PERSPECTIVES OF SEISMIC DESIGN CRITERIA FOR HIGHWAY BRIDGES IN JAPAN

TOSHIO IWASAKI

Civil Engineering Research Laboratory
3-3-4 Kanda Kaji-cho, Chiyoda-ku, Tokyo, 101 Japan

Paper No. 2110. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

ABSTRACT

Damage characteristics of highway bridges due to the 1995 Hanshin-Awaji Earthquake are first introduced. The current bridge seismic criteria (1990) and the supplementary specifications for reconstruction of damaged bridges (1995) are briefly described. Finally, important issues for revising the criteria are discussed.

KEYWORDS

Bridge Failure Mechanism, Earthquake Response Spectra, Foundation Movement, Hanshin-Awaji Earthquake, Highway Bridge, Hyogo-ken Nanbu Earthquake, Kobe Earthquake, Lateral Ground Flow, Seismic Criteria, Seismic Damage, and Soil Liquefaction.

INTRODUCTION

The Hanshin-Awaji, or Hyogo-ken Nanbu or Kobe, earthquake of Richter magnitude 7.2 attacked the city of Kobe and environs early in the morning of January 17, 1995, and brought about catastrophic damage to various engineering structures. Among them, highway bridges sustained serious damage, including fall of girders and collapse of reinforced concrete piers and steel piers.

After experiencing bridge damage caused by the Kanto Earthquake of 1923 (M7.9), Japan first introduced seismic design for highway bridges in 1926. Since then, design criteria were repeatedly improved in view of succeeding damage by strong earthquakes such as the 1948 Fukui (M7.3), the 1964 Niigata (M7.5), and the 1978 Miyagi-ken Oki (M7.4).

An analysis on bridge damage by the previous 14 major earthquakes which hit Japan between 1923 and 1984 pointed out that (1) a limited number of bridges (about ten) fell down due to direct seismic effects (a similar number of bridges fell because of fire), and (2) major types of bridge damage were dislocation of girders, failures at bearing supports, settlement and dislocation of substructures as rigid bodies, and cracking in the middle and base sections of reinforced concrete pier columns (Iwasaki et al, 1972; PIARC, 1995).

In the 1995 Hanshin-Awaji, however, a significant amount of vibrational effects on bridge structures was observed. Nine bridges, with some 30 spans of elevated bridge girders, fell down because of direct seismic effects. It is concluded that these effects were the result of very large horizontal ground accelerations exceeding 0.8g in some areas. The major damage concentrated within a narrow region close to the epicenter of the shallow shock of magnitude 7.2 (Seismic Advisory Committee, 1995; Kawashima, 1995).

In view of the severe effects on bridge structures, the Ministry of Construction set up the Seismic Advisory Committee on Bridge Damage, consisting of ten members and chaired by the author. The committee proposed supplementary specifications for reconstruction of damaged bridges in February (MOC, 1995), issued an interim report in March (Kawashima, 1995), and released a final report in December (Seismic Advisory Committee, 1995).

With reference to these reports this paper first describes damage features of highway bridges due to the 1995 Hanshin-Awaji. Then it introduces the current seismic design criteria for bridges (JRA, 1990) and the supplementary specifications for reconstruction of bridges damaged by this earthquake. Finally it discusses important issues to be studied for revising the current criteria, in view of the lessons learned from the latest earthquake.

EFFECTS OF THE 1995 HANSHIN-AWAJI EARTHQUAKE ON HIGHWAY BRIDGES

An outline of the Hanshin-Awaji Earthquake and its effects is as follows:

1) Time of Occurrence	: 5:46am, January 17, 1995
2) Epicenter	: Northern Awaji Island (Hypocenter Depth : 14 km)
3) Magnitude	: 7.2 (Richter scale, determined by JMA)
4) Damage Statistics (as of July 19, 1995)	: Casualties : 5,504 fatalities, 41,527 persons injured Houses : 100,282 destroyed; 294,158 partially collapsed Fires : 294 separate blazes Road damage : 9403 sites (Estimated damage cost: 640 billion yen = 230 b.y. for public roads + 410 b.y. for toll roads)

Fig. 1 illustrates the epicenter, focal zone, and peak accelerations (in gals) recorded, major transportation networks, and major bridge damage sites. Evidences of surface faulting are visually found only in the northern part of Awaji Island. The fault line under the ocean and Honshu Island is estimated from aftershock epicenters, measured by the Japan Meteorological Agency and other institutes (Muramatsu, 1995).

As shown in Fig. 1, large horizontal ground accelerations as much as 800 gals were observed at several stations in Kobe and nearby. The peak ground velocity and displacement are on the order of 90 cm/s and 50 cm in some areas of Kobe and nearby. These ground motions are the highest level ever measured in Japan.

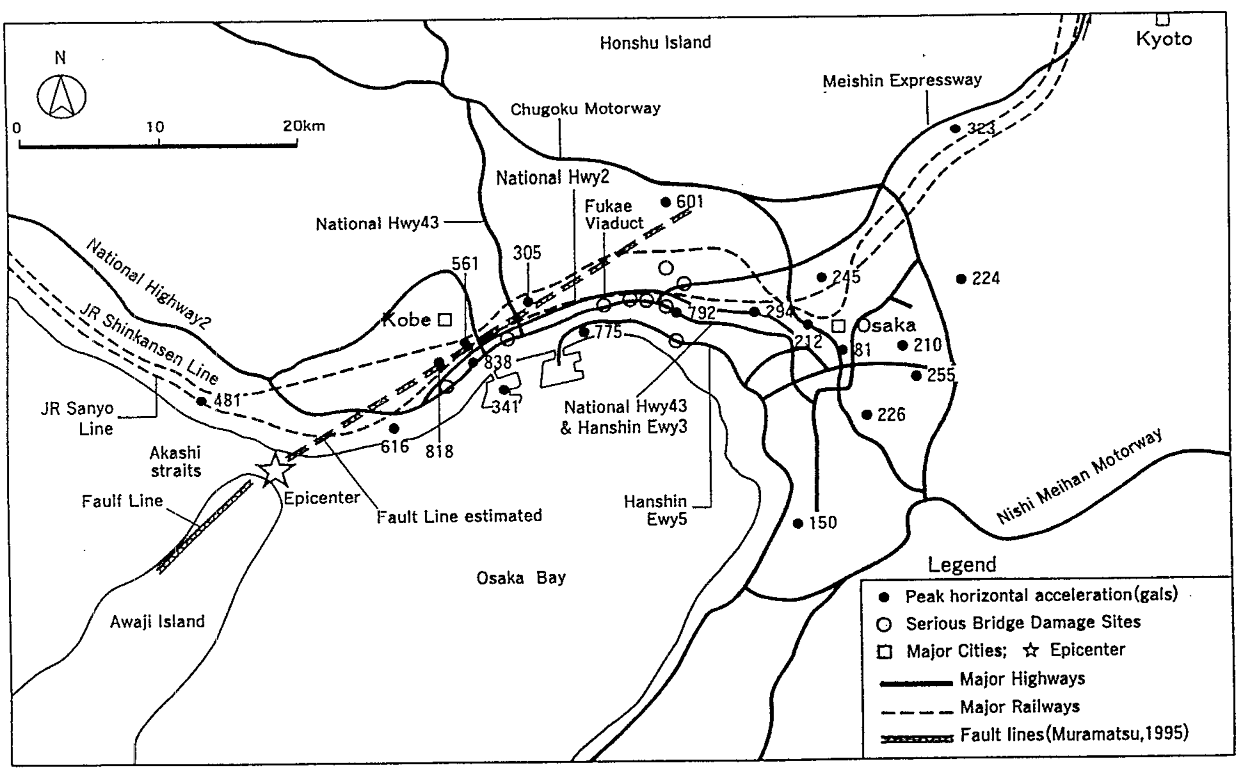

Fig. 1 Epicenter, Focal Zone, and Peak Accelerations (in gals) Recorded, Major Transportation Networks, and Major Bridge Damage Sites

Statistics of Bridge Damage

Bridge damage was observed on National Highway Routes 2, 43, 171, and 176, Hanshin Expressway Route 3 (Kobe Line) and Route 5 (Bayshore Line), and the Meishin Expressway and Chugoku Motorway of the Japan Highway Public Corporation. Damage investigations were conducted for thousands of bridges located in Kobe and six surrounding cities. The numbers of superstructures, substructures, and bearing supports investigated were 4157 spans, 3396 piers, and 5741 lines.

Table 1 indicates a relation between the numbers of bridge piers surveyed and the design criteria which were applied to the design of these piers. Figures in this table show the numbers of piers (both reinforced concrete and steel piers) that were investigated after the event. It is noted that most piers (81%) were designed according to the 1964 criteria or older ones, and a small number of piers (2%) located only on Hanshin Expressway Route 5 were designed by the current 1990 criteria. As described later, the 1964 criteria and older ones specify only a lateral force requirement by means of the seismic coefficient method, but the 1980 and 1990 criteria include detailed requirements such as girder-fall prevention devices, soil liquefaction, and reinforced concrete pier ductility.

Table 1 Relation between Numbers of Bridge Piers Investigated and Design Criteria Applied to the Piers

Routes	Criteria	1964 Criteria or Older	1971 Criteria	1980 Criteria	1990 Criteria	Total
National Highways (Ministry of Construction)	Route 2	43(37%)	72(63%)	-	-	115(100%)
	Route 43	152(100%)	-	-	-	152(100%)
	Route 171	158(100%)	-	-	-	158(100%)
	Route 176	13(65%)	2(10%)	5(25%)	-	20(100%)
	Sub-Total	366(82%)	74(17%)	5(1%)	-	445(100%)
Hanshin Expressway	Route 3	890(80%)	216(20%)	-	-	1,106(100%)
	Route 5	-	-	289(84%)	56(16%)	345(100%)
	Sub-Total	890(61%)	216(15%)	289(20%)	56(4%)	1,451(100%)
JHPC	Meishin	1,039(100%)	-	-	-	1,039(100%)
	Chugoku	461(100%)	-	-	-	461(100%)
	Sub-Total	1,500(100%)	-	-	-	1,500(100%)
Grand Total		2,756(81%)	290(9%)	294(9%)	56(2%)	3,396(100%)

In order to clarify the bridge damage characteristics, degrees of damage to superstructures, substructures, and bearing supports are classified into five groups, from As (Catastrophic) to D (Slight or no damage) according to severity of damage. As an example, definitions of pier damage degree are shown in Table 2.

Table 2 Definitions of Damage Degree for Bridge Piers

Damage Degree	Definitions
As=Catastrophic	Collapse, extensive failure or deformation resulting in lack of bearing capacity of piers
A=Very serious	Extensive cracks, buckling, rupture of main reinforcements, and large deformation
B=Serious	Local buckling of main reinforcements, large cracks of concrete, and local buckling of steel webs and flanges
C=Moderate	Spalling of surface concrete, small cracks of concrete, and residual deformation of steel webs and flanges
D=Slight or no damage	Slight damage or no damage

Table 3 shows a distribution of numbers of piers according to damage degree for all piers investigated. In this table structural materials (steel or reinforced concrete) of the piers and applied design criteria are classified.

Table 3 Distribution of Numbers of Bridge Piers according to Damage Degree

Damage Degree: Piers	Damage Degree: Criteria	As=Catastrophic	A=Very Serious	B=Serious	C=Moderate	D=Slight or no	Total
Steel Piers	1964(+)	4(4%)	11(10%)	9(8%)	72(67%)	12(11%)	108(100%)
	1971	-	-	14(13%)	45(43%)	45(43%)	104(100%)
	1980	-	-	11(9%)	19(15%)	97(76%)	127(100%)
	1990	-	-	2(13%)	2(13%)	12(75%)	16(100%)
	Subtotal	4(1%)	11(3%)	36(10%)	138(39%)	166(47%)	355(100%)
Reinforced Concrete Piers	1964(+)	78(3%)	155(6%)	143(5%)	672(25%)	1,600(60%)	2,648(100%)
	1971	1(1%)	1(1%)	15(8%)	63(34%)	106(57%)	186(100%)
	1980	-	-	1(1%)	22(13%)	144(86%)	167(100%)
	1990	-	-	-	-	40(100%)	40(100%)
	Subtotal	79(3%)	156(5%)	159(5%)	757(25%)	1,890(62%)	3,041(100%)

Notes: 1) 1964(+) includes 1964 criteria and older ones.
2) Total number of piers investigated is 3,396 (=355+3,041).

From the comprehensive investigations for the large number of bridges (4157 spans, 3396 piers, and 5741 lines) and some additional studies the following findings are indicated:

1) Ratios of seriously damaged piers (damage degree of As, A, and B) to the total examined were 14% (=51/355) for steel, and 13% (=394/3041) for reinforced concrete. These ratios do not differ.

2) Older piers designed by the 1964 criteria and older ones sustained heavier damage than those designed by the 1980 criteria or newer.

3) The ratio of seriously damaged bearing supports was comparatively high, 28% (=1604/5741).

4) For Hanshin Expressway Route 3, materials of superstructures (steel or reinforced concrete), span lengths, pier heights, and damage degree of bearing supports did not affect the damage degree of piers.

Lateral Foundation Movements

Large lateral movements of bridge foundations mainly caused by the effects of soil liquefaction and resulting ground flows, were also observed carefully. Fig. 2 illustrates a relationship between ground flows near bridge foundations and lateral movements of foundations on Hanshin Expressway Route 5. The largest ground flows and foundation movements are some 220 cm and 90 cm, and foundation movements are on the order of half or less of the ground flows.

From the studies for bridge foundations the following are apparent:

1) Many bridge foundations located close to waterways moved laterally toward the water, up to 90 cm. Movements of rigid foundations (caisson and diaphragm wall foundations) were smaller; those of flexible foundations (pile foundations) were larger.

2) Few bridges fell down due to these lateral movements of foundations caused by soil liquefaction and resulting ground flows. It seems probable that installed girder-fall prevention devices, were very effective in preventing girder fall.

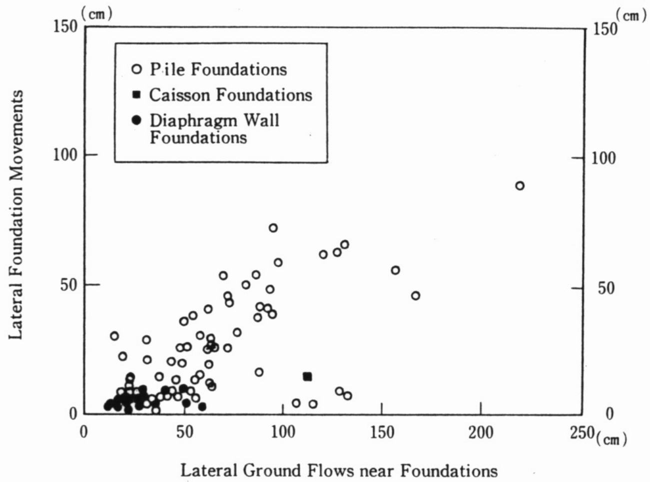

Fig. 2 Relation between Ground Flows and Foundation Movements (Seismic Advisory Committee, 1995)

Photo 1 Collapse of Viaduct at Fukae, Kobe on Hanshin Expressway Route 3

Estimation of Failure Mechanisms

In order to clarify the causes of bridge damage, failure mechanisms of six heavily damaged bridges were also investigated. As an example, collapse of the viaduct at Fukae, Higashi-nada-ku, Kobe on Hanshin Expressway Route 3 is shown in Photo 1. Over the length of 635 m, 18 prestressed concrete girders together with 17 reinforced concrete piers overturned to the north perpendicular to the bridge axis. The viaduct had been completed in 1969, with use of the 1964 design criteria. In seismic design of this bridge the conventional seismic coefficient method, 0.2 horizontal and 0.1 vertical, was applied, based on the allowable stress approach.

The columns have diameters of 3.1 and 3.3 meters, and heights of 9.9 to 12.4 meters above the footings. The number of longitudinal reinforcements in a typical pier is 180 at the column bottom, and is reduced to 120 at the section 2.5 meters above the footing.

To estimate the dynamic behavior during the event, a linear response analysis for all of the spans and a non-linear analysis for one pier column and the mass of its supporting superstructure were conducted, with use of the acceleration record (peak acceleration of 818 gals) obtained at Kobe JMA Observatory during this earthquake. From these analyses the following were pointed out:

1) Design and construction : From re-designing of the piers with use of the 1964 criteria and testing of materials (concrete pieces and steel bars) taken from the actual columns, original design and construction are found to be appropriately performed.

2) Failure mechanism : The first flexural cracks of the concrete pier column developed at the section 2.5 m above the footing where one-third of the longitudinal reinforcement bars end without enough anchoring length. The flexural cracks propagated laterally and turned to diagonal cracks. As ground motions and seismic forces increased, concrete failed considerably along the diagonal cracks and the column initiated its tilt toward north. Thus the dead weight of the superstructure affected its standing stability, and finally the column overturned toward north. Failure of tie bars and rupture of longitudinal bars at pressure welding points were presumably generated during this process.

3) Concluding estimation : Catastrophic failures of this bridge and five other bridges investigated are estimated to be essentially caused by the enormous seismic forces applied, which are much stronger than the design forces specified in the conventional criteria, including the current 1990 criteria.

CURRENT SEISMIC DESIGN CRITERIA FOR HIGHWAY BRIDGES (1990)

Seismic design for highway bridges was first introduced in 1926 after the Kanto Earthquake caused extensive bridge damage in 1923. The seismic coefficient method developed in Japan was then employed for a fairly long period. Design criteria were repeatedly changed in view of succeeding damage by strong earthquakes such as the 1948 Fukui, the 1964 Niigata, and the 1978 Miyagi-ken Oki. Table 4 briefly introduces Japan's history of seismic design for highway bridges.

Current seismic bridge design is based on the 1990 Specifications for Highway Bridges, Part V. Seismic Design, issued by the Japan Road Association and authorized by the Ministry of Construction. The Specifications have 8 chapters and reference materials relating to seismic design of bridges, and are summarized as follows:

1) Chapter 3 gives procedures for modeling a bridge as a single-degree-of-freedom system to calculate the natural period. Evaluation for soil liquefaction potential and reduction of soil stiffness or spring constant which can be determined according to the degree of liquefaction potential are also specified.

2) Chapter 4 provides the seismic coefficient for designing substructures as:

$$Kh = Cz\ Cg\ Ci\ Ct\ Kho \quad (1)$$

where Kh : horizontal seismic coefficient for design (normally Kh=0.1-0.3)
Kho: standard horizontal seismic coefficient (=0.2)
Cz : zone factor (0.7, 0.85, and 1.0 for three different zones)
Cg : soil factor (0.8, 1.0, and 1.2 for three categories)
Ci : importance factor (0.8 and 1.0 for two categories)
Ct : natural period factor (shown in Fig. 3)

Table 4 History of Seismic Bridge Criteria in Japan

Year of Issue	Criteria	Design Procedures	Affecting Earthquakes
1926	Recommendations for Design of Roads, Road Laws (MHA)	Seismic Coefficient Method (Kh=0.15-0.4)	1923 Kanto (M7.9)
1939	Specifications for Design of Steel Hy. Bridges (MHA)	Seismic Coef. Meth. (Kh=0.2,Kv=0.1)	
1956	Revision of Spec. for Design of St. Hw. Br.(JRA)	Seismic Coef. Meth. (Kh=0.15-0.35,Kv=0.1)	1946 Nankai (M8.1) 1948 Fukui (M7.3)
1964	Spec. for Design of Substructures of Hy. Br.(JRA)	Same as above, plus Detailed Calculation Meth.	
1971	Spec. for Earthquake Resistant Design of Hy. Bridges (JRA)	SCM (Kh=0.1-0.24:Rigid), Modified SCM (Kh=0.05-0.3: Flexible), Soil Liquefaction, Restrainers	1964 Niigata (M7.5)
1980	Spec. for Hy. Bridges, Part V. Seismic Design (JRA)	Same as above, plus Deformation Capacity of RC Piers, Dynamic Analysis	1978 Miyagi-ken Oki (M7.4)
1990	Spec. for Hy. Bridges, Part V. Seismic Design (JRA)	SCM (Kh=0.1-0.3), Soil Liquefaction, Restrainers, Ductility of RC Piers, Dynamic Analysis, Structural Details	1982 Urakawa (M7.1) 1983 Nihonkai Chubu (M7.7)
1995	Supplementary Specifications for Reconstruction of Damaged Bridges (MOC), and Reference Materials (JRA)	Same as above, plus Non-linear Dynamic Anal., Menshin Design, Ductility Check, Bearing Supports, Falling Prevention Device, Gravelly Soil Liquefaction	1995 Hanshin-Awaji (M7.2)

3) Chapter 5 requires a check for ductility of reinforced concrete pier columns. For this check the seismic coefficient defined below should be used:

$$Khc = Cz\ Ci\ Cr\ Khco \qquad (2)$$

where Khc : seismic coefficient for ductility check of RC columns (1.0 or less)
Khco: standard seismic coefficient for ductility check (=1.0)
Cz, Ci: zone factor, importance factor, same as in the seismic coefficient method
Cr : dynamic response factor as shown in Fig. 4

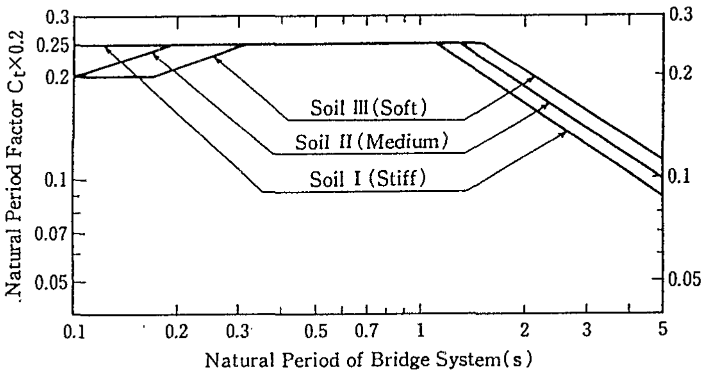

Fig. 3 Natural Period Factor in the Seismic Coefficient Method (JRA, 1990)

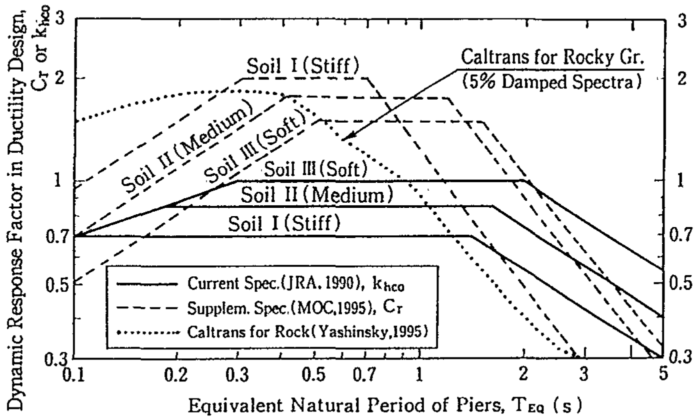

Fig. 4 Dynamic Response Factor in the Ductility Check of Piers (JRA, 1990; MOC, 1995; Yashinsky, 1995)

SUPPLEMENTARY SPECIFICATIONS FOR RECONSTRUCTION OF DAMAGED BRIDGES (1995)

With the recommendations of the Seismic Advisory Committee, the Ministry of Construction provided the supplementary specifications on February 27, 1995 for the reconstruction of heavily damaged bridges. The supplementary specifications are summarized as follows:

1) Basic concept : Reconstructing bridges should withstand a future earthquake similar to the Hanshin-Awaji. To attain this target the following supplementary items should be carefully considered, as well as the 1990 seismic design criteria.

2) Non-linear analysis : Besides dynamic linear analysis specified in the 1990 criteria, dynamic behavior of a bridge shall be examined by performing non-linear analysis and its result shall be reflected in the design. As input motions for the analysis ground acccelerations recorded during the latest event shall be employed, in order to ensure bridge safety against a similar event in the future.

3) Ductility of piers : Ductility of reinforced concrete piers, steel piers, and foundations shall be checked. Applied forces for this check can be obtained with use of response spectra illustrated by broken lines in Fig. 4. In comparison, Caltrans' elastic spectra of 5% damped for 0.7g peak rock acceleration with 0-3 meters alluvium is shown by a dotted line in Fig. 4 (Yashinsky, 1995). Moreover, to assure pier ductility sudden discontinuity of longitudinal reinforcement bars in RC pier columns is prohibited.

4) Menshin design : Menshin (or base isolation) design shall be effectively employed. The use of rubber bearings is especially recommended.

5) Liquefaction and lateral ground flow : Lateral ground flows mainly caused by the effects of soil liquefaction shall be incorporated in foundation design. Soil involving coarse materials up to diameter 20 mm may liquefy during a strong earthquake.

6) Bearings and falling-off devices : Bearing supports and falling-off devices should have sufficient strength and ductility, in order to prevent girder fall.

CONCLUSIONS

The Hanshin-Awaji Earthquake was the first occasion for Japan's modern cities to experience destructive powers of a shallow near-source magnitude-7 event. Many engineering structures designed for required seismic forces were destroyed. Among them, highway bridges sustained serious damage, including girder fall and pier collapse.

The Seismic Advisory Committee on Bridge Damage conducted comprehensive investigations on features and mechanisms of bridge failure, and proposed the supplementary specifications for reconstruction. In view of the lessons learned from the analysis of the extensive bridge damage, the Japanese bridge community is expected to revise the current design criteria (1990) within the year 1996. The following subjects should be thoroughly studied in the revision:

a) Seismic design forces induced by shallow near-source events,
b) Definition of tolerable damage level and importance of bridges, in consideration of construction cost, probability of suffering from a strong earthquake, necessity for emergency use, and duration and cost for repairing,
c) Strength and ductility of reinforced concrete piers and steel piers,
d) Non-linear dynamic analysis of bridges when subjected to large ground motions caused by shallow near-source events (for instance, 800 gals in the latest event),
e) Menshin or base isolation design for enhancing seismic performance of bridges,
f) Strength and deformation capacity of bearing supports, and application of rubber bearings,
g) Development of more effective girder-fall prevention devices, and
h) Soil liquefaction and lateral ground flow, to be incorporated for designing flexible foundations on soft soil layers and reclaimed lands.

ACKNOWLEDGMENTS

The author expresses his sincere appreciation to the members of the Seismic Advisory Committee on Bridge Damage (MOC) for their enthusiastic discussions on bridge failure caused by the 1995 Hanshin-Awaji Earthquake. Many thanks go to Professor K. Kawashima of Tokyo Institute of Technology for his enormous efforts for the accomplishment of the Committee. The author is also grateful to the staff of the Ministry of Construction's Road Bureau and Public Works Research Institute for their invaluable information on seismic effects on highway bridges.

REFERENCES

Iwasaki, T., J. Penzien, and R. W. Clough (1972): Literature Survey- Seismic Effects on Highway Bridges, EERC Report 72-11, University of California, Berkeley.
JRA (1990): Specifications for Highway Bridges, Part V. Seismic Design, Japan Road Association.
Kawashima, K. (1995): Impact of Hanshin-Awaji Earthquake on Seismic Design and Seismic Strengthening of Highway Bridges, U.S. National Conference on Bridges and Highways.
MOC and JRA (1995): Supplementary Specifications for Reconstruction of Damaged Bridges, and Reference Materials, Ministry of Construction, and Japan Road Association.
Muramatsu, I.(1995): On the JMA Seismic Intensity, JSEEP News 145, Japan Society for Earthquake Engineering Promotion.
PIARC Working Group G2 (1995): Comprehensive Report on Natural Disaster Reduction for Roads, PIARC Publication 72.01.B, World Road Association (Formerly PIARC).
Seismic Advisory Committee on Bridge Damage (1995): Investigation Report on Highway Bridge Damage Caused by the Hyogo-ken Nanbu Earthquake, Ministry of Construction.
Yashinsky, M. (1995): Seismic Design and Retrofit Procedures for Highway Bridges in Japan, What's Shakin'?, September, 1995, Office of Earthquake Engineering, State of California, Department of Transportation (Caltrans).