Outline of Japanese Design Specifications for Highway Bridges in 2012

by

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ABSTRACT

This paper presents the outline of revisions of the Japanese design specifications for highway bridges issued by Ministry of Land, Infrastructure, Transportation and Tourism (MLIT) in February 2012, and the commentary for the specifications was published by Japan Road Association (called "JRA" in the following text) in March 2012 [1]. The revised specifications incorporated the latest research achievements, many lessons learned from the recent earthquakes including the 2011 Great East Japan Earthquake and the durability related damages of existing bridges. Based on these lessons, design earthquake ground motions corresponding to the subduction-type earthquake were revised, and the requirements for easy and secure inspection and repair works for the bridges were clearly specified.

KEYWORDS: Japanese Design Specifications for Highway Bridges, Maintenance, Seismic Design

1. INTRODUCTION

Japanese Design Specifications for Highway Bridges (JRA specifications) are applied for Japanese road bridges and consist of five parts: Part I Common, Part II Steel Bridges, Part III Concrete Bridges, Part IV Substructures, and Part V Seismic Design. These specifications have been revised several times on technical progress and changes of social needs. In recent years, the 1996 specifications were revised to enhance seismic design mainly triggered by severe damages suffered from the 1995 Hyogo-ken Nanbu, Japan, earthquake. In the 2002 specifications, the performance-based design concept was introduced, and design requirements were clearly specified and the conventional detailed design methods including analytical methods and the allowable limits were used as verification methods and the examples of acceptable solutions. Additionally, the design considerations for durability were enhanced so as to design the sustainable structures [2].

The 2012 revised specifications were issued by (MLIT) in February 16, 2012, and the specifications and the commentary for the specifications was published by JRA in March. These revised specifications are improved based on the technical research achievements in terms of safety, serviceability and durability of bridges. These examples include introducing of integral abutment bridges and the use of higher strength rebar in comparison with conventional one. Moreover, many lessons learned from the recent earthquakes such as the 2011 Great East Japan Earthquake and from the damages of existing bridges due to aged deterioration have also been accumulated. Based on these lessons, design earthquake ground motions corresponding to the subduction-type earthquake were revised, and the requirements for easy and secure maintenance (inspection and repair works) for the bridges were clearly specified. This paper summarizes the main points of revisions in the 2012 specifications.

2. FUNDAMENTAL PREICIPAL OF MAINTENANCE

There are about 650 thousand road bridges which

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bridge length are 2m or more in Japan and aging rapidly. Focusing on the road bridges which bridge length are 15m or more (approx. 160, thousand bridges), there are approximately 30% of the bridges more than 40 years after construction, and approximately 50% of the bridges more than 30 years after construction as shown in Figure 1. As aged bridges increase, the bridges with damage due to deterioration such as fatigue, salt damage, alkali-silica reaction (ASR) have been increasing. However, budgets for maintenance, which is needed to keep road bridges healthy for a long time, keep decreasing so that it is important to reduce the maintenance and operation costs through countermeasures such as preventive maintenance of highway bridges.

The performance of the road bridges has changed by applying for various factors such as live load, seismic, and environmental effects in their service period. Therefore, it is important to perceive change of bridge condition by inspection and to be repaired or retrofitted as it needed timely and surely. However, it is not easy to do these things because most of existing bridges are difficult to inspect due to design concept without a view to inspecting, lack of inspection equipments such as inspection ladders, walkways, workspace, and so on. Consequently, a lot of existing bridges where appropriate measures have not been made remain even if damage of the bridge such as the corrosion of end of girder and bearing become significant.

On the basis of these lessons, it is clearly required as a fundamental principal of bridge design that structural systems of which maintenance is expected to be difficult and insecure should be avoided. It is also required that the bridges should be designed in consideration of maintenance methods such as periodic or emergency inspection, and repair, retrofitted works. The maintenance equipments such as inspection ladders, walkways, as shown in Figure 2, shall be provided to access easily and securely as they need. Visual inspections near the important structural parts are effective to be judged the bridge safety and quickly not only at normal inspection to use for a long time but also at emergency event such as the extreme earthquake. These equipments are helpful for the visual inspection more easily and securely. Strengthening of main girder in advance for temporary jack up is effective to replace the bearing in the future. Consideration of temporary support for replacement of bearings in the structural design is also effective.

There are a lot of existing bridges with unknown structural details such as foundation type, bar arrangement, especially in old bridges. In these cases, it is very difficult not only to examine the performance of the bridge appropriately but also to examine effective measures to repair or retrofit. Therefore, it is clearly required that various records on bridges about the investigation, design, construction, quality control should be preserved accurately and succeeded following stages to utilize not only for construction but also for maintenance. These kinds of information are indispensable to examine performance evaluation, repair or reinforcement method of the bridges in-service period.

3. FUNDAMENTAL PRINCIPAL OF DESIGN

In recent years, severe damages such as fracture of diagonal bridge bracing in steel truss bridges as shown in Figure 3, severe fatigue cracking of steel main girders, and fracture caused by corrosion of prestressed concrete bridge tension members were occurred in Japan. Fortunately, no bridge collapse has occurred while the I- 35W bridge fell in Minnesota, U.S. in 2007. The collapse of the I-35 bridge implies that fracture of specific member might cause the catastrophic damage of the bridge. Therefore, it is enhanced that the bridge should be designed not to collapse of whole bridge caused by damage of such critical members. For example, from a point of view of redundancy, in design of abutment foundation where located on the slope, it is recommended that the number of piles is arranged in more than four piles and more than two rows. This is because the abutment supported by multi rows of piles is more stable even if slope might be collapse due to landslide.

4. SEISMIC DESIGN

4.1 Revision of Design Earthquake Ground Motion Corresponding to Subduction-Type Earthquake

The Japanese design specifications for highway bridges consider two levels of earthquake ground motion (Level 1 and Level 2) and two types in Level 2 earthquake motion (Type I and Type II). Level 1 earthquake motion represents ground motion highly probable to occur during service period of bridges and its target seismic performance is set to have no structural damage. Level 2 earthquake motion is defined as ground motion with high intensity with less probability to occur during the service period of bridges. The target seismic performances against Level 2 earthquake motion is set to limited damage for function recovery in short period for high importance bridges and to prevent fatal damage for bridges such as unseating of a superstructure or collapse of a bridge column for standard importance bridges. Type I of Level 2 earthquake motion represents ground motion from large-scale subduction-type earthquakes, while Type II from near-field shallow earthquakes that directly strike the bridges.

In the revision in 2012, the Type I of Level 2 earthquake motion was revised considering earthquake motions from the 2011 Great East Japan Earthquake as well as the anticipated great earthquake along the Nankai Trough, of which the occurrence impends [3].

Design earthquake motions for highway bridges are set by multiplying zone factor, which will be described later, to the standard acceleration response spectra. A damping factor of 5% is considered. The standard acceleration response spectra are set for each soil profile type as shown in Figure 4. The soil profile type I, II, and III correspond to stiff, medium, and soft soil conditions, respectively. Type I earthquake motion is based on the ground motion in Tokyo area during the 1923 Kanto Earthquake (M_w =7.9). They had been introduced into seismic design of highway bridges in 1990, prior to Type II in 1996, and were revised for the first time in 2012 using recently developed attenuation relationships, and the strong motion records during the 2011 off Tohoku, Japan, earthquake (great east Japan earthquake, M_w =9.0) as well as the 2003 off Tokachi, Hokkaido, Japan, earthquake (M_w=8.0). Response spectra specified in the previous specifications are larger in soft soil (Soil profile type III) and smaller in stiff soil (Soil profile type I) because damage of structures by large earthquakes prior to the Kobe earthquake tends to be more significant in soft soil condition, while the relationship is reversed because earthquake motions recorded during recent large earthquakes show the intensive ground shaking tends to be more amplified in stiff soil condition than in soft soil condition.

Zone factors for Type I earthquake motion are also revised along with the standard acceleration response spectra. There had been three zones, A, B, and C, with zone factors 1.0, 0.85, and 0.7, respectively, and they had been employed for both Level 1 and 2 earthquake motions. As shown in Figure 5, zone A was divided into two zones, A1 and A2, as well as zone B into B1 and B2, while zone C was not changed in this revision. Zone factor for Type I earthquake motion, c_{Iz}, was set to be 1.2 for zones A1 and B1, 1.0 for A2 and B2, and 0.8 for C.

Figure 6 presents source regions of major plate boundary earthquakes that are taken into account in the revision. The moment magnitude M_w of off the Pacific coast of Hokkaido and Tokai-Tonankai-Nankai-Hyuganada earthquakes are assumed to be 9.0 besides off the Tohoku earthquake. Zones A1 and B1 were set based on the area where ground motion intensity is estimated larger than that in Tokyo area during the 1923 Kanto Earthquake.

Figure 7 compares acceleration time history, which is used for dynamic response analysis for seismic design, of Type I earthquake ground motion before and after the revision. Very long duration is considered based on the record obtained from the 2011 Great East Japan Earthquake.

4.2 Design Considerations of Effect of Tsunami, Large-scale Landslide, etc. on Structural Planning of Bridges

In recent earthquakes occurred in Japan, extreme events associated with a large earthquake, but not strong earthquake shaking, have caused collapse of bridges as shown in Figure 8. A bridge was collapsed by large-scale landslide around its abutment during the 2008 Iwate-Miyagi inland, Japan, earthquake [4], and many bridges were washed away by extreme tsunami during the 2011 Great East Japan Earthquake [5]. Although the large fault movement did not cause fatal damage to bridges in Japan recently, that caused fatal damage to bridges in the 1999 Chi-Chi, Taiwan, earthquake and the 1999 Kocaeli, Turkey, earthquake.

Although the extreme events listed above have critical effect on the performance of bridges, these events are not directly considered, but the effect of a strong earthquake motion is only considered in the seismic design of bridges according to the Japanese design specifications for highway bridges. This is because design philosophy for these events, which means the scale of external force considered, and the required performance, etc., has not yet been determined. Therefore, only design considerations to mitigate the effects of these events have been introduced in this revision.

Against tsunami, in particular, it is specified in the specifications that the local plan for disaster prevention shall be considered in planning of road, and in structural planning and structural design of bridges. For prevention of collapse of important bridges due to extreme tsunami, it is recommended that sufficient clearance for wave height of tsunami is ensured for bridge superstructures. For mitigation of the effect of tsunami, considerations in structural design to mitigate the tsunami force to bridge superstructure, and preparation of a recovery plan, which is also effective to mitigate the effect of tsunami, are recommended.

4.3 Revision of Ductility Design Method of Reinforced Concrete Bridge Columns

To improve the accuracy of evaluation of ductility capacity of reinforced concrete bridge columns, limit states of reinforced concrete bridge column are redefined considering required seismic performance of bridges, nonlinear cyclic behavior and damage progress of reinforced concrete bridge columns, and a new evaluation method of ductility capacity including a new equation that estimates plastic hinge length and allowable tensile strain of longitudinal reinforcement, which determines limit state of reinforced concrete bridge column, is proposed considering buckling behavior of longitudinal reinforcement [6].

Table 1 summarizes the seismic performance of bridges, and the proposed definition of limit states of reinforced concrete bridge columns. The damage condition at each seismic performance level (called “SPL” in the following text) is also shown in the table.

The SPL 2 requires that bridges sustain limited damages after an earthquake and are capable of functional recovery in short period, which means damage of structural members is limited and the structural members sustain its capacity of lateral force and energy absorption. Based on these requirements, the limit state of reinforced concrete bridge columns at the SPL 2 is defined at the point where significant degradation of energy absorption capacity has not yet been observed and damage is easily repairable in short period because significant damage such as spalling of cover concrete or buckling of longitudinal reinforcement has not yet occur. The limit state at the SPL 3 is defined at the point just before significant degradation of lateral force capacity is observed, which is the definition same as the method specified in the 2002 specifications (hereinafter referred to as the conventional method).

In this revision, a new equation estimating the plastic hinge length L_p (Eq. (1)), and those estimating tensile allowable strain for the repairable limit state, ε_{st2}, and the ultimate limit state, ε_{st3}, were introduced (Eq. (3) and

(4)).

$$L_p = 9.5\sigma_{sy}{}^{1/6}\beta_n{}^{-1/3}\phi \qquad (1)$$

$$\beta_n = \beta_s + \beta_c \qquad (2)$$

$$\varepsilon_{st2} = 0.025L_p{}^{0.15}\phi^{-0.15}\beta_s{}^{0.2}\beta_c{}^{0.22} \qquad (3)$$

$$\varepsilon_{st3} = 0.035L_p{}^{0.15}\phi^{-0.15}\beta_s{}^{0.2}\beta_c{}^{0.22} \qquad (4)$$

where σ_{sy} is the yield strength of longitudinal reinforcement, ϕ is the diameter of longitudinal reinforcement, β_s is the stiffness of the spring that represents restraint of transverse reinforcement, and β_c is the stiffness of the spring that represents restraint of cover concrete.

Using the plastic hinge length given by Eq. (1) and the allowable tensile strain given by Eqs. (3) and (4), the displacement at each limit state was computed, and compared to the test results. Figure 9 shows the relation of lateral displacement at the ultimate limit state obtained from the cyclic loading tests and from the computation. The accuracy on evaluation of ultimate ductility of reinforced concrete bridge column is improved from 36.5% to 17.5% by using the proposed method.

4.4 Applicability of Reinforced Concrete Bridge Columns with Hollow Sections for Plastic Hinge Region

Reinforced concrete bridge columns with hollow sections have been used for tall bridge columns constructed in mountain area in order to reduce the self weight of bridge column, and to reduce the inertia force induced in its foundation. Based on the cyclic loading test results for columns with hollow sections conducted after the Kobe earthquake, the same ductility design method to the solid section has been used in seismic design. However, the structural conditions have been changed over 15 years. For example, the wall thickness has become thinner, the axial stress has become larger, and the amount of longitudinal reinforcement has become larger, which generally result in smaller ductility capacity and severe damage.

To evaluate the effects of such structural change, a series of cyclic loading tests have been conducted at PWRI [7]. It is found from the tests that the conditions listed above causes severe damage in the compression flange and also severe damage at the inside wall as shown in Figure 10. Besides, it is not easy to inspect the damage of the inside wall after an extreme earthquake, and a method has not yet been available to evaluate the damage of the inside wall from the damage of the outside wall.

Based on these results and considerations, it is recommended in the specifications as shown in Figure 11 that a hollow section shall not be used in the plastic hinge region, and haunches shall be provided at four corners inside the hollow sections and at region around the end of hollow section to prevent severe damage.

4.5 Introducing of Design and Construction Principals of Approach Embankment

The damage of main structural members of bridges caused by the recent major earthquakes has been decreased. This is because the newly bridges were designed by the upgraded design specification and seismic retrofit of the existing bridges, which were retrofitted the piers and installed unseating prevention systems, have been progressed. On the other hand, difference in level between abutment and backfill soil, and damage of pier beam by applied for inertia force of superstructure though bearings or unseating prevention devices became remarkable as critical causes of emergency operation after the earthquakes. It is easy to repair the difference in level of road surface in comparison with the other structural members. However, lessons learned from the 2011 Great East Japan Earthquake, they need a lot of time to repair in case that a lot of damages would occur in wide area at once even if each damage might be small. Therefore, it is newly prescribed the approach embankment whose part should be designed and constructed to keep the continuity of the road surface between the bridge and the embankment adjacent to the abutment. The approach embankment should be constructed using soil material that compacts well and ensures sufficient stability and drainage.

5. NEWLY INTRODUCED POINTS BASED ON THE RECENT RESEARCHS

5.1 Introducing of Design and Construction about Integral Abutment Bridge

To reduce the total investment cost of road bridges, it is important to reduce the maintenance costs in addition to the initial costs. One reason contributing to high maintenance costs is damage to bearings at the abutment and the expansion joints. Particularly, the ratio of costs for bearings and expansion joints relative to the total cost of the road bridges is high for short and medium class bridges. To resolve these problems, it is effective to introduce the integral abutment structures which can omit bearings and expansion joints.

Integral abutment bridges are not widespread in Japan although this type of bridge was first introduced experimentally about 20 years ago. The reason is the serious maintenance challenges such as the cracks in the pavement between the abutment and the approach embankment, and the lack of adoption of systematic design methods. Moreover, seismic design of the structure is also a key factor in Japan, just as it is in some U.S. states. However, it was not clear in earlier periods whether the seismic performance verification methods for integral abutment bridges were appropriately executed, especially for extreme earthquake events such as Level 2 earthquakes. Against this background, in 2006, PWRI commenced research of design and construction methods for integral abutment bridges as a cooperative program involving four technical associations. This work led to the issue of the new guideline in 2012 [8].

Based on this research, it is newly prescribed about fundamental principal of design and construction of the structure that an abutment jointed to a superstructure rigidly. In addition to an integral abutment, a portal frame bridge is also targeted at this regulation as shown in Figure12. For example, since the backfill is expected to provide resistance, the specifications of the approach embankment of the integral abutment bridge are higher than the other type of abutments in the area and the control standard values for soil compaction. The detail matters about the design and the construction of integral abutment is described in the guideline as mentioned before.

5.2 Fatigue Durability of Steel Bridge

The fatigue cracks to penetrate a deck plate in a weld of shut section (U rib) and deck plate have been increased in the existing steel slabs. When this crack progresses, traffic operation function might be decreased due to damage of pavement and a cave-in of the road. According to the damage example investigation, it was reported that this crack occurred in case of 12mm of minimum thickness of deck plate. Additionally, fatigue loading experimental tests using full scaled wheel and FE model analyses were carried out to evaluate fatigue durability by difference of the structural detail. These results showed that it is most effective to make thicker the deck plate. Based on these results, it is improved to normalize that thickness of deck plate where wheel load of heavy vehicle is always loaded is more than 16mm in case of steel deck by using U rib.

5.3 Design of Connection of Composite Structure

For the purpose of reducing the cost and rational design, composite structures which connect between the concrete member and the steel member such as corrugated steel plate or steel truss member gradually increase in recent years. It is necessary to assume that the connection of the composite structure have enough durability during an in-service period. However, damage examples due to the corrosion were reported at the connection between the concrete member and the steel member. Therefore, the fundamental requirements of design at the connection of composite structure to secure safety and durability were prescribed. Particularly, it is important that water does not stagnant by establishing a cross grade or the draining off aperture, and appropriate rust prevention, waterproofing at an interface and the embedding part of connection between concrete and steel members. Furthermore, it is also important to design the bridge in consideration of maintenance such as easily and securely inspection in service life.

5.4 Introducing High Strength Steel Rebar

A bar arrangement of RC members, especially the substructure, tends to become overcrowded by strengthening the seismic performance of the bridge after the 1995 Hyogo-ken Nanbu earthquake. As a results, construction quality might be deteriorated by the arrangement of the rebar becoming difficult. As for a head of steel pipe pile, reinforcing rebar is installed to connect the pile to footing rigidly. The reinforcing rebar is connected by welding outside of the pipe in case that strength is insufficient only with reinforcing rebar inside the pipe. However, the construction environment of welding is not good as shown in Figure 13.

To improve these problems, it is effective to use high strength steel rebar. Experimental studies such as cyclic loading tests of pier, pile foundation, and investigation about structural details such as bending radius, were carried out. Furthermore, it become easy to obtain high strength rebar in comparison with the past. Based on these results and background, the upper limit of the yield strength of steel rebar as a normal use was improved from 345N/mm^2 (SD345) to 490N/mm^2 (SD490). This effect contributes to the improvement of the bending strength and ductility. Moreover, welding work at a head of steel pipe pile was not necessary by using high strength rebar as shown in Figure 14. However, it does not contribute to shear strength because performance verification for shear is not enough based on the truss method.

6. SUMMARIES

This paper presents the outline of revisions of the Japanese design specifications for highway bridges. Main topics of this revision are as follows,

- Enhancement about designing the bridge in consideration of the maintenance and redundancy
- Seismic Issues such as revision of design earthquake ground motions and tsunami
- Introduction or improvement of specifications based on recent research results
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We continue to examine to introduce the load and resistance factor design concept.

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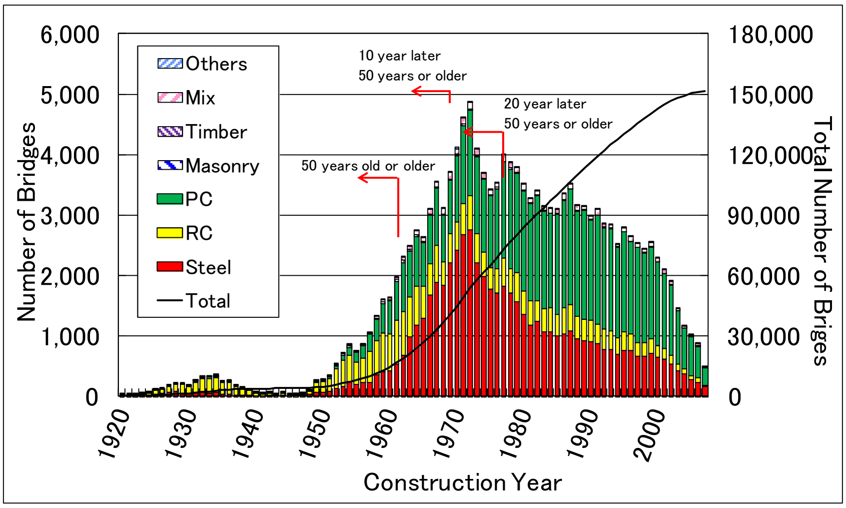
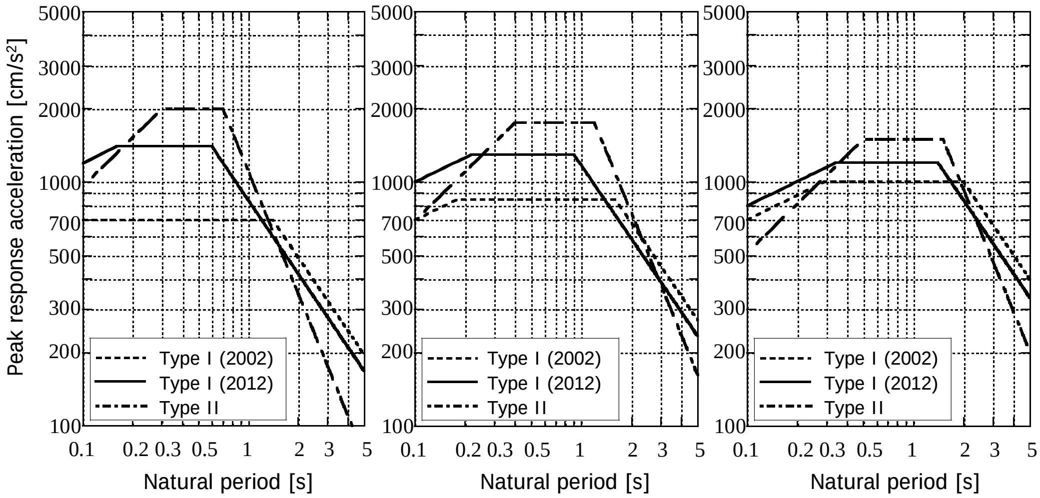


Figure 1 Construction Year Distribution of Road Bridges in Japan

Figure 2 Walkway for Inspection

Figure3 Fracture of Diagonal Bridge Bracing in Steel Truss Bridges



(a) Soil profile type I (b) Soil profile type II (c) Soil profile type III

Figure 4 Comparison of standard acceleration response spectra for Level 2 earthquake motions.

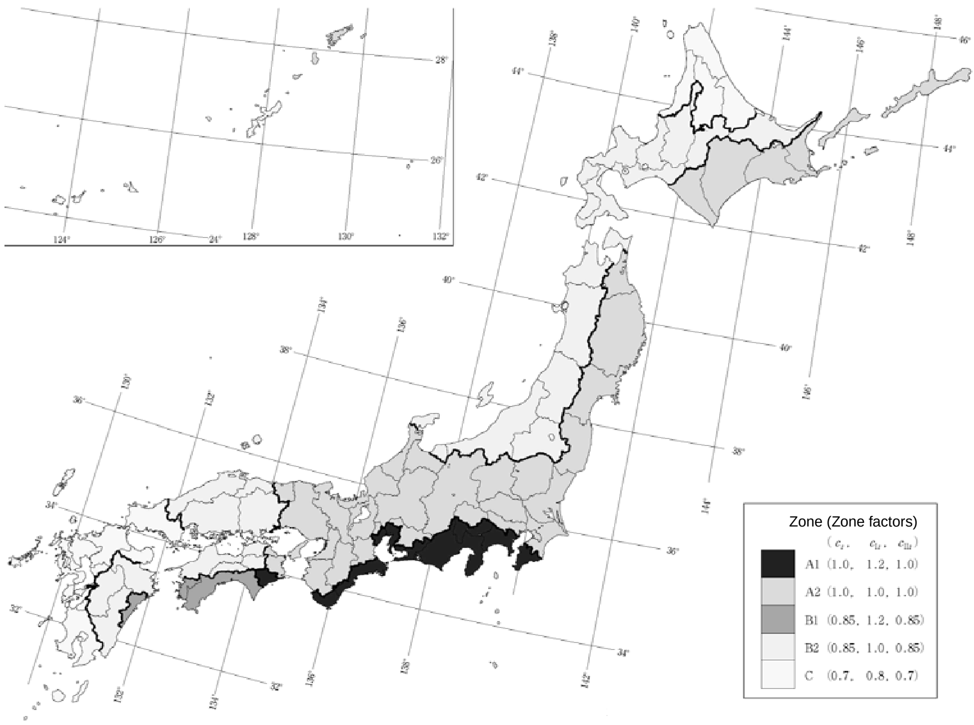


Figure 5 Regional classification for zone factors

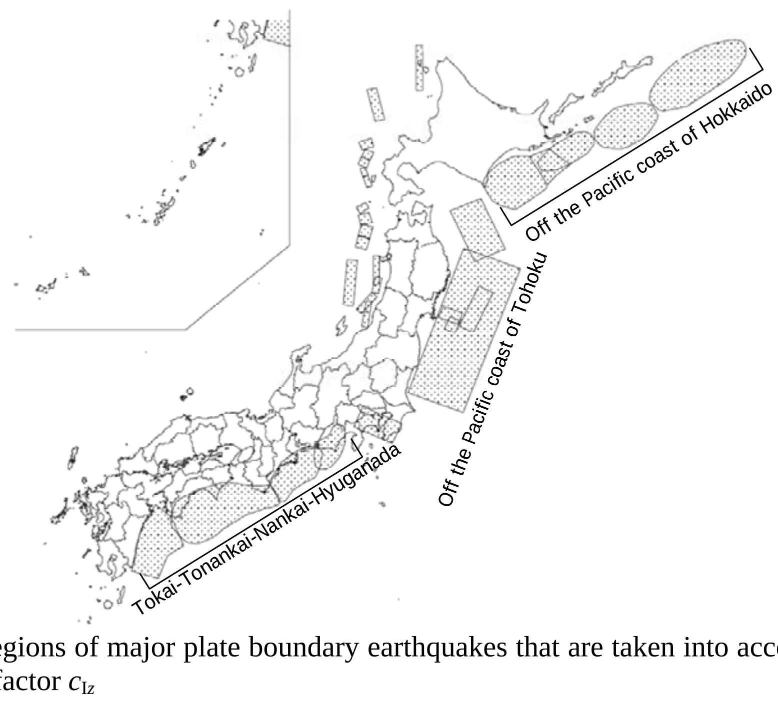
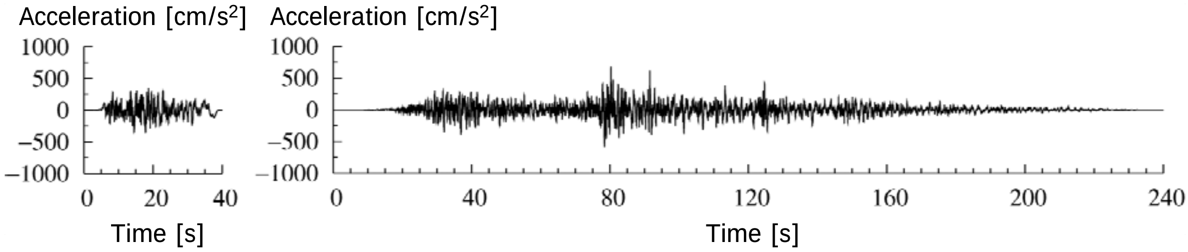


Figure 6 Source regions of major plate boundary earthquakes that are taken into account in the revision of the zone factor c_{Iz}



(a) Before revision (b) After revision

Figure 7 Comparison of acceleration waveforms prepared for time history response analysis.
(These examples correspond to soil profile type II.)

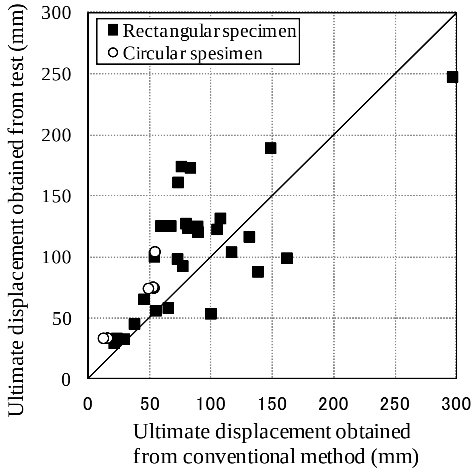
(a) Collapse of bridge due to land slide (b) Collapse of bridge due to tsunami

Figure 8 Collapse of bridge caused by effect that are not mainly affected by ground shaking

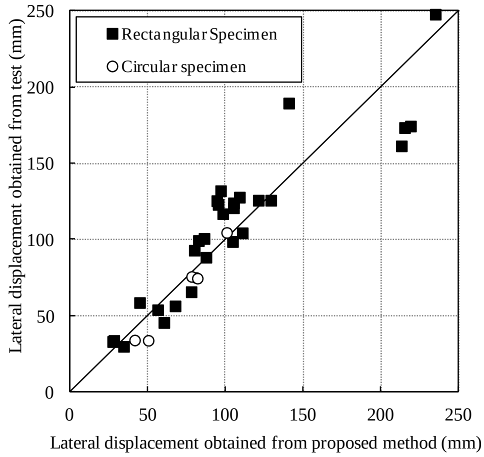
Table 1 Seismic performance of bridge and limit states of reinforced concrete bridge column

	SPL 2	SPL3
Seismic performance of bridge	Limited damage for function recovery in short period	Prevent fatal damage such as unseating of superstructure/ collapse of column
Limit state of bridge	**Repairable limit state**	**Ultimate limit state**
Limit state of reinforced concrete bridge column	State within a range of easy functional recovery and no significant degradation of energy absorption and lateral force capacity (Condition of 2) in Fig. 2)	State just before significant degradation of lateral force capacity occurs (Condition of 3) in Fig. 2)
Damage condition of reinforced concrete bridge column	Residual flexural cracks or minor spalling of cover concrete	Condition just before buckling of longitudinal reinforcement becomes noticeable after spalling of cover concrete

*) SPL1: Fully operational is required. Limit state of bridge is **serviceability limit state**.
Negligible structural damage and non-structural damage are allowed.



(a) Conventional method



(b) Proposed method

Figure 9 Accuracy of estimation of lateral displacement at limit state for ultimate limit state

(a) Damage of compression flange

(b) Damage at inside wall

Figure 10 Damage of reinforced concrete bridge column specimen with hollow section under high axial stress and high longitudinal reinforcement ratio

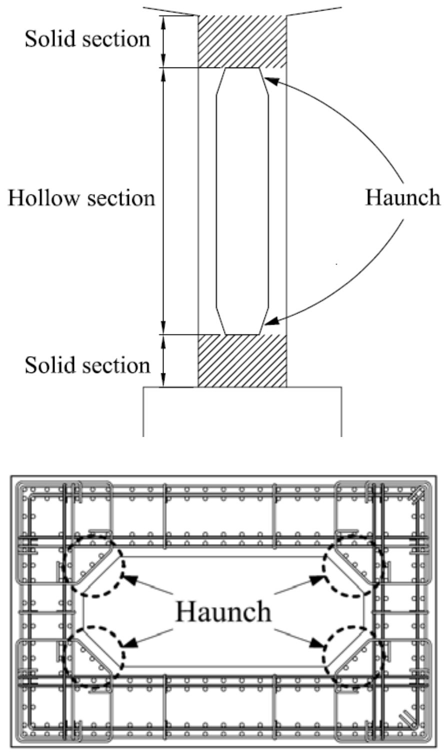
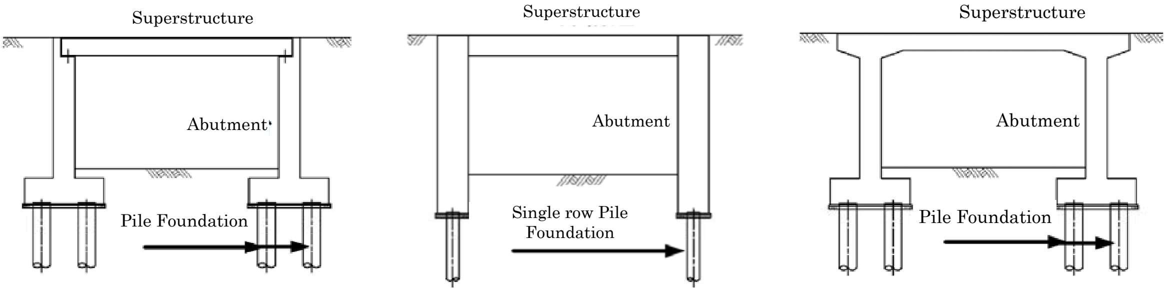


Figure 11 Design recommendations for reinforced concrete bridge columns with hollow section

Figure 12 Comparison of bridge structural characteristics

Structural type	a) Conventional bridge	Jointless Structure	
		b) Integral abutment	c) Portal frame
Bearing support	Install	Uninstalled (rigid frame)	Uninstalled (rigid frame)
Expansion joint	Install	Uninstalled (omit)	Uninstalled (omit)
Girder adjustment of expansion from thermal changes	Deform by the foundation	Deform by the flexible pile foundation	Resist by the rigidity of backwall and foundation



a) Conventional bridge b) Integral abutment bridge c)Portal Frame bridge

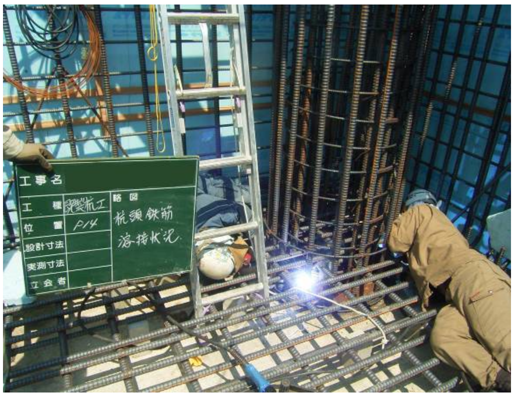


Figure 13 Welding at the head of pile

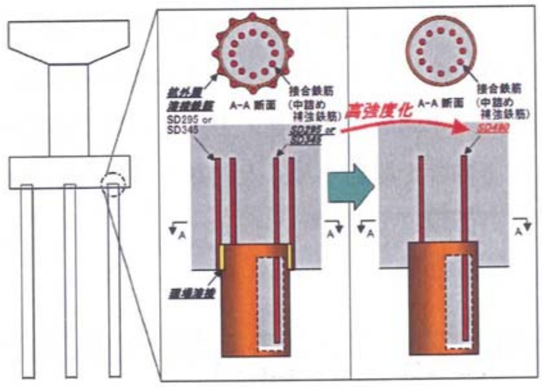


Figure 14 Improvement of Structural Detail at the Head of pile