EVALUATION METHOD OF DULABILITY IN CONCRETE STRUCTURES

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SUMMARY

In general, concrete is quite a durable construction material so that a countless number of public structures and buildings have been constructed by using it worldwidely. However, when improper selections of used materials and mix proportion of concrete are carried out, or when the structure is exposed to very severe environment, some of deterioration may occur on the concrete structures of which functions decrease in their levels less than the required ones. Therefore, an evaluation of durability in concrete structure throughout its service life is very important at the design stage not only for selecting appropriately the materials and the mix proportion of concrete and deciding structural details of the structure, but also for ensuring the reliability of performance of the designed structure.
This paper describes the present state of evaluation methods for durability of concrete structures as well as durability design concepts which are based on the design methods of the JSCE Standard Specifications for Concrete Structures.

Keywords: *Evaluation of durability; durability design; deterioration of concrete; carbonation; reinforcement corrosion due to the ingress of chloride ions; freezing and thawing action; chemical attack; alkali aggregate reaction; JSCE Standard Specification.*

INTRODUCTION

Concrete is one of the most important materials employed in public works and building construction projects and a countless number of concrete structures have been constructed worldwide. The conventional design framework for concrete structures is primarily based on safety but currently focused on the aspects of durability. Under the conditions where problems on improper selections of used materials and mix proportion of concrete have arisen and some of the concrete structures are affected by hard deterioration actions in very severe environment, the durability of concrete structures surely be threatened to reduce its performance. Furthermore, it is obvious that environmental aspects should be also

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incorporated into the design of concrete structures. From an environmental viewpoint, it can generally be thought that life-extension of a structure is directly related to the reduction of environmental impact.

Consequently, the establishment of reasonable durability designs for concrete structures is very important, and now a day, many efforts have been made to evaluate the durability of concrete structures. In 2002, the JSCE Standard Specification for Concrete Structures (JSCE 2002) provided tools for verifying durability of concrete structures numerically, which was the first real code for the durability design in the world.

This paper outlines the present state of durability design, then describes the durability design methods of the JSCE 2002.

PRESENT STATE OF DURABILITY DESIGN

We have been facing various and sometimes serious durability problems in concrete structures. In general, durability is defined as the ability of concrete to resist weathering action, chemical attack, abrasion, and other conditions of service. The severity of environmental, chemical and physical attacks on concrete depends on the properties of concrete and its exposure conditions. The actual deterioration phenomena of concrete structures include corrosion of reinforcement in concrete due to the ingress of chloride ions, carbonation of concrete, freezing and thawing, alkali-aggregate reaction, etc.

As the corrosion of reinforcement due to the ingress of chloride ions which are supplied from marine environment, use of deicing salt scattered on the road in the winter season, etc., is a typical and one of the most severe deterioration on concrete structures, it has been understood that concrete cover and its quality are the key and many efforts have been made to evaluate it quantitatively. The carbonation of concrete due to carbon dioxide has a disadvantage in reinforced concrete because the lower pH in carbonated concrete can not keep the passive condition on steel and introduces corrosion on the reinforcement. Freezing and thawing resistance can be secured by introducing an appropriate air-void system in concrete. The alkali-aggregate reaction can be prevented by several countermeasures such as the setting of threshold content of alkali in cement, the utilization of blast furnace slag and fly ash as mineral admixtures, etc. Nevertheless, most of the existing durability design codes do not provide a tool for evaluating the ingress of chloride ions in concrete, the carbonation of concrete, and other deteriorations.

Durability design may be categorized into three levels as follows:

(1) prescriptive design
(2) performance-type design
(3) performance-based design

In the prescriptive design for the durability of concrete, for example, the maximum water cement ratio and minimum cement content are provided depending on the exposure conditions. Table 1 indicate an example of the recommended limiting values for composition and properties of concrete provided in the EN206 (BS EN206-1 2000). All requirements are stipulated in a prescriptive manner. It may be said that prescriptive design is based on the simplification of safety side from real performance. However, the background of most provisions is unclear. For example, the water-cement ratio for corrosion protection of reinforcement in concrete cannot be easily determined because it is also directly related to the concrete cover.

Therefore, it is more reasonable and accurate to consider performance with time. In principle, the required performance should be verified through a direct analysis of time-dependent behavior of a concrete structure under the assumed environmental actions. At present, however, it is difficult to predict the durability performance of a structure directly throughout the lifespan because of the inadequacy of the models necessary for calculations. Further development of research on numerical approaches will pave the way to the realization of performance-based design. Under such the current situations, what we can do to verify the durability in our codes is to introduce a “performance-type design” in which principal durability performance is considered with time. The JSCE 2002 provided such a design concept for durability design.

JSCE STANDARD SPECIFICATION FOR CONCRETE STRUCTURES

Framework

The JSCE Standard Specification stipulates its fundamental concepts at the design stage of structures as follows:
“At the design stage, structural details such as the shape, size, reinforcement arrangement, required properties of concrete and reinforcing material, method of construction (in situ, pre-cast, etc.) and maintenance plan should be decided also taking into account the economic consideration. It should also be ensured that the required performances in terms of serviceability, safety, durability and compatibility with the environment, etc. are satisfied over the service life of the structure.”

Based on these concepts, the following four parts were prepared in the Specifications:

(1) Structural performance verification
(2) Seismic performance verification
(3) Materials and construction
(4) Maintenance

The procedure for verifying mechanical performance of concrete structures is given in the parts of the structural performance verification and the seismic performance verification.

The materials and construction part provides the general framework and sets standards for general concrete construction with issues related to use of special concrete or the construction of special structures. Additionally, this part deals with the performance verification for degradation of a structure on account of the deterioration of concrete and reinforcement. The performance of concrete structures varies over time due to environmental conditions and other factors. The examination on whether such change is in acceptable range is described here.

Once the construction is completed, it is difficult to repair, strengthen or renovate concrete structure, so thorough investigation at the beginning stage of design, accurate prediction for possible problem in service life and future maintenance are of great importance. The maintenance part provides basic knowledge for the maintenance of concrete structures.

Figure 1 indicates the framework of the contents to be covered in each specification.

Durability Verification of Concrete Structures

General concept

The performance of concrete structures has to remain the required one throughout its design service life. Conversely meaning, there will be no problems when utilized under a certain condition in which the required performance of the concrete structure is satisfied even if concrete or reinforcements partially deteriorate. This is a fundamental concept of the durability design that requires the performance verification of durability for concrete structures, not for concrete. In the durability verification of concrete structures in the JSCE Standard Specifications, this concept is also provided while the concept in the existing prescriptive durability design methods is completely different from it.

In the Specification, the performance verification methods for the deterioration of structure due to carbonation, the ingress of chloride ions, cyclic freezing and thawing action, chemical attack, and alkali aggregate reaction are stipulated as well as the verification method for water tightness and fire resistance of the structure are dealt with. In the following sections, these verification methods are described in detail.

Verification for carbonation

As the carbon dioxide from the atmosphere penetrates into concrete, the pH in the cover concrete reduces. If such a zone of reduced pH reaches the location of the reinforcing bars in the concrete, they are rendered susceptible to corrosion. Once the corrosion is initiated, the formation and deposition of expansive corrosion products on the bars may cause the formation of longitudinal cracks along the reinforcing bars, which in turn could accelerate further corrosion and spalling of the cover concrete, and finally a significant reduction in the cross-section of the reinforcement. Therefore, it must be ensured in the durability design that the performance of the structure is not allowed to fall below the required level by the corrosion of the reinforcing bars. Based on this concept, to check that reinforcement corrosion due to carbonation in concrete does not occur, is comparatively easy and essentially on the conservative side. It is sufficient to verify that the depth of carbonation is less than the critical depth to initiate steel corrosion.

Consequently, the verification of a structure for carbonation is conducted by ensuring that

$$\gamma_i \frac{y_d}{y_{\text{lim}}} \leq 1.0 \qquad (1)$$

Where,

γ_i **:** Factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.

y_{lim} **:** Critical carbonation depth of steel corrosion initiation.

y_d **:** Design value of carbonation depth.

It is understood in general that corrosion of steel reinforcement may begin before the carbonation depth actually becomes equal to the cover thickness (i.e. the carbonation front reaches the location of the reinforcement). It means that the critical carbonation depth of steel corrosion initiation y_{lim} may be obtained from Equation 2.

$$y_{\text{lim}} = c - c_k \qquad (2)$$

Where,

c **:** Expected value of cover thickness (mm). In general, it may be taken as the design cover thickness (mm)

c_k **:** Remaining non-carbonated cover thickness (mm).

The remaining non-carbonated cover thickness is defined as a thickness of uncarbonated concrete still remaining in the neighborhood of the steel reinforcement even when the reinforcement corrosion just initiates, as shown in Figure 2. Usually, there are indeed very rare cases in which corrosion is able to impair the performance of a structure having so long as the remaining non-carbonated cover was more than 10mm. However, when chloride ions are present in concrete, corrosion may start even at larger remaining non-carbonated cover thickness (more than the 10mm) because the chloride ions can easily cause damage to the passivation film of the reinforcement in the condition having the high alkalinity. Therefore, in such cases, higher remaining non-carbonated cover thickness (i.e. 10 to 25mm) is recommended.

For estimating the depth of carbonation, the 'square-root' law (i.e. the depth of carbonation varies linearly with related to the square-root of time) is normally used, as it is considered to agree with previous studies and is easy to use. Design value of carbonation depth y_d, therefore, may be obtained from Equation 3.

$$y_d = \gamma_{cb} \cdot \alpha_d \sqrt{t} \tag{3}$$

Where,

α_d **:** Design carbonation rate ($mm/\sqrt{year}$),

t **:** Designed service life of structure (year).

γ_{cb} **:** Safety factor to account for the variation in the design value of carbonation depth. Normally it may be taken as 1.15. In the case of high fluidity concrete, it may be taken as 1.1.

In the JSCE Specification, the design carbonation rate α_d is also given as Equation 4.

$$\alpha_d = \alpha_k \cdot \beta_e \cdot \gamma_c \tag{4}$$

Where,

α_k **:** Characteristic value of carbonation rate ($mm/\sqrt{year}$)

β_e **:** Coefficient representing the extent of environmental action. It may be taken as 1.0 for environments in which the surface of structure is difficult to be dried out, or for the north-facing surface. It may be increased to 1.6 for environments in which structures can be easily dried out or for the south-facing surface

γ_c **:** Factor to account for the material properties of concrete. In general it may be taken as 1.0, but should be taken as 1.3 for upper portions of the structure.

Figure 3 shows the relation between the effective binder ratio and coefficient of carbonation speed. The data consist of different types of binder, including fly ash and blast furnace slag. Although there is large scattering in the data, the following equation can be introduced to estimate the characteristic value of carbonation rate α_k:

$$\alpha_k = -3.57 + 9.0\ W/B \tag{5}$$

where, W/B: water binder ratio

It is obvious that the carbonation of concrete is dependent on the exposure environment of the structure. To consider the effect of exposure environment, environmental factor β_e is introduced in Equation 4. As indicated in Figure 4, the values of β_e for concrete in a dry environment or when concrete faces south direction and that in a wet environment or when the concrete faces north direction are 1.6 and 1.0, respectively.

Figure 5 shows the required minimum cover thickness at different water cement ratios and service years calculated according to the verification method mentioned above, in which the corrosion of reinforcing bars due to carbonation of concrete is prevented.

Verification for reinforcement corrosion due to the ingress of chloride ions

Chloride ions can easily penetrate into concrete from outer environment and cause the corrosion of steel reinforcements in concrete. In cases of the concrete structures in marine environment, the ones on which the deicing agents are used, and etc., it is very important to verify the reinforcement corrosion due to the ingress of chloride ions.

As well as the matter mentioned in the clause of the verification for carbonation, if the structural performance does not appear to be impaired, it may still be considered serviceable even if there are some signs of reinforcement corrosion on the structure. In other words, the structural integrity of the structure may considered to be intact so long as corrosion induced longitudinal cracks are not formed along the reinforcing bars, even if there are some other signs of reinforcement corrosion. However, as far as the verification of the performance of a structure with respect to chloride penetration is concerned, a condition that chloride induced corrosion of the reinforcement should not occur during the service life of the structure, is relatively easy to understand and on the safe side. It may be sufficient to carry out to ensure during verification that the chloride ion concentration at the location of the reinforcement is below the critical concentration that could initiate corrosion in the reinforcement.

In the Standard Specification, therefore, the verification of a structure for reinforcement corrosion due to the ingress of chloride ions is conducted by ensuring that

$$\gamma_i \frac{C_d}{C_{\lim}} \leq 1.0 \qquad (6)$$

Where,

- γ_i **:** Factor representing the importance of the structure. In general, it may be taken as 1.0, but should be increased to 1.1 for important structures
- $C_{\lim}$ **:** Critical chloride concentration for initiation of steel corrosion.
- C_d **:** Design value of chloride ion concentration at the depth of reinforcement.

The critical chloride concentration $C_{\lim}$ in the neighborhood of the reinforcement, which could initiate corrosion in the reinforcement, has been reported to be about 0.3-1.2 kg/m^3 of concrete by the past research works. For example, values of about 0.3-0.6 kg/m^3 have been reported from accelerated tests carried out with chlorides added to the fresh concrete, and values of 1.2-2.4 kg/m^3 from exposure tests carried out in the field. These differences in the critical chloride concentrations reported on the basis of accelerated and exposure tests, etc., can be attributed to factors such as differences in the water-cement ratio of the concrete, cover to the reinforcement, etc. The effect of high temperatures sometimes used in accelerated tests can also influence conclusions. Taking all the factors into account, the Specification sets a limit of 1.2 kg/m^3 (of chloride per cubic meter of concrete) as the critical chloride concentration $C_{\lim}$, which can initiate reinforcement corrosion of which level is considered detrimental to the performance of the structure.

Mathematical formulations based on the diffusion theory are most commonly used to model chloride penetration in concrete. When using the equations such as Equation 7 based on the Fick's second law of diffusion to model, the design value of chloride ion concentration at the depth of reinforcement C_d is considered satisfactory.

$$C_d = \gamma_{cl} \cdot C_0 (1 - erf(\frac{0.1 \cdot c}{2\sqrt{D_d \cdot t}})) \qquad (7)$$

Where,

C_0: Assumed chloride ion concentration at concrete surface (kg/m^3). Generally, it may be obtained from Table 2

c: Expected value of concrete cover thickness (mm). In general, the designed cover thickness may be selected.

t : Design service life of the structure (year).

γ_{cl}: Safety factor, to account for the variation in the design value of the chloride ion concentration at the depth of reinforcement C_d. Normally, it may be set at 1.3, but in the case of high fluidity concrete, a value of 1.1 may be selected.

D_d : Design value of diffusion coefficient of chloride ions into concrete (cm^2/year)

erf(s) **:** Error function, defined as $erf(s) = \frac{2}{\pi^{1/2}} \int_0^s e^{-\eta^2} d\eta$

This equation is based on the concept that chloride ions diffuse equally toward steel. When bending crack is generated on concrete cover, chloride ion concentration also depends on the distance from the crack, even though the variation of concentration of chloride ion may trend to be small when crack width is very narrow. In the JSCE Specification, therefore, such influence of the crack on the diffusibility of chloride ions can be evaluated by introducing a concept of the average diffusion coefficient of chloride ion on cover concrete, described as Equation 8, in which the influence of both concrete quality and concrete crack are considered.

$$D_d = \gamma_c \cdot D_k + \left(\frac{w}{l}\right) \cdot \left(\frac{w}{w_a}\right)^2 \cdot D_0 \qquad (8)$$

Where,

γ_c: Factor to account for the material properties of concrete. In general, it may be set at 1.0 but should be taken as 1.3 for upper portions of the structure. However, if there is no difference in the quality of concrete in structure and that of specimens cured in laboratory, this value may be taken as 1.0 even for all portions of the structure.

D_k: Characteristic value of diffusion coefficient of chloride ion in concrete (cm^2/year)

D_o: Constant to express the influence of crack on the movement of chloride ions into concrete (cm^2/year). In general, it may be taken as 200cm^2/year

w**:** Crack width (mm).

w_a**:** Allowable crack width (mm).

w/l**:** Ratio of crack width to crack interval

The ratio of crack width to crack interval, w/l is introduced in order to express the influence of crack averagely. This averaging method is effective when crack width is controlled smaller than allowable crack, but the resistance to ingress of chloride ions may be rapidly influenced by the crack if crack width is large. $(w/w_a)^2$ in the equation indicates the influence of crack width.

In the calculation, the chloride diffusion coefficient must be set as the characteristic value D_k. In the Specification, the chloride diffusion coefficient is essentially determined on the basis of the actual measurement of chloride ion concentration distributing into concrete. However, from the values reported on the basis of the surveys from a large number of structures as shown in Figure 6, it is clear that there is a large variation in the diffusion coefficient determined from the chloride concentration distributions in these structures. This means that the actual concrete has a diffusion coefficient for which the characteristic value for design is on the safety side as described in Equation 9.

$$\gamma_p \frac{D_p}{D_k} \leq 1.0 \qquad (9)$$

Where,

D_p **:** Predicted value of diffusion coefficient of actual concrete (cm^2/ year).

γ_p **:** Safety factor to account for the accuracy in determining D_p

In the Specification, the following equation for the prediction of D_p, were introduced as the predicted values:

(a) When ordinary Portland cement is used;

$$\log D_p = -3.9(W/C)^2 + 7.2(W/C) - 2.5 \qquad (10)$$

(b) When blast furnace slag cement is used;

$$\log D_p = -3.0(W/C)^2 + 5.4(W/C) - 2.2 \qquad (11)$$

Figure 7 shows the required minimum cover thickness at different water cement ratios for concrete structures suffering from the chloride attack under 50 years of design service life. In the calculations, Equation (10) and (11) is used for predicting the diffusion coefficients of the concretes made of ordinary Portland cement and blast furnace slag cement, respectively.

Verification for cyclic freezing and thawing action

In cold regions, a cyclic freezing and thawing action is essential as the cause of deterioration on concrete structures, and raises pop-outs, scaling, and formation of micro cracks at the concrete surface. The degree of these damages on concrete structures depends not only on the quality of concrete but also several other factors, such as the number of cycles of freezing and thawing actions, lowest temperature and the degree of water saturation of the concrete. However, up to now, virtually no quantitative information is available to relate these damages to any change in the performance of the structure under the freezing and thawing action. This means that it is difficult to specify a threshold level of ‘allowable’ deterioration and to utilize such the permissible level as indices or criteria for the verification of the required performance of the structures subjected to this action. Only a realistic manner is to examine whether or not deterioration due to the freezing and thawing action is likely to occur in a structure, or how much degree of deterioration will progress inside concrete.

In general, it can be considered that the structures will keep their required performance when the concrete has sufficient resistance to cyclic freezing and thawing action. Therefore, in the normal concrete structures under the action, a deterioration level in which some deterioration may occur on concrete but any degradation in the functions of the structure does not arise can be considered as a standard threshold level to keep the durability in structures.

The relationship between the results of the accelerated tests and the change in the level of performance in an actual structure subjected to cyclic freezing and thawing action is somewhat better understood on the basis of past research works and field data. It recommends that parameters such as the relative dynamic modulus of elasticity, which are measured in the accelerated tests, can be used as indices for the verification of the resistance of concrete to the action. Consequently, in the JSCE Specification, the verification of a structure for cyclic freezing and thawing action is conducted by ensuring that

$$\gamma_i \frac{E_{min}}{E_d} \leq 1.0 \tag{12}$$

Where,

γ_i **:** Factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.

E_d **:** Design value of relative dynamic modulus of elasticity and is given as Equation 13.

$$E_d = \frac{E_k}{\gamma_c} \tag{13}$$

where, E_k **:** Characteristic value of the relative dynamic modulus of elasticity.

γ_c **:** Material factor. In general, it may be set at 1.0, but should be taken as 1.3 for upper positions of the structure. However, if there is no difference in the quality of concrete in the structure (in situ) and that of laboratory-cured specimens, this value may be set at 1.0 for all positions.

$E_{\min}$ **:** Critical minimum value of relative dynamic modulus of elasticity to ensure required performance of the structure under cyclic freezing and thawing action. In general, it may be obtained from Table 3.

As for predicting the relative dynamic modulus of elasticity, the accelerated test in accordance with the method specified in JIS A 1148 (A method)"The freeze-thaw test method of concrete (the freeze-thaw test method of concrete underwater)" is recommended. The relationship between the characteristic value of relative dynamic modulus of elasticity of concrete E_k and the predicted one by the accelerated test E_p is shown in Equation 14.

$$\gamma_p \frac{E_p}{E_k} \leq 1.0 \tag{14}$$

Where,

E_p **:** Predicted value of relative dynamic modulus of elasticity of concrete.

γ_p **:** Safety factor to account for the accuracy in determining E_p. When tests are carried out in accordance with JIS A 1148 (A method), it may be taken to be 1.0

If actual freezing and thawing conditions are more severe than the condition stipulated in JIS A 1148 (A method), or if the structure is being designed for a very long service life, the test conditions in terms of the freezing and thawing temperatures, the time durations for the freezing and thawing cycles, etc. should be appropriately changed in the acceleration test condition.

In order that dams and other important structures satisfy the level of durability required, it may be essential to not allow the relative dynamic modulus of elasticity to fall below 80%. Further, in cases where it is required that the structure retains its original appearance and soundness over the entire design service life, no change in the value of the relative dynamic modulus of elasticity may be acceptable. In such cases, the relative dynamic modulus of elasticity may be kept at a level higher than 95%.

Verification for chemical attack

When aggressive chemicals come in contact with or penetrate the concrete, they may react with the hydration products of cement causing dissolution of the concrete or formation of expansive products which causes cracking in concrete and is sometime followed by spalling of the cover concrete etc. However, due to the limited understanding on how the deterioration of concrete under chemical attack results in the degradation of the structure performance, unfortunately quantitative evaluation has not been realized. Therefore, only conceptual provisions are introduced in the JSCE Specification for the verification of a structure for chemical attack.

The degradation in performance and the change in performance with time in the structure under the chemical attack are basically related to the change in the performances of the concrete itself as a constituent material. Thus, in order to ensure the durability of the structure under the chemical attack, it may be enough to ensure that the deterioration of concrete is kept below a certain pre-determined level (so as not to cause unacceptable levels of changes in the structural performance). It also be convenient and rational to actually set a certain level of chemical resistance in the concrete in order to preserve the integrity of the structure. Consequently, in case the concrete meets the criteria for resistance against chemical attack, the structure may be assumed that its performance will not be impaired on account of chemical attack

In an actual verification for resistance to chemical attack in concrete, accelerated tests, exposure tests, or any other suitable tests shall be performed on concrete specimens at the conditions as close to the actual conditions as possible. Then, the verification should be carried in a manner to ensure that there is no notable deterioration in concrete on account of the chemical attack, or that the deterioration is confined to levels that do not significantly affect the performance of the structure.

In general, the required performance level for structures in a relatively mild chemical environment, such as acid rain, marine structures, etc., may be set as a level such that any deterioration due to chemical attack is not easily seen by the naked eye. However, in cases when the chemical attack is likely to be severe, such as in the case of structures in contact with sewage or in the hot springs, etc., the required performance level may be set at a level, where the deterioration does not affect the structural performance of the structure. In all cases the importance of the structure should be duly accounted for in the process of determining the required level of performance.

When it is required that the deterioration of concrete is just within the level that does not affect the required performance of the structure, it is allowed in the specification to use the following maximum water-cement ratio, instead of the verification on the resistance to chemical attack of concrete directly:

(a) When concrete are in contact with soil or water, which contain 0.2% or more of a

sulfate such as SO_4, maximum water cement ratio is 50%.
(b) When deicing salts are used, maximum water cement ratio is 45%.

When the chemical attack on the concrete is very severe, it may be difficult to secure the performance of the structure against chemical attack simply by increasing the concrete cover thickness and increasing the resistance of concrete. Certain sewer facilities and structures near hot springs could be examples of such concrete structures. In such cases, it may be more realistic and rational to recommend the use of corrosion resistant reinforcement materials and/or providing surface coating for the concrete.

Verification for alkali-aggregate reaction

Depending on the kind of reactive mineral present in the aggregate that reacts with the alkali in the concrete, alkali aggregate reaction can be divided basically into two types, which are alkali silica reaction and alkali-carbonate reaction. However, from the occurrence of alkali aggregate reaction in different regions and countries in the world, it is clear that most of the alkali aggregate reaction is in fact the alkali-silica reaction, so that, it has been assumed that it may be sufficient to carry out appropriate verification for the alkali-silica reaction.

Deterioration due to alkali aggregate reaction in concrete is characterized as the deterioration to develop cracks in concrete by the formation of reaction products which are generated from the reaction of alkali in the concrete with the reactive minerals in the aggregate and then absorb water accompanying an expansion of the product. However a clear relationship between the degree of the cracking in concrete and the degradation rate in performance or its changes with time of the concrete structure has not been established yet. There are also reports that the structural load-carrying capacity of reinforced concrete structures is not impaired by the occurrence of alkali aggregate reaction, providing that they have at least a certain amount of reinforcement.

However, it has been considered appropriately to set the minimum required performance level of a structure susceptible to alkali aggregate reaction as a level which does not cause onset of the reaction induced cracks in concrete during the service life of the structure. The following implications of the crack formation have been kept in mind in fixing this limit – (i) onset of these cracks accompanies an appearance of a white gel-like substance which is aesthetically unappealing, the cracks provide accelerated access to other deleterious material, such as gases, water and ions, and (iii) it is very difficult to stop the alkali aggregate reaction through repair methods once cracks have been formed. Furthermore, in some structure, breakage of the reinforcing steels has been found out due to the large expansion of concrete.

As same as in the case of degradation in the performance of a structure under chemical attack described in the previous clause, the degradation in performance and the changes in performance with time in a structure due to the alkali aggregate reaction are also basically related to the changes in the performance of the concrete itself as a constituent material. Thus, in order to ensure the durability of a structure susceptible to alkali aggregate reaction, it may be enough to ensure that the deterioration in the properties of concrete is kept below a certain pre-determined level so as not to cause unacceptable levels of changes in the structural performance. In the JSCE Specification, therefore, it is stipulated that in case the concrete meets criteria for resistance against alkali aggregate reaction, the structure may be assumed that its performance will not be impaired on account of alkali aggregate reaction.

The most reliable method to verify the resistance of a structure to alkali aggregate reaction is to cast concrete specimens in the same conditions as the real structure, then expose them in similar environmental conditions and confirm the possibility of crack formation. However, considering various involving factors, such as the testing time as well as the required expense, the need to test concrete for various kinds of materials and proportions, etc., this real exposure test is not always feasible.

Therefore, at present the verification for the resistance to alkali aggregate reaction is usually carried out on the basis of the accelerated tests using concrete specimens. The JCI AAR-3 "Test method for evaluation of aggregate reactivity in concrete" is one of such methods that may be used as a reference in carrying out accelerated tests. It has been confirmed from laboratory experiments and field investigations that an expansion level of less than 0.1% at the age of 6 months in the test carried out based on this method may not cause appreciable degradation in the performance of concrete structures. This level can thus, be used as a criterion to decide whether or not the expansion due to alkali aggregate reaction could be detrimental. Namely, the verification for resistance to alkali aggregate shall be conducted out by insuring that:

$$\gamma_p \frac{L_p}{L_{\max}} \leq 1.0 \qquad (15)$$

Where,

L_p **:** Predicted expansion of concrete due to alkali aggregate reaction (%). Generally, this is set equal to the expansion at 6-month of test periods in accordance with JCI AAR-3.

$L_{\max}$ **:** Maximum permissible expansion rate at which concrete still satisfies the required resistance against alkali aggregate reaction. Generally, it may be set at 0.10%.

γ_p **:** Safety factor to account for the accuracy in determining. When tests are carried out in accordance with JCI AAR-3, it may be taken to be 1.0

When using aggregates which have been conformed to non-reactive as per the test method for evaluation of aggregate reactivity in concrete, and when restraining alkali aggregate reaction in concrete using appropriate cement, the verification for the resistance to alkali aggregate reaction can be omitted. On the other hand, it should be borne in mind that degradation in performance of a structure suffering from alkali aggregate reaction could be accelerated in the presence of chloride ions penetrating into concrete from outside. The fact should be adequately accounted for a higher level of resistance to alkali aggregate reaction required of such a concrete.

CONCLUDING REMARKS

In 2002, the JSCE Standard Specification was the first to provide a concept for durability design of concrete structures as a part of the performance-based design concept, while most of the existing design codes in the world have not yet provided such tools. This paper introduced the outline of the verification methods of the durability described in the Specification. However, the method mentioned here is not a final goal but the advanced methods for predicting directly the durability performance of the structure throughout the lifespan is required for the realization of performance-based design.

Concrete structures and their components should be planned, designed, constructed, and operated, inspected, maintained and repaired in such a way that they maintain their required performance during their design lives with sufficient reliability for the safety and the intended use of the structure, etc., under expected environmental conditions. As a result, the predicted service lives, t_S, of the structure and its components should meet or exceed their design lives, t_D, though t_S may be possible to be less than t_D for the components that are inspectable and replaceable. Figure 7 shows one of the mathematical models for predicting the service life. In the model, the verification of the durability of the structure in the performance-based design concept is conducted by ensuring that

$$P\{f\}_{t_D} = P\{R(t_D) - S(t_D) < 0\} < P_{target} \tag{16}$$

or

$$P\{f\}_{t_D} = P\{t_S \le t_D\} \le P_{target} \tag{17}$$

Where,

$R(t_D)$**:** Resistance capacity of the structural component at the design life, t_D

$S(t_D)$**:** Cumulative degradation of the component at the design life, t_D

$P\{f\}_{t_D}$**:** Probability of failure in the designed structure at the design life, t_D

P_{target}**:** Specified target probability

The probability of failure in the designed structure $P\{f\}_{t_D}$ in Equation 16 is indicated in Figure 8 by the shaded overlap area of the probability density curves for R(t) and S(t) on a vertical axis, while the $P\{f\}_{t_D}$ in Equation 17 indicated by the shaded area of the probability density function shown on the horizontal axis**.** For completion of this model, however, it is important that R(t) and S(t), which are the models evaluating the time-dependent behaviors in the performances of the structure and the progress of the degradation due to deterioration actions respectively, have to be developed as quantitative analysis models on the basis of the probability theory. Furthermore, the concept of life cycle cost is also important as the factors deciding the specified target probability P_{target}, so that a quantitative evaluation method of the life cycle cost should be established as soon as possible as well.

There is no need to mention that concrete structures have contributed to the social and economic activities of human beings. On the other hand, civil engineering and building structures consume enormous resources and emit huge amount of greenhouse gases. Durability problems of concrete structures are directly linked to environmental impacts because the shortening of the lifespan will result in the wasteful utilization of limited natural resources. Thus, in the viewpoints not only of the safety of the structures in their service lives but also of the preservation of environmental conditions, it will become more important to improve durability design methods.

REFERENCES

British Standard EN206-1, “Concrete — Part 1: Specification, performance, production and conformity,” 2000.

Japan Society of Civil Engineers, “Environmental impact evaluation of concrete (Ⅱ),” Concrete Engineering Series 62, 2004.

ISO 13823 (WD), “General Principles on the Design of Structures for Durability”, April 2005

Table 1 An example of recommended limiting values for composition and properties of concrete〔EN206〕

	Exposure classes																	
	No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environments		
						Sea water			Chloride other than sea water									
	X0	XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XF1	XF2	XF3	XF4	XA1	XA2	XA3
Maximum *w/c*		0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m^3)	–	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	–	–	–	–	–	–	–	–	–	–	–	–	4.0[1)]	4.0[1)]	4.0[1)]	–	–	–
Other requirements												Freeze/thraw resisting aggregates in accordance with the recommendations in prEN 12620:1996					Surfate-resisting cement[2)]	

1) Where the concrete is not air entrained, the performance of concrete should be tested according to ISO FFF-1 in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven.

2) When SO_4 1eads to exposure classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate-resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulfate-resisting cement should be used exposure class XA3.

Table 2 Chloride ion concentration at concrete surface C_0 (kg/m^3)

Tidal and splash zone	Distance from the coastline (km)				
	Near coastline	0.1	0.25	0.5	1.0
13.0	9.0	4.5	3.0	2.0	1.5

Considering the effect of elevation above the water surface, an increase of 1m in elevation may be considered to be equivalent to a horizontal distance of 25m. The equivalent C_0 may then be calculated from the above table.

Table 3 Minimum critical level, E_{min}(%), of relative dynamic modulus of elasticity to ensure a satisfactory performance of the structure under cyclic freezing and thawing action

Climate / Section / Exposure of structure	Severe weather conditions or frequent cyclic freezing and thawing action		Not so severe weather conditions, atmospheric temperature rarely drop to below 0°C	
	Thin [2]	General	Thin [2]	General
(1) Immersed in water or often saturated with water [1]	85	70	85	60
(2) Not covered in item (1) above and subjected to normal exposure conditions	70	60	70	60

1) Structures close to the water surface or in contact with water such as waterways, water-tanks, abutments of bridge, bridge piers, retaining walls, tunnel linings, etc. Besides, structures such as slabs, beams etc not close to the water surface but may be exposed to snow, water flow, spray, etc., also belong to this category.
2) Members with thickness less than 20cm may be considered ‘thin’.
3) Generally, the verification in section (2) may be omitted in cases when the characteristic value of relative dynamic modulus of elasticity, E_k, is higher than 90.

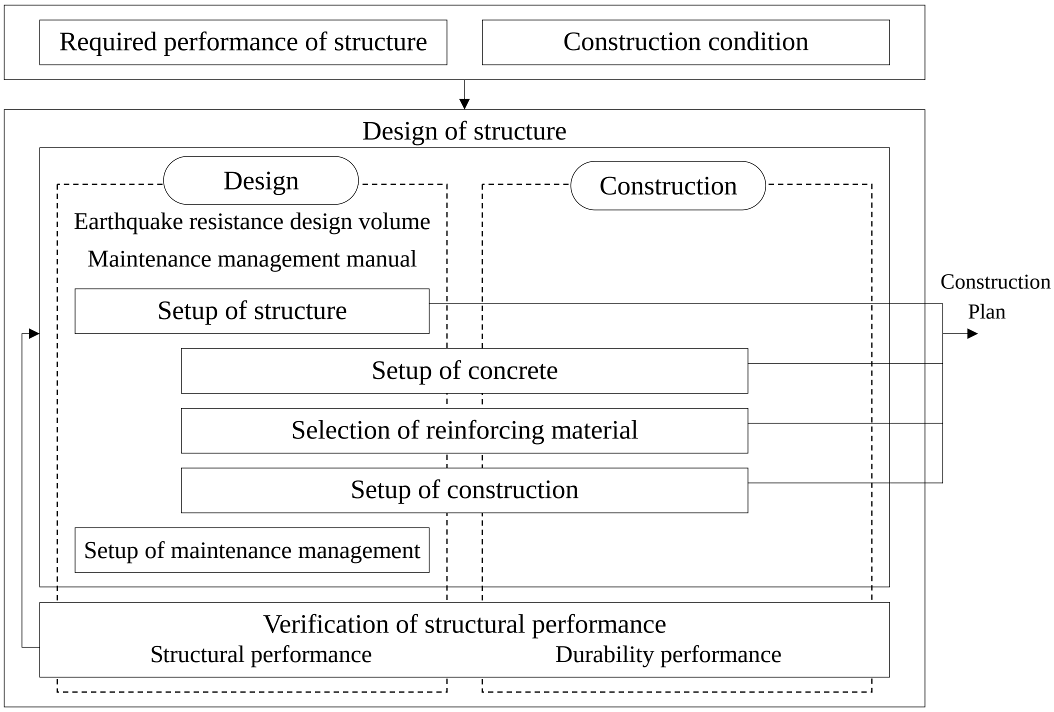


Figure 1 Works at design stage

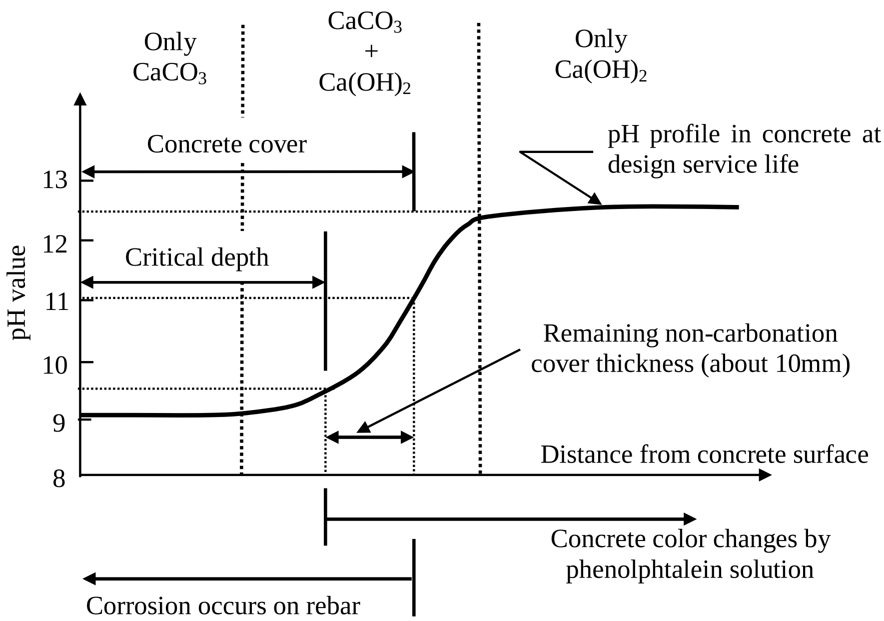


Figure 2 Schematic figure on pH distribution in carbonated concrete

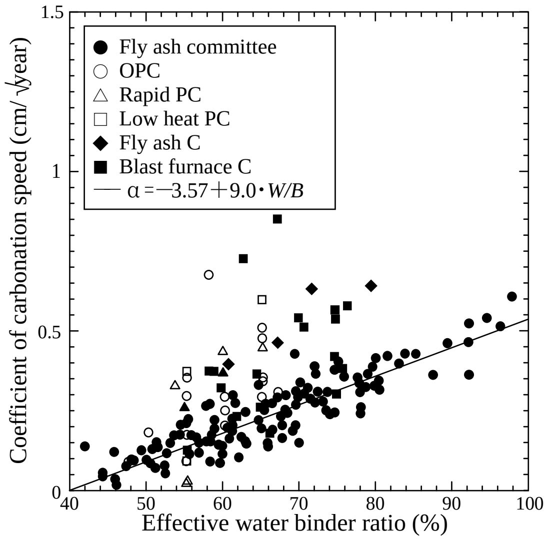


Figure 3 Relation between water binder ratio and carbonation speed

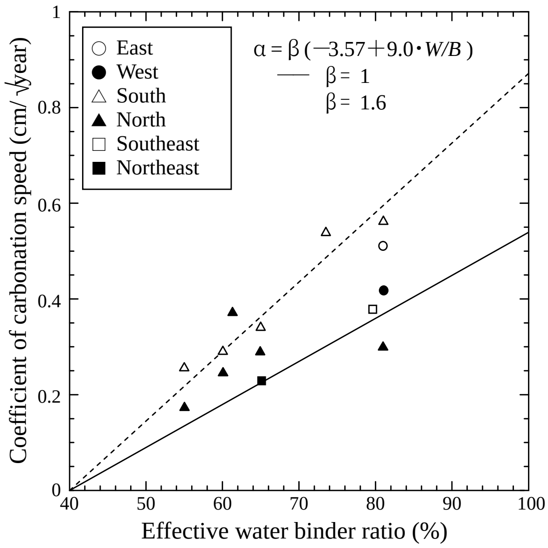
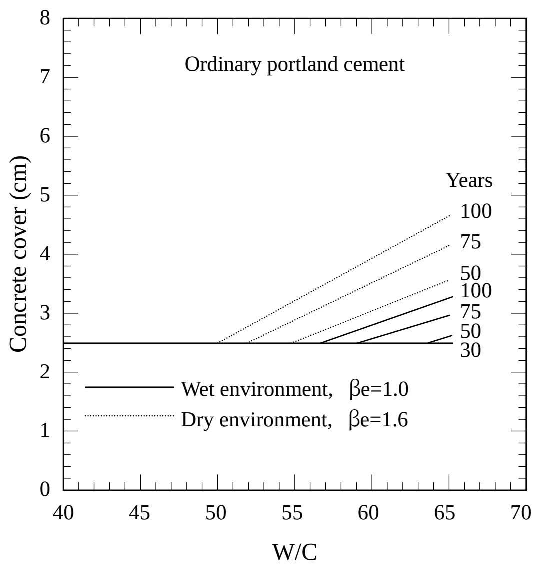


Figure 4 Relation between environment and carbonation speed



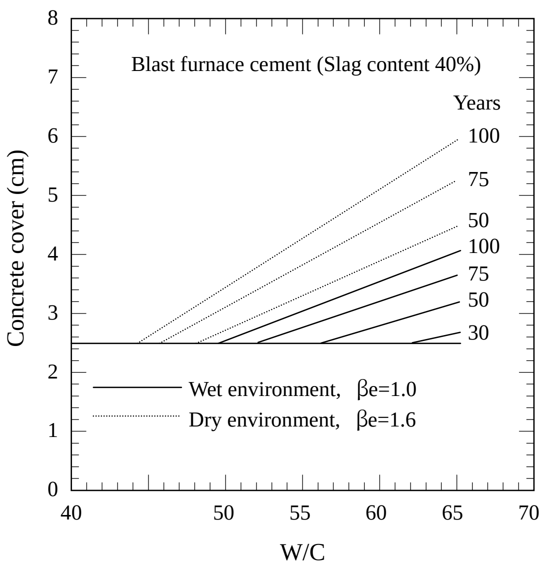


Figure 5 Minimum cover at different water cement ratio and service years for concrete structures under carbonation

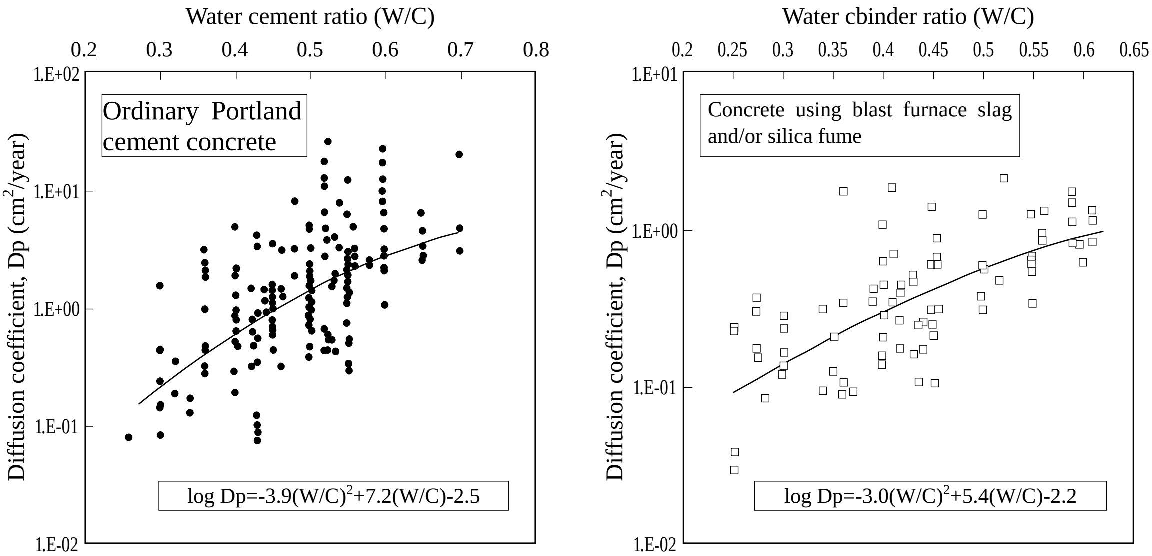


Figure 6 Diffusion coefficient

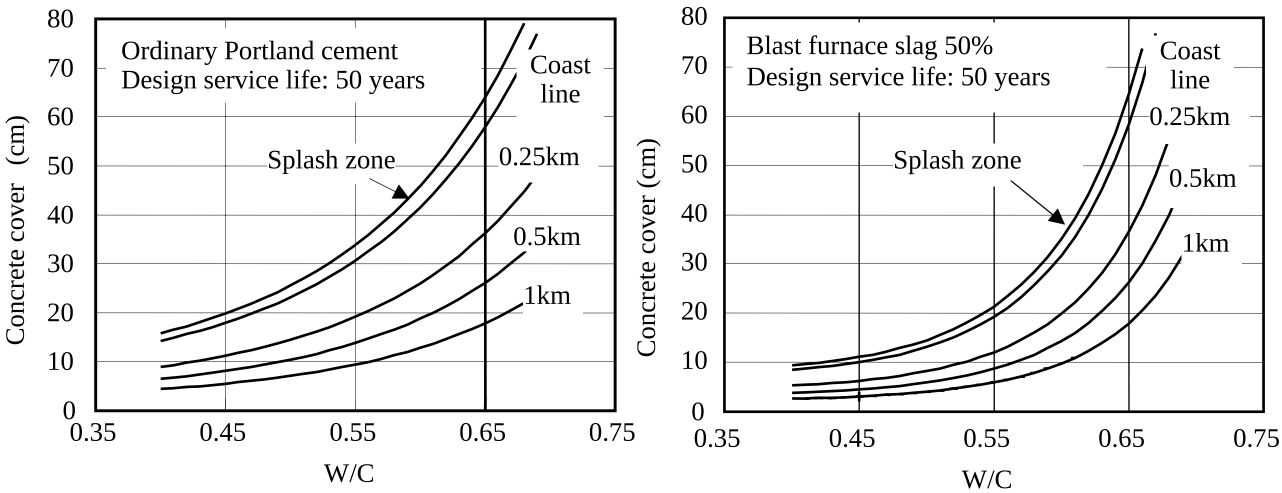


Figure 7 Minimum cover at different water cement ratio required for concrete structure under chloride attack

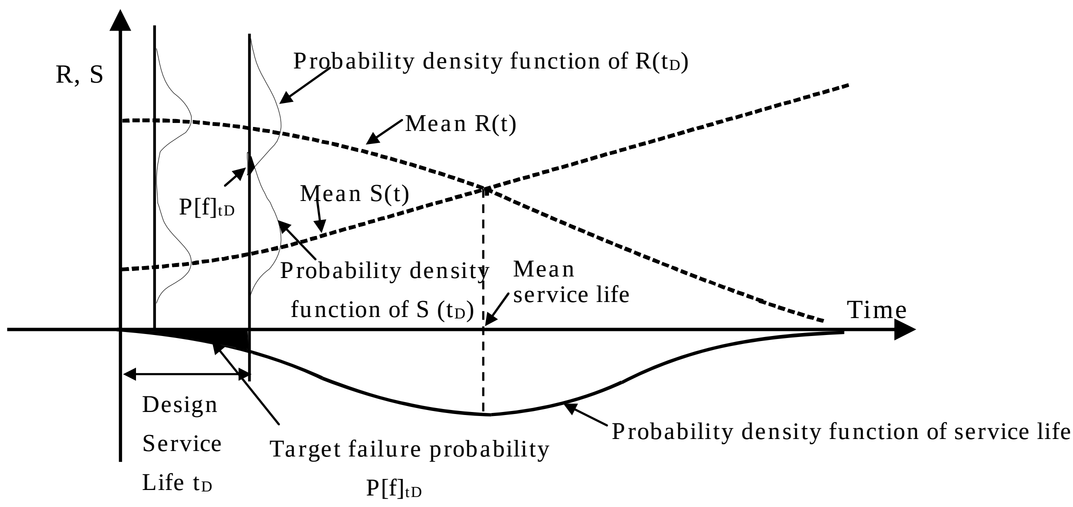


Figure 8 An example of mathematical model for predicting service life of structures