conducted by appropriate methods by using certain formats that are easily legible.

Commentary: Because the structure will be in service for a long period, changes in the organization that performs maintenance and in the persons in charge must be considered. Therefore, it is important that the series of assessment results be described using a certain manner. To do so, it is advisable to set methods of recording in accordance with the maintenance category, the content of maintenance, and the type of structure in advance. In principle, it is also important to consider formats that allow description of accurate and objective data.

8.3 Recording items

(1) When assessment of a structure is conducted, in principle, the names of the maintenance manager and persons performing the assessment, primary specifications of the structure, load and surrounding environmental conditions, maintenance category, methods and findings of inspections in the assessment, methods and results of prediction of deterioration, and results of determining the necessity of performance evaluation and remedial measures are recorded.

(2) When remedial measures such as repairs or retrofit have been implemented, information including the names of the maintenance manager and the contractors implementing the remedial measures is recorded, in addition to records concerning design, construction work, and inspections.

Commentary: Regarding (1) and (2): Points of particular note are shown below, according to the type of assessment.

Recording in initial assessment: In the initial inspection, in addition to items recorded during inspections, the position and condition of any initial defects are recorded. The content of performance evaluations and judgments based on the inspection findings are also recorded.

Recording in periodic assessments: In routine inspections, it is normally sufficient to record the date and time of inspection, the name of the person in charge of inspection, and the presence or absence of anomalies. When anomalies are found, the type, position, and condition of these must be recorded, along with the content of evaluations and judgments concerning them. In periodic inspections, in addition to items recorded during inspections, if deterioration or other issues are found, the type, position, and degree of any anomalies, the presence or absence of progress in the anomalies, and the content of evaluations and judgments based on inspection findings are recorded.

Recording in extraordinary assessments: Items to be recorded in standard investigations in extraordinary inspections and emergency inspections include, in addition to items recorded in inspections, details of the unexpected situation, the purpose of the inspection, the position and condition of damage, and the content of evaluations and judgments concerning performance.

When a detailed investigation has been conducted, the investigation items, investigation method, and the type, position, condition, etc. of the anomalies investigated are recorded in as much detail as possible, along with the results of prediction of deterioration, evaluations, and judgments based on the investigation results. The names of the maintenance managers and, if the assessment has been outsourced, the names of contractors performing the assessment are also recorded.

When remedial measures have been implemented, it is necessary to create records concerning conditions prior to the remedial measures, the method of the remedial measures, and the status of implementation. In the case of repairs and retrofit in particular, it may not be possible to perform visual checks of the frame of an existing structure or to estimate conditions prior to repairs or retrofit. Therefore, it is important to record in detail the conditions prior to implementing remedial measures. Furthermore, depending on the condition of the structure, it may not be possible to perform inspections according to plans. Therefore, it is important to record the implementation conditions in detail. Because it is possible that some materials used in repairs could be prone to the effects of weather, temperature, etc. when implementing remedial measures, creating records of this information is also important.

8.4 Storage of records

In principle, the period during which the structure is in service shall be the retention period for records.

Commentary: Because records of the maintenance of a structure constitute data for efficient and rational maintenance, in principle they should be stored for as long as the structure is in service. However, as the content of records may be of use in the maintenance of similar structures, if possible it is advisable to retain them even after the period of service has ended.

Standards Appendix

Volume 1
Mechanisms of Deterioration

Chapter 1 General Rules

1.1 Scope of application

Volume 1 of "Maintenance: Standards Appendix" (hereinafter "this volume") presents matters to keep in mind when performing maintenance on structures with the characteristics of deterioration mechanisms taken into consideration, when the types of deterioration mechanism that may lead to or have led to deterioration in concrete structures have been identified.

Commentary: In the case of structures for which mechanisms of deterioration have been identified, formulation of a maintenance plans, assessments, remedial measures, and recording must be carried out with full consideration of the characteristics of deterioration, with reference to Chapters 2 to 8 of this volume.

For the deterioration mechanisms presented in Chapters 2 to 8, deterioration processes are divided into the initiation stage, propagation stage, acceleration stage, and deterioration stage, based on the assumed progress of deterioration. **Figure C1.1.1** conceptually depicts the relationship between the degree of deterioration in individual mechanisms of deterioration and the decline in performance in the structures or structural members, plotted on the time axis. With regard to the decline in performance of the structures or structural members accompanying the progress of deterioration, individual deterioration mechanisms are not represented by a single curve; rather, the shape of the curve typically varies significantly with the aspect of performance in problems.

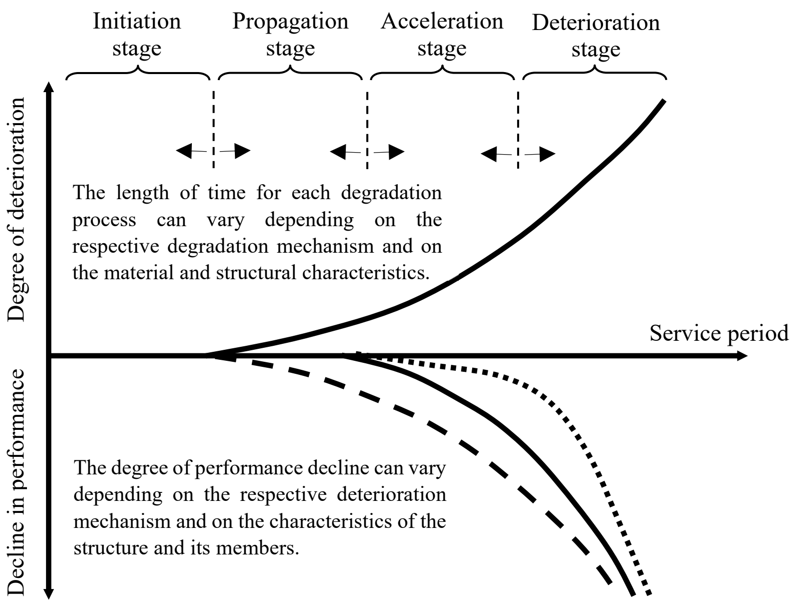


Figure C1.1.1 Conceptual diagram of the degradation progression process according to the degradation mechanisms in Chapters 2-8

Chapter 2 Carbonation

2.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to steel corrosion associated with carbonation and water penetration should be conducted with consideration of the characteristics of steel corrosion associated with carbonation and water infiltration, as presented in this chapter.
(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): Deterioration due to carbonation is frequently a problem in parts prone to cyclic wetting and drying in structures that use non-dense concrete and for which appropriate concrete cover is not ensured. Peeling cracks caused by steel corrosion are particularly likely to occur in parts with little concrete cover as these deteriorate first, leading to concern over hazards to third parties.

As shown in **Table C2.1.1**, the carbonation progress and the processes of deterioration caused by associated steel corrosion are divided into an initiation stage, propagation stage, acceleration stage, and deterioration stage. The effects of deterioration phenomena on the performance of a structures differ during each of these periods, and the degree of decline in performance corresponding to the progress of deterioration differs by the aspect of performance in question. For this reason, assessment (inspection, prediction of the progress of deterioration, evaluation, and judgment), remedial measures, and recording differ for each deterioration process.

Table C2.1.1 Definition of deterioration processes

Deterioration process	Definition	Factor determining the stage
Initiation stage	Period until the limit state for the occurrence of corrosion due to carbonation and water penetration	Carbonation rate and water supply
Propagation stage	From the initiation of corrosion of steel until cracking due to corrosion	Rate of steel corrosion
Acceleration stage	Stage in which steel corrodes at a high rate due to cracking due to corrosion	Rate of steel corrosion with cracks
Deterioration stage	Stage in which load bearing capacity is reduced considerably due to increased steel corrosion	

2.2 Maintenance plan

When conducting maintenance on structures that are subject to effects of steel corrosion associated with carbonation and water penetration, a maintenance plan and maintenance limits should be set with consideration of the progressive processes of deterioration.

Commentary: In structures that are subject to effects of steel corrosion associated with carbonation and water penetration, the progress of deterioration is greatly affected by not only the carbonation depth of the cover concrete but also the supply of water and oxygen. Maintenance limits and an appropriate maintenance plan must be set with consideration of these factors.

The items (i) to (iii) below are required for structures placed under maintenance category A. Note that (iii) is a factor affecting (i) and (ii) and is indirectly included in carbonation depth and in the state of steel corrosion, but is difficult to assess accurately at the design stage. It was selected as an item to be noted at the maintenance stage due to its necessity in the consideration of preventive maintenance.

(i) Quantitative assessment and future prediction of carbonation depth

(ii) Quantitative assessment and future prediction of the state of steel corrosion in concrete

(iii) The water supply condition to concrete and the water content of concrete

Conversely, for structures placed under maintenance category B, performance may be evaluated from the relationship between standard decline in performance and the processes of deterioration due to steel corrosion associated with carbonation, as shown in **Table C2.2.1**.

Table C2.2.1 Deterioration processes and standard performance degradation

Deterioration process	Load bearing capacity / Toughness	Deformation / Vibration	Peeling / Spalling	Cracking / Discoloration
Initiation stage	–	–	–	–
Propagation stage	–	–	–	–
Early acceleration stage	–	–	Occurrence of peeling or spalling	Cracking, rust water, exposed steel
Late acceleration stage	–	Increase in deformation, occurrence of vibration ・Decrease in steel cross-sectional area ・Decrease in adhesion between steel and concrete ・Reduction of concrete cross section due to delamination / peeling		
Deterioration stage	Degradation of load bearing capacity / toughness ・Decrease in steel cross-sectional area ・Decrease in adhesion between steel and concrete ・Reduction of concrete cross section due to delamination / peeling			

※Refer **Table C2.3.1** for the distinction between the early acceleration stage and the late acceleration stage.

Normally, maintenance limits should be set in line with quantitative performance. By contrast, the performance of a structure is often evaluated through reference to its appearance grade. However, when maintenance is performed using such methods, it is reasonable to also set the state of deterioration apparent in the external appearance of the structure as a maintenance limit. As examples, the following stages of deterioration may be set as maintenance limits:

(i) Corrosion has been confirmed in the steel inside concrete

(ii) Rust water has been confirmed on the surface of concrete

(iii) Cracking in the axial direction of steel has been confirmed on the surface of concrete

(iv) Spalling has been confirmed in the cover concrete

2.3 Assessment

2.3.1 General

In the Assessment of structures in which performance has declined or is likely to decline due to steel corrosion associated with carbonation and water penetration, it is necessary to properly conduct inspections, evaluation of current condition, prediction of the progress of deterioration, and judgment of the need for remedial measures, based on maintenance plans.

2.3.2 Inspections

2.3.2.1 General

(1) For inspection of structures that are subject to effects of steel corrosion due to carbonation and water penetration, initial inspection, routine inspections, and regular inspections should be properly conducted, based on the set maintenance categories and maintenance limits.

(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): In structures that are subject to effects of steel corrosion associated with carbonation and water penetration, the carbonation depth, contact with water, and the state of corrosion of internal steel are important indicators for the presence or absence of deterioration and the status of its progress. When deterioration has progressed considerably, inspections must be carried out to directly evaluate the performance of the structure. Appearance grades for cases of deterioration due to steel corrosion associated with carbonation and water penetration are shown in **Table C2.3.1**. However, as Grade I and Grade II are categorized according to the presence or absence of corrosion, categorization of grade is not possible through visual observation alone.

Table C2.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initial stage	No anomaly found in appearance. Before steel corrosion.
Grade II	Propagation stage	No anomaly found in appearance. Corrosion starts.
Grade III−1	Early acceleration stage	Cracking occurs due to steel corrosion.
Grade III−2	Late acceleration stage	With the progress of cracking due to corrosion, peeling or spalling is found. No loss of steel cross-section.
Grade IV	Deterioration stage	Peeling and spalling are found with cracking due to corrosion. Loss of steel cross-section .

It is necessary to conduct investigations mainly for assessing the carbonation progress and contact with water when a structure has been determined to be or is presumed to be in the initiation stage, or investigations mainly for assessing the progress of steel corrosion when the structure has been determined to be or is presumed to be in the propagation stage or later. Of the items to be investigated in each process of deterioration, for those that are related to prediction of the progress of deterioration, such as environmental actions, carbonation depth, steel corrosion, and the position of steel (i.e., concrete cover) it is advisable to collect data at the initial stage of maintenance if possible. Because carbonization progresses inward from the concrete surface, it is particularly important to assess the position of steel (i.e., concrete cover) at an early stage.

When a possibility exists of deterioration due to steel corrosion associated with carbonation and water penetration, the following points should be taken into consideration when selecting the location and scope of inspections.

(i) Environmental factors that affect the carbonation rate include carbon dioxide concentration, temperature, and ease of drying. Specific locations include railing, the undersides of deck slabs on overpasses on roads with heavy traffic, and the undersides of main girders. The carbonation rate is generally greater on the south and west sides where sunlight facilitates drying, which should be taken into account when selecting locations to investigation.

(ii) When locations with a greater carbonation rate in (i) are subject to effects of contact with water, deterioration associated with steel corrosion is likely to occur. Locations to which carbon dioxide and water are supplied and that are prone to drying due to sunlight include railings, the undersides of overhanging deck slabs on overpasses, and areas close to the ends of girders where water leakage from expansion joints occurs.

2.3.2.2 Initial inspection

(1) In an initial inspection, standard investigations are conducted as appropriate for newly constructed structures, existing structures, and structures after large-scale repair or reinforcement, to assess the initial state in maintenance of the structures.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult based on standard investigations in the initial inspection specified in the maintenance plan, detailed investigations should be conducted.

Commentary: Regarding (1): Particularly important items in the initial inspection of a structure subject to effects of carbonation include the following:

(i) The mix proportions of concrete, particularly the water-to-cement ratio (water-to-binder ratio)

(ii) The type of cement and the presence or absence, type, and amount of mineral admixture

(iii) Concrete cover

(iv) Environmental conditions

(v) The presence or absence of initial defects

(vi) The condition of repair and retrofit locations

Because (i) and (ii) above are input values for Equation (C2.3.2), they are indispensable items in prediction of the carbonation progress.

2.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of the early detection of deterioration and the assessment of its progress. These are based on the confirmation of water leakage, displacement, deformation, and other anomalies in appearance, as well as confirmation of concrete cracking, peeling and spalling of cover concrete, rust water, efflorescence, discoloration, and other abnormalities in the concrete surface.

(2) When unexpected anomalies have been found in standard investigations during the routine inspections specified in the maintenance plan, regular inspections should be conducted ahead of schedule, or detailed investigations should be conducted.

2.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail standard investigations that are more detailed than those in routine inspections, based on investigations of external appearance.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on the results of standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Investigations are generally performed through close-up visual observation of external appearance. However, investigations of not only the external appearance of a structure but also the carbonation depth in concrete, the state of corrosion of steel, or other factors may be required in some cases.

2.3.2.5 Detailed investigations

(1) For standard investigations conducted within the initial inspection and within regular inspections specified in the maintenance plan, when assessment of the current state of deterioration and the prediction, evaluation, and judgment of the progress of deterioration are difficult in a structure that is subject to effects of steel corrosion caused by carbonation and water penetration, or when standard investigations conducted within initial inspection, routine inspections, or regular inspections have revealed parts or structural members in which the progress of deterioration is significant, detailed investigations shall be performed to obtain more detailed information on the structure.
(2) The investigated items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

2.3.3 Prediction

2.3.3.1 General

(1) As a general rule, in the maintenance of structures in which steel has corroded due to carbonation and water penetration or in which steel corrosion caused by carbonation and water penetration is a concern, the performance of the structure at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of a structure requires quantitatively predicting deterioration due to steel corrosion associated with carbonation and water penetration.
(3) When (1) and (2) are difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the carbonation progress and the progress of steel corrosion.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

2.3.3.2 Prediction of the carbonation progress

(1) Prediction of the carbonation progress shall be made with appropriate consideration of the properties of the concrete and the effects of the environmental conditions under which the structure is used.
(2) Either of the following methods may be used in predicting the carbonation progress.

(i) $\sqrt{t}$ -law
(ii) Use of accelerated testing
(iii) Use of physicochemical models

Commentary: Regarding (2): There are three main types of technique for predicting the carbonation progress, as follows:

(i) $\sqrt{t}$ -law

Numerous studies have confirmed that carbonation depth is proportional to the square root of the carbonation period, as shown in the following equation.

$$y = b\sqrt{t} \qquad \text{(C2.3.1)}$$

where, y : carbonation depth (mm)

t : carbonation period (years)

b : coefficient for carbonation rate (mm/$\sqrt{\ }$years)

Therefore, when a measured value exists for carbonation depth, the coefficient of the carbonation rate should be obtained from the measurement and used to make predictions.

When inspection findings are not available, it is necessary to make predictions using a reliable equation for carbonation rate, with appropriate consideration of the concrete materials and mix and the environmental conditions. Many equations for carbonation rate use the water-to-cement ratio (water-to-binder ratio) and compressive strength as parameters representing the concrete properties. In making predictions, it is advisable to use equations intended for the same or similar materials, mix, and environmental conditions as those of the structure in question. When no such equation is available, the following equation may be used.

$$y = \gamma_{cb} \cdot (-3.57 + 9.0W/B)\sqrt{t} \qquad \text{(C2.3.2)}$$

where, W/B : effective water-to-binder ratio $= W/(C_p + k \cdot A_d)$

W : mass of water per unit volume

B : mass of effective binder per unit volume

C_p : mass of Portland cement per unit volume

A_d : mass of admixture per unit volume

k : constant set according to type of admixture

For fly ash: $k = 0$

For ground granulated blast furnace slag: $k = 0.7$

γ_{cb} : safety coefficient related to precision of prediction

As this equation does not take into account the effects of concrete curing or the environment, following the start of service, it is advisable to use inspection findings as needed to revise predictions of the progress of deterioration. Note that in the equation, γ_{cb} is a safety coefficient for the overall precision of prediction, and may generally be set to 1.0.

(ii) Use of accelerated testing

When estimating the carbonation rate through accelerated testing, the acceleration factor used in the testing should be confirmed by simultaneously performing accelerated testing on the target concrete as well as on concrete specimens made with a mix identical to concrete for which carbonation depth has been made clear in structures and through natural exposure testing.

(iii) Use of physicochemical models

Physicochemical models offer an advantage in allowing highly accurate predictions if the parameters for calculation are set appropriately. The parameters must be carefully set with reference to test results and other information.

2.3.3.3 Prediction of the progress of steel corrosion

(1) Prediction of the progress of steel corrosion associated with carbonation and water penetration shall be made with appropriate consideration of the concrete properties and the effects of the environmental conditions.
(2) As a general rule, the starting period of steel corrosion is determined from carbonation residue and the state of water supply.
(3) Any of the following methods may be used to predict the progress of steel corrosion before the occurrence of corrosion cracking:
(i) Methods based on the amount of corrosion found in inspections
(ii) Methods based on the corrosion rate of steel
(4) Any of the following methods may be used to predict the occurrence of corrosion cracking:
(i) Methods of determination based on the amount of corrosion
(ii) Methods using mechanical models
(5) Prediction of the steel corrosion progress after the occurrence of corrosion cracking shall be performed after appropriately evaluating the effects of cracking on mass transfer.

Commentary: Regarding (2): Based on the results of previous studies and investigations of structures, the criteria for determining the start of corrosion may be set to 10 mm of carbonation residue in a general environment of contact with water.

Depending on the conditions of the supply of water and oxygen, the carbonation progress and steel corrosion may be unrelated. When concrete is left in a relatively dry state, the carbonation rate increases. However, when the water content of concrete is below the level required for corrosion to occur, it can be assumed that major steel corrosion that would affect the performance of the structure will not occur. Moreover, when the concrete is in a continually wet state without cyclic wetting and drying, oxygen is not supplied and carbonation does not proceed, in which case it can be assumed that major steel corrosion will not occur.

Regarding (3): Techniques for the prediction of steel corrosion associated with carbonation and water penetration are summarized below.

(i) Methods based on the amount of corrosion found in inspections

When the change over time in the amount of corrosion has been measured through inspections, the progress of corrosion can be predicted though regression of the results. In particular, when the amount of corrosion and the starting time of corrosion can be found through inspections, subsequent prediction will be relatively easy. Even if the starting time of corrosion is unknown, the curve of the change over time can be estimated when measurements of the amount of corrosion for two or more material ages are available.

(ii) Methods based on the rate of steel corrosion

When measurements for amount of corrosion are not available, existing equations for calculation of the rate of corrosion may be used to appropriately evaluate the rate of corrosion and make predictions. Highly accurate predictions can be expected when the parameters of models can be appropriately set using electrochemical models that enable consideration of the effects of the rate of oxygen supply and of the water content in the concrete.

Regarding (4): As prediction of the occurrence of cracks boils down to determining the amount of corrosion at the time the cracking occurs, the following method should be used.

(i) Methods based on the amount of corrosion

The results of electrolytic corrosion testing in which corrosion progresses uniformly serve as reference for steel corrosion associated with carbonation and water penetration. By combining the results of investigations of steel corrosion in the target structure and the results of electrolytic corrosion testing, it is possible to predict when corrosion cracking will occur.

(ii) Methods using mechanical models

A number of methods of elasticity analysis and elasto-plasticity analysis that take into account the effects of concrete cover, concrete strength, steel diameter, and other factors have been proposed for the amount of corrosion at the onset of cracking. These may be used with model parameters appropriately determined.

Regarding (5): Most techniques for the prediction of steel corrosion following the onset of corrosion cracking are the subject of research at present and are not yet established techniques.

2.3.3.4 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the deterioration progress shall be revised following investigation of the causes of the differences. Changes should also be made to the maintenance plan as necessary.

2.3.4 Evaluation and judgment

(1) When evaluating the performance of structures undergoing steel corrosion associated with carbonation and water penetration, the characteristics of steel corrosion associated with carbonation and water penetration presented in this chapter should be considered.

(2) Judgment of the need for remedial measures must be made with consideration of the degree of decline in performance due to steel corrosion associated with carbonation and water penetration, maintenance limits, and remaining planned service period.

Commentary: Regarding (1): In evaluating the performance of a structure, the performance of the structure at the time of inspections and at the end of the planned service period must be quantitatively evaluated after first quantitatively evaluating the individual states of deterioration of the concrete and the steel that make up the structure.

Conversely, evaluation of the future aspects of performance of a structure combines the results of performance evaluation at the time of inspection with predictions of the deterioration progress, including the corrosion rate of steel and the carbonation rate, carried out in reference to **2.3.3**. However, it is often difficult to evaluate this with sufficient precision.

Quantitatively evaluating various aspects of performance of structures can also be difficult. In such case, it was decided that performance can be evaluated using **Table C2.3.1** as reference and using the appearance grades shown in **Table C2.3.2**.

Table C2.3.2 Appearance grades and factors of performance degradation

Appearance grade	Deterioration process	Load bearing capacity / Toughness	Deformation / Vibration	Peeling / Spalling	Cracking / Discoloration
Grade I	Initiation stage	–	–	–	–
Grade II	Propagation stage	–	–	–	–
Grade III−1	Early acceleration stage	–	–	Occurrence of peeling or spalling	Cracking, rust water, exposed steel
Grade III−2	Late acceleration stage	–	Increase in deformation, occurrence of vibration ・Decrease in steel cross-sectional area ・Decrease in adhesion between steel and concrete ・Reduction of concrete cross section due to delamination / peeling		
Grade IV	Deterioration stage	Degradation of load bearing capacity / toughness ・Decrease in steel cross-sectional area ・Decrease in adhesion between steel and concrete ・Reduction of concrete cross section due to delamination / peeling			

2.4 Remedial measures

2.4.1 Selection of remedial measures

As a general rule, when measures to remedy the decline in performance of a structure due to steel corrosion associated with carbonation and water penetration have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When using appearance grade as an indicator for the selection of remedial measures, the selection will depend on the type and importance of the structure, the rate of deterioration progress, the maintenance category, and the remaining planned service period. Remedial measure methods should be selected with reference to **Table C2.4.1**.

Table C2.4.1 Appearance grades and remedial measures

Appearance grade	Deterioration process	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	Initiation stage	○**	○**		
Grade II	Propagation stage	○	○		
Grade III−1	Early acceleration stage	◎	◎		
Grade III−2	Late acceleration stage	◎	◎*	○	
Grade IV	Deterioration stage		◎*	○	○

◎：Standard remedial measures（◎*：Including the restoration of mechanical performance），○：Remedial measures in some cases,
○**: Remedial measures implemented preventively

2.4.2 Repairs

(1) To obtain the desired effects from repair, work methods and materials shall be selected with consideration of decline in performance due to steel corrosion associated with carbonation and water penetration.

(2) After the implementation of repairs, it is necessary to confirm that the structural members satisfy the specified performance.

Commentary: Regarding (1): Repairs to structures in which performance has declined due to steel corrosion associated with carbonation and water penetration are divided into the work methods shown in **Table C2.4.2**, according to expected effects. **Table C2.4.3** should also be used as reference for the correspondence between appearance grades and remedial measure work methods.

Table C2.4.2 Expected effects and methods of repair or retrofit

Expected effect	Examples of methods
Control of the carbonation progress	Surface treatment (including spalling prevention), Crack injection
Reduce the carbonation depth to zero	Patching (including corrosion protection and coating), Re-alkalization
Control of the progress of steel corrosion	Surface treatment (Including spalling prevention), 【Electrical protection】, Patching, Re-alkalization, Corrosion protection, Water treatment
Control of the water penetration	Surface treatment (including surface impregnation and spalling prevention), Crack injection, Patching, Water treatment
Decrease in degree of hazards to third parties	Surface treatment (mainly spalling prevention)
【Restore load bearing capacity】	【Adhesion of steel or FRP plates, Lining, Increase of thickness】

【 】: Select in cases with a high rate of steel corrosion or a high volume of steel corrosion

※: In all deterioration processes, the combined use of water treatment is effective in controlling the progress of steel corrosion.

Table C2.4.3 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Example of methods: Load bearing capacity, Toughness, Deformation, Vibration	Example of methods: Peeling, Spalling
Grade I	Initiation stage	Surface treatment* (Including spalling prevention*), Re-alkalization*, Increase of thickness*	
Grade II	Propagation stage	Surface treatment (Including spalling prevention), 【Patching】, Re-alkalization	
Grade III−1	Early acceleration stage	【Electrical protection】, Re-alkalization, Patching	Surface coating (mainly spalling prevention)
Grade III−2	Late acceleration stage	Patching	Surface coating (mainly spalling prevention)
Grade IV	Deterioration stage	Patching, 【Adhesion of steel or FRP plates, Lining, Increase of thickness】	Surface coating (mainly spalling prevention), 【Adhesion of steel or FRP plates】

*: Preventive method

【 】: Select in cases with a high rate of steel corrosion or a high volume of steel corrosion

※: In all deterioration processes, the combined use of water treatment is effective in controlling the progress of steel corrosion.

2.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of deterioration is confirmed, remedial measures shall be promptly considered.

2.5 Recording

When creating records of inspections, prediction of deterioration progress, evaluations, judgments, remedial measures, and so on, matters specific to steel corrosion associated with carbonation and water penetration shall also be recorded.

Chapter 3 Chloride Attack

3.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to chloride attack should be conducted with consideration of characteristics of chloride attack, as presented in this chapter.
(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): The maintenance of structures in which performance has declined or is likely to decline due to chloride attack should be based on "Maintenance: Standards."

As outlined in **Table C3.1.1**, the deterioration in structures subjected to chloride attack can be divided into a initiation stage, propagation stage, acceleration stage, and deterioration stage.

Table C3.1.1 Definition of deterioration processes

Deterioration process	Definition	Factor determining the stage
Initiation stage	Period until the initiation of steel corrosion	Diffusion of chloride ions, initially contained chloride ion concentration
Propagation stage	Period from the initiation of steel corrosion until cracking due to corrosion	Rate of steel corrosion
Acceleration stage	Stage in which steel corrodes at a high rate due to cracking due to corrosion	Rate of steel corrosion with cracks
Deterioration stage	Stage in which load bearing capacity is reduced considerably due to the increase of corrosion amount	

3.2 Maintenance plan

When conducting maintenance on structures that are subject to effects of chloride attack, a maintenance plan and maintenance limits should be set with consideration of the progressive processes of deterioration.

Commentary: The maintenance plan for structures subject to the effects of chloride attack should be formulated according to Chapter 2 of "Maintenance: Standards." As the progress of deterioration due to chloride attack is greatly affected by factors including the source of the supply of chloride ions and the amount of chloride ions supplied to concrete during service, appropriate management limits taking this into consideration must be set and a management plan must be formulated. For structures in maintenance category A, however, the state of chloride ions in the concrete and the state of steel must be assessed as accurately as possible during the service period, taking note of the points below, and the maintenance plan must restrict the stage to the initiation stage (i.e., the state in which the steel in the concrete does not rust) or the propagation stage.

(i) Quantitative assessment and future prediction of the supply of chloride ions to structures

(ii) Quantitative assessment and future prediction of the state of penetration and accumulation of chloride ions in concrete

(iii) Confirmation and future prediction of the presence or absence of corrosion in steel in concrete

Conversely, for structures classified as maintenance category B, maintenance limits should be set and maintenance should be performed so that the performance of the structure does not fall below the required performance level.

As shown in **Table C3.2.1**, performance may be evaluated based on the general relationship between processes of deterioration due to chloride attack and decline in performance.

Table C3.2.1 Deterioration processes and standard performance degradation

<table>
<tr><th>Deterioration process</th><th>Load bearing capacity / Toughness</th><th>Deformation / Vibration</th><th>Peeling / Spalling</th><th>Cracking / Discoloration</th></tr>
<tr><td>Initiation stage</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Propagation stage</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Early acceleration stage</td><td>—</td><td>—</td><td>・Cracking, delamination</td><td>・Cracking, rust water
・Peeling, spalling</td></tr>
<tr><td>Late acceleration stage</td><td rowspan="2">Decrease in load bearing capacity or toughness
・Decrease in cross section of steel
・Decrease in adhesion between concrete and steel
・Decrease in cross section of concrete due to spalling, etc.</td><td rowspan="2">Decrease in stiffness
・Decrease in cross section of steel
・Decrease in adhesion between concrete and steel
・Decrease in cross section of concrete due to spalling, etc.</td><td rowspan="2">・Peeling, spalling</td><td rowspan="2">・Exposure of steel</td></tr>
<tr><td>Deterioration stage</td></tr>
</table>

In actual maintenance, it is often rational to set the following conditions for deterioration visible in the appearance of structures as maintenance limits.

(i) Rust water

(ii) Cracks in the axial direction of steel

(iii) Delamination, peeling, and spalling of cover concrete

(iv) Reduction in cross section of steel due to corrosion

3.3 Assessment

3.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to chloride attack, it is necessary to properly conduct inspections, evaluation of current condition, prediction of the progress of deterioration, and determination of the need for remedial measures on the basis of maintenance plans.

Commentary: In the maintenance of structures that are subject to chloride attack, initial assessment and regular assessments should be considered necessary actions. Extraordinary assessments are performed as needed and are not addressed here.

3.3.2 Inspections

3.3.2.1 General

(1) In the inspection of structures subject to chloride attack, the initial inspection, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance plan.
(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): The main items that should be assessed in inspections of structures that are subject to chloride attack are the distribution of chloride ion concentration in the concrete and the state of corrosion of internal steel.

The frequency and investigated locations of inspections should be determined with consideration of factors including the maintenance category, the environmental conditions, the structural form, and the state of deterioration of the structure. The main items to be investigated, methods, and points of note are explained below.

(i) Observation of the appearance of structures

Observation of the appearance of structures that are subject to chloride attack is mainly performed visually, primarily to check for rust water, peeling, spalling, and cracking associated with steel corrosion. In some cases, this may be combined with tapping to investigate delamination and peeling. Appearance grades for chloride attack are as indicated in **Table C3.3.1**. As appearance grades I and II are dependent on whether steel corrosion has begun, damage cannot be determined from observation of appearance alone.

Table C3.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	No anomalies in appearance, chloride ion concentration below the steel corrosion limit
Grade II	Propagation process	No anomalies in appearance, chloride ion concentration above the steel corrosion limit, initiation of steel corrosion
Grade III-1	Early acceleration stage	Occurrence of corrosion cracking or delamination, rust water
Grade III-2	Late acceleration stage	Numerous corrosion cracks with large width and length, partial peeling and spalling of cover concrete due to corrosion crack propagation, no significant reduction in cross section of steel
Grade IV	Deterioration stage	Large scale peeling/spalling due to corrosion crack propagation, significant reduction in cross section of steel, large displacement/deflection

(ii) Chloride ion concentration

The distribution of chloride ion concentration in concrete should be measured by taking concrete cores or concrete drill powder samples from the structure. When the chloride ion permeation depth is shallow, EPMA-based element analysis of the surface should be performed.

Estimation of the state of chloride ion permeation using sensors placed in cover concrete is another available method.

(iii) Steel corrosion

In investigations of steel corrosion, the cover concrete should be chipped away and the presence of steel corrosion, its position, its area, the reduction in cross section, and the presence of pitting corrosion should be checked. The state of steel corrosion can also be simply classified into grades, as shown in **Table C3.3.2**. Corrosion grades I, II, III, and IV in the table can be considered as roughly corresponding to the appearance grades I, II, III, and IV, respectively, shown in **Table C3.3.1**.

Table C3.3.2 Corrosion grade and state of steel

Corrosion grade	State of steel
Grade I	Mill scale. Thin, compact rust layer all over the steel. No rust adhesion to concrete surface.
Grade II	Swelling rust exists at some locations but is spotty in small areas.
Grade III	Loss of section cannot be confirmed visually. Swelling rust, however, exists all around the steel or throughout the length of steel.
Grade IV	Loss of cross section of steel.

(iv) Electrochemical indicators for steel corrosion

When no corrosion cracking is present on the surface of a structure and the location and degree of steel corrosion in the concrete cannot be assessed from appearance, electrochemical indicators can be effectively used.

Note that when the state of deterioration and decline in performance of the structure cannot be adequately evaluated through standard investigations, detailed investigations should be conducted. See 3.3.2.5 for information on detailed investigations.

3.3.2.2 Initial inspection

(1) In an initial inspection, standard investigations based on investigation of the overall structure are conducted through means such as visual inspection, tapping, and investigations of documents pertaining to design and construction work, to assess the initial state in maintenance of the structures.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on the results of standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Important items to be investigated in the initial inspection of structures that are subject to chloride attack include the following:

(i) The mix proportions of concrete (particularly the water-to-cement ratio (water-to-binder ratio)); compressive strength

(ii) The type of cement

(iii) The presence or absence, types, and amounts of mineral admixtures

(iv) The initial chloride ion content

(v) Concrete cover

(vi) Environmental conditions

(vii) The presence or absence of initial defects

(viii) The condition of repaired and retrofitted locations

3.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of early detection of deterioration and assessment of its progress. These are based on confirmation of water leakage, displacement, deformation, and other changes in appearance, as well as confirmation of cracking, peeling and spalling of cover concrete, rust water, efflorescence, discoloration, and other anomalies on the concrete surface.

(2) When unexpected anomalies have been found in standard investigations during the routine inspections specified in the maintenance plan, regular inspections should be conducted ahead of schedule or detailed investigations should be conducted.

Commentary: Regarding (1): As routine inspections mainly involve visual observation, basing evaluations and judgments on the findings of routine inspections is generally difficult for structures in maintenance category A. In this case, performing inspection in combination with appropriate sensor-based monitoring is also effective.

3.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail more detailed standard investigations conducted mainly through close visual observation.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on the results of standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): When measuring the distribution of chloride ion concentration in regular inspections, measurements should be taken multiple times during the predicted initiation stage to enable accurate prediction of the starting period of steel corrosion. Because the amount of steel corrosion in an actual structure cannot be directly measured, it is common to estimate the amount of corrosion in steel from measurements of the amount of reduction in cross section, from the rate of corrosion estimated from the polarization resistance, etc.

3.3.2.5 Detailed investigations

When prediction, evaluation, and judgment of the progress of deterioration in a structure are difficult based on only the results of standard investigations conducted in the initial inspection, routine inspections, and regular inspections, detailed investigations should be conducted to obtain more detailed information. The investigated items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

3.3.3 Prediction

3.3.3.1 General

(1) As a general rule, in the maintenance of structures that are subject to chloride attack, the performance of the structure at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of a structure requires quantitatively predicting the progress of deterioration due to chloride attack.
(3) When quantitative prediction of the progress of deterioration is difficult, methods using appearance grades may be used.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: In some cases, it is possible to estimate the performance of a structure during its planned service period by classifying the results of visual investigations of the structure into appearance grades or processes of deterioration based on **Table C3.3.2** and, taking into consideration the progress of steel corrosion and the permeation of chloride ions into concrete as discovered through inspections, by predicting the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage.

3.3.3.2 Prediction of chloride ion diffusion

(1) Prediction of the diffusion of chloride ions should be performed with appropriate consideration of the quality of the concrete and the environmental conditions of the structure.
(2) Either of the following methods may be used to predict the diffusion of chloride ions.
(i) Method of prediction based on apparent diffusion phenomena
(ii) Method of prediction based on the mechanism of chloride ion reaction and transport

Commentary: Regarding (2): The permeation of chloride ions into concrete accompanies the fixation, adsorption, etc. of chloride ions to cement hydration products and cement components, as well as advection caused by water transport and diffusion dependent on the concentration gradient of chloride ions. Prediction of the diffusion of chloride ions can be roughly divided into (i) methods of predicting diffusion by considering complex permeation phenomena as apparent diffusion phenomena; and (ii) methods of prediction based on mechanisms of transport of chloride ions together with capillary water due to cyclic wetting and drying and due to reactions such as the fixation and adsorption of chloride ions to cement hydrates, etc.

3.3.3.3 Prediction of the progress of steel corrosion

(1) Prediction of the progress of steel corrosion due to chloride attack should be made with appropriate consideration of the quality of the concrete and the environmental conditions.

(2) The starting period of steel corrosion can be determined using the concentration of chloride ions in the concrete on the surface of steel.

(3) Any of the following methods may be used to predict the progress of steel corrosion up to the occurrence of corrosion cracking.

(i) Methods based on the amount of corrosion found in inspections

(ii) Methods based on the rate of corrosion reaction in steel

(4) Any of the following methods may be used to predict the occurrence of corrosion cracking.

(i) Methods based on the amount of corrosion

(ii) Methods using mechanical models

(5) Prediction of the progress of steel corrosion after the occurrence of corrosion cracking is performed after appropriately evaluating the effects of the cracking on mass transfer.

Commentary: Regarding (1): Prediction of the progress of corrosion in steel must be performed after appropriate evaluation of environmental conditions such as cyclic wetting and drying, and with consideration of the balance between the amount of oxygen supplied and the water content of the concrete.

Regarding (2): As a general rule, the chloride ion concentration as a steel corrosion limit in structures subject to chloride attack should be set according to the relationship between the state of corrosion of the steel and the chloride ion concentration in the concrete on the steel surface, based on inspection findings for the structure.

Regarding (3): The following are two main methods for predicting the progress of steel corrosion up to the occurrence of corrosion cracking.

(i) Methods based on the amount of corrosion found in inspections

When the rate of corrosion of steel can be estimated through means such as the non-destructive polarization resistance testing method, the results may be used to predict the change in amount of corrosion over time. When it is possible to directly assess quantitative changes over time such as the surface area of corrosion or amount of reduction in cross section of steel by chipping away cover concrete, it is possible to predict the progress of steel corrosion from the results.

(ii) Methods based on the rate of corrosion reaction in steel

In addition to the diffusion of chloride ions, methods that use the amount of oxygen supplied and other parameters to estimate the corrosion reaction rate in steel are being studied.

Regarding (4): Prediction of the occurrence of corrosion cracking boils down to determining the amount of corrosion at the time the cracking occurs by predicting the progress of steel corrosion. The following prediction methods are available.

(i) Methods based on the amount of corrosion

Electrolytic corrosion testing is a method that can be used to derive the amount of corrosion at the onset of cracking.

(ii) Methods using mechanical models

A number of methods of elasticity analysis and elasto-plasticity analysis that take into account the effects of cover, concrete strength, steel diameter, and other factors have been proposed for deriving the amount of corrosion at the onset of cracking. These may be used with mechanical model parameters appropriately determined.

Regarding (5): A method for quantitatively predicting the progress of steel corrosion after the onset of corrosion cracking has not been established. As a practical remedial measure, prediction may be performed by replacing the apparent diffusion coefficient for chloride ions, the oxygen transfer rate, the water transfer rate, and other parameters in the varied corrosion models that have been proposed for cases in which corrosion cracking is not present, with the values for cases in which corrosion cracking is present.

3.3.3.4 Revision of predictions

When the status of deterioration obtained from inspection findings differs from the predicted value, predictions shall be revised following investigation of the causes of the difference. Changes should also be made to the maintenance plan as necessary.

3.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure subject to chloride attack, the characteristics of chloride attack presented in this chapter should be taken into consideration.
(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by chloride attack, maintenance limits, and remaining planned service period.

Commentary: Regarding (1): The evaluation of structures in which chloride attack has occurred should be conducted based on Chapter 6 of "Maintenance: Standards."

When evaluating the mechanical performance of a structure, the reduction in cross section of steel, the depth of pitting corrosion, the condition of anchored portions, the degree of delamination, peeling, and spalling in the concrete, the compressive strength of the concrete, the yield strength of the steel, and other mechanical properties obtained from inspection and from the results of prediction of the progress of deterioration should be substituted into the evaluation equations used at the design stage, and should be input into finite element analysis or other nonlinear analysis. By doing so, the displacement/deformation, load bearing capacity, toughness, and other properties of structural members during inspection and during the remaining planned service period can be evaluated.

Conversely, with regard to the degree of hazards to third parties, if a model (i.e., evaluation equations) pertaining to the corrosion grade or amount of corrosion in steel and to the delamination, peeling, and spalling of concrete can be constructed, then quantitative inspection findings, the results of predictions of the progress of deterioration, and other information concerning the degree of concrete deterioration and steel corrosion can be used to evaluate performance at the time of inspection and during the remaining planned service period.

In the case of chloride attack, classifying visual inspection findings into appearance grades based on **Table C3.3.1** or classifying inspection findings concerning the state of concrete deterioration and steel corrosion into processes of deterioration with reference to **Table C3.3.3** allows evaluation of the performance of structures at the time of inspection.

Table C3.3.3 Appearance grades and factors of performance degradation

<table>
<tr><th>Appearance grade</th><th>Deterioration process</th><th>Load bearing capacity / Toughness</th><th>Deformation / Water tightness</th><th>Peeling / Spalling</th><th>Cracking / Discoloration</th></tr>
<tr><td>Grade I</td><td>Initiation stage</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Grade II</td><td>Propagation process</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Grade III-1</td><td>Early acceleration stage</td><td>—</td><td>—</td><td rowspan="3">・Cracking, delamination

・Peeling, spalling</td><td rowspan="3">・Cracking, rust water
・Peeling, spalling

・Exposure of steel</td></tr>
<tr><td>Grade III-2</td><td>Late acceleration stage</td><td rowspan="2">Decrease in load bearing capacity or toughness
・Decrease in cross section of steel
・Decrease in adhesion between concrete and steel
・Decrease in cross section of concrete due to spalling, etc.</td><td rowspan="2">Decrease in load bearing capacity or toughness
・Decrease in cross section of steel
・Decrease in adhesion between concrete and steel
・Decrease in cross section of concrete due to spalling, etc.</td></tr>
<tr><td>Grade IV</td><td>Deterioration stage</td></tr>
</table>

Regarding (2): The need for remedial measures should be determined using the maintenance limits set in the maintenance plan, based on the degree of decline in performance in the structure due to chloride attack according to the evaluation performed in (1). If the performance of the structure is found through inspection to have reached the maintenance limit or is expected to reach the limit within its remaining planned service period, remedial measures should be considered in accordance with 3.4.

3.4 Remedial measures

3.4.1 Selection of remedial measures

As a general rule, when remedial measures to remedy the decline in performance of a structure due to chloride attack have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When using appearance grade as an indicator for the selection of remedial measures, the selection will depend on the type and importance of the structure, the rate of progress of deterioration, the maintenance category, and the remaining planned service period. Remedial measure methods should be selected with reference to **Table C3.4.1**.

Table C3.4.1 Appearance grades, deterioration processes and examples of remedial measures

Appearance grade	Deterioration process	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	Initiation stage	○	○**		
Grade II	Propagation process	○	○		
Grade III-1	Early acceleration stage	◎	◎		
Grade III-2	Late acceleration stage	◎	◎*	○	
Grade IV	Deterioration stage		○*	◎	◎

◎ : Standard remedial measures (◎* : Including the restoration of mechanical performance)
○ : Remedial measures in some cases (○* : Including the restoration of mechanical performance),
○** : Preventive remedial measures

3.4.2 Repairs

3.4.2.1 Design of repairs and repair work

(1) Repairs shall be appropriately designed through the selection of work methods and materials that yield the desired effect, taking into consideration factors including the decline in performance of the structure due to chloride attack, local environmental conditions, the duration of the efficacy of the work methods, the period of extended lifespan of the structure, and economic efficiency.
(2) Repair work shall be carried out so that the effect yielded by the designed repair method will be demonstrated for the specified duration.

Commentary: Regarding (1): Repairs on structures that have deteriorated due to chloride attack can be divided into the work methods shown in **Table C3.4.2**, according to expected effect. When selecting a work method, together with **Table C3.4.2**, the current state of the decline in performance of the structure must be taken into consideration. When selecting a work method with consideration of appearance grade, **Table C3.4.3** should be used as reference.

Table C3.4.2 Expected effects and methods of repair

Expected effect	Examples of methods	Related guideline
Reduction in the ingress amount of chloride ion	Surface treatment	CL119, CL137
Removal of chloride ion	Demineralization, patching	CL107, CL119, CL123
Anticorrosion of steel	Cathodic protection	CL107
Restoration of mechanical property	Patching, other	CL119, CL123, CL95, CL101

CL: Concrete Library of JSCE

Table C3.4.3 Appearance grades and examples of standard methods of repair

Appearance grade	Deterioration process	Standard methods
Grade I	Initiation stage	Surface treatment
Grade II	Propagation process	Surface treatment, demineralization, cathodic protection, patching
Grade III-1	Early acceleration stage	Patching, demineralization, cathodic protection
Grade III-2	Late acceleration stage	Patching (including restoration of mechanical property)
Grade IV	Deterioration stage	Patching (including restoration of mechanical property)

3.4.2.2 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of deterioration is confirmed, remedial measures shall be promptly considered.

3.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to chloride attack shall also be recorded.

Chapter 4 Freezing-and-thawing Damage

4.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to freezing-and-thawing damage should be conducted with consideration of characteristics of freezing-and-thawing damage, as presented in this chapter.
(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): In structures that are subjected to freezing-and-thawing damage, scaling and microcracking caused by the deterioration of cement paste, pop-outs connected to the quality of the aggregate, and other defects become apparent. Moreover, when water containing chlorides is supplied to concrete through deicing agents or seawater spray, scaling is accelerated.

Scaling, microcracking, and pop-outs caused by freezing-and-thawing damage reduce the resistance of concrete to material penetration. In maintenance, it is generally not advisable to ignore deterioration caused by freezing-and-thawing damage until the acceleration and deterioration stages shown in **Table C4.1.1** are reached.

Table C4.1.1 Definition of deterioration processes

Deterioration process	Definition
Initiation stage	Period until scaling, microcracks and pop-outs occur due to freezing and thawing cycles
Propagation stage	Period until scaling, microcracks and pop-puts occur, coarse aggregate is exposed, or multiple microcracks propagate on the surface
Acceleration stage	Period during scaling and microcracks propagate in depth direction, and spalling of coarse aggregate occurs
Deterioration stage	Stage during spalling of cover concrete occurs, leading to exposure and corrosion of steel

4.2 Maintenance plan

When conducting maintenance on structures that are subject to effects of freezing-and-thawing damage, a maintenance plan and maintenance limit should be set with consideration of the progressive processes of deterioration.

Commentary: In addition to environmental conditions, the materials and concrete mix of a structure have a large effect on the freezing-and-thawing damage. In particular, entrainment of minute air bubbles in concrete due to AE agents or other factors greatly enhances freezing resistance in concrete. In existing structures in particular, it is important to investigate whether AE agents are used.

When setting maintenance criteria, it is often reasonable to use the anomalies in appearance i) to iv) below as indicators.

i) Concrete surface scaling, pop-outs, and corner microcracks (D-cracking)

ii) Exposure of aggregate due to the progress of scaling and pop-outs, with the above-noted crack propagation

iii) Spalling of coarse aggregate

iv) Spalling of cover concrete, etc.

4.3 Assessment

4.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to freezing-and-thawing damage, it is necessary to properly conduct inspections, evaluation of current condition, prediction of the progress of deterioration, and determination of the need for remedial measures on the basis of maintenance plans and maintenance limit.

4.3.2 Inspections

4.3.2.1 General

(1) In inspections of structures that are subject to freezing-and-thawing damage, the initial inspections, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limit.

(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): In the case of freezing-and-thawing damage, it is advisable to focus inspections on areas that are subject to considerable moisture and sunlight. Grading of appearance is a method for appropriately evaluating the locations and degree of anomalies based on inspection findings. **Table C4.3.1** presents examples of appearance grades under deterioration caused by freezing-and-thawing damage.

Table C4.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	Subjected to freeze-thaw cycles, but no anomalies in appearance are observed
Grade II	Propagation stage	Scaling, microcracks and pop-outs occur on the surface
Grade III	Acceleration stage	Scaling and microcracks propagate in depth direction, and spalling of coarse aggregate occurs
Grade IV	Deterioration stage	Spalling of cover concrete occurs, leading to exposure and corrosion of steel

4.3.2.2 Initial inspection

(1) In initial inspections, standard investigations are conducted as appropriate for newly built structures, existing structures, and structures after large-scale repair or reinforcement, to assess the initial state in maintenance of the structures.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): The followings are important items in the initial inspection of structures under freezing-and-thawing damage.

a) Quality of concrete: Quality of aggregate, mix proportions of concrete, air content in fresh concrete, etc.

b) Quality of construction work: Cover, initial defects

c) Environmental actions, etc.: Minimum temperature, number of freezing and thawing cycles, supply of moisture, chloride supply

4.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of early detection of deterioration and assessment of its progress. These are based on the confirmation of scaling, microcracks, pop-outs, and other anomalies on the concrete surface, as well as the state of water supply (degree of water leakage in concrete) and other changes in appearance.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, regular inspections should be conducted ahead of schedule or detailed investigations should be conducted.

Commentary: Regarding (1): The main investigation items are changes in the state of the concrete surface due to scaling, microcracks, pop-outs, etc. specific to freezing-and-thawing damage. Even if deterioration is not found in the concrete, deterioration can be expected to proceed at locations where water is supplied. Therefore, it is advisable to check the state of water supply during inspection.

4.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail more detailed investigations of anomalies, the state of water supply, etc., mainly through close visual observation of appearance. It is also necessary to combine tapping-based methods and, as necessary, methods using non-destructive testing equipment, testing of collected cores, etc. in inspections, and to assess environmental conditions.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult based on standard investigations in regular inspections specified in the maintenance plan, detailed investigations must be conducted.

Commentary: Regarding (1): Because freezing-and-thawing damage progresses in winter when moisture in concrete freezes, to accurately assess the progress of deterioration in regular inspections, avoiding inspections during winter is advisable.

4.3.2.5 Detailed investigations

(1) In standard investigations conducted within the initial inspection and within regular inspections specified in the maintenance plan, when assessment of the current state of deterioration and the prediction, evaluation, and judgment of the progress of deterioration are difficult in a structure subjected to freezing-and-thawing damage, or when standard investigations conducted within initial inspection, routine inspections, or regular inspections have revealed parts or structural members in which the progress of deterioration is significant, detailed investigations shall be performed to obtain more detailed information on the structure.

(2) The investigation items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

4.3.3 Prediction

(1) As a general rule, in the maintenance of structures subject to freezing-and-thawing damage, the performance of the structure at the time of inspection should be quantitatively evaluated and future performance should be predicted.
(2) Predicting the decline in performance of a structure requires quantitatively predicting deterioration due to freezing-and-thawing damage.
(3) It is advisable to predict the progress of deterioration caused by freezing-and-thawing damage based on scaling depth and the state of cracking.
(4) When (3) is difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of deterioration.
(5) When the status of deterioration obtained from inspections differs from that predicted, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to the maintenance plan as necessary.

Commentary: Regarding (1) and (2): Predicting the decline in performance of a structure subject to freezing-and-thawing damage requires quantitatively predicting the decrease in concrete cross section caused by the propagation of cracking and scaling and caused by the progress of corrosion in the steel material in the concrete.

Regarding (3): Scaling depth and cracking as shown in **Figure C4.3.1** should be used as indicators for the deterioration of concrete due to freezing-and-thawing damage. As shown in **Figure C4.3.2**, it should be kept in mind that the rate of progress of scaling varies greatly before and after regulation of the use of spiked tires. The figure indicates the progress of scaling using a straight line, but measurements from multiple years are required as prediction based on a single year is difficult.

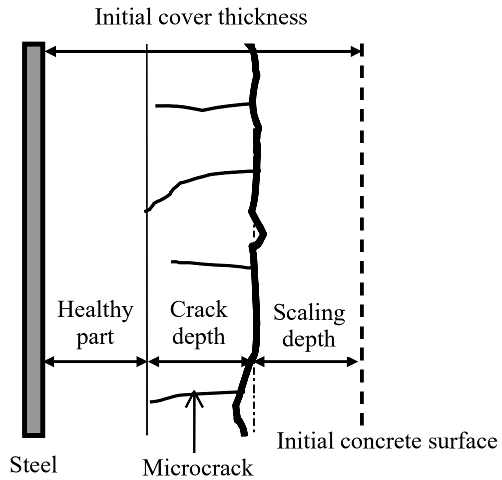


Figure C4.3.1 Conceptual diagram of deterioration due to freezing-and-thawing damage

When the occurrence of steel corrosion is suspected based on appearance, deterioration of the structure of the concrete will already have begun, and the concrete cannot be expected to serve as a protective layer for the steel material. Moreover, when chloride ions are supplied to concrete, there is concern that the rate of permeation of chloride ions in concrete subjected to freezing-and-thawing damage will be significantly greater than the case where no damage is present.

Regarding (4): When it is difficult to predict the progress of deterioration with consideration of the deterioration mechanisms of freezing-and-thawing damage, the use of probabilistic methods such as the Markov chain model enables ready prediction of the progress of deterioration in appearance grades.

Regarding (5): To accurately predict freezing-and-thawing damage, it is advisable to collect measurements over several years and to revise predictions whenever inspection data has been collected.

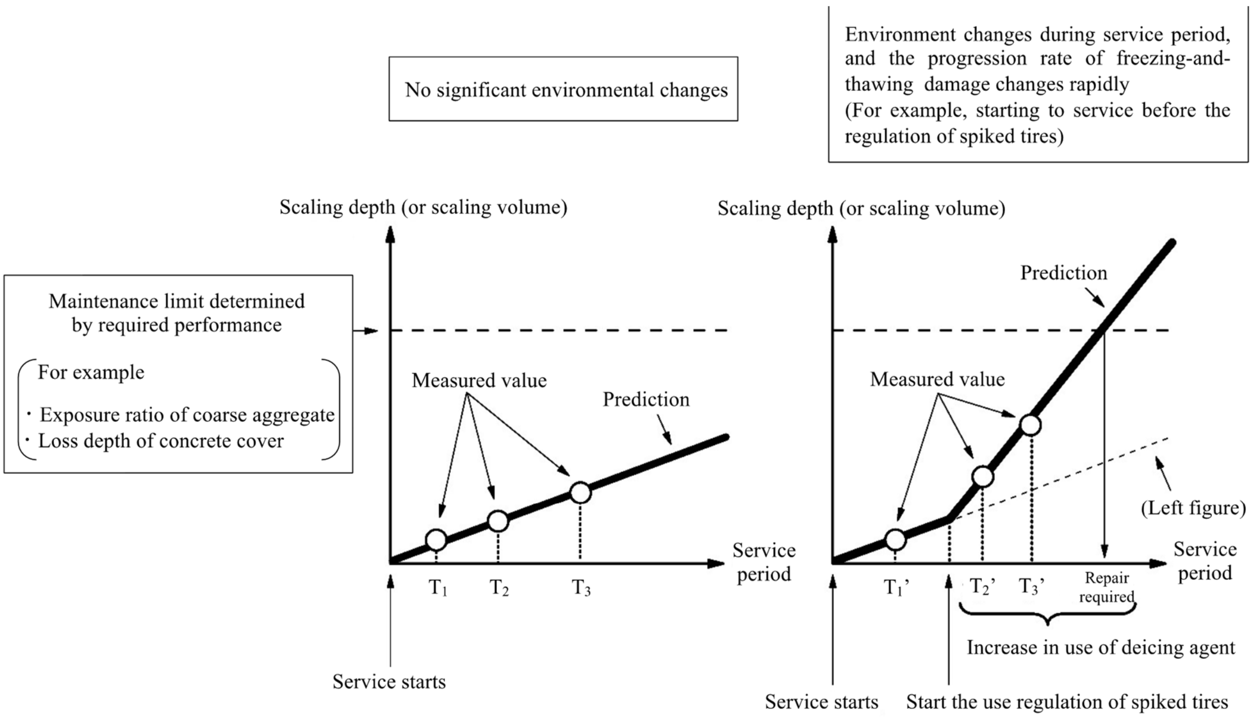


Figure C4.3.2 Schematic diagram of scaling progression prediction

4.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure in which freezing-and-thawing damage has occurred, the characteristics of freezing-and-thawing damage presented in this chapter should be taken into consideration.

(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by freezing-and-thawing damage, the maintenance limit, and the remaining planned service period.

Commentary: Regarding (1): An example of progressive states of deterioration is shown in **Table C4.3.2**. It should also be kept in mind that steel corrosion may be induced when scaling or microcracking have occurred in an environment in which chloride ions are supplied, such as from deicing agents.

Table C4.3.2 Appearance grades and standard performance degradation

<table>
<tr><th>Appearance grade</th><th>Deterioration process</th><th>Load bearing capacity / Toughness</th><th>Deformation / Vibration</th><th>Peeling / Spalling</th><th>Cracking / Discoloration</th></tr>
<tr><td>Grade I</td><td>Initiation stage</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Grade II</td><td>Propagation process</td><td>—</td><td>—</td><td>—</td><td rowspan="2">・Scaling
・Microcracks
・Pop-outs</td></tr>
<tr><td>Grade III-1</td><td>Early acceleration stage</td><td>—</td><td>—</td><td>・Scaling
・microcracks</td></tr>
<tr><td>Grade III-2</td><td>Late acceleration stage</td><td>—</td><td>・Propagation of cracks</td><td rowspan="2">・Exposure of coarse aggregate
・Propagation of cracks
・Steel corrosion
・Decrease in adhesion between steel and concrete</td><td rowspan="2">・Exposure of coarse aggregate
・Propagation of cracks
・Exposure of steel
・Steel corrosion
・Change in surface color such as rust water</td></tr>
<tr><td>Grade IV</td><td>Deterioration stage</td><td>・Spalling of cover concrete
・steel corrosion
・Decrease in steel cross-sectional area
・Decrease in adhesion between steel and concrete
・Decrease in mechanical property due to cracking</td><td>・Spalling of cover concrete
・steel corrosion
・Decrease in steel cross-sectional area
・Decrease in adhesion between steel and concrete
・Decrease in mechanical property due to cracking</td></tr>
</table>

4.4 Remedial measures

4.4.1 Selection of remedial measures

As a general rule, when measures to remedy the decline in performance of a structure due to freezing-and-thawing damage have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When selecting remedial measures with appearance grade used as an indicator, **Table C4.4.1** should be used as reference. Moreover, if initial inspection or detailed investigations find that the target structure does not use AE agents in an area with severe weather conditions or if there are parts subject to significant water contact, inspections should be strengthened.

Table C 4.4.1 Appearance grades and remedial measures

Appearance grade	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	○	○		
Grade II	◎	○	○	
Grade Ⅲ	◎	◎	○	
Grade Ⅳ		◎	◎	○

◎ : Standard remedial measures, ○ : Remedial measures in some cases

4.4.2 Repair

(1) To obtain the desired effects from repair, work methods and materials shall be selected with consideration of decline in performance in the structure and life cycle costs.
(2) After the implementation of remedial measures, it is necessary to confirm that the structures satisfy the specified performance.

Commentary: Regarding (1): The objective of freezing-and-thawing damage repair is the removal of deteriorated parts and the restoration of performance. Examples of repair methods are presented in **Table C4.4.2**. Freezing damage typically causes significant deterioration of physical properties in deteriorated parts of concrete. Remedial measures that prevent the supply of water and that replace locations that have been subjected to freezing-and-thawing damage are effective. As shown in **Table C4.4.3**, the methods to be selected differ with the degree of deterioration. In general, however, implementation at the initial stage of deterioration enables simpler remedial measures. Cutting off the supply of water is also effective.

Table C4.4.2 Expected effects and methods of repair (including preventive methods)

Expected effect	Examples of methods
Control the water supply	Surface treatment, Grout injection into cracks, Draining treatment
Control the chloride supply	Surface treatment, Grout injection into cracks, Draining treatment
Recovery of cross section	Patching, Grout injection into cracks

Table C4.4.3 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Standard method
Grade I	Initiation stage	Surface treatment*, Draining treatment
Grade II	Propagation stage	Surface treatment, Draining treatment
Grade Ⅲ	Acceleration stage	Patching, Surface treatment, Grout injection into cracks, Draining treatment
Grade Ⅳ	Deterioration stage	Patching, Draining treatment

*：Preventive method

Regarding (2): A structure on which remedial measures have been implemented consists of the concrete main body, the repair materials, and the interfaces between the two. After the implementation of repairs, it is necessary to evaluate whether the work has been carried out according to the repair design and, as a result, whether the specified performance is satisfied.

4.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of deterioration is confirmed, remedial measures shall be promptly considered.

4.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to freezing-and-thawing damage shall also be recorded.

Chapter 5 Chemical Attack

5.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to chemical attack should be conducted with consideration of characteristics of chemical attack, as presented in this chapter.
(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): Examples of chemical corrosion-inducing acids, inorganic salts, hydrogen sulfide, and corrosive gases such as sulfurous acid gas are shown in **Table C5.1.1**.

Table C5.1.1 Examples of acids, inorganic salts and corrosive gases that cause chemical attack

Deterioration	Acids	Inorganic salts	Corrosive gases
Deterioration by converting hydrates into soluble substances	Hydrochloric acid Sulfuric acid Sulfurous acid Nitric acid Hydrofluoric acid Carbonic acid Acetic acid Formic acid butyric acid	Sodium hydroxide (more than 10% concentration) Potassium hydroxide (more than 10% concentration) Ammonium hydroxide (more than 10% concentration) Magnesium chloride Zinc chloride Copper chloride Calcium chloride Sodium chloride (high concentration)	Sulfur dioxide Hydrogen fluoride Nitric monoxide Hydrogen chloride Chlorine Hydrogen sulfide
Deterioration by generating expansive compounds through a reaction with hydrates	Sulfuric acid	Sodium sulfate Magnesium sulfate Ammonium sulfate	

(Note) The above is merely an example, and it should be noted that the extent of concrete erosion caused by these substances varies greatly depending on the type and concentration of acids, inorganic salts, and corrosive gases.

As shown in **Table C5.1.2**, the processes of deterioration in structures caused by the progress of chemical attack and accompanying steel corrosion can be divided into an initiation stage, propagation stage, acceleration stage, and deterioration stage.

Table C5.1.2 Definition of deterioration processes

Deterioration process	Definition in case without protective layer	Definition in case with protective layer	Main factors determining the stage
Initiation stage	Period until the alteration of concrete due to ingress of chemically aggressive substances into concrete	Period until the alteration of concrete underlying concrete due to ingress of chemically aggressive substances through the protective layer and concrete	Rate of ingress of chemically aggressive substances into concrete or protective layer of concrete
Propagation process	Period until concrete cracking occurs or period until the exposure and peeling of aggregates in the concrete start		Rate of concrete erosion
Acceleration stage	Period until initiation of steel corrosion due to the increase of deteriorated depth by chemical attack and reach of deterioration factors to steel		Rate of concrete erosion
Deterioration process	Stage in which load bearing capacity is reduced considerably due to loss of cross section of concrete or steel		Rate of concrete erosion Corrosion rate of steel

5.2 Maintenance plan

When conducting maintenance on structures subject to chemical attack, a maintenance plan and maintenance limit should be set with consideration of the progressive processes of deterioration.

Commentary: Examples of setting maintenance limit based on the progressive processes of deterioration due to chemical attack are shown in **Table C5.2.1**.

Table C5.2.1 Examples of maintenance limit

Progress of deterioration due to chemical attack	Deterioration process
Penetration of deterioration factors into protective layer and concrete (before occurrence of change of properties in concrete)	Initiation stage
Exposure of aggregate (before occurrence of spalling of aggregate)	Propagation stage
Occurrence of delamination or spalling of aggregate on surface part	Acceleration stage
Deterioration factors reach to the position of steel and steel corrosion occurs	
Loss of cross section of concrete and decrease in load bearing capacity due to steel corrosion	Deterioration stage

5.3 Assessment

5.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to chemical attack, it is necessary to properly conduct inspections, evaluation of current condition, prediction of the progress of deterioration, and judgment of the need for remedial measures on the basis of maintenance plans.

5.3.2 Inspections

5.3.2.1 General

(1) In inspections of structures that are subject to chemical attack, the initial inspections, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limit.
(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): Investigation items in inspections differ according to the progressive processes of deterioration. It is necessary to conduct investigations mainly for assessing the state of the permeation of deterioration factors into concrete protective layers and into concrete when the structure has been determined or is presumed to be in the initiation stage; investigations mainly for assessing the progress of chemical attack when the structure has been determined or is presumed to be in the propagation stage or acceleration stage; or investigations mainly for assessing the progress of chemical attack or the loss of cross section in concrete when the structure has been determined or is presumed to be in the deterioration stage or later. When the performance of a structure cannot be quantitatively evaluated, anomalies in appearance provide useful information for the evaluation of performance. Appearance grades when deterioration has occurred due to chemical attack are shown in **Table C5.3.1**.

Table C5.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration without protective layer	State of deterioration with protective layer
Grade I	Initiation stage	No anomalies found in appearance of concrete	No anomalies found in protective layer of concrete
Grade II	Propagation stage	Cracks or rough condition found on concrete surface	Anomalies found in concrete protective layer and also anomalies found inside concrete
Grade III	Acceleration stage	Cracks and loss of cross section in concrete are significant and exposure or spalling of aggregate found	
Grade IV	Deterioration stage	Loss of concrete cross section and cracks propagate to the position of steel, and significant displacement or deformation due to decrease in steel cross section found	

5.3.2.2 Initial inspections

(1) In initial inspections, standard investigations are conducted as appropriate for newly constructed structures, existing structures, and structures after large-scale repair or retrofit, to assess the initial state in maintenance of the structures.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Particularly important items to be investigated in the initial inspection of structures subject to chemical attack include the following:

(i) Environmental conditions

(ii) Presence and type of concrete protective layer

(iii) Type of cement (binding material)

(iv) The concrete mix proportions, particularly the water-to-cement ratio (water-to-binder ratio) and concrete strength

(v) Presence, type, and amount of mineral admixture used

(vi) Concrete cover

(vii) The presence or absence of initial defects

(viii) The condition of repaired and retrofitted locations

In (i) above, the types of deterioration factors and their concentrations must be confirmed. (ii) is important for predicting the progress of the initiation stage, while (iii), (iv), and (v) are important for predicting the carbonation rate and permeation rate of deterioration factors in concrete. (vi) is important as a factor that affects the start and the rate of steel corrosion. (vii) has a major effect on the durability of structures, which makes its discovery important during the initial inspection. For (viii), it is important to investigate whether anomalies are present in the location of repair or retrofit, and if so, to investigate the applied repair or retrofit methods and their status.

5.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted to confirm the presence of odor and to visually confirm water leakage, displacement/deformation, and other anomalies in appearance, as well as concrete surface anomalies (change of properties, concrete cracking, peeling, spalling, rust water, efflorescence, etc.) and anomalies in the concrete protective layer (change of properties, cracking, spalling, peeling, etc.).

(2) When standard investigations have revealed signs of anomalies or deterioration not anticipated in the maintenance plan, regular inspections should be conducted ahead of schedule or detailed investigations should be conducted.

5.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail standard investigations of anomalies, water leakage, etc. more detailed than those in routine inspections, based on investigations of appearance.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Investigation items of not only the appearance of a structure but also the depth of permeation of deterioration factors, the depth of carbonation, the state of steel corrosion, and other factors may be required in some cases.

5.3.2.5 Detailed investigations

When prediction, evaluation, and judgment of the progress of deterioration in a structure subjected to chemical attack are difficult based on only the standard investigations conducted in the initial inspection, routine inspections, and regular inspections, detailed investigations should be conducted to obtain more detailed information. The investigation items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

5.3.3 Prediction

5.3.3.1 General

(1) In the maintenance of structures subject to chemical attack, the performance of the structure at the time of inspection is quantitatively assessed and future performance is predicted.

(2) As a general rule, predicting the decline in performance of a structure requires quantitatively predicting deterioration due to chemical attack.

(3) When quantitative prediction of deterioration is difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of steel corrosion and the progress of chemical attack in the concrete and the concrete protective layer.

(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: Regarding (3): The factors that determine processes of deterioration and their timing are as shown in **Table C5.1.2**. During the initiation stage, this calculation is made from the rate of permeation into concrete or the concrete protective layer by gases or by liquids containing substances that cause chemical attack; during the propagation stage and acceleration stage, from the rate of attack of concrete due to deterioration factors; and during the deterioration phase, from the rate of steel corrosion and the amount of steel corrosion by which load bearing capacity declines.

Regarding (4): When information on environmental conditions cannot be obtained for a new structure, or when there are no inspection findings for existing structures, etc., reference may be made to inspection findings for adjacent structures or structures in similar environments.

5.3.3.2 Prediction of the progress of chemical attack

(1) Prediction of the progress of chemical attack should be made with appropriate consideration of the quality of the concrete protective layer and the concrete, and the effects of the environmental conditions under which the structure will be used.

(2) The following methods may be used in predicting the progress of chemical attack.

(i) Methods based on inspection findings

(ii) Methods based on accelerated testing

(iii) Methods based on numerical analysis that considers environmental conditions and the transport and reactions of deterioration factors

Commentary: Regarding (1): Because visual or other observation is possible only for anomalies in the appearance of concrete, it is advisable to predict the depth of erosion of the concrete. When making more detailed predictions, the carbonation depth and permeation depth of deterioration factors, etc. should also be predicted. Note that, when a resin lining or other concrete protective layer is provided on the concrete surface, prediction of the progress of deterioration must consider the quality of the material used for the concrete protective layer as well as the quality of the concrete.

Regarding (2): There are three main types of method for predicting the progress of chemical attack, as follows:

(i) Methods based on inspection findings

In soil or in an environment with no water flow, under conditions not conducive to peeling or under deterioration due to sulfates, the erosion depth in concrete can be expressed using Equation (C5.3.1).

$$y = \gamma_c (a \cdot \sqrt{t} + b) \qquad \text{(C5.3.1)}$$

where, y : erosion depth in concrete (mm)

t : duration of exposure to substances that cause chemical attack (year)

a : coefficient for rate of erosion in concrete (mm/$\sqrt{\ }$year)

b : coefficient (a coefficient determined by the length of the initiation stage, with b = 0 (mm) when deterioration progresses from the start)

γ_c : safety coefficient for precision of. prediction (may be set to 1.0)

Conversely, in a water channel or other environment with water flow, when peeling is likely or deterioration due to acidic substances is present, the erosion depth in concrete can be expressed using Equation (C5.3.2).

$$y = \gamma_c (c \cdot t + d) \qquad \text{(C5.3.2)}$$

where, c : coefficient for rate of erosion in concrete (mm/year)

d : coefficient (a coefficient determined by the length of the initiation stage, with d = 0 (mm) when deterioration progresses from the start)

Note that when multiple measurements have been obtained and the measurements fall approximately between Equation (C5.3.1) and Equation (C5.3.2), or when a sufficient approximation cannot be made using Equation (C5.3.1) or Equation (C5.3.2), the use of Equation (C5.3.3) may enable better approximation of measurements.

$$y = \gamma_c (e \cdot t^f + g) \qquad \text{(C5.3.3)}$$

where, e : coefficient for rate of erosion in concrete. (mm/year^f)

f: coefficient ($0.5 < f < 1$)

g : coefficient (a coefficient determined by the length of the initiation stage, with g = 0 (mm) when deterioration progresses from the start) (ii) Methods based on accelerated testing

Because chemical attack progresses gradually from the concrete protective layer or the concrete surface, deterioration is easily assessed through visual inspection. However, while assessing the environment of the structure, it is advisable to predict the erosion depth in concrete or the permeation depth of deterioration factors by conducting accelerated testing of deterioration factors that are expected to come into contact with the structure using appropriate concentrations within a range that does not cause changes to the deterioration mechanisms.

(iii) Methods based on numerical analysis that considers environmental conditions and the transport and reactions of deterioration factors

A method for predicting the progress of chemical attack has been proposed using a numerical analysis method able to quantitatively evaluate the chemical reactions between deterioration factors and cement hydrates and accompanying changes in the rate of transport of substances. Because the generation of hydrogen sulfide gas is a factor that governs the rate of deterioration of concrete in sewage pipes, etc., the use of analytical methods that incorporate environmental conditions is advisable. It is also advisable to perform analysis taking into account the effects of the phenomenon by which reaction products expand and cause the structure to become porous, as in deterioration caused by sulfates.

5.3.3.3 Prediction of the progress of steel corrosion

(1) Prediction of the progress of steel corrosion caused by chemical attack should be made with appropriate consideration of the quality of the concrete and the effects of the environmental conditions under which the structure will be used.
(2) The starting period of steel corrosion should be determined from carbonation residue.
(3) The following methods may be used in predicting the progress of steel corrosion.
 (i) Methods based on the amount of corrosion found in inspections
 (ii) Methods based on accelerated testing

Commentary: Regarding (3): Methods for predicting the progress of steel corrosion caused by chemical attack are shown below:

(i) Methods based on the amount of corrosion found in inspections

When the change over time in the amount of corrosion has been measured through inspections, it is advisable to estimate the rate of corrosion from the curve of change over time regressed using measurements of the amount of corrosion at three or more time points.

(ii) Methods based on accelerated testing

When inspection findings are not available, it is advisable to make predictions of the rate of corrosion through accelerated testing, with appropriate consideration of the type of steel, the quality of the concrete, and the environmental conditions under which the structure will be used.

5.3.3.4 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to the maintenance plans as necessary.

5.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure in which chemical attack has occurred, the characteristics of chemical attack presented in this chapter should be taken into consideration.
(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by chemical attack, the maintenance limit, and the remaining planned service period.

Commentary: Regarding (1): In a structure subject to chemical attack, the aspects of performance to be evaluated for each process of deterioration must be appropriately selected.

In evaluating the performance of a structure, the performance of the structure at the time of inspections and at the end of the remaining planned service period must be quantitatively evaluated, after first quantitatively evaluating the individual states of deterioration of the concrete, the concrete protective materials, and the steel that make up the structure.

However, because a method for quantitatively evaluating the performance of structures has not been established at present, the appearance grades shown in **Table C5.3.1** may be used to evaluate the performance of structures in inspections, with reference made to **Table C5.3.2**.

Table C5.3.2 Appearance grades and factors of performance degradation

<table>
<tr><th>Appearance grade</th><th>Deterioration process</th><th>Load bearing capacity / Toughness</th><th>Deformation / Vibration</th><th>Peeling / Spalling</th><th>Cracking / Discoloration</th></tr>
<tr><td>Grade I</td><td>Initiation stage</td><td>—</td><td>—</td><td></td><td>—</td></tr>
<tr><td>Grade II</td><td>Propagation stage</td><td>—</td><td>—</td><td rowspan="3">Occurrence of peeling or spalling</td><td rowspan="2">・Change of properties of concrete
・Cracks
・Peeling or spalling</td></tr>
<tr><td>Grade III</td><td>Acceleration stage</td><td>Decrease in load bearing capacity
・Decrease in cross section of concrete</td><td>Increase in deformation or occurrence of vibration
・Decrease in cross section of concrete
・Decrease in cross section smoothness of concrete (decrease in flow rate)</td></tr>
<tr><td>Grade IV</td><td>Deterioration stage</td><td>Decrease in load bearing capacity or toughness
・Decrease in cross section of concrete
・Decrease in cross section area of steel</td><td>Increase in deformation or occurrence of vibration
・Decrease in adhesion between steel and concrete
・Decrease in cross section of concrete
・Decrease in cross section area of steel</td><td>・Change of properties of concrete
・Cracks
・Peeling or spalling
・Rust water
・Exposure of aggregate</td></tr>
</table>

5.4 Remedial measures

5.4.1 Selection of remedial measures

As a general rule, when remedial measures to remedy the decline in performance of a structure due to chemical attack have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When using appearance grade as an indicator for the selection of remedial measures, the selection will depend on the type and importance of the structure, the rate of deterioration progress, the maintenance category, and the remaining planned service period. Remedial measure methods should be selected with reference to **Table C5.4.1**.

Table C5.4.1 Appearance grades and remedial measures

Appearance grade	Deterioration process	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	Initiation stage				
Grade II	Propagation stage	◎	◎		
Grade Ⅲ	Acceleration stage	◎	◎	○	
Grade Ⅳ	Deterioration stage	○	○	◎	◎

◎：Standard remedial measures, ○：Remedial measures in some cases

5.4.2 Selection of repair methods and materials

To obtain the desired effects from repair, work methods and materials shall be selected with consideration of decline in performance caused by chemical attack and of life cycle costs.

Commentary: Repairs for structures deteriorated due to chemical attack can be divided into the methods shown in **Table C5.4.2** according to their expected effect. When selecting a work method, along with **Table C5.4.2**, the current state of the decline in performance of the structure must be taken into consideration. **Table C5.4.3** should also be used as a reference for the correspondence between appearance grades and remedial measure work methods.

Table C5.4.2 Expected effects and methods of repair and retrofit

Expected effect	Examples of method
Control the progress of chemical attack	Surface treatment (lining coating, resin lining, mortar lining), Patching, FRP adhesion, Embedded form, Ventilation, Washing, Thickness increasing
Control the progress of steel corrosion	Surface treatment, Patching, Rust-preventing coating
Improvement of load bearing capacity	FRP adhesion, thickness increasing, Jacketing

Table C5.4.3 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Standard method
Grade I	Initiation stage	Surface treatment, Ventilation, Washing
Grade II	Propagation stage	Surface treatment, Patching, Embedded form, Ventilation, Washing
Grade Ⅲ	Acceleration stage	Patching, Surface treatment, Thickness increasing, Embedded form, Ventilation, Washing
Grade Ⅳ	Deterioration stage	FRP adhesion, Patching, Surface treatment, Thickness increasing, Jacketing, Embedded form

5.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence is confirmed, remedial measures shall be promptly considered.

5.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to chemical attack shall also be recorded.

Chapter 6 Alkali-Silica Reaction

6.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to alkali-silica reaction (hereinafter "ASR" in this chapter) should be carried out with consideration of characteristics of ASR, as presented in this chapter.
(2) This chapter applies mainly to structures in maintenance category B.

Commentary: Regarding (1): Anomalies due to ASR, and their causes: This chapter mainly applies to reinforced concrete subjected to effects of ASR.

Regarding the progress of deterioration due to ASR: As shown in **Table C6.1.1**, the progress of deterioration of structures due to ASR can be divided into an initiation stage, propagation stage, acceleration stage, and deterioration stage.

Table C6.1.1 Definition of deterioration processes

Deterioration process	Definition	Factor determining the stage
Initiation stage	Period in which ASR occurs but neither expansion nor resultant cracking has yet occurred	Rate of alkali-silica gel formation (type and quantity of reactive minerals, quantity of alkalis)
Propagation stage	Period in which expansion occurs continuously and cracking occurs in the presence of water and alkalis, but steel corrosion does not occur	Formation and water-induced expansion rate of alkali-silica gel (supply of water and alkalis)
Acceleration stage	Period in which cracking propagates due to ASR and steel corrosion may or may not occur	
Deterioration stage	Period in which the width and density of cracks increases and the integrity of structural members is lost, the steel cross-section area decreases because of corrosion, the load bearing capacity decreases considerably because of steel damage, etc.	Water-induced expansion rate of alkali-silica gel (supply of water and alkalis) Rate of steel corrosion

Regarding (2): At present, the following can be noted with regard to structures that have been or are likely to be affected by ASR, and the maintenance of those structurs.

(i) It is difficult to identify in advance when anomalies (cracking, etc.) due to ASR will occur. Even when reactive aggregates are used or the quantity of alkalis is large, it cannot be known whether expansion will occur or not.

(ii) Even when structures and structural members in which anomalies due to ASR will occur can be estimated with accuracy, no measures have been established to keep the state of the structure within the initiation stage.

(iii) Based on (i) and (ii), because of the possibility that deterioration due to ASR will occur in structures that are in the initiation stage (i.e., all structures for which anomalies due to ASR are not observed), many uncertainties exist in the application of remedial measures.

6.2 Maintenance plan

When conducting maintenance on structures affected by ASR, a maintenance plan and maintenance limits should be set with consideration of the progressive processes of deterioration.

Commentary: Maintenance must be performed with consideration of decline in performance from the acceleration stage onward when the load bearing capacity of structures and structural members and the peeling and spalling of concrete are the focus, or from the propagation stage onward when deformation, vibration, and appearance are the focus. In the case of ASR, phenomena that enable confirmation of the progress of deterioration include the following:

(i) Cracking occurs; leaching of alkali-silica gel and discoloration are visible

(ii) The width and density of cracks increase; rust water due to steel corrosion is visible

(iii) The width and density of cracks further increase; level differences, gaps, and partial peeling of cover concrete are visible

(iv) Cracking and damage to steel due to external force are visible

(v) Displacement and deformation increase

With reference made to these, maintenance limits should be determined based on factors including the performance required of structures and structural members and on methods of implementing remedial measures.

6.3 Assessment

6.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to ASR, it is necessary to properly conduct inspections, evaluate current conditions, predict the progress of deterioration, and judge the need for remedial measures on the basis of maintenance plans and maintenance limits.

6.3.2 Inspections

6.3.2.1 General

(1) In inspections of structures affected by ASR, the initial inspection, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limits.

(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): Appearance grades when decline in performance has occurred due to ASR are as shown in **Table C6.3.1**. As items used for the quantitative evaluation of anomalies in appearance, crack width, depth, density, and other indicators are used in investigations of cracking.

Table C6.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	The expansion and associated cracking due to ASR have not occurred yet, and no visible anomalies are observed in the appearance.
Grade II	Propagation stage	Under the supply of moisture and alkali, continuous expansion progresses and minor cracking occurs. Discoloration and exudation of alkali-silica gel may be observed. However, no rust water due to steel corrosion is observed.
Grade III	Acceleration stage	The cracking due to ASR progresses, and the width, density, and extent of the cracks increase. In some cases, rust water due to steel corrosion may also be observed.
Grade IV	Deterioration stage	The width and density of the cracks further increase, and partial peeling or spalling of cover concrete, steps, displacement, and overlaps occur. Steel corrosion progresses, and rust water is observed. In some cases, cracks due to external force or damage to the steel may be observed. Displacement and deformation become significant.

6.3.2.2 Initial inspection

(1) In initial inspection, standard investigations are conducted as appropriate for newly constructed structures, existing structures, and structures after large-scale repairs or retrofit, to assess the initial state in maintenance of the structures.
(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Initial inspections of structures affected by ASR are mainly conducted to clarify whether a possibility exists of ASR occurring, or, when deterioration is present in the structure, whether it is caused by ASR, as well as to determine the necessity of remedial measures. Particularly important items in an initial inspection include the following:

(i) The origins and the mineral types of aggregates
(ii) The total quantity of alkalis in the concrete
(iii) The type of cement and whether mineral

admixtures are used

(iv) The state of exposure to sunlight and rain and whether seawater or deicing agents are supplied

(v) The presence of cracking, discoloration, leaching of alkali-silica gel, etc.

(vi) Displacement, deformation

(vii) The condition of repaired locations

6.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of early detection of deterioration and assessment of its progress. These are based on factors including confirmation of the state of water supply and investigations of displacement, deformation, gaps, level differences, anomalies in joint filler, and other anomalies in appearance, as well as confirmation of cracking, discoloration, alkali-silica gel leaching, rust water, peeling, spalling, and other anomalies in the concrete surface.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

6.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail more detailed investigations of anomalies, water leakage, etc. mainly through close visual observation of appearance. It is also necessary to combine tapping-based methods and, as necessary, methods using non-destructive testing equipment, testing of collected cores, etc. in inspections, and to assess environmental conditions.

(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult based on standard investigations in regular inspections specified in the maintenance plan, detailed investigations shall be conducted.

Commentary: Regarding (1): To assess the progress of deterioration and predict the progress of expansion in a structure in which anomalies are occurring due to ASR, inspections are conducted to assess cracking (width, length, density, and scope) and changes over time in expansion behavior. The inspection findings are helpful in considering the need for remedial measures, their timing, and selection of the materials to be used.

6.3.2.5 Detailed investigations

(1) In standard investigations conducted within the initial inspection and regular inspections specified in the maintenance plan, when assessment of the current state of deterioration and the prediction, evaluation, and judgment of the progress of deterioration in a structure subject to effects of ASR are difficult, or when standard investigations conducted within initial inspection, routine inspections, or regular inspections have revealed parts or structural members in which the progress of deterioration is significant, detailed investigations shall be performed to obtain more detailed information on the structure.
(2) The investigation items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

6.3.3 Prediction

6.3.3.1 General

(1) In the maintenance of structures subject to the effects of ASR, the performance of the structure at the time of inspection is quantitatively assessed and, as needed, future performance is predicted.
(2) To predict the decline in the performance of a structure, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with reference to the progress of ASR.
(3) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: Regarding (1): Prediction should give particular focus to the expansion of concrete (and accompanying propagation of cracks), the deformation of structural members, the progress of steel corrosion, and the occurrence of damage to steel.

Regarding (2) and (3): Because the progress of expansion due to ASR is complexly affected by a variety of factors, such as the type and amount of reactive minerals in the aggregate, the reinforcing bar arrangement in structural members, and the environmental conditions (temperature and supply of water), it is difficult to predict quantitatively. Therefore, it is practical to predict the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage with consideration of the progress of deterioration due to ASR in the concrete. The factors that determine processes of deterioration and their timing are as shown in **Table C6.1.1**.

6.3.3.2 Prediction of the progress of expansion in concrete due to ASR

(1) Prediction of the progress of expansion in concrete due to ASR must be made with appropriate consideration of the quality of the concrete and the effects of the environmental conditions under which the structure is used.

(2) Either of the following methods may be used in predicting the progress of expansion in concrete due to ASR.

(i) Methods based on inspection findings

(ii) Methods combining testing results with analysis, etc.

Commentary: Regarding (1) and (2): While it is difficult to accurately predict the progress of concrete expansion due to ASR, when judging the need for remedial measures in structures that have deteriorated due to ASR and when considering the materials to be used for repairs, the possibility of future expansion of the concrete must be considered by some method.

(i) Regularly measuring changes over time in deformation and crack propagation in structural members using contact gauges or other means is a method for predicting future deformation and crack propagation based on inspection findings.

(ii) Methods combining testing results with analysis, etc. predict the progress of expansion by performing analysis on combined data that includes accelerated testing data, structural specifications, and environmental conditions.

6.3.3.3 Prediction of the progress of steel corrosion

Prediction of the progress of steel corrosion in a structure affected by ASR shall be made with appropriate consideration of anomalies caused by ASR and the effects of the environmental conditions under which the structure is used.

6.3.3.4 Prediction of the occurrence of damage to steel

Prediction of the occurrence of damage to steel due to ASR shall be made with appropriate consideration of the quality of the concrete, the arrangement of reinforcing bars, the types and forms of structural members, and the effects of the environmental conditions under which the structure is used.

6.3.3.5 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to maintenance plans as necessary.

6.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure in which ASR has occurred, the characteristics of ASR presented in this chapter should be taken into consideration.
(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by ASR, the maintenance limits, and the remaining planned service period.

Commentary: Regarding (1) and (2): Quantitative evaluation of varied aspects of performance in structures is ideal as a method of performance evaluation under a performance verification-based design system. At present, however, no such method has been established. Therefore, a realistic approach is to determine the appearance grade shown in **Table C6.3.1** based on anomalies in the appearance of the structure and to evaluate the performance of the structure with reference to **Table C6.3.2**. However, the mechanical performance of a structure is affected by which parts or structural members have deteriorated, a factor which must be taken into consideration when using appearance grade to evaluate the performance of the structure.

Table C6.3.2 Appearance grades and factors of performance degradation

<table>
<tr><th>Appearance grade</th><th>Deterioration process</th><th>Load bearing capacity / Toughness</th><th>Deformation / Water tightness</th><th>Peeling / Spalling</th><th>Cracking / Discoloration</th></tr>
<tr><td>Grade I</td><td>Initiation stage</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>Grade II</td><td>Propagation process</td><td>—</td><td rowspan="2">Increase of deformation
・Deformation of member due to the progression of expansion
・Deformation caused by the decrease in the elastic modulus of concrete
Decrease in water tightness
・Crack</td><td>—</td><td>・Crack
・Discoloration
・Exudation of alkali-silica gel</td></tr>
<tr><td>Grade III</td><td>Acceleration stage</td><td>Decrease in load bearing capacity or toughness
・Decrease in concrete strength
・Steel corrosion</td><td rowspan="2">Occurrence of peeling or spalling</td><td rowspan="2">In addition to the above
・Rust water</td></tr>
<tr><td>Grade IV</td><td>Deterioration stage</td><td>In addition to the above
・Decrease in adhesion of steel
・Damage of steel</td><td>In addition to the above
・Displacement
・Step</td></tr>
</table>

6.4 Remedial measures

6.4.1 Selection of remedial measures

As a general rule, when remedial measures to remedy the decline in performance of a structure due to ASR have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When using appearance grade as an indicator for the selection of remedial measures, the selection will depend on the type and importance of the structure, the rate of progress of deterioration, the maintenance category, and the remaining planned service period. Remedial measure methods should be selected with reference to **Table C6.4.1**.

Table C6.4.1 Appearance grades and remedial measures

Appearance grade	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	○	○**		
Grade II	○	◎		
Grade III	◎	◎	○	
Grade IV	◎	◎*	◎	◎

◎ : Standard remedial measures (◎* : Including the restoration of mechanical performance), ○ : Remedial measures in some cases, ○** : Preventive remedial measures

6.4.2 Repairs

(1) To obtain the desired effects from repair, work methods and materials shall be selected with consideration of decline in performance due to ASR and life cycle costs.
(2) After the implementation of remedial measures, it is necessary to confirm that the structures satisfy the specified performance.

Commentary: Regarding (1): Repairs on structures deteriorated due to ASR can be divided into the methods shown in **Table C6.4.2** according to their expected effect.

Water control (cutoff, drainage), surface treatment, crack injection, and other means of preventing the external supply of water to a structure are effective in inhibiting the progress of ASR. However, water exists in concrete prior to remedial measures, and blocking the absorption of water from outside can be difficult in some locations, such as abutments and the back side of retaining walls. When sufficient moisture to enable the progress of ASR remains in concrete, completely stopping the progress of ASR is difficult.

Table C6.4.2 Expected effects and methods of repair

Expected effect	Examples of methods
Control of the progress of ASR	Water treatment (cutoff, draining treatment), Crack injection, Surface treatment (covering, impregnation)
Restraining of ASR-induced expansion	Installation of prestressing, Jacketing (steel plate, PC, continuous fiber)
Removal of deteriorated portion	Patching
Control of the steel corrosion	Crack injection, Crack filling, surface treatment (coating, impregnation)
Elimination of the degree of effects on third party	Prevention of spalling
Restoration of the load bearing capacity	Bonding (steel plate, continuous fiber), Installation of prestressing, Jacketing (steel plate, PC, continuous fiber), External tendon, Repair of damaged portion of steel

When selecting a method, along with **Table C6.4.2** the current state of the decline in performance of the structure must be taken into consideration. **Table C6.4.3** should be used as a reference for the correspondence between appearance grades and remedial measure work methods.

Table C6.4.3 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Expected expansion	Standard methods
Grade I	Initiation stage	—	Water treatment (cutoff, draining treatment)*, Surface treatment (covering, impregnation)*
Grade II Grade III	Propagation stage Acceleration stage	Small	Water treatment (cutoff, draining treatment), Crack injection, Surface treatment (covering, impregnation), Prevention of spalling
		Large	Water treatment (cutoff, draining treatment), Crack injection, Surface treatment (covering, impregnation), Prevention of spalling, Patching, Installation of prestressing, Bonding (steel plate, continuous fiber), Jacketing (steel plate, PC, continuous fiber), External tendon
Grade IV	Deterioration stage	—	Water treatment (cutoff, draining treatment), Patching , Surface treatment (covering), Prevention of spalling, Installation of prestressing, Bonding (steel plate, continuous fiber), Jacketing (steel plate, PC, continuous fiber), External tendon, Repair of damaged portion of steel

*：Preventive method

6.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of is confirmed, remedial measures shall be promptly considered.

6.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to ASR shall also be recorded.

Chapter 7 Fatigue

7.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to fatigue should be conducted with consideration of characteristics of fatigue, as presented in this chapter.

(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): The maintenance of structures in which performance has declined or is likely to decline due to fatigue should be based on "Maintenance: Standards."

The main processes of deterioration due to fatigue in beams and in planar members are shown in **Table C7.1.1** and **Table C7.1.2**, respectively. Distinguishing between the initiation stage and propagation stage in particular can be difficult in actual maintenance, depending on the survey method and its accuracy. Prediction, evaluation, judgment, and implementation of remedial measures should be considered without making this distinction.

Table C7.1.1 Definition of deterioration processes of fatigue in beams

Appearance grade	Deterioration process	Definition	Factor determining the stage
Grade I	Initiation stage	・State at which normal flexural cracking occurs on the tensile side of the concrete ・State at which slip has occurred in the steel at the crystalline level of the metal due to cyclic action, but before the initial cracking that can be observed in general ・State at which deflection changes are negligible ・State at which no effect on load bearing capacity is expected	Maximum value, amplitude, frequency and cumulative number of cyclic actions, fatigue crack growth rate of steel
Grade II	Propagation stage	・State at which the amount of opening and closing of flexural cracks on the concrete surface is gradually increasing compared to the initial state, but is not recognized as anomalies ・State at which a part of flexural cracks extend in shear direction near the fulcrum, or cracks that appear to be shear cracks (not flexural cracks) begin to be observed ・State at which initial cracks are observed in the steel and the size of the cracks becomes large enough to be observed by using magnetic powder testing ・State at which the crack propagation rate depends on the material composition of the steel ・State at which the cross section loss due to fatigue cracking of steel is small and has little effect on the load bearing capacity of beams ・State at which almost no change in deflection is observed ・State at which almost no effect on load bearing capacity is expected	
Grade III	Acceleration stage	・State at which the amount of flexural crack width or shear crack misalignment on the concrete surface increases, causing angular dropout on the crack surface and cracks in the concrete at the compression zone ・State at which fatigue cracks in the steel are close to the limit crack length and some of them are close to fatigue fracture ・State at which the amplitude of deflection increases and residual deflection is confirmed by measurement ・State at which the condition in which the load bearing capacity is also assumed to have decreased	
Grade IV	Deterioration stage	・State at which an increase in the width of flexural cracks or the amount of misalignment of shear cracks on the concrete surface can be visually confirmed, or cracks can be seen in the concrete in compression zone ・State at which fatigue fracture occurs in the main steel or stirrup, and fatigue fracture progresses further in other steels in the section ・State at which deflection deformation increases ・State at which significant decrease in load bearing capacity	

Table C7.1.2 Definition of deterioration processes of fatigue in planar members

Appearance grade	Deterioration process	Definition	Factor determining the stage
Grade I	Initiation stage	・State at which normal flexural cracking occurs on the tensile side of the concrete ・State at which slip has occurred in the steel at the crystalline level of the metal due to cyclic action, but before the initial cracking that can be observed in general (only in case of planar member with a relatively small amount of steel) ・State at which deflection changes are negligible ・State at which no effect on load bearing capacity is expected	Maximum value, amplitude, frequency and cumulative number of cyclic actions, fatigue crack growth rate of steel
Grade II	Propagation stage	・State at which the amount of opening and closing of flexural cracks on the concrete surface is gradually increasing compared to the initial state, but is not recognized as a anomalies ・State at which initial cracks are observed in the steel and the size of the cracks becomes large enough to be observed by using magnetic powder testing (only in case of planar member with a relatively small amount of steel) ・State at which the crack propagation rate depends on the material composition of the steel (only in case of planar member with a relatively small amount of steel) ・State at which the cross section loss due to fatigue cracking of steel is small and has little effect on the load bearing capacity of planar members ・State at which almost no change in deflection is observed (only in case of planar member with a relatively small amount of steel) ・State at which almost no effect on load bearing capacity is expected	
Grade III	Acceleration stage	・State at which the flexural crack width on the concrete surface increases and radial propagation is observed, and cracks are observed in the concrete in the compression zone ・State at which fatigue cracks in the steel are close to the limit crack length and some of them are close to fatigue fracture (only in case of planar member with a relatively small amount of steel) ・State at which the amplitude of deflection increases and residual deflection is confirmed by measurement ・State at which the condition in which punching shear cracks are assumed to have developed inside the concrete ・State at which the condition in which the load bearing capacity is also assumed to have decreased	
Grade IV	Deterioration stage	・State at which an increase in the width of flexural cracks or the amount of misalignment of shear cracks on the concrete surface can be visually confirmed ・State at which fatigue fracture occurs in steel, and fatigue fracture progresses further in other steels in the cross section (only in case of planar member with a relatively small amount of steel) ・State at which deflection deformation increases ・State at which punching shear failure occurs ・State at which significant decrease in load bearing capacity	

7.2 Maintenance planning

When conducting maintenance on structures that are subject to effects of fatigue, a maintenance plan and maintenance limits should be set with consideration of the progressive processes of deterioration.

Commentary: Maintenance plans for structures that are subject to effects of fatigue should be formulated according to Chapter 2 of "Maintenance: Standards."

Maintenance plans to address fatigue should be formulated with reference to the condition of deterioration due to fatigue in similar structures or structures sited in similar environments.

In the case of structures to which maintenance category A is applied, maintenance must be performed to keep the stage of the structure within the initiation stage. Keeping comparisons with design conditions in mind, it is important to set the maintenance plan so that the structure states indicated below can be assessed.

i) The magnitude of loading, number of cycles, position of loading, and predictions

ii) The width of cracks on the concrete surface

iii) The amplitude of stress in steel

iv) The state of water ingress (particularly at structural member junctions (anchorage zones embedded in steel))

Conversely, for structures placed under maintenance category B, maintenance should be performed so that the performance of the structure does not fall below the required performance level.

7.3 Assessment

7.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to fatigue, it is necessary to properly conduct inspections, evaluate the current condition, predict the progress of deterioration, and determine the need for remedial measures on the basis of maintenance plans and maintenance limits.

Commentary: The "Maintenance" volume stipulates three types of assessments for proper implementation of the maintenance of structures: initial assessments, regular assessments, and extraordinary assessments. Of these, initial assessments and regular assessments are indispensable for the maintenance of structural members in which performance has or is expected to deteriorate due to fatigue. Conversely, extraordinary assessments should be conducted as necessary and are not addressed in this chapter.

7.3.2 Inspections

(1) In inspections of structures that are subject to effects of fatigue, the initial inspection, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limits.
(2) When performing inspections, standard surveys should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed surveys should also be conducted as necessary.

Commentary: Regarding (1) and (2): The items and methods of the surveys that make up inspections are mainly visual observation of anomalies in appearance. **Table C7.3.1** shows appearance grades and states of deterioration that may occur in beams and surface members.

Table C7.3.1 Appearance grades and state of deterioration in beams and planar members

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	A few cracks are observed in the direction perpendicular to the main bending due to bending action
Grade II	Propagation stage	The amount of opening and closing of flexural crack width on the concrete surface is gradually increasing compared to the initial condition
Grade III	Acceleration stage	There is a clear increase in the width of flexural cracks on the concrete surface, cracks in the concrete in compression zone, shear cracks in beams, and radial cracks in planar members
Grade IV	Deterioration stage	Fatigue fracture of steel or punching shear failure of planar members

7.3.3 Prediction

7.3.3.1 General

(1) As a general rule, in the maintenance of structures in which deterioration due to fatigue has occurred or is a concern, the performance of the structure at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of a structure requires quantitatively predicting deterioration due to fatigue.
(3) When (1) and (2) are difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of deterioration.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: Regarding (1) and (2): In predicting the progress of fatigue in beams and surface members, methods of indirectly estimating the amplitude of stress by measuring deflection in structural members, observing the state of cracking in appearance, etc. are used. Decline in performance can also be predicted by focusing on the fatigue performance of steel in the design.

7.3.3.2 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to maintenance plans as necessary.

7.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure with respect to fatigue, the characteristics of fatigue presented in this chapter should be taken into consideration.

(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by fatigue, the maintenance limits, and the remaining planned service period.

Commentary: Regarding (1): As a general rule, the evaluation of structures with respect to fatigue should be conducted based on Chapter 6 of "Maintenance: Standards." Regarding the evaluation and judgment of fatigue, techniques to quantitatively evaluate decline in performance due to fatigue are not sufficiently established. For this reason, performing evaluation through appearance grade, etc. is considered acceptable, but clear confirmation of anomalies in appearance due to fatigue is often not possible until the acceleration stage is reached. Matching design conditions with actual loading history is a conceivable method for evaluation at or before the propagation stage.

7.4 Remedial measures

7.4.1 Selection of remedial measures

As a general rule, when remedial measures to remedy the decline in performance of a structure due to fatigue have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: For fatigue in beams, repair and retrofit methods and materials are categorized as shown in **Table C7.4.1** according to their expected effect. When selecting work methods and materials, the state of deterioration of structural members must be considered. **Table C7.4.2** shows the relationship between appearance grades and standard work methods.

Table C7.4.1 Expected effects and methods of repair in fatigue of beam

Expected effect	Examples of methods
Restoration of load bearing capacity	Steel plate/FRP adhesion, external cable
Restoration of stiffness	Steel plate adhesion

Table C7.4.2 Appearance grades and examples of standard methods in fatigue of beam

Appearance grade	Deterioration process	Standard methods
Grade I	Initiation stage	–
Grade II	Propagation stage	Steel plate/FRP adhesion
Grade III	Acceleration stage	Steel plate/FRP adhesion, external cable
Grade IV	Deterioration stage	

7.4.2 Repairs

(1) To obtain the desired effects from repair, methods and materials shall be selected with consideration of decline in performance due to fatigue and life cycle costs.

(2) After the implementation of remedial measures, it is necessary to confirm that the structures satisfy the specified performance.

Commentary: Regarding (1): When repairs are carried out as a remedial measure, the expected effects of the repairs and the required repair performance necessary to achieve said effects (i.e., what degree of performance to maintain and for how many years) must be made clear.

7.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of deterioration is confirmed, remedial measures shall be promptly considered.

7.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to fatigue shall also be recorded.

Chapter 8 Abrasion

8.1 General rules

(1) The maintenance of structures in which performance has declined or is likely to decline due to abrasion should be conducted with consideration of characteristics of abrasion, as presented in this chapter.
(2) This chapter applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): Abrasion is a phenomenon by which concrete cross section of concrete is lost to wear or impact from water flows, wheels, or other sources. **Table C8.1.1** presents examples of the mechanisms of abrasion and structures for which abrasion is a problem. This chapter focuses on concrete in which deterioration is occurring due to water flow.

Table C8.1.1 Structures where abrasion is a problem and affected performance

Mechanism of abrasion	Target structures	Phenomena caused by abrasion	Affected performance	Remarks
Erosion due to water flow or ocean waves	River and harbor structures Waterways, underwater bridge piers and abutments, etc.	Reduced cross section, unevenness	Flow rate Appearance	Main subject of this chapter
Cavitation due to water flow	Overflow parts of dams, etc. Jogs, steps, etc. in waterways			
Impact and rolling due to water flow containing conglomerate, etc.	Overflow and apron of dams, etc. Bottom of waterway, etc.	Reduced cross section, unevenness Cracks, peeling/spalling (in case of impact)	Flow rate, water flow Appearance	
Erosion due to flying sand	Structures near the coast	Reduced cross section, unevenness	Appearance	Refer to this chapter
Impact and slip due to ice or floating ice	Harbor structures in cold region	Reduced cross section, unevenness Cracks, peeling/spalling (in case of impact)	Appearance	
Rolling and sliding of wheels, etc.	Pavements, floors	Reduced cross section, unevenness	Flatness Appearance	Refer to standard specifications for pavements (JSCE)

Because the reduction of concrete cover precedes steel exposure in actual structures, abrasion is rarely ignored until such a state develops. For this reason, this chapter mainly addresses maintenance up to the point before the processes of deterioration that affect steel material are reached. As shown in **Table C8.1.2**, the processes of deterioration due to the progress of abrasion can be divided into stages from an initiation stage to a deterioration stage. Aspects of performance affected by abrasion are typically flow turbulence and a decrease in flow rate due to increase in the roughness coefficient, as well as appearance. Therefore, this chapter focuses only on water permeability and appearance as performance aspects, and does not address decline in load bearing capacity due to steel corrosion during the deterioration stage.

Table C8.1.2 Definition of deterioration processes

Deterioration process	Definition	Factor determining the stage
Initiation stage	Period until concrete abrasion becomes apparent	Abrasion rate
Propagation stage	・Period until the exposure of coarse aggregate ・Period until the occurrence of cracking by impacts	
Acceleration stage	・Period until the falling of coarse aggregate ・Period until the occurrence of peeling/spalling by impacts	
Deterioration stage	Period of significant performance loss caused by section loss due to coarse aggregate loss or concrete peeling/spalling	

8.2 Maintenance plan

When conducting maintenance on structures that are subject to effects of abrasion, a maintenance plan and maintenance limit should be set with consideration of the progressive processes of deterioration.

Commentary: For structures placed under maintenance category A, the maintenance limit must be set so that the structure remains within the latency stage during which abrasion is not apparent. Therefore, the following actions are necessary.

(i) Assessment of the state of actions by abrasion deterioration factors on the surface of the structure

(ii) Quantitative evaluation and prediction of the state of abrasion in the concrete protective layer or the main concrete body

Conversely, for structures in maintenance category B, maintenance should be performed so that deterioration does not progress to the deterioration stage when the focus is on securing cross section, flow rate, or other aspects of usability, or does not progress to the acceleration stage when the focus is on appearance. The following examples of the emergence of deterioration may be set as maintenance limit.

(i) Abrasion on the concrete surface

(ii) Exposure of coarse aggregate and cracking due to shock

(iii) Detachment of coarse aggregate and peeling/spalling due to shock

8.3 Assessment

8.3.1 General

In the assessment of structures in which performance has declined or is likely to decline due to abrasion, it is necessary to properly conduct inspections, evaluate the current condition, predict the progress of deterioration, and determine the need for remedial measures on the basis of maintenance plans and maintenance limit.

8.3.2 Inspections

8.3.2.1 General

(1) In inspections of structures that are subject to effects of abrasion, the initial inspection, routine inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limit.

(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): In structures that are subject to effects of abrasion, the main items that should be assessed through inspection are depth and extent of abrasion, the condition of the abraded surface, and the rate of abrasion. "Rate of abrasion" refers to the progress of depth of abrasion per unit of time. In addition, anomalies in the appearance of structures that are subject to abrasion, as shown in **Table C8.3.1**, provide useful information for evaluation of performance.

Table C8.3.1 Appearance grades and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	No apparent abrasion of concrete
Grade II	Propagation stage	Occurrence of abrasion, but no exposure of coarse aggregate
Grade III	Acceleration stage	・Exposure of coarse aggregate ・Occurrence of cracking due to impacts
Grade IV	Deterioration stage	・Falling of coarse aggregate ・Peeling/spalling due to impacts

8.3.2.2 Initial inspection

(1) In an initial inspection, standard investigations are conducted as appropriate for newly constructed structures, existing structures, and structures after large-scale repair or retrofit, to assess the initial state in maintenance of the structures.
(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

Commentary: Regarding (1): Important investigation items in the initial inspection of structures that are subject to abrasion include the following:

(i) Environmental actions
(ii) The materials, mix proportions, and strength of the concrete
(iii) The presence or absence, types, and amounts of mineral admixtures
(iv) The arrangement of steel and concrete cover
(v) The presence or absence of initial defects
(vi) The depth and extent of abrasion, the condition of abraded surfaces, cracking due to shock, peeling/spalling
(vii) The condition of repaired locations

8.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of early detection of deterioration and assessment of its progress. These are based on visual investigations with attention to the depth and extent of abrasion, the state of abraded surfaces, anomalies in concrete surfaces (cracking, peeling, spalling), and other changes in appearance.
(2) When standard investigations have revealed signs of anomalies or deterioration not anticipated in the maintenance plan, regular inspections should be conducted ahead of schedule or detailed investigations should be conducted.

Commentary: Regarding (1): In addition to inspections focused on locations where deterioration was deemed likely to progress in the initial inspection, it is advisable to also perform inspections on other locations, if less frequently. For example, in the case of a water channel or other structure in which the main mechanism of deterioration is abrasion, the frequency of inspections of the concrete surface may be low.

8.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail more detailed investigations of anomalies, water leakage, etc. mainly through close visual observation of appearance. It is also necessary to combine tapping-based methods and, as necessary, methods using non-destructive testing equipment, testing of collected cores, etc. in inspections, and to assess environmental conditions.
(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult based on standard investigations in regular inspections specified in the maintenance plan, detailed investigations shall be conducted.

Commentary: Regarding (1): When measuring the depth and extent of abrasion, the width and length of cracks, the depth and extent of peeling and spalling, and other items during regular inspections, measurements should be taken at least two times during the periods of the expected processes of deterioration.

8.3.2.5 Detailed investigations

(1) In standard investigations conducted within the initial inspection and within regular inspections specified in the maintenance plan, when assessment of the current state of deterioration and the prediction, evaluation, and judgment of the progress of deterioration are difficult in a structure subject to effects of abrasion, or when standard investigations conducted within initial inspection, routine inspections, or regular inspections have revealed parts or structural members in which the progress of deterioration is significant, detailed investigations shall be performed to obtain more detailed information on the structure.
(2) The investigation items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

8.3.3 Prediction

8.3.3.1 General

(1) As a general rule, in the maintenance of structures in which abrasion has occurred or is a concern, the performance of the structure at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of a structure requires quantitatively predicting deterioration due to abrasion.
(3) When (1) and (2) are difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of concrete abrasion.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: Regarding (1) and (2): Compared to other mechanisms of deterioration, abrasion is highly linear with respect to time. Therefore, when environmental actions do not change significantly, prediction of the progress of deterioration based on the rate of abrasion may be considered relatively easy.

Regarding (3): When it is difficult to quantitatively predict deterioration due to abrasion based on inspection findings, quantitative prediction of the progress of deterioration by predicting the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be performed instead, with consideration of the progress of concrete abrasion.

8.3.3.2 Prediction of the progress of abrasion

(1) Prediction of the progress of abrasion should be made with appropriate consideration of the quality of concrete and environmental actions.

(2) Any of the following methods may be used in predicting the progress of abrasion.

(i) Methods based on the amount of abrasion found in inspections

(i) Methods based on inspection findings from similar structures in identical environments

(i) Methods based on abrasion testing

Commentary: Regarding (1): Elements that determine the rate of abrasion include the mix proportions and the strength of the concrete, flow velocity, the mixture of earth and sand, the strength and frequency of waves, and the presence of shock.

Regarding (2): The following are the three main types of prediction techniques for the progress of abrasion :

(i) Methods based on the rate of abrasion found in inspections

When the rate of abrasion has been measured through inspections, the progress of abrasion can be predicted though regression of the results. Under identical environmental conditions, the rate of abrasion for each deterioration process can be considered constant if environmental conditions are identical.

(ii) Methods based on inspection findings from similar structures in identical environments

(iii) Methods based on abrasion testing

When inspection findings are not available, the rate of abrasion can be predicted based on abrasion testing, with appropriate consideration of the concrete mix, environmental action (flow velocity, the mixture of earth and sand, the presence of shock, etc.).

8.3.3.3 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to maintenance plans as necessary.

8.3.4 Evaluation and judgment

(1) When evaluating the performance of a structure subjected to the effects of abrasion, the characteristics of abrasion presented in this chapter should be considered.
(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance caused by abrasion, the maintenance limit, and the remaining planned service period.

Commentary: Regarding (1): Specific techniques for evaluation of performance at the time of inspection include methods of deriving the flow rate and methods of evaluating appearance from cracking, extent and density of peeling and flaking, and exposure of coarse aggregate. A realistic approach is to grade appearance according to **Table C8.3.1** based on anomalies and to evaluate the performance of the structure at the time of inspection with reference to **Table C8.3.2**.

Table C8.3.2 Appearance grades and factors of performance degradation

Appearance grade	Deterioration process	Exposure/falling, etc. of coarse aggregate	Other
Grade I	Initiation stage	−	−
Grade II	Propagation process	・Unevenness of surface	・Decrease in flow rate
Grade III	Acceleration stage	・Exposure of coarse aggregate ・Crack	
Grade IV	Deterioration stage	・Falling of coarse aggregate ・Peeling, spalling	・Change of flow (Vortex generation, meandering, etc.) ・Water leakage

8.4 Remedial measures

8.4.1 Selection of remedial measures

As a general rule, when remedial measures to remedy the decline in performance of a structure due to abrasion have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the structure following implementation.

Commentary: When using appearance grade as an indicator for the selection of remedial measures, the selection will depend on the type and importance of the structure, the rate of progress of deterioration, the maintenance category, and the remaining planned service period. Reference should be made to **Table C8.4.1**.

Table C 8.4.1 Appearance grades and remedial measures

Appearance grade	Deterioration process	Intensified inspection	Repair	Restriction in service	Demolition / removal
Grade I	Initiation stage	◎	○*		
Grade II	Propagation process	○	◎		
Grade III	Acceleration stage		◎		
Grade IV	Deterioration stage		◎	○	○

◎：Standard remedial measures,　○：Remedial measures in some cases ［○*：Preventive remedial measures］

8.4.2 Repairs

(1) To obtain the desired effects from repair, methods and materials shall be selected with consideration of decline in performance due to abrasion and life cycle costs.

(2) After the implementation of remedial measures, it is necessary to confirm that the structures satisfy the specified performance.

Commentary: Regarding (1): Repairs on structures that have deteriorated due to abrasion can be divided into the methods shown in **Table C8.4.2** according to their expected effect. **Table C8.4.3** should also be used as reference for the correspondence between appearance grades and remedial measures.

Table C8.4.2 Expected effects and methods of repair

Expected effect	Examples of methods
Control the progress of abrasion	Surface treatment, adhesion of abrasive-resistant materials, installation of protective layer
Reduce roughness coefficient	Surface treatment, patching
Secure member cross section	Patching

Table C8.4.3 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Standard methods
Grade I	Initiation stage	Surface treatment*
Grade II	Propagation process	Patching, surface treatment*
Grade III	Acceleration stage	Patching, surface treatment*
Grade IV	Deterioration stage	Patching, replacing, surface treatment*

＊：Preventive remedial measures

8.4.3 Post-repair maintenance

After the application of repair work, whether the anticipated effects have been obtained shall be confirmed through regular inspections. If unexpected recurrence of deterioration is confirmed, remedial measures shall be promptly considered.

8.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to abrasion shall also be recorded.

Volume 2
Concrete Structures/Structural Members

Chapter 1 Prestressed Concrete

1.1 General rules

(1) This chapter presents items that are particularly necessary in the maintenance of prestressed concrete structures or structural members.

(2) Maintenance of prestressed concrete structures shall be conducted with attention paid to particulars of prestressed concrete, in addition to matters relating to reinforced concrete structures.

Commentary: Regarding (1): Prestressed concrete incorporates PC steel, anchorage zones, and other materials and components that are important in maintenance. In existing prestressed concrete, durability may not have been given sufficient consideration at the time of design and construction. Therefore, in maintenance, the structural characteristics, specific deterioration, and other aspects of prestressed concrete should be fully taken into consideration.

Regarding (2): Because the structural properties of prestressed concrete differ from those of reinforced concrete, the following types of deterioration may occur:

(i) Corrosion of PC steel, anchorage zones, and deflecting sections

When prestressing is lost due to the fracture of PC steel, performance may decline and the structure and structural members may undergo failure. For this reason, particular attention must be paid to corrosion of PC steel.

(ii) Anchorage zones, deterioration around deflecting sections

When cracking occurs due to localized stress occurring in anchorage zones and water, etc., is supplied, PC steel may undergo corrosion and adequate anchorage may be lost.

In prestressed concrete, in which cracking is considered unacceptable in verification of usability, the amplitude of stress in PC steel due to fluctuations in live loads, etc. is usually small. Therefore, fatigue is likely not an issue in prestressed concrete.

(iii) Corrosion and fracture of PC steel associated with insufficient grouting in post-tensioned PC, etc.

With post-tensioning, the Bernoulli hypothesis of linear strain distribution cannot be applied to locations with insufficient PC grouting. Therefore, when insufficient grouting is extensive, the design flexural load bearing capacity will differ from assumptions made at the design stage.

(iv) Deterioration caused by characteristics of the method of erection

At the time of erection, vulnerabilities with respect to durability may occur in parts including backfill parts of anchorage zones, joints of erection blocks, and junctions of precast concrete structural members.

1.2 Maintenance plan

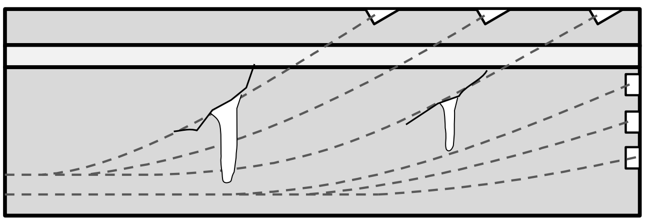
(1) Maintenance plans shall be formulated for each targeted structure and structural member with consideration of the characteristics of prestressed concrete, technological changes, and other factors, in addition to matters relating to reinforced concrete structures.

(2) The maintenance limit should be appropriately set with consideration of the maintenance category, the characteristics of the target structure or structural members, and other matters, and should be revised as needed based on the results of assessments.

Commentary: Regarding (1): The history of prestressed concrete spans a relatively short 60 years. When formulating maintenance plans, it is important to assess technical characteristics of the era in which the relevant structure was constructed, and to draft the maintenance plans in accordance with the technology of the time.

With post-tensioning, maintenance plans must be drafted on the assumption that grading of appearance may not be applicable. This is because it may not be possible to assess the presence or absence of corrosion in PC steel inside the sheath based on appearance, or because, when PC steel at locations of insufficient PC grouting undergoes corrosion, corrosion byproducts such as rusty water can accumulate inside the sheath and corrosion cracking may not occur.

In formulating a maintenance plan, it is important to assess the condition of the PC steel first. Investigation plans should be drafted first for post-tensioned prestressed concrete constructed in an age when the risk of steel anomalies can be considered high. **Figure C1.2.1** presents an example of deterioration due to infiltration of water from the bridge surface in girders anchored at the upper edge. In a girder anchored at the upper edge, the backfilled part of the anchorage zone is directly subject to upper load. Therefore, in addition to a high risk of deterioration, visual inspection of the anchorage zone may not be possible and deterioration may progress markedly.



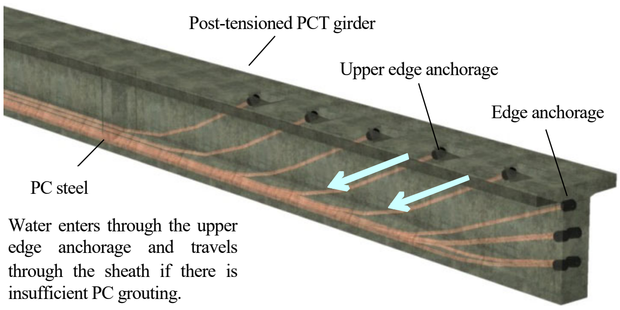


Figure C1.2.1 Example of deterioration of a girder anchored at the upper edge due to water ingress (example of a PCT girder constructed before about 1980)

Regarding (2): The occurrence of cracks perpendicular to the direction of prestressing should not be used as a maintenance threshold indicator. This is because of the concern that, when flexural cracking occurs in a simple T-girder bridge that consists of numerous prestressed concrete main girders and cross girders, PC steel may have already fractured in a considerable number of main girders.

1.3 Inspection

1.3.1 General

In addition to matters relating to reinforced concrete structures, inspections shall pay attention to the characteristics of prestressed concrete.

Commentary: The state of PC grouting and of the corrosion in PC steel should be investigated first.

1.3.2 Initial inspection

In addition to matters relating to reinforced concrete structures, initial inspection shall address the anomalies characteristic of prestressed concrete.

Commentary: In prestressed concrete, initial defects greatly affect durability. The initial inspection should focus on defects including insufficient PC grouting, corrosion of PC steel, cracks caused by localized stress associated with prestressing, cracks caused by stress distribution during erection, and backfilling around anchorage zones.

1.3.3 Routine inspections

In addition to matters relating to reinforced concrete structures, routine inspections should generally address fracturing/protrusion of PC steel; anomalies in anchorage zones, deflecting sections, and surrounding concrete; openings in joints; and water leakage, displacement/deformation, and other anomalies in appearance.

Commentary: Special notes for prestressed concrete in routine inspections are shown below.

(i) Investigation items

The presence or absence of anomalies related to corrosion in PC steel should be confirmed. Decline in performance associated with the fracturing of PC steel in particular greatly affects safety.

(ii) Inspection locations

With regards to corrosion in PC steel, attention should be paid to the presence or absence of rust fluid along the sheath and the occurrence of cracking in the concrete. With regards to the presence or absence of PC steel fracturing, attention should be paid to protrusion of the steel, peeling of cover concrete, etc.

(iii) Frequency of inspections

If corrosion of PC steel is a concern, detailed investigations must be conducted and the frequency of routine inspections must be reviewed.

1.3.4 Regular inspections

Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and, in addition to matters relating to reinforced concrete structures, target anomalies specific to prestressed concrete.

Commentary: Special notes for prestressed concrete in regular inspections are shown below.

(i) Investigation items

Regular inspections are generally conducted close to the structure, and thus allow accurate assessment of cracking, water leakage, and efflorescence as well as the presence or absence of fracturing and corrosion in PC steel.

(ii) Inspection locations

Regular inspections focus on checking for anomalies specific to prestressed concrete. Specific examples of anomalies are shown below.

a) Flexural cracks and shear cracks occurring in PC structural members

Because the structure of prestressed concrete is generally not prone to cracking, cracking found in structural members is likely caused by a decrease in prestressing due to corrosion of the PC steel, a decrease in load bearing capacity, etc.

b) Cracks along PC steel

In locations with insufficient PC grouting, cracks along the PC steel and water leakage can sometimes be found relatively early following construction. When PC steel is anchored to the upper edge of a main girder, attention should be paid to the infiltration of drainage water from the bridge surface.

c) Water leakage from construction joints (segment joints)

Cracks may appear at construction joints, and PC steel may undergo corrosion and fracture due to effects of water leakage. When reinforcing bars are not continuous at precast concrete junctions and PC steel is undergoing corrosion due to insufficient PC grouting in an internal cable structure, care must be taken as this leads directly to a decrease in cross-sectional load bearing capacity.

(iii) Frequency of inspections

When concern exists over decline in load bearing capacity associated with corrosion in PC steel, detailed investigations must be conducted and the frequency of regular inspections must be reviewed.

1.3.5 Detailed investigations

In addition to matters relating to reinforced concrete structures, detailed investigations should be conducted to obtain more detailed information on structures in which anomalies specific to prestressed concrete have occurred.

1.4 Estimation and prediction of mechanisms of deterioration

(1) Mechanisms of deterioration shall be estimated based on inspection findings, with consideration given to environmental conditions and usage conditions of the prestressed concrete in addition to matters relating to reinforced concrete structures. As a general rule, the estimated mechanisms of deterioration are divided into deterioration specific to prestressed concrete and other deterioration.

(2) Predictions should be made using appropriate techniques based on the findings of inspections, targeting estimated mechanisms of deterioration or deterioration specific to prestressed concrete, based on Chapter 5 of "Maintenance: Standards."

Commentary: Regarding (1): When deterioration progresses to the point that corrosion of PC steel is accelerated in prestressed concrete, the process by which load bearing capacity declines is extremely short. Therefore, the decision was made to distinguish the content of "Maintenance: Standards" and Volume 1 from deterioration specific to prestressed concrete.

Regarding (2): Because deterioration specific to prestressed concrete directly affects load bearing capacity, the progress of deterioration cannot be appropriately predicted through visual inspection alone. Therefore, appropriate models should be examined based on the results of detailed investigations and changes in the state of deterioration, and predictions should be made through analysis or other means. In making predictions, care should be taken to note that not only do anomalies in the structure have effects as direct factors, but the structural properties of prestressed concrete also have effects as indirect factors (see **Table C1.4.1**). For example, even when PC steel has fractured, flexural cracking may not occur immediately in cases such as the following:

(i) The superstructure consists of multiple simple T girders and PC steel is fractured only in some of the main girders

(ii) The girder span is small

(iii) The fracture positions of the PC steel are not located at the critical sections

Table C1.4.1 Examples of factors that structural properties influence load bearing capacity (in the case of PC bridge)

Example of items	Example of influence factor	Example of focal points
Super structure	Structural type: Simple girder, continuous girder, rigid-frame Cross-section: I-girder, T-girder, hollow girder, box girder, hollow slab	Differences in cross-sectional force redistribution
Prestressed	Installation method: pre-tensioning method, post-tensioning method PC steel arrangement: inner cable method, outer cable method	Availability of PC grouting and ease of maintenance
Girder Manufacturing Method	Cast-in-place, precast	With or without joints
PC steel	PC steel wire, PC steel stranded wire, PC steel, fill rate of PC grouting	Difference in corrosion fracture morphology
Position of anchorage zones	Upper edge anchorage, edge anchorage	Ease of water ingress
Bridge face waterproofing	Yes, no	Differences in water supply
Scale of girder span	Span length: short (below 30m), medium (30-60m), long	Occurrence time of flexural crack
Deteriorated part	Precast girder: girder, interfiling Box girder: slab, web, under slab	Degree of decrease in stiffness and load bearing capacity
Structural design	Analysis method: methods based on beam theory and plate theory, static elastic analysis (framework analysis based on micro-deformation theory, FEM analysis), static nonlinear analysis, dynamic nonlinear analysis Verification methods: Prestressed concrete and PRC, design-determined and other cross sections	Difference in the amount of margin of load bearing capacity

1.5 Evaluation and judgment

(1) In addition to matters relating to reinforced concrete structures, evaluations of performance should be carried out with attention paid to the characteristics of prestressed concrete.

(2) In addition to matters relating to reinforced concrete structures, judgment of the need for remedial measures should be carried out with attention paid to the characteristics of prestressed concrete.

Commentary: Regarding (1): When corrosion occurs in PC steel, quantitatively estimating the safety of prestressed concrete in advance is difficult. When a decline in the load bearing capacity of prestressed concrete is confirmed, the cause is often salt damage.

1.6 Remedial measures

1.6.1 General

When remedial measures have been deemed necessary, in addition to matters relating to reinforced concrete structures, attention shall be paid to the characteristics of prestressed concrete and appropriate remedial measures shall be selected and implemented in accordance with Chapter 7 of "Maintenance: Standards."

Commentary: Of the five remedial measures in maintenance categorized in Chapter 7 of "Maintenance: Standards," repairs and retrofit must be conducted with due consideration given to the fact that concrete has been prestressed.

1.6.2 Repairs and retrofit

In the repair and retrofit of prestressed concrete, methods and materials shall be selected so as to satisfy the targeted performance, with appropriate consideration of the structural characteristics of the target structure and the effects of prestressing.

Commentary: The following is an overview of the PC grout re-injection method, a repair method that is specific to prestressed concrete and that is not shown in **Figure C7.3.1** of "Maintenance: Standards."

PC grout re-injection method: In post-tensioned prestressed concrete, the rate of corrosion in PC steel protected by PC grout is extremely low. In locations with insufficient PC grouting, however, a passivation film does not form over the steel surface and the locations are prone to corrosion. Therefore, grout must be re-injected at locations of insufficient PC grouting to ensure the integrity of the PC steel and the concrete structural members.

1.7 Recording

In addition to matters relating to reinforced concrete structures, recording shall address matters specific to prestressed concrete.

Chapter 2 Highway Bridge Deck Slabs

2.1 General rules

(1) The maintenance of highway bridge deck slabs in which performance has declined or is likely to decline due to the effects of repeated moving load and the spreading of deicing agents should be conducted with consideration of characteristics of highway bridge deck slabs, as presented in this chapter.

(2) This chapter applies to highway bridge deck slabs in maintenance category A or maintenance category B.

Commentary: Regarding (1): In 2.2., this chapter first presents maintenance methods for addressing fatigue in deck slabs. In 2.3, it then presents maintenance methods to address compound deterioration from chloride attack, freezing-and-thawing damage, alkali-silica reaction, and fatigue in deck slabs on which deicing agents are used. Premised on the content of 2.2, section 2.3 describes points of note in maintenance necessitated by deterioration caused by the use of deicing agents.

Regarding (2): Maintenance categories for deck slab fatigue and deck slab deterioration due to deicing agents are described in 2.2.1 (2) and 2.3.1 (2), respectively.

2.2 Fatigue of deck slab

2.2.1 General

(1) The maintenance of deck slabs in which performance has declined or is likely to decline due to fatigue should be conducted with consideration of the characteristics of fatigue in deck slabs as shown in this section.

(2) This section applies to deck slabs in maintenance category A or maintenance category B.

Commentary: Regarding (1): The three components of deck slabs, surface waterproof layers, and pavement should work as one to resist the water ingress into deck slabs. The performance required of each of these should be made clear and comprehensive remedial measures must be implemented. Note that surface waterproof layers may undergo deterioration or damage due to a variety of factors, and may be stripped off during replacement of pavement, necessitating remedial measures that assume this occurrence.

As shown in **Table C2.2.1** and **Figure C2.2.1**, the processes of deterioration in structures that are subjected to fatigue can be divided into an initiation stage, propagation stage, acceleration stage, and deterioration stage.

When water ingress into deck slabs is significant, even if the lower surface is in the initiation or propagation stage, anomalies could appear on the upper surface. Specifically, these are occurrences of potholes and horizontal cracks alongside upper reinforcing bars (**Figure C2.3.1**).

Table C2.2.1 Definition of deterioration processes

Appearance grade	Deterioration process	Definition	Factor determining the stage
Grade I	Initiation stage	At this stage, primarily due to drying shrinkage, a few unidirectional cracks are occurring in the main girder in the orthogonal direction. Depending on the restraint conditions of the main girder, these cracks may further propagate due to temperature changes and other factors.	Applied design standards: Thickness of deck slab Amount of distributed steel Span length of deck slab Construction: Drying shrinkage Service condition: Traffic volume Vehicle weight (axle load) Vehicle location Surface waterproof layer
Grade II	Propagation stage	At this stage, due to the action of the main girder, bending cracks are progressing in the orthogonal direction of the main girder, and cracks caused by bending of the deck slab in the main girder direction are also beginning to propagate, forming a grid-like network of cracks. The increase in crack density is significant, but the continuity of the floor plate (two-way plate) is maintained.	
Grade III	Acceleration stage	At this stage, a grid-like network of cracks develops on the underside of the deck slab, and some cracks penetrate through both the upper and lower surfaces of the deck slab, becoming through cracks. Subsequently, crack opening and rubbing against each other occur, resulting in smoothing and corner breakage of the crack surfaces. As a result, the resistance of the concrete cross section cannot be relied upon, and the punching shear capacity of the deck slab begins to rapidly decrease.	In addition to the above factors Environmental conditions: Influence of penetrated water Implemented remedial measures: Surface waterproof layer Repair/retrofit
Grade IV	Deterioration stage	At this stage, the occurrence of penetration cracks in the cross section of the deck slab causes the continuity of the deck slab to be lost, and the resultant beam-like members separated by the penetrating cracks have to resist wheel loads. The ultimate strength of the member is affected by such factors as penetrating crack spacing, concrete strength and the amount of steel. Consideration must also be given to factors such as rainwater ingress and steel corrosion.	All factors mentioned above

Initiation stage   Propagation stage   Acceleration stage   Deterioration stage

Figure C2.2.1 Cracking progression on underside of deck slab

2.2.2 Maintenance plan

When conducting maintenance on deck slabs that are subject to effects of fatigue, a maintenance plan and maintenance limit should be set with consideration of the progressive processes of deterioration.

Commentary: It is important to formulate maintenance plans with appropriate consideration of deck slab usage conditions (traffic volume, ratio of large vehicles, axle load, and other live load conditions) from the start of service to the present and extending to the foreseeable future.

At the stage of maintenance plan formulation, accurately predicting the condition of deck slabs throughout their service life generally entails great difficulties. Therefore, maintenance plans should be formulated with reference to the state of fatigue-based deterioration in similar deck slabs or deck slabs placed in similar environments.

It is advisable to use required performance as criteria for setting the maintenance limit for deck slab fatigue. In setting the maintenance limit, the appearance grades shown in **Table C2.2.1** should be used as indicators.

When maintenance category A has been set for fatigue of deck slabs, the following points must be assessed.

・Evaluation and prediction of axle load, total weight, number of cycles, travel position of passing vehicles, etc.

・Effects of rainwater or other water

Conversely, for deck slabs in maintenance category B, maintenance plans should be formulated under an approach of taking some form of action after deterioration has become apparent.

In the case of fatigue of deck slabs, after the stage in which unidirectional cracks occur in the direction orthogonal to the main girders on the lower surface of deck slabs, a progression from i) to iii) below, in that order, generally occurs. The maintenance limit should be set appropriately, with reference to these phenomena.

i) The stage at which a grid-like network of cracks begins to form on the lower surface of deck slabs

ii) The stage at which a grid-like network of cracks develops, cracks penetrate into the cross section of the deck slab, and water leakage and efflorescence occur

iii) The stage at which, as a result of opening/closing and friction among cracks, smoothing of crack surfaces on the lower surface of deck slabs and corner falls at the end surfaces have occurred

After iii), repeated occurrence of potholes or other damage to the pavement surface may appear in conjunction with the progress of the deterioration in deck slabs as described above or due to the effects of water ingress. In such cases, not only the lower surface of deck slabs but also the pavement surface must be checked and the condition of damage must be appropriately assessed.

Because it is difficult to confirm the progress of deterioration from appearance in deck slabs on which retrofitting has been implemented from the lower surface, as in the case of steel plate bonded deck slabs, the maintenance limit must be set as appropriate based on the characteristics of the retrofit structure.

Even if maintenance category B is applied to fatigue of deck slabs, it is advisable to set the maintenance limit to enable repairs at stage i) above.

2.2.3 Assessment

2.2.3.1 General

In the assessment of deck slabs in which performance has declined or is likely to decline due to fatigue, it is necessary to properly conduct inspections, evaluation of current condition, prediction of the progress of deterioration, and determination of the need for remedial measures on the basis of maintenance plans and maintenance limit.

2.2.3.2 Inspections

2.2.3.2.1 General

(1) In inspections of deck slabs subject to effects of fatigue, the initial inspections, general inspections, and regular inspections should be properly conducted based on the set maintenance categories and maintenance limit.
(2) When performing inspections, standard investigations should be conducted with inspection items, methods, frequencies, and scope set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

Commentary: Regarding (1) and (2): The items and methods of the investigations that make up inspections of deck slabs are mainly visual observations of appearance anomalies. Examples of methods of investigations for each item of investigation are shown in **Table C2.2.2**. It is advisable to also conduct inspections of anomalies on the upper surface of the deck slab concrete during procedures such as pavement replacement. Appearance grades on the lower surface of deck slabs can be classified as shown in **Table C2.2.3**.

Table C2.2.2 Examples of main investigation methods for each investigation item

General investigation items	Examples of information	Examples of main investigation method	Related standards
Anomalies in appearance	・State of crack (direction, density, width, corner falls) ・Water leakage, efflorescence, discoloration of concrete	・Visual observation, simple measurement, photography	NDIS 3418
	・Abnormal sound	・Hammering method	
Anomalies on road surface	・Crack, cave-in	・Visual observation, photography	NDIS 3418
Crack	・Depth, opening/closing, surface unevenness	・Contact gauge, etc.	
	・Propagation, density of crack, direction	・Document investigation (record on the density of crack, etc.)	
Cross-sectional quantities	・Concrete strength	・Rebound hammer	JIS A 1155 JSCE-G 504
		・Core test	JIS A 1108 JIS A 1149
	・Steel arrangement, position of steel, cover depth	・Electromagnetic wave measurement	
		・Ultrasonic radar method, etc.	NDIS 2426-1
	・Thickness of deck slab	・Ultrasonic radar method, etc.	NDIS 2426-1
	・Bending stiffness	・Deflection measurement	
Traffic characteristics	・Traffic volume, large vehicle percentage, lane-by-lane traffic volume percentage, type of vehicle, axle load	・Investigation of traffic volume, load measurement	
Surface waterproof layer	・Presence or absence・Type・Condition	・Visual observation	NDIS 3418
Deicing agent	・Presence or absence・Type・Amount, amount of snow, temperature data, etc.	・Document investigation (record on the use amount of deicing agent, etc.)	
Horizontal crack	・Presence or absence and extensity	・Electromagnetic radar method	NDIS 3429

Table C2.2.3 Appearance grades of deck slab and state of deterioration

Appearance grade	Deterioration process	State of deterioration
Grade I	Initiation stage	Primarily due to drying shrinkage, a few unidirectional cracks are occurring in the main girder in the orthogonal direction.
Grade II	Propagation stage	Due to the action of the main girder, bending cracks are progressing in the orthogonal direction of the main girder, and cracks caused by bending of the deck slab in the main girder direction are also beginning to propagate, forming a grid-like network of cracks.
Grade III	Acceleration stage	Grid-like network of cracks develops on the underside of the deck slab, and some cracks penetrate through both the upper and lower surfaces of the deck slab, becoming through cracks.
Grade IV	Deterioration stage	The occurrence of penetration cracks in the cross section of the deck slab causes the continuity of the deck slab to be lost, and the resultant beam-like members separated by the penetrating cracks have to resist wheel loads.

2.2.3.2.2 Initial inspections

(1) In initial inspections, standard investigations are conducted as appropriate for newly installed deck slabs, existing deck slabs, and deck slabs after large-scale repair or retrofit, to assess the initial state in maintenance of the deck slabs.
(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

2.2.3.2.3 Routine inspections

(1) In routine inspections, standard investigations are conducted with the goal of early detection of deterioration and assessment of its progress. These are based on the confirmation of water leakage, displacement, deformation, and other anomalies in appearance, as well as confirmation of cracking, peeling and spalling of cover concrete, rust water, efflorescence, discoloration, and other anomalies in the concrete surface. Anomalies in pavement are also investigated.
(2) When prediction, evaluation, and judgment of the progress of deterioration are difficult to perform based on standard investigations, detailed investigations should be conducted.

2.2.3.2.4 Regular inspections

(1) Regular inspections are conducted with the aim of obtaining information that is difficult to obtain in routine inspections, and entail more detailed investigations of anomalies, water leakage, etc. mainly through close visual observation of appearance. It is also necessary to combine hammer tapping and, as necessary, methods using non-destructive testing equipment, testing of concrete cores removed during inspections, and to assess environmental conditions. In addition, when anomalies in pavement have been confirmed from the road surface and the results have been compared with those from investigations of the lower surface of deck slabs with the conclusion that deterioration of the upper surface of deck slabs is suspected, detailed investigations should be carried out.
(2) When reinforcing material is installed on deck slabs, standard investigations of appearance properties and other properties of the reinforcing material should be conducted.
(3) When prediction, evaluation, and judgment of the progress of deterioration are difficult based on standard investigations in regular inspections specified in the maintenance plan, detailed investigations shall be conducted.

Commentary: <u>Regarding (1)</u>: In regular inspections of the lower surface of deck slabs, standard investigations of the deck slabs should in principle be close-up investigations. The density (direction), width, and behavior of cracks, deflection properties, and other properties should be measured through simple methods such as visual inspection and hammer tapping; the actions of operating vehicles and other traffic properties should also be investigated and monitored. When measuring crack density in regular inspections, it is important to assess the directionality, width, and behavior of cracks and to confirm crack propagation. Cracks that allow easy visual observation in the absence of load may be selected as the cracks to be measured.

2.2.3.2.5 Detailed investigations

(1) In standard investigations conducted within the initial inspection and within regular inspections specified in the maintenance plan, when assessment of the current state of deterioration and the prediction, evaluation, and judgment of the progress of deterioration are difficult in deck slabs subject to the effects of fatigue, or when standard investigations conducted during initial inspections, routine inspections, or regular inspections have revealed parts or structural members in which the progress of deterioration is significant, detailed investigations shall be performed to obtain more detailed information on the deck slabs.
(2) The investigated items, methods, and locations in a detailed investigation shall be appropriately selected with consideration of factors including the objectives of the investigation and the accuracy of the results.

2.2.3.3 Prediction

2.2.3.3.1 General

(1) As a general rule, in the maintenance of deck slabs in which deterioration due to fatigue has occurred or is a concern, performance at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of deck slabs requires quantitatively predicting deterioration due to fatigue.
(3) When (1) and (2) are difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of deterioration in the lower surface of the deck slabs.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.

Commentary: Regarding (1) and (2): In a performance verification-based maintenance system, predictions of changes (decline) in aspects of the performance of deck slabs are normally made. When using 3D finite element analysis to estimate fatigue life, nonlinearity must be taken into account through appropriate analytical models using methods with demonstrated applicable scope and accuracy.

Regarding (3) and (4): Quantitative prediction of the progress of deterioration in deck slabs due to fatigue may be uncertain due to the difficulty of identifying environmental conditions and other factors. In this case, the progress of deterioration may be predicted by considering factors that affect the progress of fatigue with respect to the condition of the lower surface or upper surface of deck slabs, as ascertained through visual observation.

Predicting the processes of fatigue deterioration in deck slabs requires observation of factors including the width and depth of cracks; the amount of opening/closing, amount of level differences, and other behavioral aspects of cracks; the state of water leakage and efflorescence appearance; and anomalies in the road surface.

Conversely, for deck slabs in which the lower surface is reinforced with steel plates, etc., or deck slabs in which compound deterioration due to deicing agents, etc. is occurring, it is advisable to assess the presence and extent of potholes and cracking based on the progress of cracking in not only the lower surface but also the upper surface of deck slabs (i.e., the pavement surface), and to take these progressive processes into consideration in predicting the progress of deterioration based on the upper surface of the deck slabs.

2.2.3.3.2 Revision of predictions

When the status of deterioration obtained from inspection findings differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to maintenance plans as necessary.

Commentary: With regard to fatigue in deck slabs, the chloride ingress under the use of deicing agents incorporates the effects of diffusion of chloride ions as well as the effects of water ingress due to gravity. In making predictions, equations for predicting of the progress of deterioration must be updated appropriately in line with inspection findings.

2.2.3.4 Evaluation and judgment

(1) When evaluating the performance of deck slabs with respect to fatigue, the characteristics of fatigue presented in this section should be taken into account.

(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance of deck slabs caused by fatigue, the maintenance limit, and the remaining planned service period.

Commentary: Regarding (1): The aspects of performance to be evaluated in each process of deterioration can be judged to some extent based on the state of cracking on the lower surface of deck slabs, as shown in **Figure C2.2.1**. Because quantitatively evaluating the performance of deck slabs is difficult at present, performance may be evaluated in inspections using the deck slab appearance grades presented in **Table C2.2.4**, with reference to **Table C2.2.3**.

Table C2.2.4 Appearance grades of deck slab and standard performance degradation

Appearance grade	Deterioration process	Load bearing capacity/ Toughness	Deformation/Vibration	Peeling/Spalling	Cracking/ Discoloration
Grade I	Initiation stage	–	–	–	Cracks, rust water, exposure of steel
Grade II	Propagation stage	–	–	–	
Grade III	Acceleration stage	–	Increase in deformation, occurrence of vibration ・Cracks on road surface, potholes ・Smoothing and corner fall of the crack surfaces ・ Penetration of cracks, rainwater ingress	Occurrence of peeling/spalling	
Grade IV	Deterioration stage	Decrease in load bearing capacity/toughness ・Decrease in cross-sectional area and adhesion of steel due to corrosion ・Extent of horizontal crack ・Gravelization			

2.2.4 Remedial measures

2.2.4.1 Selection of remedial measures

As a general rule, when measures to remedy the decline in performance of deck slabs due to fatigue have been deemed necessary, remedial measures should be selected so as to satisfy required performance in the deck slabs following implementation.

Commentary: When remedial measures have been judged necessary, one of i) strengthening of inspections, ii) repair, iii) retrofit, iv) restriction of service, and v) demolition/removal must be selected. When selecting remedial measures with deck slab appearance grade used as an indicator, **Table C 2.2.5** should be used as reference.

Table C2.2.5 Appearance grades of deck slab and remedial measures

Appearance grade	Deterioration stage	Intensified inspection	Repair	Retrofit**	Restriction in service	Partial replacement /Replacement
Grade I	Initiation stage	○	(○)	※		
Grade II	Propagation stage	◎	◎	※		
Grade Ⅲ	Acceleration stage	◎	◎*	※		
Grade Ⅳ	Deterioration stage	○	◎*	※	○	○

◎：Standard remedial measures［◎*: Including the restoration of mechanical performance］,
○：Remedial measures in some cases［(○)：Preventive remedial measures］,
※：Remedial measures implemented according to criteria other than appearance grade
**：Remedial measures in cases improving mechanical performance beyond the initial performance

2.2.4.2 Repairs and retrofit

(1) To obtain the desired effects from repairs and retrofit, work methods and materials shall be selected with consideration of decline in performance due to fatigue, and design must be carried out appropriately.
(2) After the implementation of remedial measures, it is necessary to confirm that the deck slabs satisfy the specified performance.

Commentary: Regarding (1): Repair and retrofit methods and materials are categorized as shown in **Table C2.2.6** according to their expected effect. When selecting methods and materials, the state of deterioration of deck slabs must be considered. **Table C2.2.7** shows standard work methods.

Table C2.2.6 Examples of methods of repair/retrofit and expected effects

Expected effect	Examples of methods
Improvement in the degree of effects on third party and appearance	Surface treatment
Improvement in fatigue resistance achieved by eliminating the effects of water	Installation of surface waterproof layer
Improvement in fatigue resistance achieved by controlling crack opening	FRP adhesion, prestressing
Restoration of cross-sectional stiffness achieved by installing members at extreme tension fiber	Adhesion of steel plates, etc. on underside of slab, overlay to enlarge cross section of reinforced concrete, installation of additional girders
Improvement in fatigue resistance achieved by improving shear stiffness of compression zone of cross section.	Overlay on upper surface of slab, (partial) replacement

Table C2.2.7 Appearance grades and examples of standard methods of repair and retrofit

Appearance grade	Deterioration process	Standard methods
Grade I	Initiation stage	Draining[※1], surface waterproof layer[※1]
Grade II	Propagation stage	Draining[※2], surface waterproof layer[※2], steel plate/FRP sheet adhesion, overlay on upper surface, overlay on underside, additional girder
Grade III	Acceleration stage (With influence of water)	Draining[※2], surface waterproof layer[※2], steel plate adhesion, overlay on upper surface
Grade III	Acceleration (Without influence of water)	Draining[※2], surface waterproof layer[※2], steel plate adhesion, overlay on upper surface, additional girder
Grade IV	Deterioration stage	Restriction in service, replacement, (partial) replacement[※2], replacement of upper layer

※1 : Preventive method
※2 : Implement in conjunction with other repair/reinforcement methods

With regard to deck slab fatigue, when a high risk of decline in performance is expected in cases of maintenance category A, the application of waterproofing to the upper surface of deck slabs before fatigue cracks propagate is effective in inhibiting the progress of fatigue. When water has an effect, methods which replace the upper layer of deck slabs are applied. It is advisable to temporarily attach support members to deck slabs in the deterioration stage.

2.2.4.3 Maintenance after remedial measures

(1) When remedial measures are implemented for deck slabs in which performance has declined due to fatigue, appropriate maintenance shall be performed with consideration of the decline in performance of the deck slab units, the repair/retrofit materials, and the interfaces between these, to ensure that the remedied deck slabs satisfy required performance over the remaining planned service period.

(2) When significant repeat deterioration occurs in deck slabs that have undergone remedial measures and the deck slabs are assumed to reach the maintenance limit again within their remaining planned service period, appropriate remedial measures shall be taken.

Commentary: Regarding (1): In principle, all deck slabs for which remedial measures have been implemented should be subjected to maintenance under maintenance category A. Conversely, in the case of deck slabs, etc., for which remedial measures were implemented at a stage in which deterioration has progressed considerably, maintenance plans must be formulated for maintenance category B, remedial measures must be implemented as necessary, and required performance must be satisfied.

Regarding repair/retrofit structural members, general maintenance must be performed so that repair/retrofit materials and deck slabs are integrated in resisting actions.

Regarding (2): Repair/retrofit can have negative effects, such as concrete deck slab lower surfaces being immersed in water or becoming no longer directly visible. Appropriate work methods and materials must be selected with consideration of these.

Table C2.2.8 provides considerations for the evaluation of repaired and reinforced deck slabs in which causes of decline in fatigue resistance are clear, such as those in which water has penetrated into the deck slab units and corrosion is occurring in steel plates, or in which rainwater is leaching from the ends of steel plates.

Table C2.2.8 Considerations for the quantitative evaluation of performance degradation in fatigue of repaired/retrofitted deck slab

Performance	Consideration
Load bearing capacity, toughness	Extent of horizontal cracks in the deck slab Gravelization of deck slab and its size and range Leaching of rainwater from the edge of the repaired/retrofitted material Reduction in cross section of steel members due to corrosion Extensive lifting of reinforcement Decrease in adhesion between material of repair/retrofit and deck slab body
Deformation, vibration	Crack on road surface, pothole, caving
Peeling, spalling	Occurrence of peeling/spalling of concrete
Appearance	Water leakage, efflorescence Deterioration of paint on steel members

2.2.4.4 Replacement of deck slab

(1) In the replacement of a deck slab, an appropriate plan shall be formulated and methods and materials shall be selected with consideration of factors including the surrounding environment, workability, methods of disposal of dismantled deck slabs, the construction period, decline in performance due to fatigue in newly installed deck slabs, life cycle cost, and time and space constraints during service, based on the premise that the bridge as a whole will satisfy required performance.
(2) Maintenance plans for replaced deck slabs should be in accordance with 2.2.2, with maintenance performed for maintenance category A.
(3) When replaced deck slabs differ in structure from those prior to demolition, assessment and remedial measures shall be performed with said structural characteristics taken into account.

Commentary: Regarding (1): When performing renewal using replacement precast deck slabs, sufficient consideration must be given to the fatigue resistance of cast-in-place parts, including joint parts.

Regarding (2): When deterioration that was not assumed at the time of design has become apparent, remedial measures for maintenance category B should be implemented quickly. In this case, the remaining planned service period and required performance must be made clear and satisfaction of required performance must be ensured over a long period of time, such as through use in combination with appropriate retrofit methods.

2.2.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to fatigue in deck slabs shall also be recorded.

2.3 Deterioration of deck slabs under the use of deicing agents

2.3.1 General

(1) The maintenance of deck slabs in which performance has declined or is likely to decline due to compound deterioration consisting of varied types of deterioration and fatigue, arising due to the use of deicing agents, should be conducted with consideration of the characteristics of deterioration in deck slabs under the use of deicing agents as shown in this section.
(2) This section applies to structures in maintenance category A or maintenance category B.

Commentary: Regarding (1): Premised on the content of 2.2, this section describes points of note in maintenance to address deterioration caused by using deicing agents.

Table C2.3.1 presents definitions of processes of deterioration related to deck slab fatigue, as well as factors determining their timing, related to the use of deicing agents. When water infiltrates into deteriorated parts and repeated traffic action (i.e., fatigue) occurs, this accelerates damage specific to deck slabs, such as the occurrence of horizontal cracks on the upper surface of upper reinforcing bars and the gravelization of cover concrete, as shown in **Figure C2.3.1**. When the upper surface of deck slabs deteriorates due to gravelization, even when the situation corresponds to the propagation stage as presented in **Table C2.2.1** and **Figure C2.2.1**, anomalies on the lower surface of deck slabs may have reached the deterioration stage. Assessment, remedial measures, and recording must be implemented with this taken into consideration.

Table C2.3.1 Deterioration processes of deck slab under the application of deicing agent and factors determining the stage

<table>
<tr><th>Appearance grade</th><th>Deterioration process</th><th>Definition</th><th>Factor determining the stage</th></tr>
<tr><td>Grade I</td><td>Initiation stage</td><td>Chloride ions penetrate with water from the upper surface of deck slab through cracks.</td><td rowspan="2">Applied design standards:
Thickness of deck slab
Amount of distributed steel
Span length of deck slab

Construction:
Drying shrinkage

Service condition:
Traffic volume
Vehicle weight (axle load)
Vehicle location

Amount of splayed deicing agent
Surface waterproof layer</td></tr>
<tr><td>Grade II</td><td>Propagation stage</td><td>One or more of the following types of deterioration can develop from the upper surface of the deck slab: freeze-and-thawing damage (scaling), chloride attack (steel corrosion), and alkali-silica reaction depending on the aggregate used.</td></tr>
<tr><td>Grade III</td><td>Acceleration stage</td><td>The synergistic actions of steel corrosion, scaling, alkali-silica reaction, water ingress, and fatigue cause horizontal crack and gravelization near the upper steel.</td><td>In addition to the above factors
Environmental conditions:
Influence of penetrated water

Implemented remedial measures:
Surface waterproof layer
Repair/retrofit</td></tr>
<tr><td>Grade IV</td><td>Deterioration stage</td><td>The fatigue resistance rapidly declines due to horizontal crack and gravelization.</td><td>All factors mentioned above</td></tr>
</table>

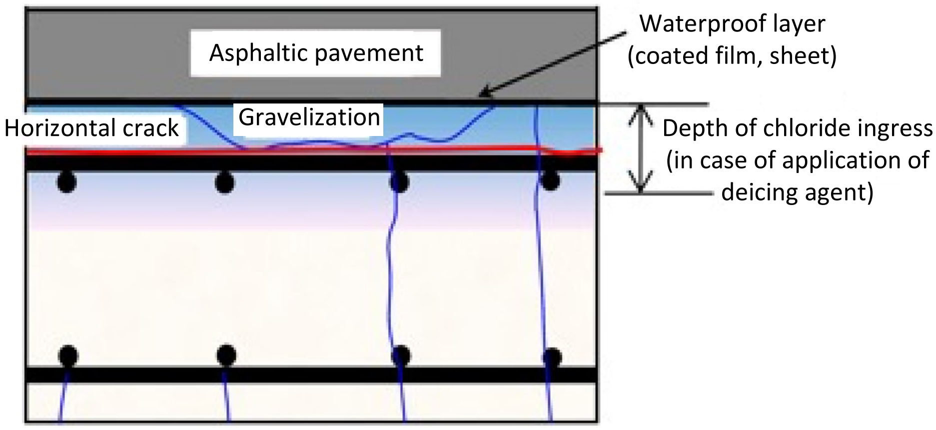


Figure C2.3.1 Anomalies of upper surface of the deck slab

2.3.2 Maintenance plan

When conducting maintenance on deck slabs under the use of deicing agents, a maintenance plan and maintenance limit should be set with consideration of the progressive processes of deterioration.

Commentary: When maintenance category A has been set, remedial measures must be implemented so that deterioration does not emerge. It is important to prevent inadvertent water ingress containing sodium chloride into deck slabs, necessitating a variety of remedial measures.

When maintenance management category B has been set, remedial measures must be implemented at the stage in which cracks and potholes on the pavement surface, scaling on the upper surface of deck slabs, steel corrosion, cracking due to alkali-silica reaction, horizontal cracks, gravelization, and other defects have occurred.

2.3.3 Assessment

2.3.3.1 General

In the assessment of deck slabs in which performance has declined or is likely to decline under the use of deicing agents, it is necessary to properly conduct inspections, evaluate the current condition, predict the progress of deterioration, and determine the need for remedial measures on the basis of maintenance plans and maintenance limit.

2.3.3.2 Inspections

(1) Initial inspections, routine inspections, and regular inspections of deck slabs under the use of deicing agents should be properly conducted based on the maintenance categories of the structures.
(2) Inspections should be conducted with inspection items, methods, frequencies, and scope properly set in advance in the maintenance plan. Detailed investigations should also be conducted as necessary.

2.3.3.3 Prediction

(1) As a general rule, in the maintenance of deck slabs under the use of deicing agents, performance of the deck slabs at the time of inspection should be quantitatively assessed and future performance should be predicted.
(2) Predicting the decline in performance of deck slabs requires quantitatively predicting deterioration due to fatigue, based on assessment of the state of deterioration due to chloride attack, freezing-and-thawing damage, and alkali-silica reaction.
(3) When (1) and (2) are difficult, the durations of the initiation stage, propagation stage, acceleration stage, and deterioration stage may be predicted with consideration of the progress of deterioration.
(4) As a general rule, the progress of deterioration should be predicted based on inspection findings.
(5) When the status of deterioration due to chloride attack, freezing-and-thawing damage, alkali-silica reaction, and fatigue, as obtained from inspection findings, differs from predicted values, predictions of the progress of deterioration shall be revised following investigation of the causes of the differences. Changes should also be made to maintenance plans as necessary.

Commentary: Regarding (1) and (2): When deicing agents are used, it is advisable to assess the presence and extent of potholes and of cracking based on the progress of cracking not only in the lower surface but also the upper surface of deck slabs (i.e., the pavement surface), and to take these progressive processes into consideration in predicting the progress of deterioration from the upper surface of the deck slabs.

2.3.3.4 Evaluation and judgment

(1) When evaluating decline in performance in deck slabs under the use of deicing agents, reference should be made to the characteristics of deterioration in deck slabs under the use of deicing agents as shown in this section.
(2) Judgment of the need for remedial measures shall be made with consideration of the degree of decline in performance of deck slabs under the use of deicing agents, the maintenance limit, and the remaining planned service period.

2.3.4 Remedial measures

(1) As a general rule, when remedial measures to remedy the decline in performance of deck slabs under the use of deicing agents have been deemed necessary, remedial measures should be selected so as to satisfy required performance in structural members following implementation.

(2) To obtain the desired effects from repair and retrofit, work methods and materials shall be selected with consideration of decline in performance due to chloride attack, freezing-and-thawing damage, alkali-silica reaction, and fatigue, and design shall be carried out appropriately.

(3) After the implementation of remedial measures, it is necessary to confirm that the deck slabs satisfy the specified performance.

Commentary: Regarding (1): In deck slabs in which pumping phenomena are occurring on the pavement surface, gravelization is likely progressing over a wide area. When the application of work methods such as overlaying of the upper surface is difficult solely through removal of deteriorated locations on the upper surface of deck slabs, it is important to plan the replacement of deck slabs. When implementing remedial measures from the lower surface of deck slabs, beginning from the lower surface will allow rainwater, etc. to pool inside the deck slabs. Therefore, waterproofing work should first be implemented on the upper surface, followed by remedial measures implemented from the lower surface.

Regarding (2): For deck slabs in which performance has declined under the use of deicing agents and which have been placed under maintenance management category A, remedial measures that integrate floor slabs, waterproof layers of the surface, and pavement must be implemented to maximally prevent water that contains deicing agents from penetrating into deck slabs. Conversely, remedial measures for maintenance category B may include the removal of parts weakened by horizontal cracks and gravelization and replacement with appropriate concrete.

Regarding (3): In addition to the post-remedial-measure maintenance of deck slabs in which performance has declined due to fatigue, when deterioration combining chloride attack, freezing-and-thawing damage, and alkali-silica reaction is suspected, attention must be paid to maintenance after implementing remedial measures to address the causes of deterioration.

2.3.5 Recording

When creating records of inspections, prediction of progress of deterioration, evaluations, judgments, remedial measures, and so on, matters specific to deck slabs under the use of deicing agents shall also be recorded.

Volume 3
Changes in Required Performance Level

Chapter 1 General Rules

1.1 General

This volume provides basic points on the planning, assessment, design, method selection, execution, maintenance after remedial measures, and recording for ensuring performance in existing structures when the required performances for structures are reviewed or when the methods for performance verification are modified.

Commentary: If the required performances for structures are revised or if the actions and limit values used for performance verification are revised, it is necessary to verify the performance of existing structures based on the new methods or standards. Additionally, remedial measures such as retrofit should be applied as needed. The representation of the performance of a structure in terms of mechanical indicators such as load bearing capacity, displacement, etc. is referred to as the performance level. Therefore, the required performance level refers to the minimum values of these mechanical indicators, generally representing the limit of verification values set during design.

Changes to the required performance level of existing structures may involve the following measures: retrofit, replacement, service restrictions, and demolition/removal.

(i) Response to changes in the required performance level for accidental actions:

Seismic retrofit (including the installation of seismic isolation and damping devices) and retrofit due to re-evaluation of assumed tsunami heights.

(ii) Response to changes in the required performance level for variable actions:

Retrofit, replacement, service restrictions, demolition, and removal in response to re-evaluation of variable actions such as changes in live loads (e.g., increase in vehicle size).

(iii) Response to changes in environmental actions:

Response to sea level rise due to climate change, abnormal rainfall, and other related factors.

(vi) Response to other changes in use:

Retrofits, including widening, installation of fences or walls, or raising the height due to changes in purpose.

If seismic design standards are revised during the service life of a structure, it may result in the structure not satisfying the required performance demanded by the latest standards. In such cases, it is crucial to comprehensively consider the following factors to accurately assess the priority of seismic retrofitting.

(i) The importance of the target structure and the social impact in the event of damage occurring

(ii) The design standards that the target structure complied with

(iii) The probability of occurrence of seismic or other actions

(iv) The vulnerability of the structural members to failure (failure modes of the members)

Chapter 2 Planning of Remedial Measure

2.1 General

(1) As a general rule, when implementing measures the plan should satisfy the latest standards. If there is a need to prioritize the implementation of remedial measures, it is necessary to consider not only the various performance aspects of the existing structure but also the societal impacts, such as the availability of alternatives.
(2) Generally, the planning of remedial measures should include items such as assessment, design and method selection, execution, maintenance after remedial measures, and recording.
(3) When formulating a plan for remedial measures, it is necessary to select the optimal method based on the required performance level, characteristics of the target structure or members, and the surrounding environment, ensuring the desired effects are achieved.
(4) During the formulation of the plan, maintenance management after implementing remedial measures shall be examined by considering the remaining service life, and it should be carried out to facilitate post-measure inspections or further actions.

Chapter 3 Assessment

3.1 General

When considering remedial measures for changes in the required performance level, it is necessary to assess the existing performance of the structure. To determine the necessity of measures, appropriate assessment shall be implemented by considering the importance, urgency, economy, and other factors of the structure.

Commentary: Assessment can range from simple estimation of performance based on the design philosophy at the time of construction considering the design and construction period of the structure, to more advanced methods involving direct investigation of the structure and conducting detailed evaluations based on the results.

3.2 Investigation for assessment

The standard method for assessment is an investigation of the design documents and other relevant records. However, if the quantity or accuracy of the obtained information is insufficient or if the structure has anomalies, it is necessary to conduct on-site inspections.

Commentary: Methods as described in Chapter 6 of the "Maintenance: Standard" are applied for the evaluation of the existing performance of structures or members conducted during assessment. On-site inspections for assessment are conducted in the following cases:

(i) When there are no design documents or as-built drawings available, and data such as member dimensions, steel arrangement, material strengths, etc. cannot be obtained

(ii) When deterioration has occurred in the structure

For case (ii), it is recommended to follow Chapter 6 of the "Maintenance: Standard."

3.3 Evaluation of the performance and Judgment on the necessity of remedial measures

Evaluation of the performance and judgment of the necessity of remedial measures can be implemented by following Chapter 6 of the "Maintenance: Standard".

Chapter 4 Selection of Design of Remedial Measure and Method

4.1 General

(1) The design of remedial measures shall be conducted in an appropriate manner to satisfy the target performance.
(2) The method of remedial measures shall be appropriately selected based on the existing performance of the target structure or member, the target performance, the importance, urgency, and economy of the structure, and the environment or local conditions.

Commentary: Regarding (1): In the design of remedial measures, it is important to note that the improvement of performance through retrofit or other measures in existing structures is effective only against the additional loads that act after the implementation of those measures. Additionally, it is necessary to overlay and verify the response values of the existing members before the remedial measures and the response values of the members after the additional load increment. Alternatively, it is necessary to examine the difference between the limit values after the remedial measures and the limit values before the remedial measures with the response values caused by the additional load increment.

For seismic retrofit, it is necessary to carefully consider the differences in the load bearing capacity of each member after retrofit and the differences in load bearing capacity between the devices added for retrofit and the attachment points or existing members that support them. Taking into account the ease of post-earthquake inspection and repair work, it is necessary to perform seismic retrofit on each member of the structure within a range that ensures an appropriate difference in load bearing capacity. By considering the maximum strength exhibited by the devices added for seismic retrofit and the difference in load bearing capacity between them and the attachment points or existing members that support them, it is necessary to ensure that the expected functionality of the devices is reliably manifested during earthquakes. When implementing remedial measures such as widening or raising the structure, it is necessary to appropriately evaluate the behavior of both the existing members and the components added for widening or raising and design the structure as a whole system to achieve the optimal result. If the remedial measures increase the stiffness of specific parts, the balance of load distribution in the overall structure system may be disrupted, resulting in localized load concentration. Therefore, caution is required, particularly for statically indeterminate structures.

The safety factors for structural analysis coefficients and other parameters used in the design of remedial measures should be appropriately set based on the chosen execution method. For newly added members or parts, the same safety coefficients (material coefficients) as those used for newly constructed structures should be applied. For the materials used in existing members or parts, it is recommended to adopt either values consistent with those used for new structures or values based on actual measurements obtained through on-site investigations or similar means.

Regarding (2): There are various methods to improve the performance level, including increasing the bending and shear strength of members, enhancing deformation capacity, and reducing the actions and responses. Accordingly, there are various remedial measures that can be employed, such as overlay, jacketing, adhesion, installing reinforcement materials, prestressing, and implementing seismic isolation or seismic dampers. In some cases, remedial measures like raising the structure's height may also be taken to address tidal levels, wave heights, or rising water levels.

Table C4.1.1 provides an explanation of the characteristics and application examples of representative remedial measures.

Table C4.1.1 Characteristics and applications of typical remedial measure methods

Methods	Characteristics	Applications
Concrete overlaying, jacketing	・Relatively large increase in dimension and deadweight ・Relatively simple maintenance	・ Relatively large cross-sectional members ・In case architectural restrictions or obstruction of rivers are not a concern
Steel plate adhesion, jacketing	・Relatively small increase in dimension and deadweight ・Corrosion protection and other maintenance required	・ Relatively small cross-sectional members ・In case there are constraints due to architectural restrictions or obstruction of rivers.
Continuous fibers adhesion, jacketing	・Almost no increase in dimension or deadweight ・Simple maintenance due to high corrosion resistance ・Possible to construct manually by human ・Flexural reinforcement of the base of bridge piers and similar structures is difficult to accomplish alone	・ High places or confined areas, locations where heavy machinery cannot be utilized ・In case there are constraints due to architectural restrictions or obstruction of rivers.
Mortar overlaying, jacketing	・The increase in dimensions and deadweight is smaller compared to concrete. ・Relatively simple maintenance	・In case there are constraints due to architectural restrictions or obstruction of rivers.
Installation of reinforcing materials	・By installing new members, it is possible to reduce the sectional forces ・ Verification is required to assess the changes in sectional force distribution	・Planar members such as deck slab
Introduction of prestressing	・ By generating negative bending moments, it is possible to reduce the sectional forces caused by dead loads ・Verification of concrete strength is necessary when introducing prestressing	・Rod members such as girders and beams (for bending reinforcement)
Seismic isolation, damping devices	・By installing seismic isolation or damping devices, it is possible to reduce the inertia forces and vibrations ・In seismic isolation, due to larger displacements of the members, displacement limits become necessary	・In case methods like jacketing are impractical
Additional members, widening, raising	・Widening and raising of the structure can be achieved by adding new members ・In order to shorten the construction period and ensure space under girders, precast members or steel-concrete composite structures may be applied. However, the joints with existing structures are often cast-in-place concrete.	・ Widening of bridges, widening of retaining walls, raising of coastal breakwaters, etc.

Existing structures that are subject to remedial measures are generally facilities that are already in service.

Here are some examples of retrofit methods for typical structures.

(i) Retrofit of girders, beams, columns, and rigid frame bridge structures:

・Overlaying and jacketing method using concrete or mortar reinforced with steel or other reinforcing materials

・Adhesion and jacketing method using steel plates or continuous fibers

・Method of placing steel bars or segmented steel plates around the outer perimeter of columns

・Method of bonding thin steel plates in multiple layers

・Method of jacketing with concrete segments and steel strand

・Method of retrofitting only one side of a column using steel plates and steel bars

・Method of installing rubber, dampers, or braces

(ii) Retrofit of bridge piers:

・Jacketing method using concrete or mortar reinforced with steel or other reinforcing materials

・Jacketing method using steel plates or continuous fibers

・Method of execution using mechanical joints in underwater sections

・Method of installing concrete-filled steel tubes from above ground

・Method of installing straight steel poling board from above ground

・Method of inserting steel members afterward

(iii) Retrofit of side walls and deck slabs of underground box culverts (for execution from the inside space):

・Overlaying method using concrete or mortar reinforced with steel or other reinforcing materials

・Adhesion method using steel plates or continuous fibers

・Method of installing support

・Method of inserting shear reinforcement materials afterward

Chapter 5 Execution of Remedial Measure

5.1 General

(1) The execution of remedial measures for changing the required performance level shall be carried out appropriately using materials that have had their quality verified.

(2) During and after the execution of remedial measures for changing the required performance level, inspections shall be conducted according to a rational inspection plan and using appropriate inspection methods. This is to verify that the retrofit effects intended in the retrofit design are effectively implemented during execution.

Commentary: Regarding (1): The execution of remedial measures should be carried out appropriately using materials that have been verified for quality, following “Construction” and Chapter 7 of “Maintenance: Standard.”

For remedial measures, various methods such as overlaying, jacketing, adhesion, installation of reinforcing materials, and introduction of prestressing are commonly used. These methods often involve operations such as drilling into existing members. To ensure proper anchorage and sufficient adhesion at the joints between existing concrete and newly added concrete for retrofit, as well as adhesive surfaces of steel plates and continuous fibers, it is necessary to remove any weak portions or contaminants on the concrete surface and ensure sound conditions. In recent times, water jetting has also been used as a method to remove concrete without damaging steel or the internal concrete.

Regarding (2): Ideally, the performance of the members or structures where remedial measures have been implemented should be directly confirmed after construction by inspection. In practice, it is necessary to conduct appropriate intermediate inspections at each stage of execution to ensure that the performance of the structure with implemented remedial measures meets the required standards. Additionally, it is important to confirm the strength and other properties of the materials used in the remedial measures through test certificates and quality control tests.

Chapter 6 Maintenance after Remedial Measure

6.1 General

Structures that have undergone remedial measures shall receive maintenance throughout their remaining service life to ensure that the performance of the structure does not fall below the maintenance limits.

Commentary: The maintenance of structures after remedial measures should be carried out according to the procedures outlined in "Maintenance." It is worth noting that the remedial measures for ensuring the performance of existing structures, such as retrofit, often fall under large-scale measures. Therefore, it is advisable to follow the procedures for maintenance of newly constructed structures, including cases where the remedial measures are implemented gradually.

It is important to incorporate any special considerations on maintenance related to the adopted remedial method into the establishment of maintenance limits in advance.

Furthermore, there may be cases where additional remedial measures need to be implemented if the selected remedial method is limited to urgent temporary remedial measures, or if it is discovered that separate remedial measures are necessary.

Chapter 7 Recording

7.1 General

Information regarding the implementation of remedial measures for structures shall be recorded in a suitable manner and kept for the necessary period.