https://doi.org/10.20965/jdr.2006.p0262

Review:

Seismic Design of Bridges After 1995 Kobe Earthquake

Kazuhiko Kawashima

Department of Civil Engineering, Tokyo Institute of Technology
2-12-1 O-Okayama, Meguro, Tokyo 152-8550, Japan
E-mail: kawasima@cv.titech.ac.jp
[Received July 24, 2006; accepted August 3, 2006]

The 1995 Kobe earthquake extensively damaged bridges and triggered research and review as a consequence of recent earthquakes that have led to significant advances in bridge seismic design. This paper presents how this has affected design philosophy and design codes in Japan compared to seismic design codes in EC, New Zealand, and the United States concerning design philosophy, near-field ground motions, design force and ductility requirements, linear/nonlinear static/dynamic response analysis, and treatment of liquefaction and liquefaction-induced lateral ground movement.

Keywords: seismic design, disaster prevention, design codes, design ground motions, bridges

1. Introduction

Extensive damage of bridges in the 1989 Loma Prieta, 1994 Northridge and 1995 Kobe (Hyogo-ken nanbu) earthquakes together with research triggered as a consequence of recent earthquakes have led to significant advances in bridge seismic design. Near-field ground motions developed in the Northridge and Kobe earthquakes were included in the 1996 Japanese and 1999 Caltrans design codes (JRA 1996 [11], Caltrans 1999 [4, 5]). The conventional seismic coefficient method is being replaced by ductility design method, and linear/nonlinear dynamic response analysis is being used on routine basis in design of bridges with a complex structural response. New treatment for liquefaction and liquefaction-induced lateral ground movement was included in Japanese Specifications, and verification of the ductility evaluation of reinforced concrete/steel single/frame columns is being conducted by various research organizations.

Because of the unsatisfactory performance of bridges in the 1995 Kobe earthquake, the Japanese Design Specifications of Highway Bridges was revised in 1996. The code was further revised in 2002 based on the same main contents. This paper describes the seismic design practice of bridges related to the performance criteria, seismic design force, the design of reinforced concrete columns and foundations, and the treatment of liquefaction and liquefaction-induced ground movement in the 1996 and 2002 Japanese codes. Features of the codes in comparison with the codes in Europe, the United States and New Zealand where significant contributions in developing seismic design methodology of bridges have been made are described.

2. Design Philosophy and Seismic Performance Criteria

Basic concept of design philosophy and seismic performance criteria are more or less similar among codes in Japan (JRA 1996 [11], JRA 2002 [11], Kawashima 2000 [17]), the USA (AASHTO 1995 [1], ATC 1996 [2], Caltrans 1999 [4, 5], Roberts 1999 [26], Buckle 1996 [3]), the EU (European Standard 1994 [7], Pinto 1995 [23]) and New Zealand (TNZ 1994 [28], Chapman 1995 [6], Park 1996 [22]), i.e., for small-to-moderate earthquakes bridges should be resisted within the elastic range of structural components without significant damage, and bridges exposed to shaking in large earthquakes should not collapse.

In each code, the performance requirements depend on bridge importance, which is classified into 2 to 3 categories. In EU and New Zealand codes, design force is factored by the importance factor. In AASHTO and Caltrans, importance is reflected in the evaluation of the response modification factor. In Japan, importance is reflected in the evaluation of the design ductility factor of substructures, which subsequently affects the evaluation of the response modification factor.

Table 1(1) shows the anticipated function of a bridge after a design earthquake in the Japanese code. Important bridges undergo "limited damage," in which "damage" does not exceed the stage in which the restoring force of main structural components initiates deterioration. Hence, "limited damage" is intended to be almost "immediately functional." ATC-32 and Caltrans classify the performance criteria into service and damage levels as shown in **Table 1(2)**. In service level, "immediate" implies full access to normal traffic almost immediately following the earthquake, and "limited" implies that limited access (reduced lanes, and light emergency traffic) is possible within days of the earthquake, and that full service is restorable within months. In damage level, "repairable damage" denotes damage repairable with minimum risk of losing functionality. "Significant damage" denotes

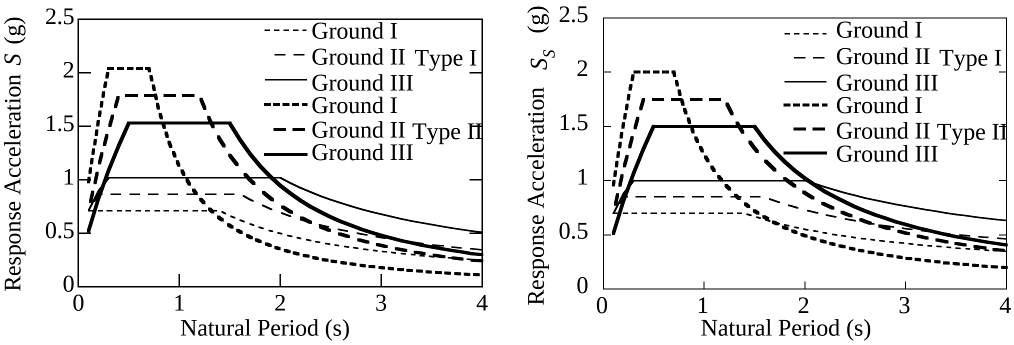
Table 1. Seismic performance criteria.

(1) JRA (1996)

Ground Motion (GM)		Ordinary Bridges	Important Bridges
GM with high probability of occurrence		Functional	Functional
GM with low probability of occurrence	Type I GM (Plate-boundary earthquakes)	Prevent critical damage	Retain limited damage
	Type II GM (Intra-plate earthquakes)		

(2) ATC32 and Caltrans (1999)

Ground Motion	Service level		Damage level	
	Ordinary Bridges	Important Bridges	Ordinary Bridges	Important Bridges
Functional evaluation GM	Immediate	Immediate	Repairable Damage	Minimum Damage
Safety evaluation GM	Limited	Immediate	Significant Damage	Repairable Damage



(a) S by Eqs. (2) and (3)

(b) S_s by Eqs. (8) and (9)

Fig. 1. Response accelerations of bridges.

a minimum risk of collapse, but would require closure for repair, while "minimum damage" denotes essentially elastic performance. "No collapse," "no major damage," "no secondary injuries or fatalities because emergency equipment cannot get through," "major important structures and lifeline routes must remain operational" are the current Caltrans performance criteria (Roberts 1999 [26]). It is important that the important bridges should provide full access to normal traffic immediately following the safety-evaluation earthquake.

In the 1995 Kobe earthquake, engineers in the organizations with jurisdiction over bridge construction and maintenance suffered damage to their families and properties. Due to a lack of information, it was days after the earthquake before even approximate damage to transportation system was identified based on systematic inspection. If damage occurred only at a bridge, temporary shoring up of the structure may have been possible within a couple of days of the earthquake, but extensive damage over wide area prevented even temporary shoring up for a number of bridges shortly after the earthquake. No stock of bearings, expansion joints or restrainers was available, and it took considerable time to fabricate them after ordering. This experience made it obvious that bridges on important routes must not sustain damage to where repairs were required after an earthquake. Otherwise, repair would be difficult within a month or even months after the earthquake. Prior to the Kobe earthquake, it was a question of whether bridges should have higher seismic performance than buildings. It is now obvious that bridges should have higher seismic performance than standard buildings, because damaged buildings cannot be restored if transportation is suspended due to collapse of bridges which have the same seismic performance as buildings.

3. Design Ground Motion

Two-level ground motion is used in seismic design of bridges in Japan as shown in **Fig. 1**, i.e., function evaluation and safety evaluation ground motions. There are Type I and Type II ground motions in the safety evaluation ground motions as follows:

<u>Function-Evaluation</u>

$$S = k_Z \cdot k_D \cdot \begin{cases} S_1 \cdot T^{1/3} & (0 < T \le T_1) \\ S_2 & (T_1 \le T \le T_2) \\ S_3/T & (T_2 \le T) \end{cases} \quad . \; . \quad (1)$$

<u>Safety-Evaluation</u>

$$S = k_Z \cdot k_D \cdot \begin{cases} S_4 T^{1/3} & (0 < T \le T_3) \\ S_5 & (T_3 \le T \le T_4) \\ S_6/T & (T_4 \le T) \end{cases} \quad . \; . \quad (2)$$

(Type I ground motion)

$$S = k_Z \cdot k_D \cdot \begin{cases} S_7 \cdot T^{2/3} & (0 \le T \le T_5) \\ S_8 & (T_5 \le T \le T_6) \\ S_9/T^{5/3} & (T_6 \le T) \end{cases} \quad . \; . \quad (3)$$

(Type II Ground Motion)

Table 2. Values of parameter in Eqs. (1)-(3).

(a) Function evaluation ground motion

Soil Group	T_1 (s)	T_2 (s)	S_1(m/s^2)	S_2(m/s^2)	S_3(m/s^2)
I	0.1	1.1	4.31	2.0	2.2
II	0.2	1.3	4.27	2.5	3.25
III	0.34	1.5	4.3	3.0	4.5

(b) Safety evaluation ground motions

Soil Group	Type I Ground Motion					Type II Ground Motion				
	T_3(s)	T_4(s)	S_4(m/s^2)	S_5(m/s^2)	S_6(m/s^2)	T_5(s)	T_6(s)	S_7(m/s^2)	S_8(m/s^2)	S_9(m/s^2)
I	0	1.4	—	7.0	9.8	0.3	0.7	44.63	20.0	11.04
II	0.18	1.6	15.05	8.5	13.6	0.4	1.2	32.24	17.5	23.71
III	0.29	2.0	15.11	10.0	20.0	0.5	1.5	23.81	15.0	29.48

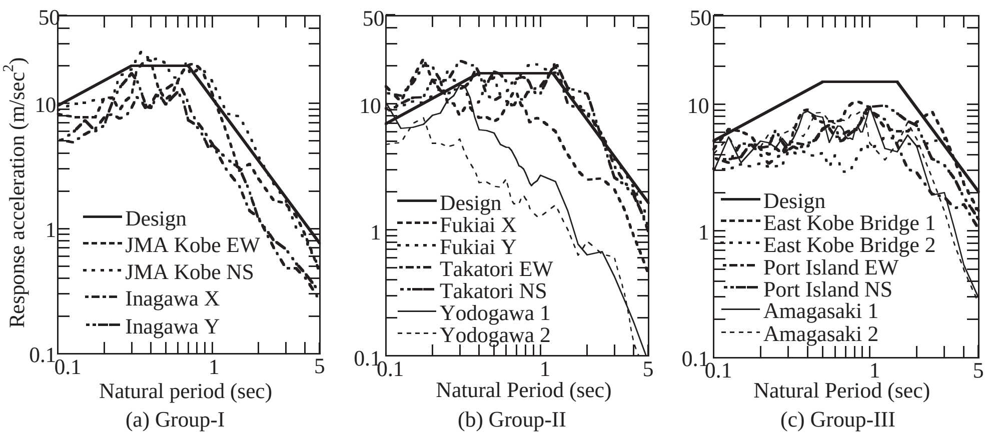


Fig. 2. Type II ground motions determined from 5% damped response accelerations of records from 1995 Kobe earthquake.

in which k_Z = zone factor (1.0, 0.85 and 0.7 based on the region and provided in a map), k_D = damping modification factor, $S_i (i = 1\text{-}9)$ = spectral acceleration (m/s) with 0.05 damping ratio, and $T_i (i = 1\text{-}6)$ = limits of constant response acceleration branch (**Table 2**). Damping modification factor k_D is provided as (Kawashima and Aizawa 1986 [13])

$$k_D = \frac{1.5}{40\xi + 1} + 0.5. \qquad (4)$$

Type I ground motion represents ground motion developed in Tokyo in the 1923 Kanto earthquake (M7.9). Because of the uniquely large number of victims, the Kanto earthquake has been regarded in Japan as a special case. Type I safety evaluation ground motion in Eq. (2) was deterministically evaluated assuming a combination of M = 8 and an epicentral distance = 50 km based on an attenuation equation of response acceleration developed based on the statistical analysis of 394 components of free-field Japanese ground motions (Kawashima et al. 1984 [12]). Other empirical experience was included in establishing Type I safety evaluation ground motion. To represent near-field ground motions, Type II safety evaluation ground motion was included in 1996 (JRA 1996 [11]). Response acceleration of 5% damping ratio of the Type II ground motion was determined from the response accelerations of records from Kobe earthquake as shown in **Fig. 2** (MOC 1995 [20]). An equivalent lateral force coefficient of 0.2-0.3 combined with an allowable stress design approach has been used since 1926.

Although bridge importance was included in design by factoring the design force in terms of the importance factor in previous Specifications, it has been eliminated from the evaluation of design force, because ground motion does not change with bridge importance. As described later, importance is accounted for in the evaluation of the degree of damage accepted in design.

A unique feature of design force in Japan is that elastic response accelerations used for equivalent static analysis includes modification of the period dependence of the damping ratio of bridges (Kawashima, 1994 [14]). In dynamic response analysis, damping ratios of a bridge are assigned by assuming damping ratios for modes. Because damping ratio is not assigned in static analysis, it must be included in design acceleration evaluation. By representing elastic response accelerations for equivalent static analysis as S_s, S_s and elastic response acceleration with a 5% damping ratio are related as

$$S_s(T) = S(T, \xi) = c_D(\xi) \cdot S(T, \xi = 0.05) \qquad (5)$$

Table 3. Values of parameter in Eqs. (7)-(9).

Soil Group	Function Evaluation			Safety Evaluation					
				Type I Ground Motion			Type II Ground Motion		
	S_{s1}(m/s^2)	S_{s2}(m/s^2)	S_{s3}(m/s^2)	S_{s4}(m/s^2)	S_{s5}(m/s^2)	S_{s6}(m/s^2)	S_{s7}(m/s^2)	S_{s8}(m/s^2)	S_{s9}(m/s^2)
I	4.22	1.96	2.09	—	6.86	8.58	43.7	19.6	12.15
II	4.18	2.45	2.92	14.8	8.33	11.37	31.8	17.15	21.85
III	4.21	2.94	3.85	14.8	9.8	15.58	23.3	14.7	25.2

Table 4. Minimum required analysis.

(a) JRA, 1999

Category		Functional Evaluation	Safety Evaluation
Bridges with Simple Response Characteristics		ESA	ISA
Bridges with Complex Response Characteristics	Equivalent Static Analysis is Applicable	ESA and EDA	ISA and IDA
	Equivalent Static Analysis is not Applicable	EDA	IDA

(b) ATC-32, 1996

Importance	Configuration	Functional Evaluation	Safety Evaluation
Ordinary Bridge	Type I	None required	ESA or EDA
	Type II	None required	EDA
Important Bridge	Type I	ESA or EDA	ESA or EDA
	Type II	EDA	EDA, ISA and IDA

in which ξ represents the bridge damping ratio. Based on the forced excitation test on many bridges, it is known that damping ratio ξ of bridges depends on natural period T as (Kuribayashi and Iwasaki, 1972 [18])

$$\xi = 0.02/T. \qquad (6)$$

Substituting Eq. (6) and Eqs. (1)-(3) into Eq. (5) yields elastic response accelerations for equivalent static analysis S_s after re-fitting by straight lines as

Functional Evaluation

$$S_s = k_Z \cdot \begin{cases} S_{s1} \cdot T^{1/3} & 0 < T \le T_1 \\ S_{s2} & T_1 \le T \le T_2 \\ S_{s3}/T^{2/3} & T_2 \le T \end{cases} \qquad (7)$$

Safety Evaluation

$$S_s = k_Z \cdot \begin{cases} S_{s4} T^{1/3} & 0 < T \le T_3 \\ S_{s5} & T_3 \le T \le T_4 \\ S_{s6}/T^{2/3} & T_4 \le T \end{cases} \qquad (8)$$

(Type I GM)

$$S_e = k_Z \cdot \begin{cases} S_{s7} T^{2/3} & 0 < T \le T_5 \\ S_{s8} & T_5 \le T \le T_6 \\ S_{s9}/T^{4/3} & T_6 \le T \end{cases} \qquad (9)$$

(Type II GM)

in which $S_{si}(i = 1\text{-}9)$ = spectral acceleration (m/s) (**Table 3**). Natural periods $T_i(i = 1\text{-}6)$ representing limits of the constant response acceleration range are the same as those in Eqs. (1) and (3), but the gradient in the constant velocity ranges in Eqs. (7)-(9) is smaller than that in Eqs. (1)-(3) by a factor of $T^{1/3}$ due to smaller damping ratio in the constant response velocity range. **Fig. 1(b)** shows response accelerations S_s by Eqs. (8) and (9).

4. Analytical Methods and Design Requirements

Analytical methods are generally classified into "Elastic Static Analysis" (ESA), "Elastic Dynamic Analysis" (EDA), "Inelastic Static Analysis" (ISA), and "Inelastic Dynamic Analysis" (INA). In ESA, the elastic response of a bridge is divided by the response modification factor, and the structural components are sized based on capacity design. The minimum analysis requirements are compared between Japan and ATC in **Table 4**. In ATC, requirements vary with the bridge category, configuration type and evaluation level. Configuration Type I includes bridges with simple response characteristics, including those with continuous superstructures, well-balanced spans, supporting bents with appropriately equal stiffness and insignificant vertical response. Configuration Type II includes bridges with more complex response characteristics that are unlikely to be represented well by ESA, including bridges with intermediate superstructure hinges, irregular configurations, bents of nonuniform stiffness, significant skew or spans likely to be excited by vertical input motion.

In Japan, pushover analysis is used for bridges, particularly for the design of moment resisting frame piers and foundations. Inelastic dynamic response analysis is widely used routinely. Since input data required for pushover analysis and EDA are almost the same as IDA, IDA is easily conducted once data is prepared. Based on advances in personal computers, IDA is being used increasingly.

5. Design of RC Piers

In the 1995 Kobe earthquake, a number of reinforced concrete piers suffered damage due to flexure-shear failure at mid-height as a consequence of premature termination of longitudinal reinforcement. Bar termination was determined based on design moment distribution, without accounting for the effect of tension shift due to diagonal shear cracking. This resulted in a short development length of reinforcements lap spliced at mid-height (JRA 1964 [10], Kawashima and Unjoh 1997 [15]). This practice was revised in 1980 (JRA 1980 [11]). Failure of the 18-spans of the collapsed Hanshin Expressway was triggered by this deficiency (MOC 1995 [20]). Allowable shear stress had been overestimated and the lateral confinement of the core concrete was poor.

Important point is that the vulnerability of reinforced concrete piers associated with the above three deficiencies was not widely recognized prior to the Kobe earthquake, since bridges had not collapsed before that. The Ministry of Construction conducted nation-wide seismic inspections for bridges five times prior to the 1995 Kobe earthquake, i.e., in 1971, 1976, 1979, 1986 and 1991, by gradually expanding normal inspection from deterioration to seismic vulnerability to collapse (JRA 1987 [11], Kawashima and Unjoh 1994 [15]). Flexure-shear failures at pier mid-height due to premature termination of longitudinal reinforcement were first recognized in the 1982 Urakawa-oki earthquake (Narita et al. 1983 [21]). Seismic evaluation for flexure-shear failure was included in the 1991 inspection, and several dozens reinforced concrete piers were retrofitted by steel jacketing in the Metropolitan and Hanshin Expressways prior to the Kobe earthquake. Retrofitted piers performed well in the Kobe earthquake (Sato 1997 [27]), but portions of the Hanshin Expressway damaged in the Kobe earthquake had not yet been retrofitted.

As a consequence of the Kobe earthquake, Design Specifications of Highway Bridges were extensively revised in 1996 (JRA 1996 [11]). The ductility check of reinforced concrete piers included in 1990 Design Specifications was upgraded to "ductility design method" applied to every structural component in which seismic effect is predominant. An ordinary bridge is designed assuming a principal plastic hinge at each pier to meet the following requirements:

$$P_a \geq \frac{S_s \cdot W}{g \cdot R} \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which P_a = lateral capacity of a pier, S_s = elastic response acceleration for the equivalent static analysis by Eqs. (8) and (9), R = response modification factor and W = tributary weight. Assuming the equal energy principle, the response modification factor is assumed to be

$$R = \sqrt{2\mu_a - 1} \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which μ_a = design displacement ductility factor of a pier. Since the maximum μ_a for a single reinforced concrete piers is 8, the response modification factor $R = \sqrt{2\mu_a - 1}$ is smaller than 3.8. Since the design displacement ductility factor is generally 4 to 6 in an ordinary reinforced concrete single pier, the response modification factor is 2.6 to 3.3.

The pier strength and the design ductility factor μ_a are determined based on the failure mode. Flexural strength is evaluated from the standard moment vs. curvature analysis. Fiber-element analysis is conducted for every pier assuming a stress vs. strain relationship for concrete and reinforcing bars. The stress vs. strain relationship for reinforcing bars is idealized by an elastic-perfect plastic model, while the stress vs. strain relation of confined concrete is given as (Hoshikuma et al. 1997 [9])

$$f_c = \begin{cases} E_c \varepsilon_c \left\{ 1 - \dfrac{1}{n} \left(\dfrac{\varepsilon_c}{\varepsilon_{cc}} \right)^{n-1} \right\} & (0 \leq \varepsilon_c \leq \varepsilon_{cc}) \\ f_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} \leq \varepsilon_c \leq \varepsilon_{cu}) \end{cases} \quad \ldots\ldots\ldots\ldots\ldots \quad (12)$$

in which f_{cc} and ε_{cc} = strength of confined concrete and strain corresponding to f_{cc}, E_c = elastic modulus of concrete, E_{des} = gradient at descending branch, and $n = E_c\varepsilon_{cc}/(E_c\varepsilon_{cc} - f_{cc})$. In Eq. (12), f_{cc}, ε_{cc} and E_{des} are provided as

$$f_{cc} = f_{c0} + 3.8\alpha\rho_s f_{sy}; \quad \varepsilon_{cc} = 0.002 + 0.033\beta \frac{\rho_s f_{sy}}{f_{c0}};$$

$$E_{des} = 11.2 \frac{f_{c0}^2}{\rho_s \cdot f_{sy}} \quad \ldots\ldots\ldots\ldots \quad (13)$$

in which f_{c0} = design strength of concrete, f_{sy} = yield strength of reinforcements, α and β = shape factors (α = 1.0 and β = 1.0 for circular piers, and α = 0.2 and β = 0.4 for rectangular piers), and ρ_s = volumetric ratio of tie reinforcements.

The ultimate displacement d_u is defined as displacement at the gravity center of a superstructure when the concrete compression strain at out-most reinforcements reaches the following ultimate strain ε_{cu}:

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{(Type-I ground motion)} \\ \varepsilon_{cc} + 0.2 f_{cc}/E_{des} & \text{(Type-II ground motion)}. \end{cases} \quad \ldots\ldots\ldots\ldots\ldots \quad (14)$$

It should be noted in Eq. (14) that the ultimate concrete strain depends on the type of ground motions. Deterioration of reinforced concrete piers at the plastic hinge region is larger under cyclic loading with large number of inelastic excursions (Type I ground motion) compared to cyclic loading with a smaller number of inelastic excursions (Type II ground motion) (e.g., Takemura and Kawashima, 1997 [29]). Taking account of such ground motion-dependent deterioration of piers, the ultimate concrete compression strain is assumed to be ε_{cc} under the Type I ground motion, while it is assumed to be strain corresponding to stress 20% deteriorating from the maximum stress f_{cc} under Type II ground motion. The ultimate displacement of a pier is larger under the Type II ground motion than the Type I ground motion.

Ultimate displacement d_u of a pier is evaluated from the yield and ultimate curvatures ϕ_y and ϕ_u as (Priestley and Park 1987 [24], Priestley et al. 1996 [25])

$$d_u = d_y + (\phi_u - \phi_y)(h - L_p/2)L_p \quad (15)$$

in which h is the height of the pier and L_p is the plastic hinge length given based on the width of the pier D as:

$$L_p = 0.2h - 0.1D; \quad 0.1D \leq L_p \leq 0.5D. \quad (16)$$

On the other hand, the shear strength of a reinforced concrete pier P_s is evaluated as

$$P_s = V_c + V_s \quad (17)$$

where

$$\begin{aligned} V_c &= k_c \cdot k_e \cdot k_{pt} \cdot v_c \cdot b \cdot h; \\ V_s &= A_w \cdot f_{sy} \cdot d \cdot (\sin\theta + \cos\theta)/(1.15 \cdot a) \end{aligned} \quad (18)$$

in which V_c and V_s = shear strength of concrete and transverse reinforcement, v_c = shear strength of concrete (e.g., 0.35 MPa and 0.37 MPa for concrete with design strength of 24 MPa and 30 MPa, respectively), k_c = cyclic loading effect factor , k_e = effective height factor (e.g., 1.0, 0.7, 0.6 and 0.5 for effective height of < 1 m, 3 m, 5 m and > 10 m, respectively), k_{pt} = modification factor depending on the tension bars ratio p_t (e.g., 0.9, 1.0, 1.2 and 1.5 for tension reinforcement ratio of 0.2%, 0.3%, 0.5% and > 1.0%), b and h = effective width and height, and A_w = area of reinforcing bars with an interval a and an angle θ. The factor k_c in Eq. (18) represents the deterioration of shear strength under cyclic loading. It is recommended to be 1.0, 0.6 and 0.8 under static loading, Type I ground motion and Type II ground motion, respectively. Shear strength of concrete P_s under static loading ($k_c = 1.0$) is denoted as P_{s0}.

Based on flexural strength P_{fu}, shear strength P_s and shear strength under static loading P_{s0}, the failure mode of a pier is classified into flexural failure, shear failure after flexural damage and shear failure as

$$\text{Failure Mode} = \begin{cases} \text{flexural failure} \cdots\cdots & P_u \leq P_s \\ \text{shear failure after flexural damage} \cdots & P_s \leq P_u \leq P_{s0} \\ \text{shear failure} \cdots\cdots & P_{s0} \leq P_u \end{cases} \quad (19)$$

and the lateral capacity P_a and the design displacement ductility factor μ_a in Eqs. (10) and (11) are provided as

$$P_a = \begin{cases} P_u \cdots\cdots \text{flexural failure} + \text{shear failure after flexural damage} \\ P_{s0} \cdots\cdots \text{shear failure} \end{cases} \quad (20)$$

$$\mu_a = \begin{cases} 1 + \dfrac{d_u - d_y}{\alpha \cdot d_y} \cdots\cdots \text{flexural failure} \\ 1 \cdots\cdots \text{shear failure after flexural damage} + \text{shear failure} \end{cases} \quad (21)$$

in which α = safety factor depending on bridge importance and the type of ground motion ($\alpha = 3.0$ and 2.4 for important and ordinary bridges, respectively, under the Type I ground motions, and $\alpha = 1.5$ and 1.2 for important and ordinary bridges, respectively, under the Type II ground motions), d_y and d_u = yield and ultimate displacement of the pier. Larger α-value is assigned to bridges with higher importance under cyclic loading with long duration (Type I ground motion). As a consequence, in addition to the determination of ultimate displacement d_u, the ground motion characteristics are also accounted for in evaluating design displacement ductility factor of a piers μ_a.

A number of reinforced concrete piers that suffered flexural failure at their bases did not collapse, but tilted and were left with large residual displacement at the top. About 100 piers with a tilt angle exceeding 1° (1.75% drift) had to be demolished and new piers were built due to the difficulty of setting superstructures back at original alignments and levels. Residual displacements in piers have been considered to be secondary to maximum ductility demand in seismic design. Residual displacement should be considered independently of maximum ductility because a wide range of residual displacement may occur for the same ductility demand (MacRae and Kawashima, 1997 [19], Kawashima et al. 1998 [16]). Hence, a requirement of residual displacement d_R developed at a pier after an earthquake was first included in 1995 Specifications as

$$d_R \leq d_{Ra} \quad (22)$$

where

$$\begin{aligned} d_R &= c_R(\mu_r - 1)(1 - r)d_y; \\ \mu_r &= 1/2 \cdot [\{S_s \ /(g \cdot P_a)\}^2 + 1] \end{aligned} \quad (23)$$

in which d_{Ra} = allowable residual displacement, r = bilinear factor defined as a ratio between the first stiffness (yield stiffness) and the second stiffness (post-yield stiffness) of a pier, c_R = factor depending on the bilinear factor r, μ_r = response displacement ductility factor of a pier, and d_y = yield displacement of a pier. c_R for reinforced concrete piers was set at 0.6 based on the residual displacement response spectrum (Kawashima et al. 1998 [16]). The d_{Ra} is 1% of the distance between the bottom of the pier and the gravity center of the superstructure (1% drift).

In design details, various requirements for the lateral confinement of concrete were added, e.g., size of tie bars must be 13 mm or larger in diameter, and they are provided in each 150 mm or shorter. In tall piers (> 30 m), it is extended to 300 mm outside 4 times the plastic hinge length, but space should be gradually changed. Cross ties with the same size as main bars are required at spacing of 150 mm or shorter (the same spacing as the tie bars). They are placed at distances of no more than 1 m.

It is interesting to evaluate how much changes were induced due to code revisions. A few bridges were designed based on past and current codes, assuming that only pier size and reinforcements were changed by keeping the superstructure, pier height and soil condition the same. **Fig. 3** compares the section of a 11 m high rein-

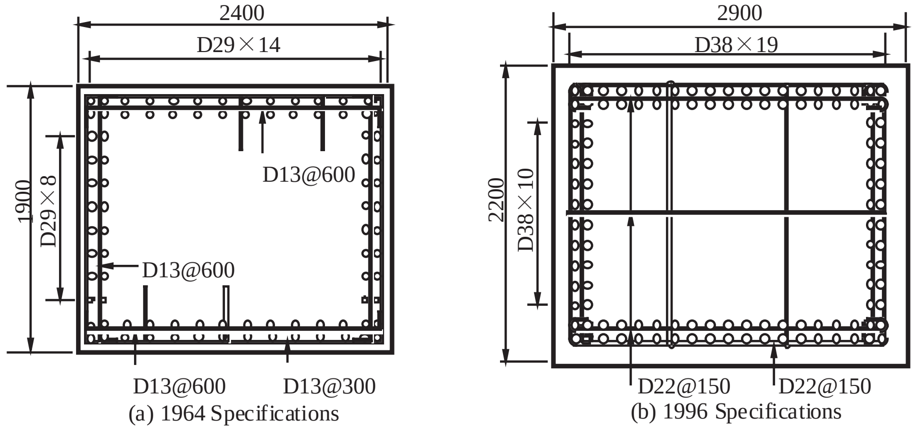


Fig. 3. Comparison of column sections designed based on 1964 and 1996 Design Specifications.

forced concrete pier that supports a part (4.97 MN weight) of a 3-span continuous steel girder bridge 3@40 m long (Yoneda et al. 1999 [32]). Ground condition was Group-II (moderate). The pier which was designed based on 1964 Specifications, used in the design of most bridges that suffered damage in the Kobe earthquake including the collapsed 18-span bridge, is 2.4 m wide (transverse direction) and 1.9 m high (longitudinal direction). It is reinforced by 88 deformed bars with 29 mm diameter (D29) in double (reinforcement ratio $= 1.24\%$), and D13 ties are provided at each 300 mm and 600 mm for outside and inside bars. The volumetric ratio of ties ρ_s is only 0.08% and 0.1% in the longitudinal and transverse directions, respectively. On the other hand, the pier design based on 1996 Design Specifications is 2.9 m wide and 2.2 m high. It is reinforced by 116 D38 bars in double (reinforcement ratio $= 2.07\%$) and D22 ties are provided at each 150 mm for both outside and inside bars. The tie reinforcement ratio ρ_s is thus 1.15% and 1.08% in the longitudinal and transverse directions, respectively.

If the piers designed based on 1964 Specifications are evaluated based on current Specifications, they fail in shear based on Eq. (19), because shear strength based on Eq. (17) is 1.78 MN while flexural strength is 1.87 MN in longitudinal direction. On the other hand, shear and flexural strengths of piers designed based on current Specifications are 11.33 MN and 4.32 MN, respectively in the longitudinal direction. Hence, it fails in flexure. Since the ultimate displacement ductility factor $\mu_u = d_u/d_y$ computed from standard moment-curvature analysis under the Type II ground motion (Type II ground motion was predominant in this example) is 5.4, design displacement ductility factor μ_a is 3.93 from Eq. (21). The natural period of the pier-superstructure system taking account of foundation flexibility is 0.73 second. Since the response acceleration for safety evaluation in Type II at this natural period is 17.5 m/s^2 from Eq. (11), dividing this value by the response modification factor $R = \sqrt{2\mu_a - 1} = 2.62$, the equivalent response acceleration $S_{es} \equiv S_s/R$ becomes 6.7 m/s^2. Since the tributary weight of this pier is 6.15 MN, the lateral force demand is 6.7×6.15 MN$/g = 4.1$ MN. Since the flexural capacity of the pier (4.32 MN) is larger than demand, this pier is safe in the longitudinal direction. Similar evaluation shows that the pier is also safe in the transverse direction. The residual displacement from Eq. (23) is 80 mm and 90 mm in the longitudinal and transverse directions, respectively. They are smaller than the 1% drift (110 mm).

6. Design of Foundations

Pushover analysis was included for the design of foundations under the safety evaluation ground motion in 1996 Specifications. In pushover analysis, a pile foundation is idealized as supported by nonlinear soil springs as shown in **Fig. 4**. In addition to plastic deformation of piles, nonlinear deformation of soils around piles, and uplift and pull-down of piles are taken into account. Evaluation of spring stiffness and strength are presented in the Design Specifications (JRA 1996, JRA 2002 [11]).

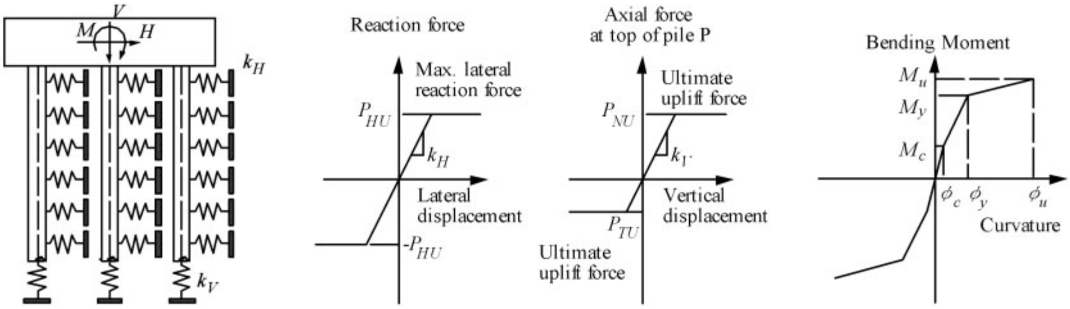
Design lateral force for foundation P_F is evaluated from the lateral capacity of the pier P_a by Eq. (20) as

$$P_F = k_F P_a \quad \text{. . . . . (24)}$$

in which k_F is the over-strength factor ($= 1.1$). Under the design lateral force, foundations are designed so that they do not yield, where the yield of foundations is defined based on the type of foundations. In a pile foundation, yield is defined as the stage when one of the following states occurs first:

- All piles yield.
- Compression force, which applies to piles at the same line, reaches ground strength.

The above conditions result in sudden deterioration of the lateral resistance of pile foundations. Although upward force, which applies to piles, is also important in lateral resistance, it is not included as the stage of yield, because this does not cause critical deterioration due to the dead weight effect.



(a) Analytical model (b) Lateral force vs. lateral displacement relation (c) Vertical force vs. vertical displacement relation (d) Moment vs. curvature relation

Fig. 4. Model of pile foundations under safety evaluation ground motions.

7. Liquefaction and Liquefaction-Induced Ground Movement

Extensive soil liquefaction occurred over a wide area in offshore reclaimed lands and natural deposits in the Kobe earthquake. Liquefaction occurred in not only sandy layers but also gravel layers with averaged grain size over 2 mm. Such gravel layers had been regarded as free from liquefaction. Liquefaction resulted in horizontal ground movement near shorelines. It was found from aerial photographs that liquefaction-induced ground movement reached 3 to 4 m (Hamada et al. 1995 [8]), extensively damaging several bridges (MOC 1996, Tamura et al. 2000 [31]).

After the Kobe earthquake, the evaluation of liquefaction potential and its treatment were extensively revised. The F_L method (Iwasaki 1985) was revised so that it is evaluated under safety evaluation ground motion. The type of ground motion dependency of the shear strength of soil was included. Soils that require the evaluation of liquefaction potential based on the F_L method was extended to include gravel layers; saturated ground within 20 m from the ground surface and where water table is within 10 m from the ground surface; soil layers with fine content FC $<$ 35%, or soils with FC $>$ 35% and the plasticity index I_p <15; and soil layers with $D_{50} < 10$ mm and $D_{10} < 1$ mm, in which D_{50} and D_{10} are the grain size corresponding to 50% and 10% on the accumulation grain size curve.

In the above soil layers, the liquefaction potential factor F_L is evaluated based on cyclic shear stress ratio R and cyclic shear strength ratio L as

$$F_L = R/L \quad (25)$$

where

$$R = c_{GM} R_L; \quad L = r_d k_{hc} \sigma_v / \sigma_v' \quad (26)$$

in which R_L = tri-axial strength ratio of soil, k_{GM} = type of ground motion factor, k_d = acceleration reduction factor in underground ($k_d = 1.0 - 0.015x$ where x is the depth (m)), a_g = acceleration at ground surface in Type I and Type II ground motions, and σ_v and σ_v' = total and effective overburden pressure. a_g is 0.3 g, 0.35 g and 0.4 g at the ground groups I, II and III, respectively, under Type I ground motion, and 0.8 g, 0.7 g and 0.6 g at ground groups I, II and III, respectively, under Type II ground motion. Ground motion dependency of the shear strength of soils is accounted for by k_{GM} as

$$k_{GM} = \begin{cases} 1.0 & \text{(Type-I ground motion)} \\ \left.\begin{array}{ll} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{array}\right\} & \text{(Type-II ground motion).} \end{cases} \quad (27)$$

The tri-axial strength ratio R_L in Eq. (23) is represented as

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (14 \leq N_a) \end{cases} \quad (28)$$

where

$$N_a = \begin{cases} c_1 \dfrac{1.7N}{\sigma_v' + 0.7} + c_2 & \text{(Sand)} \\ \{1 - 0.36\log_{10}(D_{50}/2)\} N_1 & \text{(Gravels)} \end{cases} \quad (29)$$

in which N, N_a and $N_1 = N$ value of standard penetration test, corrected N value accounting for the effect of grain size, and N value corresponding to overburden soil pressure of 0.1 MPa (1 kgf/cm^2), and coefficients c_1 and c_2 are given as

$$c_1 = \begin{cases} 1 & (0 \leq FC < 10\%) \\ (FC + 40)/50 & (10\% \leq FC < 60\%) \\ FC/20 - 1 & (60\% \leq FC) \end{cases}$$
$$c_2 = \begin{cases} 0 & (0\% \leq FC < 10\%) \\ (FC - 10)/18 & (10\% \leq FC) \end{cases} \quad (30)$$

The F_L value is evaluated usually at each 1 m. Soil layers where F_L is less than 1.0 are evaluated as liquefiable under design ground motion. When it is estimated that liquefaction occurs from Eq. (25), both spring stiffness k_{SL} and bearing capacity of soils p_{SL} in liquefying layers are reduced from the original spring stiffness k_S and bearing capacity p_S as

$$k_{SL} = c_L k_S; \quad p_{SL} = c_L p_S \quad (31)$$

Table 5. Soil parameter reduction factor associated with liquefaction c_L in Eq. (31).

F_L value	Depth x from Ground Surface (m)	Cyclic Shear Strength Ratio R	
		$R \leq 0.3$	$R > 0.3$
$F_L \leq 1/3$	$0 \leq x \leq 10$	0	1/6
	$10 < x < 20$	1/3	1/3
$1/3 < F_L \leq 2/3$	$0 \leq x \leq 10$	1/3	2/3
	$10 < x < 20$	2/3	2/3
$2/3 < F_L \leq 1$	$0 \leq x \leq 10$	2/3	1.0
	$10 < x < 20$	1.0	1.0

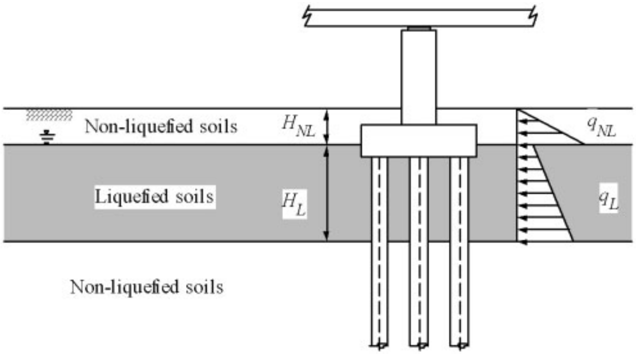


Fig. 5. Treatment of liquefaction-induced ground movement.

in which c_L is the soil parameter reduction factor associated with liquefaction, and is provided based on F_L value, depth and cyclic shear strength ratio R as shown in **Table 5**.

Typical soil profile at sites where bridges suffered damage due to liquefaction-induced ground movement consisted of reclaimed top soil resting on loose sandy layers as shown in **Fig. 5**. Liquefaction occurred in loose sandy layers, while top soil did not liquefy. Top soil moved associated with the lateral movement of the loose sandy layers. Footings were often embedded in top soil, so a large lateral force applied to footings from top soil. Although the mechanism of lateral ground movement is still under investigation, it was found in the Kobe earthquake that it was triggered by revetment failure along the shoreline, and thus the amount of ground movement was reduced as the distance from the failed revetment increased. It was also found that ground movements were larger when the sea bottom was deep from the ground surface.

When ground movement occurs, force associated with ground movement is applied to part of foundations in contact with moving ground. To represent such force, it is appropriate to idealize the foundation as a structure supported by soil springs, and prescribe the movement of ground at the end of each soil spring. In such an analytical mode, it is important how accurately the amount of ground movement can be predicted. Since the evaluation of maximum ground movement was still difficult at this stage, an analytical model in which force associated with ground movement applies to foundations was included in 1996 Design Specifications (JRA 1996 [11]). The foundation is idealized as a structure supported by soil springs. The reduction of soil springs based on Eq. (31), if any, is accounted for in the model. Distributed force applies to part of the foundation in nonliquefying ground (top soil) and liquefying ground as shown in **Fig. 5**. Distributed force which applies to part of foundations in nonliquefying and liquefying soils, q_{NL} (kN/m) and q_L (kN/m), respectively, are provided as

$$q_{NL} = c_s \cdot c_{NL} \cdot K_P \cdot \gamma_{NL} \cdot x \qquad (0 \leq x \leq H_{NL}) \quad . \; (32)$$

$$q_L = c_s \cdot c_L \cdot \{\gamma_{NL} \cdot H_{NL} + \gamma_L \cdot (x - H_{NL})\} \qquad (H_{NL} \leq x \leq H_{NL} + H_L) \quad . \; . \; (33)$$

in which K_P = passive earth-pressure coefficient, γ_{NL} and γ_L = unit weight of nonliquefying and liquefying soils, H_{NL} and H_L = thickness of nonliquefying and liquefying soils, x = depth from the ground surface, c_s = modification factor based on the distance from the shoreline, and c_{NL} and c_L = modification factor for ground movement-induced force in nonliquefying and liquefying soils, respectively. Based on several back-analyses of bridges that suffered damage from ground movement in the Kobe earthquake and shaking table tests, the modification factor c_L was recommended to be 0.3 (MOC 1995 [20], Tamura and Azuma 1997 [30], Tamura et al. 2000 [31]). Based on empirical evaluation of damage in the Kobe earthquake and a series of shaking table tests, $c_s = 1.0$, 0.5 and 0.0 corresponding to $s \leq 50$ m, $50 < s \leq 100$ m and 199 m$< s$, respectively, where s is the distance from the shoreline, and $c_{NL} = 0.0$, $(0.2P_L - 1)/3$ and 1.0 corresponding to $P_L \leq 5$, $5 < P_L \leq 20$ and $20 < P_L$, respectively, where the liquefaction index P_L defined as

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5x)dx. \; . \; . \; . \; . \; . \; . \; (34)$$

8. Concluding Remarks

In the 1994 Northridge and 1995 Kobe earthquakes, bridges designed in accordance with pre-1980 design suffered extensive damage. Extensive progress has been achieved in the last three decades from design based on old concepts that included too much simplifications and assumptions to design based on new concepts. Progress in computer analysis and experimental verification has been led to accumulate scientific knowledge on the inelastic response of bridges. Recent progress in theoretical estimation of near-field ground motions and verifications of design analysis by large-scale testing will further improve seismic design methods. However many problems remain to be studied for inelastic response of bridges with multi plastic hinges, seismic performance of bridges under destructive near-field ground motions of long duration, pounding between decks, and seismic stability under liquefaction-induced lateral ground movement.

References:

[1] AASHTO, "Standard Specifications for Highway Bridges, 16th Edition," American Association of State Highway and Transportation Officials, USA, 1995.

[2] ATC, "Improved Seismic Design Criteria for California Bridges: Provisional Recommendations," ATC 32, Applied Tech. Council, Redwood City, CA, USA, 1996.

[3] I. A. Buckle, "Overview of Seismic Design Methods for Bridges in Different Countries and Future Directions," Paper No.2113, 11 WCEE, Acapulco, Mexico, 1996.

[4] California Department of Transportation, "Bridge Memo to Designers (20-1)-Seismic Design Methodology," Sacramento, CA, USA, 1999.

[5] California Department of Transportation, "Seismic Design Criteria," Ver.1.1, Sacramento, CA, USA, 1999.

[6] H. E. Chapman, "Earthquake Resistant Bridges and Associated Highway Structures: Current New Zealand Practice," Proc. National Seismic Conf. on Bridges and Highways, San Diego, CA, USA, 1995.

[7] European Prestandard, "Eurocode 8 – Design Provisions for Earthquake Resistance of Structures," Brussels, 1994.

[8] M. Hamada, R. Isoyama, and K. Wakamatsu, "The 1995 Hyogo-ken Nanbu Earthquake, Liquefaction, Ground Displacement and Soil Condition in Hanshin Area," Association for Development of Earthquake Prediction, 1995.

[9] J. Hoshikuma, K. Kawashima, K. Nagaya, and A. W. Taylor, "Stress-Strain Model for Confined Reinforced Concrete in Bridge Piers," J. Structural Engineering, ASCE, Vol.123, No.5, pp. 624-633, NY, USA, 1997.

[10] Japan Road Association, "Design of Steel Bridges," Tokyo, Japan, 1964.

[11] Japan Road Association, "Design Specifications of Highway Bridges," Maruzen, Tokyo, Japan, 1980, 1990, 1996, 2002.

[12] K. Kawashima, K. Aizawa, and K. Takahashi, "Attenuation of Peak Ground Motions and Absolute Acceleration Response Spectra," Proc. 8WCEE, Vol.II, pp. 257-264, San Francisco, CA, USA, 1984.

[13] K. Kawashima and K. Aizawa, "Modification of Earthquake Response Spectra with Respect to Damping Ratio," 3rd US National Conference on Earthquake Engineering, Charleston, SC, USA, 1986.

[14] K. Kawashima, "Seismic Design Force of Bridges," Proc. 2nd International Workshop on Seismic Design and Retrofitting of Reinforced Concrete Bridges (Edited by R. Park), pp. 25-56, Queenstown, NZ, 1994.

[15] K. Kawashima and S. Unjoh, "The Damage of Highway Bridges in the 1995 Hyogo-ken Nanbu Earthquake and Its Impact on Japanese Seismic Design," Journal of Earthquake Engineering, Vol.1, No.3, pp. 505-541, IC Press, London, UK, 1997.

[16] K. Kawashima, G. A. MacRae, J. Hoshikuma, and K. Nagaya, "Residual Displacement Response Spectrum," Journal of Structural Engineering, ASCE, Vol.124, No.5, pp. 523-530, NY, USA, 1998.

[17] K. Kawashima, "Seismic Design and Retrofit of Bridges," Key Note Presentation, 12WCEE, Paper No.1818 (CD-ROM), Auckland, New Zealand, 2000.

[18] E. Kuribayashi and T. Iwasaki, "Dynamic Properties of Highway Bridges," Proc. 5WCEE, Roma, Italy, 1972.

[19] G. A. MacRae and K. Kawashima, "Post-earthquake Residual Displacement of Bilinear Oscillators," Earthquake Engineering and Structural Dynamics, Vol.26, pp. 701-716, 1997.

[20] Ministry of Construction, "Report on the Damage of Highway Bridges in the 1995 Hyogo-ken Nanbu Earthquake," Tokyo, Japan, 1995.

[21] N. Narita, M. Murakami, and H. Asanuma, "Earthquake Damage to Shizunai Bridge," 15th Joint Meeting, US-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, Japan, 1983.

[22] R. Park, "New Zealand Perspectives on Seismic Design of Bridges," Paper No.2111, 11WCEE, Acapulco, Mexico, 1996.

[23] P. E. Pinto, "Eurocode 8 Part 2: Earthquake Resistant Design of Bridges," Proc. 1st Japan-Italy Workshop on Seismic Design of Bridges, 72-84, Tech. Memorandum, 3369, Public Works Res. Inst., ISSN 0386-5878, Ministry of Const., Tsukuba Science City, Japan, 1995.

[24] N. M. J. Priestley and R. Park, "Strength and Ductility of Concrete Bridge Columns under Seismic Loading," ACI Structural J., pp. 61-76, 1987.

[25] M. J. N. Priestley, F. Seible, and G. M. Calvi, "Seismic Design and Retrofit of Bridges," John Wiley & Sons, New York, USA, 1996.

[26] J. E. Roberts, "Optimizing Post Earthquake Lifeline System Reliability – Seismic Design Details for Bridges," Proc. International Workshop on Mitigation of Seismic Effects on Transportation Structures (Loh, C. H. et al editors), pp. 282-293, National Center for Research on Earthquake Engineering, Taipei, Taiwan, R.O.C, 1999.

[27] T. Sato, "Seismic Performance of Bridge Piers which were Retrofitted and Unretrofitted in the Hyogo-ken Nanbu Earthquake," Vol.131, No.3, pp. 17-23, Bridges and Foundations, 1997.

[28] T. N. Z., "Bridge Manual," Transit New Zealand, New Zealand, 1994.

[29] H. Takemura and K. Kawashima, "Effect of Loading Hysteresis on Ductility Capacity of Reinforced Concrete Bridge Piers," Proc. Structural Engineering, 43A, pp. 849-858, JSCE, 1997.

[30] K. Tamura and T. Azuma, "Experimental Study of the Effects of Liquefaction-Induced Ground Flow on Bridge Foundation," Proc. 13th US-Japan Bridge Eng. Workshop, US-Japan Panel on Wind and Seismic Effects, UJNR, Tsukuba, Japan, 1997.

[31] K. Tamura, T. Azuma, and T. Hamada, "Seismic Design of Bridge Foundations against Liquefaction-Induced Ground Flow," 12WCEE, Auckland, New Zealand, 2000.

[32] K. Yoneda, K. Kawashima, G. Syoji, and Y. Fujita, "Increase of Seismic Safety of an Overcrossing Associated with Improvement of the Seismic Design Code," Proc. Structural Engineering, 45A, pp. 751-762, JSCE, 1999.

Name:
Kazuhiko Kawashima

Affiliation:
Professor, Department of Civil Engineering, Tokyo Institute of Technology

Address:
O-Okayama, Meguro, Tokyo 152-8552, Japan

Brief Career:

1972- Public Works Research Institute, Ministry of Construction
1975-76 Invited Researcher at Earthquake Engineering Research Center, University of California, Berkeley
1995- Tokyo Institute of Technology

Selected Publications:

• "Stress-Strain Model for Confined Reinforced Concrete in Bridge Piers," ASCE Journal of Structural Engineering, Vol.123-5, pp. 624-633, 1997.
• "Residual Displacement Response Spectrum," ASCE Journal of Structural Engineering, Vol.124-5, pp. 523-530, 1998.

Academic Societies & Scientific Organizations:

• Japan Society of Civil Engineers (JSCE)
• Japan Association of Earthquake Engineering (JAEE)
• American Society of Civil Engineers (ASCE)
• Earthquake Engineering Research Institute (EERI)