JSCE SPECIFICATION FOR SEISMIC PERFORMANCE VERIFICATION AND DAMAGE OF CONCRETE STRUCTURES DUE TO RECENT EARTHQUAKES

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SUMMARY

A destructive damage which leads to the loss of human life shall be prevented against a strong earthquake ground motion that has a rare probability of occurrence within the lifetime of a structure. In addition, from social and economic points of view, deterioration in functions of the structure should be avoided as much as possible, and livelihood and productive activities of inhabitants after the earthquake should be restored smoothly. The JSCE concrete committee has investigated about the seismic design and has published the Standard Specification and the Recommendations about it. This paper presents the damage of concrete structures due to recent earthquakes and introduces the JSCE Standard Specifications for Seismic Performance Verification published in 2002.

Keywords: *Seismic performance verification; recent earthquakes; JSCE Standard Specification; damage of concrete structures; finite element method; nonlinear analysis .*

INTRODUCTION

It was observed the destructive damages of many concrete structures in the Hanshin-Awaji (Kobe) Earthquake in January, 1995. As the result, the JSCE Standard Specification for 'Seismic Design' was established in 1996, in which the methods for seismic performance verification of concrete structures are described. The specification was revised in 2002 based on the concept of the performance based design and was published as renamed 'Seismic Performance Verification'. In the specification, the nonlinear analysis, in principle, is performed to verify the seismic performance with modeling of the structures and the ground. It can be mentioned that this reflect the knowledge of the seismic performance and the advancement of the analytical technique after Hanshin-Awaji Earthquake, and the social requirement to verify the seismic performance of concrete structures more reasonably for a

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strong earthquake ground motion.

After Hanshin-Awaji Earthquake, the concrete structures are damaged due to several earthquakes such as Geiyo Earthquake on March 24, 2001, South of Sanriku-Oki Earthquake on May 26, 2003, Tokachi-Oki Earthquake on September 26, 2003, and Niigata-ken Chuetsu Earthquake on October 23, 2004 and so on. Therefore, it is important to evaluate these damages and to verify the seismic performance accurately of the concrete structures.

This paper presents the damage of concrete structures due to recent earthquakes, introduces the JSCE Standard Specifications for Seismic Performance Verification published in 2002, and shows a result of damage analysis based on the specification of damaged Shinkansen Elevated Bridges due to South of Sanriku-Oki Earthquake.

DAMAGE OF CONCRETE STRUCTURES DUE TO RECENT EARTHQUAKES

After the Hanshin-Awaji Earthquake on January 17, 1995, which was JMA intensity level 7 (magnitude (Mj) of 7.2), the concrete structures are damaged several earthquakes as listed in Table 1. In this chapter, the damage of concrete structures, especially the damage of railway and highway bridges, due to recent earthquakes in Japan are presented.

Table 1 Information of recent earthquakes

	Geiyo Earthquake	South of Sanriku-Oki Earthquake	Tokachi-Oki Earthquake	Niigata-ken Chuetsu Earthquake
Date	Mar. 24, 2001	May 26, 2003	Sept. 26, 2003	Oct. 23, 2004
Magnitude (Mj)	6.7	7.1	8.0	6.8
Depth of epicenter	50 km	71km	42km	13km
Type	intraslab	intraslab	inter-plate	inland
Maximum intensity	6-	6-	6-	7-
Recorded maximum acceleration	830 cm/s^2	1106 cm/s^2	972 cm/s^2	1722 cm/s^2

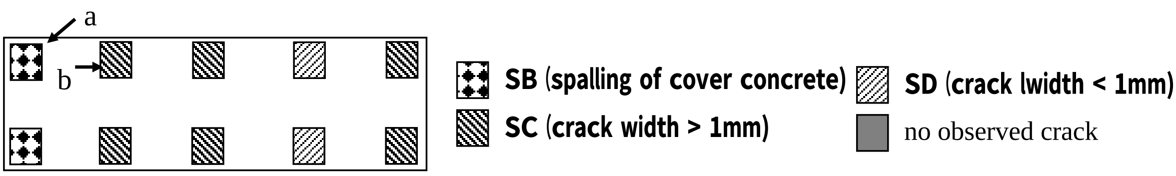
Geiyo Earthquake on March 24, 2001

The Geiyo Earthquake on March 24, 2001, occurred at Aki-nada in the Seto Inland Sea and the magnitude was 6.7. It was intraslab type earthquake caused by subduction of The Philippine plate. The epicenter was relatively deep and it was about 50 km. The recorded maximum acceleration was more than 800cm/s^2. 146 piers in RC elevated bridges of Sanyo Shinkansen were damaged. The shear failure with the spalling of cover concrete was observed by 12 piers among these. Photo.1 shows a damaged two story RC rigid frame elevated bridge. In the middle layer beam, severe diagonal shear crack was observed.

South of Sanriku-Oki Earthquake on May 26, 2003

The South of Sanriku-Oki Earthquake on May 26, 2003, occurred at Off Miyagi Prefecture and the magnitude was 7.1. It was intraslab type earthquake in The Pacific Ocean plate. The epicenter was deep and it was about 71km. The recorded maximum acceleration was more than 1100cm/s^2. The feature of the earthquake ground motion is that short-period wave is dominant. The severe damages in 5 one story RC elevated bridges of Tohoku Shinkansen were observed. The feature of these damages was that the end columns which is shorter than intermediate columns are mainly damaged. The damaged columns were repaired by shrinkage compensating mortar, injection of epoxy resin and steel jacketing.

Photo.1 A damaged two story RC rigid frame elevated bridge of Sanyo Shinkansen



(a) damage of end column (view from a) (b) damage of intermediate column (view from b)

Photo.2 A damaged one story RC rigid frame elevated bridge of Tohoku Shinkansen

Photo.2 shows damaged Dai-san Otagi Bridge R2 of Tohoku Shinkansen which was constructed in 1977 to 1978. This is four bay one story RC frame elevated bridge. The end columns has severe condition for shear failure, since they are shorter than intermediate columns. Two of the end columns failed in shear with the spalling of cover concrete, while others were observed diagonal cracks. The damage due to flexure, however, hardly observed.

Tokachi-Oki Earthquake on September 26, 2003

The Tokachi-Oki Earthquake on September 26, 2003, occurred at Kushiro-Oki (southeast offshore of Hokkaido) and the magnitude was 8.0. Tsunami was also observed. It was typical inter-plate type earthquake. The recorded maximum acceleration was about 1000cm/s^2. Around the focal area, an earthquake of magnitude 8.2 occurred on March 4, 1952, and an earthquake of magnitude 7.9 occurred on May 16, 1968 at south of this area. The feature of the earthquake ground motion was that long-period wave is dominant and the duration time is long. A fire of the oil storage tank caused by the sloshing occurred and the effect of the long-period wave was paid to attention. The concrete structures were mainly damaged in superstructure and substructure of railway and highway bridges.

Toshibetsu-gawa railway bridge of Nemuro Line was constructed in 1966 to 1969. This is 13 span girder PC bridge of 416m bridge length with circular RC piers. This bridge was damaged around the supports at Off Kushiro earthquake in 1993. Photo.3 shows damaged pier in which the spalling of concrete cover and the buckling of longitudinal bars occurred. Photo.4 shows the damage of the floor slab at the end of girder which occurred due to the collision between girders.

Photo.3 Damage of a pier

Photo.4 Damage of floor slab

Photo.5 shows the damaged Chiyoda highway bridge which consists of middle bridge of 5 span warren truss constructed in 1954 and side bridge of 5 span PC girder constructed in 1966. The flexural failure in the piers of PC girder bridge as shown in Photo.5 and the punching shear failure at support of warren truss bridge due to horizontal force from anchor as shown in Photo.6 occurred. The feature of the damage of the piers is the flexural cracks, the spalling of concrete cover and the buckling of longitudinal bars. Similar failure at support was already observed in Hanshin-Awaji Earthquake.

Photo.5 Damage of a pier

Photo.6 Damage at a support

Niigata-ken Chuetsu Earthquake on October 23, 2004

The Niigata-ken Chuetsu Earthquake on October 23, 2004, occurred at Mid Niigata Prefecture and the magnitude was 6.8. It was caused by inland active fault. The depth of the epicenter was about 13 km. The recorded maximum acceleration was more than 1500cm/s^2. The recorded maximum acceleration for this earthquake is much greater than that for the 1995 Kobe earthquake. The feature of the earthquake was that many aftershocks including some magnitude 6 class events have been following. Aftershocks were distributed along the northeast and southwest direction with a length of about 30km. Several concrete structures of railway and highway bridges were damaged.

Photo.7 shows the damage of Uono-gawa Bridge of Joetsu Shinkansen, which is three span box girder PC bridge. In 2P and 3P with circular RC piers with diameter of 6.5 m, the spalling of concrete cover and buckling of longitudinal bars occurred at the mid height. Lateral ties at that location were found to detach. Failure took place at the location of termination point of the longitudinal bars. Three steel arcs with central angle little more than 120 degree were used to make circular ties. No hooks were provided in the lateral ties.

(a) Panorama of Uono-gawa bridge

(b) Failure at mid height in pier

Photo.7 Damaged Uono-gawa Bridge of Joetsu Shinkansen

Photo.8 Damage of end column
(both sides of a column)

Photo.9 Strengthened column
by the steel jacketing

Photo.8 shows the damage of Dai-san Wanazu Bridge R1 of Joetsu Shinkansen, which is three bay one story RC frame elevated bridge. One of the end columns failed in shear as shown in Photo.8, while another one showed diagonal cracks. The end columns has severe condition for shear failure, since they are shorter than intermediate columns. This failure is the same as the one observed in the South of Sanriku-Oki Earthquake. These were designed by the same specifications for Shinkansen published in 1970 and 1972. It is noted that some columns of elevated bridge of Joetsu Shinkansen were strengthened by the steel jacketing before earthquake as shown in Photo.9. As for them, no damage was observed and the effect of strengthening was confirmed

JSCE STANDARD SPECIFICATION FOR CONCRETE STRUCTURES-2002 "SEISMIC PERFORMANCE VERIFICATION"

Outline of Seismic Performance Verification

The provisions concerning the seismic performance of concrete structures had been specified as one chapter of the JSCE Standard Specification for Design until 1991 version. A similar form was planned to be adopted at the revision in 1996. However, it was necessary to present the specification based on a new design concept, since concrete structures were damaged destructively in the Hanshin-Awaji Earthquake, 1995. Therefore, the specification for 'Seismic Design' was established in 1996, in which the methods for seismic performance verification of concrete structures was described basically.

In 'Seismic Design(1996)', the modeling and the analytical method of the structures were not described enough. In 'Seismic Performance Verification(2002)', these items were enhanced based on the knowledge of seismic performance and the advancement of the analytical technique afterwards. Especially, the items of (1)Loads (earthquake ground motion in verification), (2) Evaluation for the effect of ground, (3) Verification technique(analytical method) and (4) Structural details were enhanced. Moreover, it was systematized that the more reasonable seismic performance verification becomes possible. In order to indicate their contents clearly, the specification 'Seismic Design' was renamed as 'Seismic Performance Verification'. Fig.1 shows the general procedure to verify the seismic performance based on 'Seismic Performance Verification(2002)'.

Seismic Performance and Limit Values

The required seismic performance for concrete structures was defined in 'Seismic Design(1996)' as described in Table 2 and same ones are also defined. When setting seismic performance of a structure, it should be better to take its response characteristics into account according to the magnitude of an expected earthquake, as well as its behaviors during the earthquake and restoration ability after the earthquake. Needless to say that a destructive damage which leads to the loss of human life shall be prevented against a strong earthquake ground motion that has a rare probability of occurrence within the lifetime of a structure. In addition, from social and economic points of view, deterioration in functions of the structure should be avoided as much as possible, and livelihood and productive activities of inhabitants after the earthquake should be restored smoothly. Therefore, the seismic performances are related to the necessity of repair and/or strengthening and to the serviceability after an earthquake.

Seismic Performance 1 means that the residual deformation of a structure after an earthquake remains sufficiently small.

Seismic Performance 2 is that load carrying capacity does not deteriorate after an earthquake.

In general, this performance may be thought to be satisfied in case that each constitutive member of a structure does not fail in shear during an earthquake and response deformation does not reach its ultimate one. In case of structures such as wall type ones in which earthquake force acts mainly as in-plane force, large deformation capacity cannot be expected. In the design of such structures, it is necessary to provide sufficient shear capacity so that brittle failure may not occur.

Seismic Performance 3 requires that whole structural systems do not collapse due to self and imposed masses, earth pressure, hydraulic (liquid) pressure, and so on, even if the structure becomes un-restorable after an earthquake. In case of concrete structures, generally Seismic Performance 3 can be satisfied if each constitutive member has enough safety against shear failure. In some structural systems, however, displacement of the whole structure becomes excessive, and the additional bending moment as well as longitudinal displacement increase due to the self weight. These may lead to self-overturning or collapse mechanism.

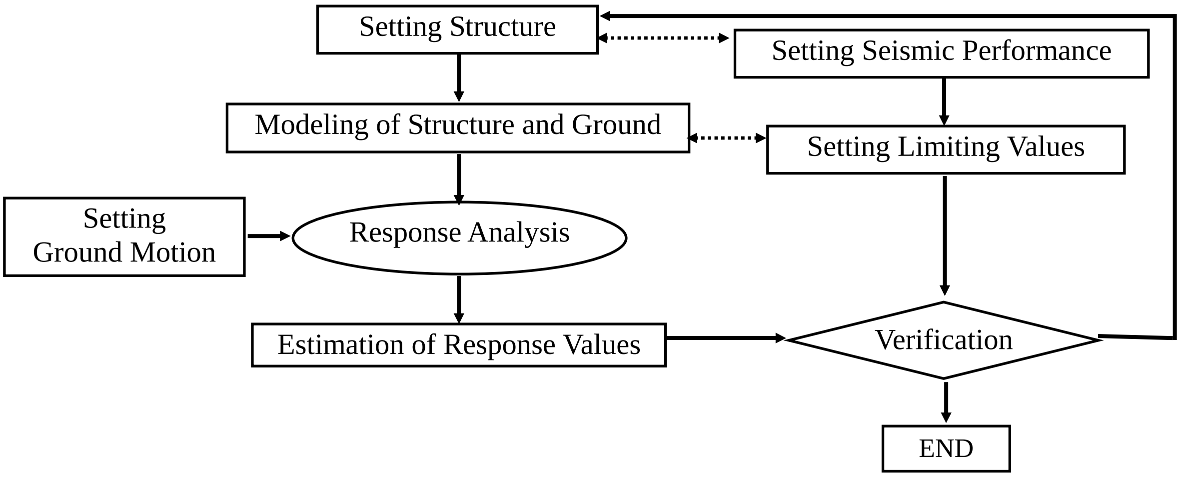


Fig.1 Procedure to verify seismic performance

Table 2 Definition of seismic performance

Seismic Performance 1	Function of the structure during an earthquake is maintained, and the structure is functional and usable without any repair after the earthquake.
Seismic Performance 2	Function of the structure can be restored within a short period after an earthquake and no strengthening is required.
Seismic Performance 3	There is no overall collapse of the structural system due to an earthquake even though the structure does not remain functional at the end of the earthquake.

When the seismic performances of structures are verified, limit values of response should be determined, which assure the defined seismic performance. Although the seismic performances of structures depend on the performances of members, it is not necessary that the seismic performances defined for structures should be directly applied to members. For example, in the case of rigid frame viaducts with two layers, when only middle layer beams failed in shear, the viaducts structure do not collapse. In general, the relationships between the seismic performances of structures and the damage of constitutive members are not always clear in many cases. Therefore, the limit values for members should be determined from the viewpoint of the safety of structures as shown in Table 3.

During an earthquake, in general, Seismic Performance 1 is satisfied unless re-bars yield.

Therefore, yield displacement of a member was decided as the limit value of Seismic Performance 1 on the assumption that members do not fail in shear.

Table 3 Limit values for members

Seismic Performance 1	displacement of a member does not exceed the yield displacement
Seismic Performance 2	shear and torsional capacity of a member, and ultimate displacement of a member are not reached
Seismic Performance 3	shear capacity of vertical members and self-weight support capacity is not exceeded

During an earthquake, unless members fail in shear or torsion, and unless the displacement reaches the ultimate displacement, Seismic Performance 2 is generally satisfied. However, in some structures, there may happen troubles in reusing them, when the residual displacement becomes remarkably large. In those cases, the residual displacement may be defined as the limit value which is determined considering time and cost for restoration and expected residual functions.

When vertical members are sufficiently safe against shear failure, Seismic Performance 3 of concrete structures is generally satisfied. In some cases, however, displacement of the whole structure becomes excessive, and additional bending moment as well as longitudinal displacement of member increase due to self-weight. These sometime may lead to self-collapse. Therefore, it should be necessary to confirm using the response analysis of structures for the earthquake ground motion that the structure does not reach to the condition of self-collapse due to self-weight.

Loads (earthquake ground motion in verification)

The earthquake ground motion in design had not been investigated concretely and the response spectrum was only described before Hanshin-Awaji Earthquake. Moreover, the design seismic coefficient was assumed as 0.2 and it was considerably small compared with the earthquake ground motion at the location that structures damaged in the Hanshin-Awaji Earthquake.

In ‘Seismic Performance Verification(2002)’, it is required that two kinds of design earthquake ground motions – Level 1 and Level 2 as given in Table 4 should be considered, which are determined considering the magnitude of the earthquake, source characteristics, structure of local stratum between the epicenter and construction site, propagation characteristics and distance between the epicenter and the construction site, and topographical, geological and ground conditions of the construction site.

Table 4 Design earthquake ground motion

Level 1 Design Earthquake Ground Motion	earthquake ground motion that is likely to occur a few times within the lifetime of a structure.
Level 2 Design Earthquake Ground Motion	very strong earthquake ground motion that has only a rare probability of occurrence within the lifetime of a structure.

Level 1 earthquake ground motion is the ground motion whose return period is about 50 years. Level 2 ground motion is a very strong motion whose return period is about 1000 years, which is chosen from the ground motion caused by an inland active fault beneath or close to the site and by large scale inter-plate earthquake occurring in the neighborhood of land, taking the one that has the larger effect.

The earthquake ground motion used for seismic performance verification should, in principle, be expressed as the time history waveform of acceleration and should be considered as acting on the engineering base layer at the construction site. It is reason that for precise evaluation of the safety of a concrete structure during an earthquake, it is necessary to perform a nonlinear dynamic response analysis using a time history waveform of an earthquake ground motion. Prediction method of the earthquake ground motions is employed in the specification. Moreover, the examples of simulated earthquake ground motion waveforms are presented for Level 1 earthquake ground motion, Level 2 earthquake ground motion for an inland type and for an inter-plate type (the off-shore type) with digital data of CD-ROM. Level 2 earthquake ground motion for an inland type and for an inter-plate type (the off-shore type) are shown in Fig.2 and Fig.3, respectively.

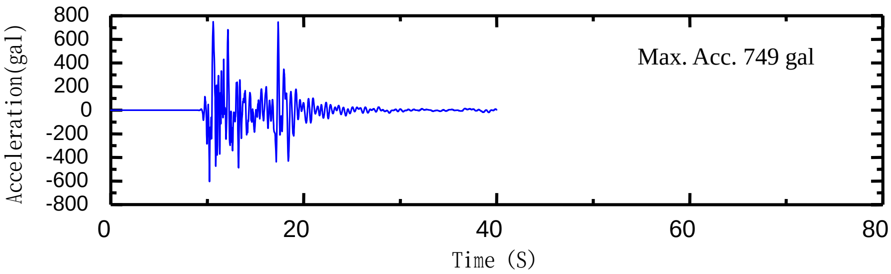
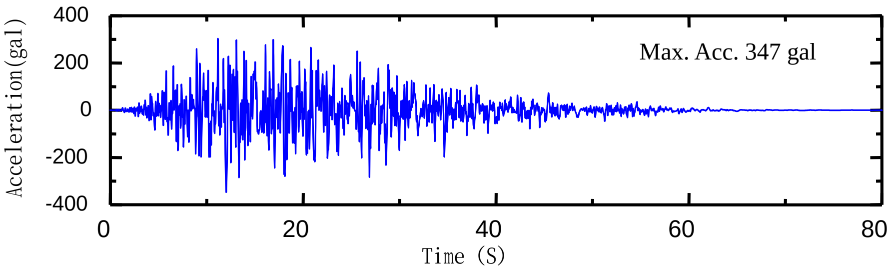


Fig.2 Examples of time history acceleration waveform of Level 2 earthquake ground motion (an inland type)

The earthquake ground motion waveform of inland type is determined according to the following process based on the past inland type earthquake records.

(1)Acceleration response spectra of many observed records are transformed to those on the fault using attenuation model. Then, they are modified such that the non-exceedance probability is 90% in the samples.

(2)The phase characteristics to generate a time history acceleration waveform according with the acceleration response spectrum is determined such that asperity and rupture starting point are changed parametrically in the fault plane of 40×20 km and failure process of the fault is considered. Then, the waveform is composed.



(b) Off-shore Type ②

Fig.3 Examples of time history acceleration waveform of Level 2 earthquake ground motion (a off-shore type)

Evaluation for the Effect of Ground

Since the response of a structure during an earthquake is strongly affected by neighboring ground and others, the whole structural system including foundation or neighboring ground should be analyzed. To do so, a coupled analysis taking the interaction between structures and soil into account directly should be used. Table 5 shows a general analytical method. Input place of the earthquake ground motion is at the engineering base layer. In the modeling of ground, the influence of distant ground and others should be sufficiently examined. An example of finite element model of a structure is shown in Fig.4.

Table 5 Method for a coupled analysis of structure and ground

Structure Type	Ground structure, Underground Structure
Analytical Method	Time History Response Analysis
Analytical Model of Structures	Finite Element Model or Beam Element Model
Analytical Model of Ground	Finite Element Model
Input Value	Time History Acceleration Wave Form
Input Place	Base Layer

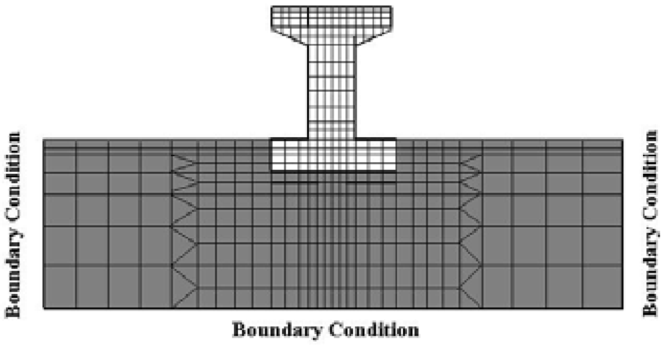


Fig.4 Example of modeling for structure and ground in a coupled analysis

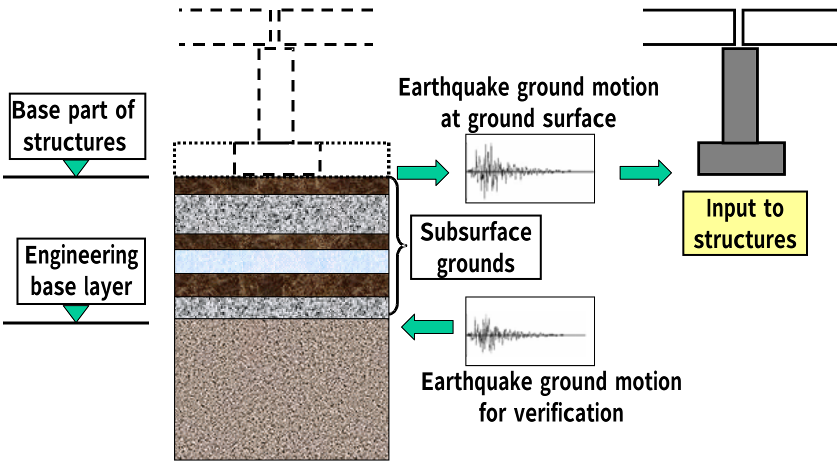


Fig.5 Methods to analyze the structure and the ground independently

According to types or characteristics of structures and ground, there are some cases which dynamic interaction between structures and ground can be neglected or modeled appropriately. In these cases, the responses of the structures and the ground may be analyzed independently, because the response can be estimated with sufficient reliability without using coupled analysis. Fig.5 shows the methods for analyzing the structure and the ground independently.

Verification technique(analytical method)

The performance verification techniques of a concrete structure that used a nonlinear analysis have improved with the advancement of analytical environment after Hanshin-Awaji Earthquake. It became possible to analyze the structure and the ground together. Analytical method is applied by finite element method based on the background of advancement of such a technology. The Modeling of structures using finite elements and modeling of ground are described concretely.

Structures should be modeled as an assembly of member models in which columns and beams are modeled as linear members and members with planar spread such as wall or floor are modeled as planer members. Basically, the linear member is modeled using beam element and planar member is modeled using plate or layered shell element. Strictly speaking, structures should be modeled three dimensionally, because it is composed of members connected in three dimensions. Two dimensional modeling of the structure may, however, be applied if only the response behavior in a two dimensional is considered according to the direction of input earthquake ground motion and characteristics of structural response. It is possible to model a linear member using plate or layered shell element. In general, however, modeling by using beam element is effective, since the computation time becomes shorter and accuracy does not decrease much. Shell structures such as tank structures should be modeled as an assembly of shell elements for the whole structure, since the structure consists of one member. The pull-out of re-bar at joint of members is usually negligible. It is, however, desirable to use a joint element between members, when it is necessary to consider the effect of the pull-out of re-bar if the diameter of the re-bar is relatively large compared with the member size.

In finite element method, mechanical model of materials are applied. Therefore, the constitutive model of concrete, reinforcing bar, and soil are described with those hysteresis and with notes to use. The hysteresis of concrete in compression and ground are shown in Fig.6 and Fig.7.

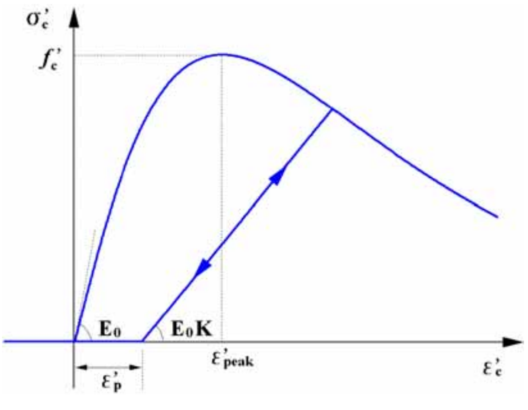


Fig.6 A simplified hysteresis model of concrete

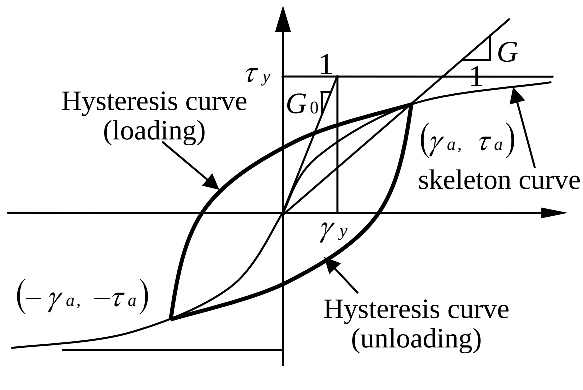


Fig.7 Example of the dynamic shear stress-strain curve of the soil

Structural Details

It was observed many damages that are related to structural details of the concrete structures in Hanshin-Awaji Earthquake. Most were the damage occurred from the point of re-bar cutoff and the damage that splices performance of the longitudinal and the lateral reinforcing bar is insufficient after the spalling of cover concrete and so on. Therefore, structural details were greatly revised from 'Seismic Design(1996)'.

For the development of longitudinal re-bar, the tensile re-bar shall be anchored into concrete sections not subjected to tensile stresses. It may, however, be anchored into concrete sections subject to tensile stresses, when the some conditions are satisfied. Then, it is noted that the maximum cross sectional force induced during an earthquake depends on actual strengths of materials and actual area of re-bars provided in a section. This point should be carefully taken into account in the assumptions used in computation of design flexural moment and design shear force. For example, it is assumed that column members in bridge piers or column and beam members in frame structures reach plastic region, when structures are designed to satisfy the Seismic Performance 2 for level 2 earthquake ground motion. Maximum flexural moment caused in columns or beams depend on actual arrangements and actual strengths of longitudinal re-bars. Furthermore, flexural moment increases with increase in stress of the re-bars if the re-bar strain reach strain hardening zone. Then, flexural moment and shear force should be computed using the characteristic strength of materials and the member factor considered about it. They are taken into consideration since shear force in the response analysis relates to the flexural moment.

For the splices of longitudinal re-bar, the longitudinal re-bar shall be spliced in a manner that the splices perform satisfactorily even under repeated stress in the zone, where plastic hinges may be formed. Lap splices shall not be provided in plastic hinge zones subjected to repeated stress. In principle, splices should be provided in a manner that the number of splices at a particular section is less than one for every two re-bars, and the splices are not concentrated at any particular section.

For spacing of lateral re-bar, the lateral re-bars, such as ties or spiral re-bars, restrain the progress of diagonal cracks, increase shear capacity, prevent buckling of longitudinal re-bars, and also provide confinement of core concrete. Therefore, from the view points of shear reinforcing and to ensure the required ductility, it is necessary to provide sufficient amount of lateral re-bar. Fig.8 and Fig.9 show the example of spacing of ties.

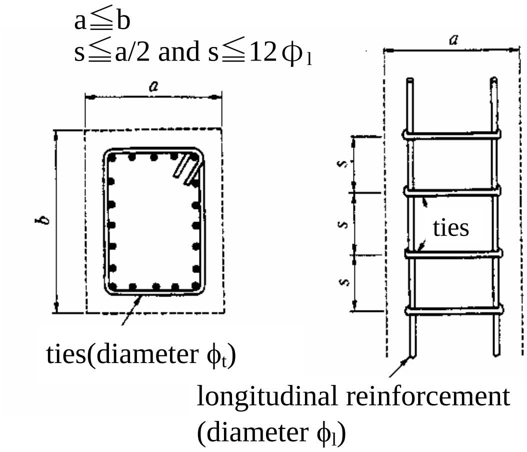


Fig.8 Spacing of ties enclosing all longitudinal re-bar

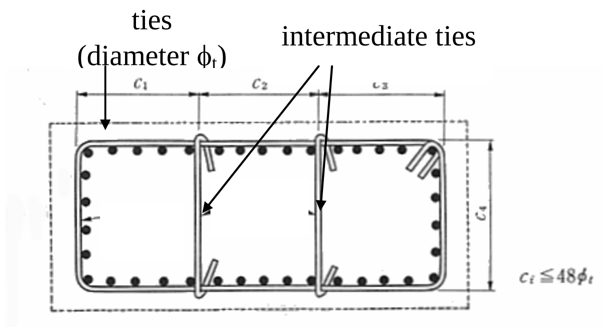


Fig.9 Spacing of ties in the cross section

For splices of ties, the ties, which enclose all longitudinal re-bars in the area where the spalling of cover concrete occurs with large deformation, should transmit full strength, even if the spalling occurs. The required condition should also be satisfied when the splices are provided for ties. Considering this requirement, flare welding or mechanical coupler are recommended. The reliability of spliced portion is less than the bare bar, and tends to be a weak point from construction viewpoint. They should, therefore, be staggered such that the spliced portion are not concentrated in member direction. Intermediate ties arranged in core concrete are not affected by the spalling of cover concrete, hence, lap splices with standard hooks may be used as shown in Fig.10.

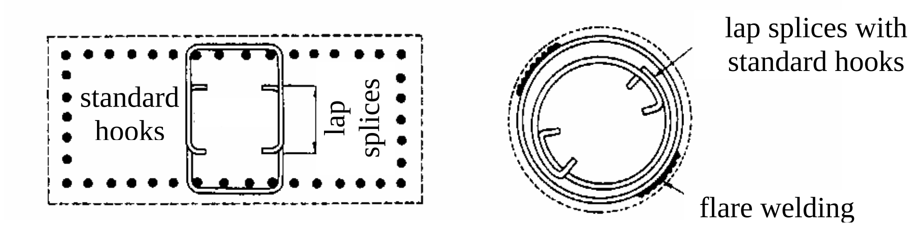


Fig. 10 Lap splices of ties

In General, the items specified as structural details are the preconditions of verification, and they shall be obedient without any examination. However, if the specified items are able to be verified due to the advance of technology, it is natural that some treatments should be done based on the results of the verification. In the specification, the method to be confirmed by experiments is adopted as the verification method, and if it is confirmed that the new structural details can assure higher performances than those of the structural details mentioned in this chapter, the new details may be used. The experiments to confirm the performances, however, should satisfy the following.
(1) The shape and the size of members, the size and the spacing of re-bar, the cover and so on should be taken as the same as those of actual members.
(2) Displacement controlled reverse cyclic loading should be carried out using the loading methods which can apply almost the same force to member sections as is likely to occur in actual members.

DAMAGE ANALYSIS OF CONCRETE STRUCTURES DUE TO THE EARTHQUAKE

In the specification, nonlinear analysis, in principle, is performed to verify the seismic performance with modeling of structures and ground as described above. Therefore, it is useful to try to find the reason of damage by the analysis for the damaged concrete structures due to recent earthquake. It can give the verification of the nonlinear analysis and give the information to enhance seismic design. Therefore, a subcommittee performed the damage analysis of concrete structures due to the earthquakes in 2003 and published Concrete Library 114(2004). In this chapter, the results of damage analysis based on the JSCE specification of a damaged Shinkansen Elevated Bridges due to South of Sanriku-Oki earthquake, which are described in Concrete Library 114, are shown.

Outline of Damage Analysis

The end frame of the damaged Dai-san Otagi Bridge R2 Shinkansen Elevated Bridges are analyzed by finite element in two dimension. The column in the frame failed in shear with the spalling of cover concrete as shown in Photo.2. Fig.11 shows the details of the end frame. The structure is modeled with the foundation and the neighboring ground according to the JSCE specification as explained above. Fig.12 shows the finite element model analyzed. The stress-strain relationships of concrete and reinforcement are used by those described in the specification. Osaki model (Osaki, 1980) is used as the shear stress-strain curve of the soil.

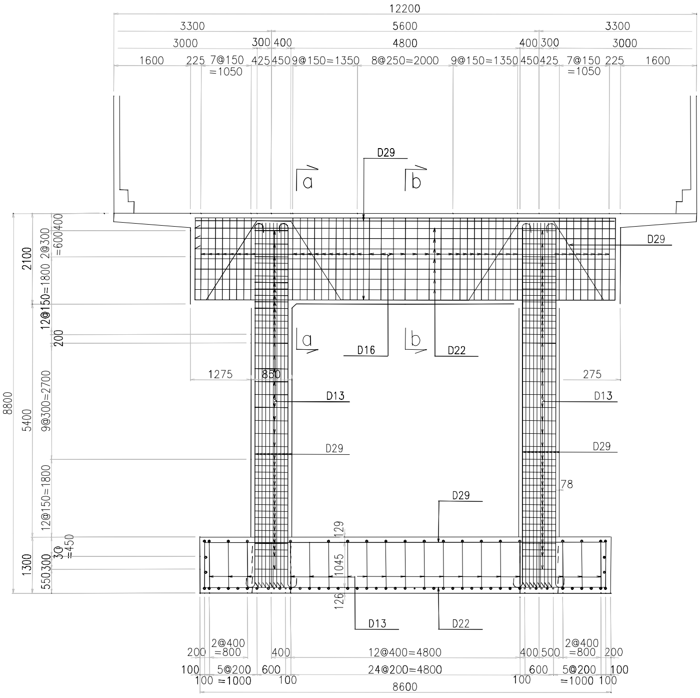


Fig.11 Details of the end frame of Dai-san Otagi Bridge

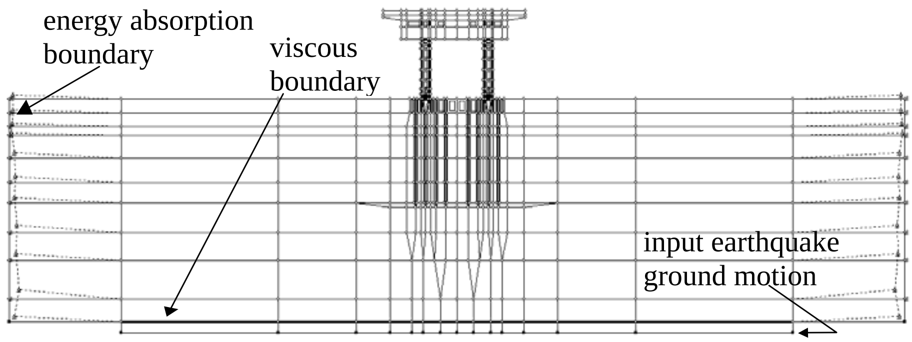


Fig.12 Analytical model

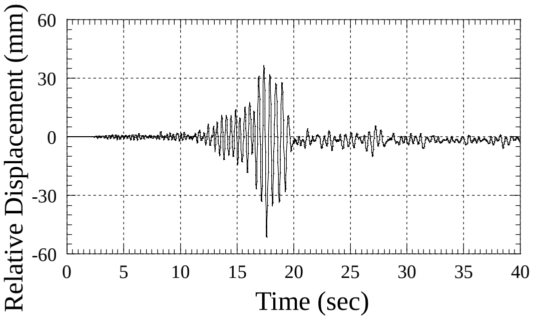


Fig.13 Time history response displacement

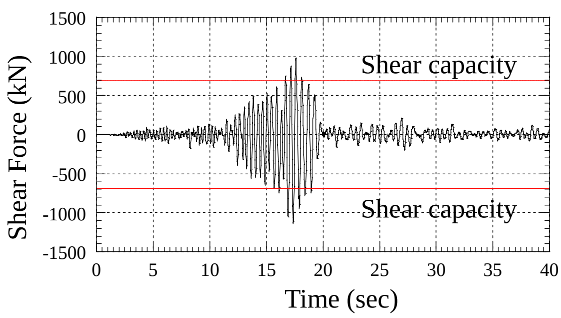


Fig.14 Time history response shear force

Input earthquake ground motion is obtained that recorded earthquake ground motion is translated to the one at engineering base layer by using FDEL(Sugito et al., 1994) considering the ground condition.

Fig.13 shows the time history response displacement at the top of frame and Fig.14 shows the time history response shear force at the column with shear capacity calculated by the JSCE equation in specification for Structural Performance Verification(2002). The response shear force at the column shows larger value than the shear capacity. This is the reason that the column failed in shear. Fig.15 shows residual crack pattern obtained from analysis after calculation. The excessive diagonal crack remain and it is the similar result with the one observed in the damaged structure.

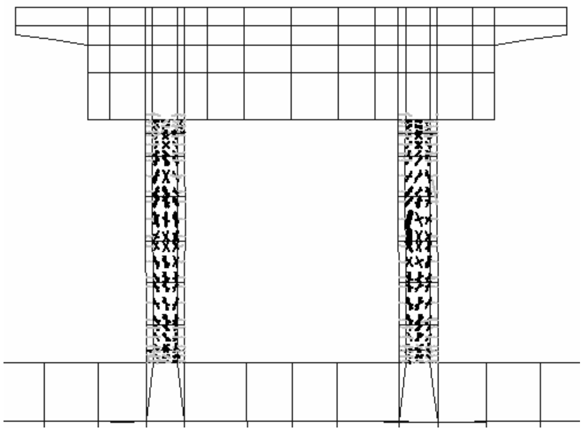


Fig.15 Residual crack pattern after calculation

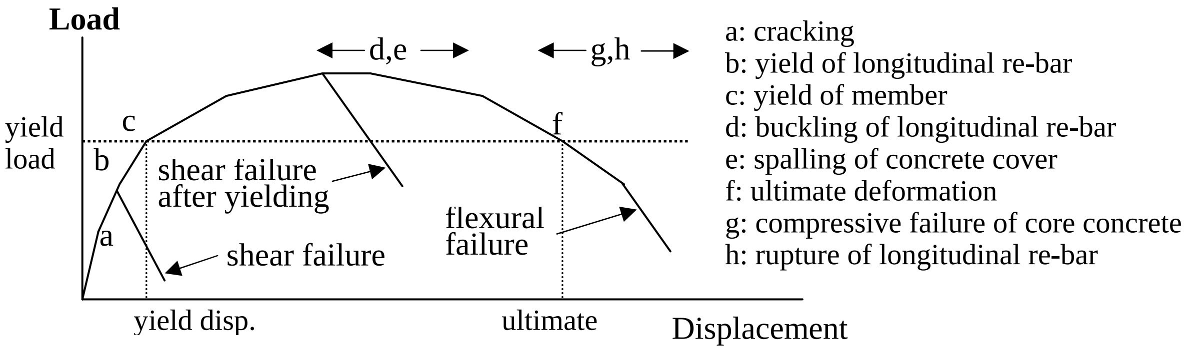


Fig.16 An example of skeleton curve of member

CONCLUDING REMARKS

The seismic design in Japan changed greatly after Hanshin-Awaji Earthquake. The damages of most concrete structures in the Earthquake are caused by the shear failure. As the result, the structural details were greatly revised and large plastic deformation has been required in flexural failure mode. Moreover, the performance based design has been adopted, and the nonlinear analysis, in principle, is performed to verify the seismic performance. Fig.16 shows the typical damage event on skeleton curve of a concrete member. Although these are the limit values for the required performance, several events remain to be evaluate. Especially, the buckling and the rupture of longitudinal reinforcement should be evaluated accurately in flexure failure, which sometimes have the possibility of becoming the trigger of ductile behavior in ultimate state.

The damage can not be prevented for a strong earthquake. Then, functions of the structure should be restored as soon as possible smoothly from social and economic points of view. To do so, restoration ability after an earthquake is important. That is, it is important to make clear the relationships between damage, restoration ability and performance after an earthquake as well as seismic performance during an earthquake.

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