THE 1996 SEISMIC DESIGN SPECIFICATIONS OF HIGHWAY BRIDGES

by

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ABSTRACT

This paper presents the revised Seismic Design Specifications of Highway Bridges in 1996 as well as the background of the revision. Damage features of bridges in the 1995 Hyogo-ken nanbu Earthquake is firstly described with emphasis on the lessons learned from the earthquake. Seismic performance levels and design methods as well as the seismic design force introduced in the 1996 Design Specifications are then described. The ductility design methods for reinforced concrete piers, steel piers, foundations, and bearings are also briefly described.

KEY WORDS : Seismic Design
Highway Bridges
Design Codes
Hyogo-ken nanbu Earthquake
Ductility Design Method

1. INTRODUCTION

Highway bridges in Japan had been considered safe even against extreme earthquake such as the Great Kanto Earthquake (M7.9) in 1923, because various past bitter experiences have been accumulated to formulate the seismic design method (Kawashima (1995)). Large seismic lateral force ranging from 0.2g to 0.3g has been adopted in the allowable stress design approach. Various provisions for preventing damage due to instability of soils such as soil liquefaction have been adopted. Furthermore, design detailings including the unseating prevention devices have been implemented.

In fact, reflecting those provisions, number of highway bridges which suffered complete collapse of superstructures was only 15 since 1923 Great Kanto Earthquake. Based on such evidence, it had been regarded that the seismic damage of highway bridges had been decreasing in recent years.

However, the Hyogo-ken nanbu Earthquake of January 17, 1995, exactly one year after the Northridge, California, USA, earthquake, caused destructive damage to highway bridges. Collapse and nearly collapse of superstructures occurred at 9 sites, and other destructive damage occurred at 16 sites (Ministry of Construction, 1995a). The earthquake revealed that there are a number of critical issues to be revised in the seismic design and seismic strengthening of bridges in urban areas.

After the earthquake the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken nanbu Earthquake" (chairman : Toshio IWASAKI, Executive Director, Civil Engineering Research Laboratory) was formulated in the Ministry of Construction to survey the damage and clarify the factors which contributed to the damage.

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On February 27, 1995, the Committee approved the "Guide Specifications for Reconstruction and Repair of Highway Bridges which suffered Damage due to the Hyogo-ken nanbe Earthquake," (Ministry of Construction 1995b) and the Ministry of Construction noticed on the same day that the reconstruction and repair of the highway bridges which suffered damage in the Hyogo-ken nanbu earthquake should be made by the Guide Specifications. It was decided by the Ministry of Construction on May 25, 1995 that the Guide Specifications should be tentatively used in all sections of Japan as emergency measures for seismic design of new highway bridges and seismic strengthening of existing highway bridges until the Design Specifications of Highway Bridges was revised.

In May, 1995, the "Special Sub-Committee for Seismic Countermeasures for Highway Bridges" (chairman : Kazuhiko KAWASHIMA, Professor of the Tokyo Institute of Technology) was formulated in the "Bridge Committee" (chairman : Nobuyuki NARITA, Professor of the Tokyo Metropolitan University), Japan Road Association, to draft the revision of the Design Specifications of Highway Bridges. The Special Sub-Committee drafted the new Design Specifications of Highway Bridges, and after the approval of the Bridges Committee, the Ministry of Construction released it November 1, 1996.

This paper summarizes the damage feature of highway bridges by the Hyogo-ken Nanbu earthquake and the new Design Specifications of Highway Bridges issued in November 1996.

2. DAMAGE FEATURES OF BRIDGES IN THE HYOGO-KEN NANBU EARTHQUAKE

Hyogo-ken Nanbu earthquake was the first earthquake which hit an urban area in Japan since the 1948 Fukui Earthquake. Although the magnitude of the earthquake was moderate (M7.2), the ground motion was much larger than anticipated in the codes. It occurred very close to the Kobe City with shallow focal depth.

Damage was developed at highway bridges on Routes 2, 43, 171 and 176 of the National Highway, Route 3 (Kobe Line) and Route 5 (Bay Shore Line) of the Hanshin Expressway, the Meishin and Chugoku Expressway. Damage was surveyed for all bridges on National Highways, Hanshin Expressways and Expressways in the area where destructive damage occurred. Total number of piers surveyed reached 3,396 (Ministry of Construction, 1995a). **Fig.1** shows Design Specifications referred to in design of the 3,396 piers. Most of piers (bridges) which suffered damage were designed according to the 1964 Design Specifications or older Design Specifications. Although the seismic design methods have been improved and amended several times since 1926 based on damage experience and progress of bridge earthquake engineering, only a requirement for lateral force coefficient was provided in the 1964 Design Specifications or older Specifications.

Fig.2 compares damage of piers on the Route 3 (Kobe Line) and Route 5 (Bay Shore Line) of the Hanshin Expressway. Damage degree was classified as A_s (collapse), A (nearly collapse), B (moderate damage), C (damage of secondary members) and D (minor or no damage). Substructures of the Route 3 and Route 5 were designed with the 1964 Design Specifications and 1980 Design Specifications, respectively. It should be noted in this comparison that the intensity of ground shaking in terms of response spectra was smaller at the Bay Area than the narrow rectangular area where JMA Seismic Intensity was VII (equivalent to Modified Mercalli Intensity of X-XI). The Route 3 was located in the narrow rectangular area while the Route 5 was located in the Bay Area. Keeping in mind such difference of ground motion, it is apparent in **Fig.2** that about 14% of the piers on Route 3 suffered A_s or A damage while no such damage was developed in the piers on the Route 5.

Although damage concentrated on the bridges designed with the older Design Specifications, it was thought that essential revision was required even in the recent Design Specifications to prevent damage against destructive earthquakes such as the Hyogo-ken nanbu earthquake. The main points requiring modifications were;
(1) it was required to increase lateral capacity and ductility of all structural components in which seismic force is predominant so that ductility of a total bridge system be enhanced. For such purpose, it was required to upgrade the "Check of Ductility of Reinforced Concrete Piers," which has been used since 1990, to a "Ductility Design Method," and to apply the Ductility Design Method to all structural components. It should be noted here that "check" and "design" is different; the check is only to verify the safety of a structural member designed by other design method, and is effective only to increase the size or reinforcements if required, while the design is an essential procedure to determine the size and reinforcements,
(2) it was required to include the ground motion developed at Kobe in the earthquake as a design force in the Ductility Design Method,
(3) it was required to specify input ground motions in terms of acceleration response spectra for dynamic response analysis more actively,
(4) it was required to increase tie reinforcements and to introduce intermediate ties for increasing ductility of piers. It was decided not to terminate main reinforcements at mid-height for preventing premature shear failure, in principle,
(5) it was suggested to adopt multi-span continuous bridge for increasing number of indeterminate of a total bridge system,
(6) it was suggested to adopt rubber bearings for absorbing lateral displacement between a superstructure and substructures. It was important to consider correct mechanism of force transfer from a superstructure to substructures,
(7) it was suggested to include the Menshin design (seismic isolation),
(8) it was required to increase strength, ductility and energy dissipation capacity of unseating prevention devices, and
(9) it was required to consider the effect of lateral spreading associated with soil liquefaction in design of foundations at the site vulnerable to lateral spreading.

3. BASIC PRINCIPLE OF SEISMIC DESIGN

Table 1 shows the seismic performance level provided in the revised Design Specifications in 1996. The bridges are categorized into two groups depending on their importance; standard bridges (Type-A bridges) and important bridges (Type-B bridges). Seismic performance level depends on the importance of bridges. For moderate ground motions induced in the earthquakes with high probability to occur, both A and B bridges should behave in an elastic manner without essential structural damage. For extreme ground motions induced in the earthquakes with low probability to occur, the Type-A bridges should prevent critical failure, while the Type-B bridges should perform with limited damage .

In the Ductility Design Method, two types of ground motions must be considered. The first is the ground motions which could be induced in the plate boundary-type earthquakes with magnitude of about 8. The ground motion at Tokyo in the 1923 Kanto Earthquake is a typical target of this type of ground motion. The second is the ground motion developed in earthquakes with magnitude of about 7-7.2 at very short distance. Obviously the ground motions at Kobe in the Hyogo-ken nanbu earthquake is a typical target of this type of ground motion. The first and the second ground motions are called as Type-I and Type-II ground motions, respectively. The recurrence time of the Type-II ground motion may be longer than that of the Type-I ground motion, although the estimation is very difficult.

4. DESIGN METHODS

Bridges are designed by both the Seismic Coefficient Method and the Ductility Design Method as shown in **Fig.3**. In the Seismic Coefficient Method, a lateral force coefficient ranging from 0.2 to 0.3 has been used based on the allowable stress design approach. No change was introduced since the 1990 Specifications in the Seismic Coefficient Method.

In the Ductility Design Method, assuming a principle plastic hinge formed at the bottom of pier as shown in **Fig.4(a)** and the equal energy assumption, a bridge is designed so that the following requirement is satisfied.

$$P_a > k_{he} W \qquad (1)$$

where

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu_a - 1}} \qquad (2)$$

$$W = W_U + c_P W_P \qquad (3)$$

in which, P_a = lateral capacity of a pier, k_{he} = equivalent lateral force coefficient, W = equivalent weight, k_{hc} = lateral force coefficient, μ_a = allowable displacement ductility factor of a pier, W_U = weight of a part of superstructure supported by the pier, W_P = weight of a pier, and c_P = coefficient depending on the type of failure mode. The c_P is 0.5 for a pier in which either flexural failure or shear failure after flexural cracks are developed, and 1.0 for a pier in which shear failure is developed. The lateral capacity of a pier P_a is defined as a lateral force at the gravity center of a superstructure.

In the Type-B bridges, residual displacement developed at a pier after an earthquake must be checked as

$$\delta_R < \delta_{Ra} \qquad (4)$$

where

$$\delta_R = c_R(\mu_R - 1)(1-r)\delta_y \qquad (5)$$

$$\mu_R = 1/2\{(k_{hc} \cdot W/P_a)^2 + 1\} \qquad (6)$$

in which δ_R = residual displacement of a pier after an earthquake, δ_{Ra} = allowable residual displacement of a pier, 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 ductility factor of a pier, and δ_y = yield displacement of a pier. The δ_{Ra} should be 1/100 of a distance between the bottom of a pier and a gravity center of a superstructure.

In a bridge with complex dynamic response, the dynamic response analysis is required to check the safety of a bridge after it is designed by the Seismic Coefficient Method and the Ductility Design Method. Because this is only for a check of the design, the size and reinforcements of structural members once determined by the Seismic Coefficient Method and the Ductility Design Methods can only be increased if necessary. It should be noted however that under the following conditions in which the Ductility Design Method is not directly applied, the size and reinforcements can be determined based on the results f a dynamic response analysis as shown in **Fig.3**. The conditions when the Ductility Design Method should not be directly used include:

(1) principle mode shapes which contribute to bridge response are different from the ones assumed in the Ductility Design Methods,

(2) more than two modes significantly contribute to bridge response,

(3) principle plastic hinges form at more than two locations, or principle plastic hinges are not known where to be formed, and

(4) response modes for which the equal energy assumption are not applied.

In the seismic design of a foundation, a lateral force equivalent to the ultimate lateral capacity of a pier P_u is assumed to be a design force as

$$k_{hp} = c_{df} \, P_u/W \quad (7)$$

in which k_{hp} = lateral force coefficient for a foundation, c_{df} = modification coefficient (=1.1), and W = equivalent weight by Eq.(3). Because the lateral capacity of a wall-type pier is very large in transverse direction, the lateral seismic force evaluated by Eq.(7) becomes in most cases excessive. Therefore if a foundation has sufficiently large lateral capacity compared with the lateral seismic force, the foundation is designed assuming a plastic hinge at the foundation and surrounding soils as shown in **Fig.4(c)**.

5. DESIGN SEISMIC FORCE

Lateral force coefficient k_{hc} in Eq.(2) is given as

$$k_{hc} = c_z \cdot k_{hc0} \quad (8)$$

in which c_z = modification coefficient for zone, and is 0.7, 0.85 and 1.0 depending on zone, and k_{hc0} = standard modification coefficient. **Table 2** and **Fig.5** show the standard lateral force coefficients k_{hc0} for the Type-I and the Type-II ground motions. The Type-I ground motions have been used since 1990 (1990 Specifications), while the Type-II ground motions were newly introduced in the 1996 Specifications. It should be noted here that the k_{hc0} at stiff site (Group I) has been assumed smaller than the k_{hc0} at moderate (Group II) and soft soil (Group III) sites in the Type-I ground motions as well as the seismic coefficients used for the Seismic Coefficient Method. The Type-I ground motions were essentially estimated from an attenuation equation for response spectra that was derived from a statistical analysis of 394 components of strong motion records. Although the response spectral accelerations at short natural period are larger at stiff sites than at soft soil sites, the tendency has not been explicitly included in the past. This was because damage has been more developed at soft sites than at stiff sites. To consider such fact, the design force at stiff sites has been assumed smaller than that at soft sites even at short natural period. However being different from such a traditional consideration, the Type-II ground motions were determined by simply taking envelops of response accelerations of major strong motions recorded at Kobe in the Hyogo-ken nanbu Earthquake. It was considered appropriate to set realistic ground motions.

Although the acceleration response spectral intensity at short natural period is higher in the Type-II ground motions than in the Type-I ground motions, the duration of extreme accelerations excursion is longer in the Type-I ground motions than the Type-II ground motions. As will be described later, such a difference of the duration has been taken into account to evaluate the allowable displacement ductility factor of a pier.

6. EVALUATION OF DISPLACEMENT DUCTILITY FACTOR OF A REINFORCED CONCRETE PIER

(1) Evaluation of Failure Mode

In the ductility design of reinforced concrete piers, the failure mode of the pier is evaluated as the first step. Failure modes is categorized to 3 types based on the bending capacity and shear capacity of the pier as

1) $P_u \leqq P_s$: bending failure
2) $P_s < P_u \leqq P_{s0}$: bending to shear failure
3) $P_{s0} < P_u$: shear failure

in which P_u = bending capacity, P_s = shear capacity in consideration of the effect of cyclic loading, and P_{s0} = shear capacity without consideration of the effect of cyclic loading.

The ductility factor and capacity of the reinforced concrete piers are determined according to the failure mode as described later.

(2) Displacement Ductility Factor

Th allowable displacement ductility factor of a pier μ_a in Eq.(2) is evaluated as

$$\mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y} \quad (9)$$

in which α = safety factor, δ_y = yield displacement of a pier, and δ_u = ultimate displacement of a pier. As well as the lateral capacity of a pier P_a in Eq.(1), the δ_y and δ_u are defined at the gravity center of a superstructure. In a reinforced concrete single pier as shown in **Fig.4(a)**, the ultimate displacement δ_u is evaluated as

$$\delta_u = \delta_y + (\phi_u - \phi_y) L_P(h - L_P/2) \quad (10)$$

in which ϕ_y = yield curvature of a pier at bottom, ϕ_u = ultimate curvature of a pier at bottom, h = height of a pier, and L_P = plastic hinge length of a pier. The plastic hinge length is given as

$$L_P = 0.2h - 0.1D \ (0.1D \leqq L_P \leqq 0.5D) \quad (11)$$

in which D is a width or a diameter of a pier.

The yield curvature ϕ_y and ultimate curvature ϕ_u in Eq.(10) are evaluated assuming a stress-strain relation of reinforcements and concrete as shown in **Fig.6**. The stress σ_c - strain ε_c relation of concrete with lateral confinement is assumed as

$$\sigma_c = \begin{cases} E_c \varepsilon_c \{1 - \dfrac{1}{n} (\dfrac{\varepsilon_c}{\varepsilon_{cc}})^{n-1}\} & (0 \leqq \varepsilon_c \leqq \varepsilon_{cc}) \\ \sigma_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{cu}) \end{cases} \quad (12)$$

$$n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - \sigma_{cc}} \quad (13)$$

in which σ_{cc} = strength of confined concrete, E_c = elastic modules of concrete, ε_{cc} = strain at maximum strength, and E_{des} = gradient at descending branch. In Eq.(12), σ_{cc}, ε_{cc} and E_{des} are determined as

$$\sigma_{cc} = \sigma_{ck} + 3.8 \alpha \rho_s \sigma_{sy} \quad (14)$$

$$\varepsilon_{cc} = 0.002 + 0.033 \beta \frac{\rho_s \sigma_{sy}}{\sigma_{ck}} \quad (15)$$

$$E_{des} = 11.2 \frac{\sigma_{ck}^2}{\rho_s \sigma_{sy}} \quad (16)$$

in which σ_{ck} = design strength of concrete, σ_{sy} = yield strength of reinforcements, α and β = coefficients depending on shape of pier (α=1.0 and β=1.0 for a circular pier, and α = 0.2 and β = 0.4 for a rectangular pier), and ρ_s = tie reinforcement ratio defined as

$$\rho_s = \frac{4A_h}{sd} \leqq 0.018 \quad (17)$$

in which A_h = area of tie reinforcements, s = space of tie reinforcements, and d = effective width of tie reinforcements.

The ultimate curvature ϕ_u is defined as a curvature when concrete strain at longitudinal reinforcing bars in compression reaches an ultimate strain ε_{cu} defined as

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{for Type I ground motions} \\ \varepsilon_{cc} + \dfrac{0.2 \sigma_{cc}}{E_{des}} & \text{for Type II ground motions} \end{cases} \quad (18)$$

It is important to note here that the ultimate strain ε_{cu} depends on the types of ground motions; the ε_{cu} for the Type-II ground motions is larger than that for the Type-I ground motions. Based on a loading test, it is known that a certain level of failure in a pier such as a sudden decrease of lateral capacity occurs at smaller lateral displacement in a pier subjected to a loading hysteresis with more number of load reversals. To reflect such a fact, it was decided that the ultimate strain ε_{cu} should be evaluated by Eq.(18), depending on the type of ground motions. Therefore, the allowable ductility factor μ_a depends on the type of ground motions; the μ_a is larger in a pier subjected to the Type-II ground motions than a pier subjected to the Type-I ground motions.

It should be noted that the safety factor α in Eq.(9) depends on the type of bridges as well as the type of ground motions as shown in **Table 3**. This is to preserve higher seismic safety in the important bridges, and to take account of the difference of recurrent time between the Type-I and the Type-II ground motions.

(3) Shear Capacity

Shear capacity of reinforced concrete piers is evaluated by a conventional method as

$$P_s = S_c + S_s \tag{19}$$

$$S_c = 10\, c_c\, c_e\, c_{pt}\, \tau_c\, b\, d \tag{20}$$

$$S_s = \frac{A_w\, \sigma_{sy}\, d\,(\sin\theta + \cos\theta)}{10 \times 1.15a} \tag{21}$$

in which P_s = shear capacity, S_c = shear capacity shared by concrete, S_s = shear capacity shared by tie reinforcements, τ_c = shear stress capacity shared by concrete, c_c = modification factor for cyclic loading (0.6 for Type-I ground motions, 0.8 for Type-II ground motions), c_e = modification factor for scale effect of effective width, c_{pt} = modification factor for longitudinal reinforcement ratio, b, d = width and height of section, A_w = sectional area of tie reinforcement, σ_{sy} = yield strength of tie reinforcement, θ = angle between vertical axis and tie reinforcement, and a = spacing of tie reinforcement.

The modification factor on scale effect of effective width, c_e, was based on the experimental study of loading tests of beams with various effective height and was newly introduced in the 1996 Specifications. **Table 4** shows the modification factor on scale effect.

(4) Arrangement of Reinforcement

Fig.7 shows suggested arrangement of tie reinforcement. Tie reinforcement should be deformed bars with a diameter equal or larger than 13 mm, and it should be placed in most bridges at a distance of no longer than 150mm. In special cases such as the bridges with pier height taller than 30m, the distance of tie reinforcement may be increased at height so that pier strength should not be sharply decreased at the section. Intermediate ties should be also provided with the same distance with the ties to confine the concrete. Space of the intermediate ties should be less than 1m.

(5) Two-Column Bent

To determine the ultimate strength and ductility factor for two-column bents, it is modeled as the frame model with plastic hinges at the both end of lateral cap beam and columns as shown in **Fig.8**. Each elastic frame member has the yield stiffness which is obtained based on the axial load by the dead load of the superstructure and the column. The plastic hinge is assumed to be placed at the end part of a cap beam and the top and bottom part of each column. The plastic hinges are modeled as spring elements with bilinear moment-curvature relation. The location of plastic hinges is half distance of the plastic hinge length off from the end edge of each member, where plastic hinge length L_P is assumed to be Eq.(11).

When the two-column bent is subjected to the lateral force in the transverse direction, axial force developed in the beam and columns is affected by the aoolied lateral force. Therefore, the horizontal force-displacement relation is obtained through the static push-over analysis considering axial force N - moment M interaction relation. The ultimate state of each plastic hinges is obtained by the ultimate plastic angle θ_{pu} as

$$\theta_{pu} = (\phi_u/\phi_y - 1)\, L_P\, \phi_y \tag{22}$$

in which ϕ_u = ultimate curvature and ϕ_y = yield curvature.

The ultimate state of the whole two-bent column is determined so that all 4 plastic hinges developed reach the ultimate plastic angle.

7. EVALUATION OF DISPLACEMENT DUCTILITY OF A STEEL PIER

(1) Basic Concept

To improve seismic performance of a steel piers, it is important to avoid specific brittle failure modes. **Fig.9** shows the typical brittle failure mode for rectangular and circular steel piers. The followings are the countermeasures to avoid such brittle failure modes and to improve seismic performance of steel piers:

1) fill the steel column with concrete

2) improve structural parameters related to buckling strength
- decrease the width/thickness ratio of stiffened palates of rectangular piers or the diameter/thickness ratio of steel pipes
- increase the stiffness of stiffeners
- reduce the diaphragm spacing
- strengthen corners using the corner plates

3) improve welding section at the corners of rectangular section

4) eliminate welding section at the corners by using round corners

(2) Concrete Infilled Steel Pier

In a concrete infilled steel pier, the lateral capacity P_a and the allowable displacement ductility factor μ_a in Eqs.(1) and (2) are evaluated as

$$P_a = P_y + \frac{P_u - P_y}{\alpha} \tag{23}$$

$$\mu_a = \left(1 + \frac{\delta_u - \delta_y}{\alpha\,\delta_y}\right)\frac{P_u}{P_a} \tag{24}$$

in which P_y and P_u = yield and ultimate lateral capacity of a pier, δ_y and δ_u = yield and ultimate displacement of a pier, and α = safety factor (refer to **Table 3**). The P_a and the μ_a are evaluated idealizing that a concrete infilled steel pier resists flexural moment and shear force as a reinforced concrete pier. It is assumed in this evaluation that the steel section be idealized as reinforcing bars and that only steel section resists axial force. A stress vs. strain relation of steel and concrete as shown in **Fig.10** is assumed. The height of infilled concrete has to be decided so that bucking is not developed above the infilled concrete.

(3) Steel Pier without Infilled Concrete

A steel pier without infilled concrete must be designed with the dynamic response analysis. Properties of the pier need to be decided based on a cyclic loading test. Arrangement of stiffness and welding at corner must be precisely evaluated so that brittle failure should be avoided.

8. DYNAMIC RESPONSE ANALYSYS

Dynamic response analysis is required in the bridges with complex dynamic response to check the safety factor of the static design. Dynamic response analysis is alos required as a "design" tool in the bridges for which the Ductility Design Method is not directly applied. In dynamic response analysis, ground motins which are spectral fitted to the following response spectra are used;

$$S_{I} = c_Z \cdot c_D \cdot S_{I0} \tag{25}$$

$$S_{II} = c_Z \cdot c_D \cdot S_{II0} \tag{26}$$

in which S_I and S_{II} = acceleration response spectrum for Type-I and Tyoe-II ground motions, S_{I0} and S_{II0} = standard acceleration response spectrum for Type-I and Type-II ground motions, respectively, c_Z = modification coefficient for zone (refer to Eq.(8)), and c_D = modification coefficient for damping ratio given as

$$c_D = \frac{1.5}{40h_i + 1} + 0.5 \tag{27}$$

Table 5 and **Fig.11** show the standard acceleration response spectra (damping ratio h=0.05) for the Type-I and Type-II ground motions.

It is recommended to use at least three ground motions per analysis, and take an average to evaluate the response.

In the dynamic analysis, modal damping ratios have to be carefully evaluated. To determine themodal damping aratios, a bridge may be divided into several sub-strucctures in which energy dissipating mechanism is essentially the same. If one can specify a damping ratio of each sub-structure for a given mode shape, the modal damping ratio for i-th mode, h_i, may be evaluated as

$$h_i = \frac{\sum_{j=1}^{n} \phi_{ij}^{T} \cdot h_{ij} \cdot K_j \cdot \phi_{ij}}{\Phi_i^{T} \cdot K \cdot \Phi_i} \tag{28}$$

in which h_{ij} = damping ratio of j-th substructure in i-th mode, ϕ_{ij} = mode vector of j-th substructure in i-th mode, k_j = stiffness matrix of j-th substructure, K= stiffness matrix of a bridge, and Φ_i = mode vector of a bridge in i-th mode, and is given as

$$\Phi_i^T = \{\phi_{i1}^T, \phi_{i2}^T, \cdots\cdots, \phi_{in}^T\} \quad (29)$$

Table 6 shows recommended damping ratios for major structural components.

9. MENSHIN DESIGN

(1) Basic Principle

Implementation of the Menshin bridges should be carefully decided from not only seismic performance but function for traffic and maintenance point of view, based on the advantage and disadvantage of increasing natural period The Menshin design should not be adopted at the following conditions;

1) sites vulnerable to lose bearing capacity due to the soil liquefaction and the lateral spreading,
2) bridges supported by flexible columns,
3) soft soil sites where potential resonance with surrounding soils could be developed by increasing the fundamental natural period, and
4) bridges with uplift force at bearings.

It is suggested that the design should be made with an emphasis on an increase of energy dissipating capability and a distribution of lateral force to as many substructures as possible. To concentrate the hysteretic deformation at not piers but bearings, the fundamental natural period of a Menshin bridge should be about 2 times or longer than the fundamental natural period of the same bridge supported by the conventional bearings. It should be noted that an elongation of natural period aiming to decrease the lateral force should not be attempted.

(2) Design Procedure

Menshin bridges are designed by both the Seismic Coefficient Method and the Ductility Design Method. In the Seismic Coefficient Method, no reduction of lateral force from the conventional design is mae.

In the Ductility Design Method, the equivalent lateral force coefficient k_{hem} in the Menshin design is evaluated as

$$k_{hem} = \frac{k_{hcm}}{\sqrt{2\mu_m - 1}} \quad (30)$$

$$k_{hcm} = c_E \cdot k_{hc} \quad (31)$$

in which k_{hcm} = lateral force coefficient in menshin design, μ_m = allowable ductility factor of a pier, c_E = modification coefficient for energy dissipating capability (refer to **Table** 7), and k_{hc} = lateral force coefficient by Eq.(8). Because the k_{hc} is the lateral force coefficient for a bridge supported by the conventional bearings, Eq.(31) means that the lateral force in the Menshin design can be reduced, as large as 30%, by the modification coefficient c_E depending on the modal damping ratio of a bridge.

Modal damping ratio of a menshin bridge h for the fundamental mode is computed as Eq.(32). In Eq.(32), h_{Bi} = damping ratio of i-th damper, h_{Pi} = damping ratio of i-th pier or abutment, h_{Fui} = damping ratio of i-th foundation associated with translational displacement, $h_{F\theta i}$ = damping ratio of i-th foundation associated with rotational displacement, K_{Pi} = equivalent stiffness of i-th pier or abutment, K_{Fui} = translational stiffness of i-th foundation, $K_{F\theta i}$ = rotational stiffness of i-th foundation, u_{Bi} = design displacement of i-th Menshin device, and H = distance from a bottom of pier to a gravity center of a deck.

In the Menshn design, the allowable displacement ductility factor of a pier μ_m in Eq.(30) is evaluated by

$$h = \frac{\Sigma K_{Bi} \cdot u_{Bi}^2 \left(h_{Bi} + \frac{h_{Pi} \cdot K_{Bi}}{K_{Pi}} + \frac{h_{Fui} \cdot K_{Bi}}{K_{Fui}} + \frac{h_{F\theta i} \cdot K_{Bi} \cdot H^2}{K_{F\theta i}}\right)}{\Sigma K_{Bi} \cdot u_{Bi}^2 \left(1 + \frac{K_{Bi}}{K_{Pi}} + \frac{K_{Bi}}{K_{Fui}} + \frac{K_{Bi} \cdot H^2}{K_{F\theta i}}\right)} \quad (32)$$

$$\mu_m = 1 + \frac{\delta_u - \delta_y}{\alpha_m \delta_y} \qquad (33)$$

in which α_m is a safety factor used in Menshin design, and is given as

$$\mu_m = 2\alpha \qquad (34)$$

where α is the safety factor in the conventional design (refre to **Table 3**). Eq.(34) means that the allowable displacement ductility factor in the menshin design μ_m should be smaller than the allowable displacemnent ductility factor μ_a by Eq.(2) in the conventional design. The reason for the smaller allowable ductility factor in the menshin design is to limit the hysteretic behavior of a pier at the plastic hinge zone so that principle hysteretic behavior occurs at the menshin devices as shown in **Fig.4(b)**.

(3) Design of Menshin Devices

Simple devices stable against extreme earthquakes have to be used. The bearings have to be anchored to a deck and substructures with bolts, and should be replaceable. The clearance has to be provided between a deck and an abutment or between adjacent decks.

Isolators and dampers must be designed for a desired design displacement u_B. The design displacement u_B is evaluated as

$$u_B = \frac{k_{hem} W_U}{K_B} \qquad (35)$$

in which k_{hem} = equivalent lateral force coefficient by Eq.(31), K_B = equivalent stiffness, and W_U = dead weight of a superstructure. It should be noted that because the equivalent lateral force coefficient k_{hem} depends on the type of ground motions, the design displacement u_B also depends on it.

The equivalent stiffness K_B and equivalent damping ratio h_B of a Menshin device are evaluated as

$$K_B = \frac{F(u_{Be}) - F(-u_{Be})}{2u_{Be}} \qquad (36)$$

$$h_B = \frac{\Delta W}{2\pi W} \qquad (37)$$

$$u_{Be} = c_B \cdot u_B \qquad (38)$$

in which F(u) = restoring force of a device at a displacement u , u_{Be} = effective design displacement, ΔW = energy dissipated per cycle, W = elastic strain energy, and c_B = coefficient to evaluate effective displacement (=0.7).

10. DESIGN OF FOUNDATION

Th evaluation methods of ductility and strength of foundations such as pile foundations and caisson foundations was newly introduced in the 1996 Specifications.

In a pile foundation, a foundation is so idealized that a rigid footing is supported by piles which are supported by soils. The flexural strength of a pier defined by Eq.(7) shall be applied as a seismic force to foundations at the bottom of the footing together with the dead weight superstructure, pier and soils on the footing. **Fig.12** shows the idealized nonlinear model of a pile foundation. The nonlinearity of soils and piles is considered in the analysis.

The safety of the foundation shall be checked so that 1) the foundation shall not reach the yield point of a foundation, 2) if the primary nonlinearity is developed in the foundations, the response displacement shall be less than displacement ductility limit, and 3) the displacement developed in the foundation shall be less than allowable limit. The allowable ductility and allowable limit of displacement were commented as 4 in displacement ductility, 40cm in horizontal displacement and 0.025rad in rotation angle.

For a caisson type foundation, the foundation is modeled as a reinforced concrete column which is supported by soil spring model and the safety is checked in the same way as the pile foundations.

11. DESIGN AGAINST SOIL LIQUEFACTION AND LIQUEFACTION-INDUCED GROUND

FLOW

(1) Estimation of Liquefaction Potential

Since the Hyogo-ken nanbu Earthquake of 1995 caused liquefaction even at coarse sand or gravel layers which had been regarded invulnerable to liquefy, a gravel layer was included in the soil layers that require liquefaction potential estimation. Soil layers that satisfies the following conditions is estimated to be potential liquefaction layers:

1) saturated soil layer which is located within 20m deep under the ground surface and in which ground water level is within 10m deep.
2) soil layer in which fine particle content ratio FC is equal or less than 35% or plasticity index I_P is equal or less than 15.
3) soil layer in which mean grain size D_{50} is equal or less than 10mm and 10% grain size D_{10} is equal or less than 1mm.

Liquefaction potential is estimated by the safety factor against liquefaction F_L as

$$F_L = R/L \tag{35}$$

where, F_L = liquefaction resistant ratio, R = dynamic shear strength ratio and L = share stress ratio during an earthquake. The dynamic shear strength ratio R may be expressed as

$$R = c_w R_L \tag{36}$$

where, c_w = corrective coefficient for ground motion characteristics (1.0 for Type-I ground motions, 1.0-2.0 for Type-II ground motions), and R_L = cyclic triaxial strength ratio. The cyclic triaxial strength ratio was estimated by laboratory tests with undisturbed samples by in-situ freezing method.

The shear stress ratio during an earthquake may be expressed as

$$L = r_d k_{hc} \sigma_v / \sigma_v' \tag{37}$$

where, r_d = modification factor shear stress ratio with depth, k_{hc} = design seismic coefficient for the evaluation of liquefaction potential, σ_v = total loading pressure, σ_v' = effective loading pressure.

It should be noted here that the design seismic coefficient for the evaluation of liquefaction potential k_{hc} is ranging from 0.3 to 0.4 for Type-I ground motions, and from 0.6 to 0.8 for Type-II ground motions.

(2) Design Treatment of Liquefaction for Bridge Foundations

When the liqeufaction occurs, the strength and the bearing capacity of a soil decreases. In the seismic design of highway bridges, soil constants of a sandy soil layer which is judged liable to liquefy are reduced according to the F_L value. The reduced soil constants are calculated by multiplying the coefficient D_E in **Table 8** to the soils constants estimated on an assumption that the soil layer does not liquefy.

(3) Design Treatment of Liqeufaction-Induced Ground Flow for Bridge Foundations

When the liquefaction-induced ground flow that may affect bridge seismicity is likely to occur, this influence was included in the revised Design Specifications in 1996. The case in which the ground flow that may affect bridge seismicity is likely to occur is generally that the ground is judged to be liquefiable and is exposed to biased earth pressure, for example, the ground behind a seaside protection wall. The effect of liquefaction-induced ground flow is considered as the static force acting on structure. This method premises that the surface soil is of the non-liqeufiable and liquefiable layers, and the forces equivalent to the passive earth pressure and 30% of the overburden pressure are applied to the structure in the non-liquefiable layer and liquefiable layer, respectively.

The seismic safety of a foundation is checked by confirming the displacement at the top of foundation caused by ground flow does not exceed an allowable value, in which a foundation and the ground are idealized as shown in **Fig.12**. The allowable displacement of a foundation may be taken as two times the yield displacement of a foundation. In this process, the inertia force of structure is not necessary to be considered simultaneously, because the liquefaction-induced ground flow may take place after the principle ground motion.

12. BEARING SUPPORTS

The bearings are classified into two groups; the first is the bearings which resist the seismic force of Eq.(2), and the second is the bearings which resist the seismic force considered in the Seismic Coefficient Methȯd. The first and the second bearings are called as the Type-B bearings and the Type-A bearings, respectively. Seismic performance of the Type-B bearings is, of course, much higher than the Type-A bearings. In the Type-A bearings, a displacement limiting device, which will be described later, has to be co-installed in both longitudinal and transverse directions, while it is not required in the Type-B bearings. Because of the importance of bearings as one of the main structural components, the Type-B bearings should be used in the menshin bridges.

The uplift force applied to the bearing supports is specified as

$$R_U = R_D - \sqrt{R_{heq}^2 + R_{veq}^2} \qquad (38)$$

in which R_U = design uplift force applied to the bearing support, R_D = dead load of superstructure, R_{heq} and R_{veq} are vertical reactions caused by the horizontal seismic force and vertical force, respectively. **Fig.13** shows the design forces for the bearing supports.

13. UNSEATING PREVENTION SYSTEMS

Unseating prevention measures are required for the highway. bridges. The measures required for the highway bridges are as:

1) the unseating prevention systems have to be so designed that unseating of a superstructure from their supports can be prevented even if unpredictable failures of the structural members occur,

2) the unseating prevention systems are consisted of providing enough seat length, a falling-down prevention device, a displacement limiting device, and a settlement prevention device,

3) enough seat length must be provided and a falling-down prevention device must be installed at the ends of a superstructures against longitudinal response. If the Type-A bearings are used, a displacement limiting device has to be further installed at not only the ends of a superstructure but each intermediate support in a continuous bridge, and

4) if the Type-A bearings are used, a displacement limiting device is requested at each support against transverse response. The displacement limiting device is not generally required if the Type-B bearings are used. But, even if the Type-B bearing is adopted, it is required in skewed bridges, curved bridges, bridges supported by columns with narrow crest, bridges supported by few bearings per piers, and bridges constructed at the sites vulnerable to lateral spreading associated with soil liquefaction.

The seat length S_E is evaluated as

$$S_E = u_R + u_G \geqq S_{EM} \qquad (39)$$

$$S_{EM} = 70 + 0.5l \qquad (40)$$

$$u_G = 100 \cdot \varepsilon_G \cdot L \qquad (41)$$

in which u_R = relative displacement (cm) developed between a superstructure and a substructure subjected to a seismic force equivalent to the equivalent lateral force coefficient k_{hc} by Eq.(2), u_G = relative displacement of ground along the bridge axis, S_{EM} = minimum seat length (cm), ε_G = ground strain induced during an earthquake along the bridge axis, and is 0.0025, 0.00375, and 0.005 for Group-I, II and III sites, respectively, L= distance which contributes to the relative displacement of ground (m), and l = span length (m). If two adjacent deck are supported by a pier, the lager span length should be l in evaluating the seat length.

Inthe menshin deisgn, in addition to the above requirements, the following copnsiderations have to be made.

1) To prevent collisions between a deck and an abutment or between two adjacent decks, enough clearance must be provided. The clearance between those structural components S_B shall be evaluated as

$$S_B = \begin{cases} u_B + L_A & \text{between a deck and an abutment} \\ c_B \cdot u_B + L_A & \text{between two adjacent decks} \end{cases} \quad (42)$$

in which u_B = design displacement of menshin devices (cm) by Eq.(35), L_A = redundancy of a clearance (generally $\pm$ 1.5cm), and c_B = modification coefficient for clearance (refer to **Table 9**). The modification coefficient c_B was determined based on an analysis of the relative displacement response spectra. It depends on a difference of natural periods $\Delta T = T_1 - T_2$ ($T_1 > T_2$), in which T_1 and T_2 represent the natural period of the two adjacent bridge systems.

2) The clearance at an expansion joint L_E is evaluated as

$$L_E = u_B + L_A \quad (43)$$

in which u_B = design displacement of menshin devices (cm) by Eq.(35), and L_A = redundancy of a clearance (generally $\pm$ 1.5cm).

14. CONCLUDING REMARKS

The preceding pages presented an outline of the new Seismic Design Specifications of Highway Bridges issued in 1996 as well as the damage features of highway bridges in the Hyogo-ken nanbu earthquake. The Hyogo-ken nanbu earthquake was the first earthquake which developed destructive damage in an urban area since the 1948 Fukui Earthquake. Because it had been considered that such destructive damage could be prevented due to the progress of construction technology in recent years, it provided a large impact on the earthquake disaster prevention measures in various fields. The "Part V Seismic Design" of the "Design Specifications of Highway Bridges" (Japan Road Association) was totally revised in 1996, and the design procedure moved from the traditional Seismic Coefficient Method to the Ductility Design Method. The revision was so comprehensive that the past revisions in the last 30 years look minor.

Major point of the revision was the introduction of explicit two-level seismic design consisting of the Seismic Coefficient Method and the Ductility Design Method. Because the Type-I and the Type-II ground motions are considered in the Ductility Design Method, three design seismic forces are totally used in design. Seismic performance for each design force was clearly stated in the Specifications.

The fact that lack of near-filed strong motion records prevented to seriously evaluate the validity of recent seismic design codes is important. The Hyogo-ken nanbu earthquake revealed that history of strong motion recording is very short, and that no near-field records have yet been measured by an earthquake with magnitude on the order of 8. It is therefore essential to have enough redundancy and ductility in a total bridge system. It is hoped that the revised Seismic Design Specifications of Highway Bridges contributes to enhance seismic safety of highway bridges.

ACKNOWLEDGMENTS

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APPENDIX

TABLE OF CONTENTS OF "Part V : SEISMIC DESIGN" OF "DESIGN SPECIFICATIONS FOR HIGHWAY BRIDGES," JAPAN ROAD ASSOCIATION, 1996

Table 1 Seismic Performance Levels

<table>
<tr><td colspan="2" rowspan="2">Type of Design Ground Motions</td><td colspan="2">Importance of Bridges</td><td colspan="2">Design Methods</td></tr>
<tr><td>Type-A
(Standard Bridges)</td><td>Type-B
(Important Bridges)</td><td>Equivalent Static Lateral Force Methods</td><td>Dynamic Analysis</td></tr>
<tr><td colspan="2">Ground Motions with High Probability to Occur</td><td colspan="2">Prevent Damage</td><td>Seismic Coefficient Method</td><td rowspan="3">Step by Step Analysis
or
Response Spectrum Analysis</td></tr>
<tr><td rowspan="2">Ground Motions with Low Probability to Occur</td><td>Type-I
(Plate Boundary Earthquakes)</td><td rowspan="2">Prevent Critical Damage</td><td rowspan="2">Limited Damage</td><td rowspan="2">Ductility Design Method</td></tr>
<tr><td>Type-II
(Inland Earthquakes)</td></tr>
</table>

Table 2 Lateral Force Coefficient k_{hc0} in the Ductility Design Method

(a) Type-I Ground Motions

Soil Condition	Lateral Force Coefficient k_{hc0}		
Group I (stiff)	$k_{hc0}=0.7$ for $T \leqq 1.4$	$k_{hc0}=0.876T^{2/3}$ for $T > 1.4$	
Group II (moderate)	$k_{hc0}=1.51T^{1/3}$ ($k_{hc0} \geqq 0.7$) for $T < 0.18$	$k_{hc0}=0.85$ for $0.18 \leqq T \leqq 1.6$	$k_{hc0}=1.16T^{-2/3}$ for $T > 1.6$
Group III (soft)	$k_{hc0}=1.51T^{1/3}$ ($k_{hc0} \geqq 0.7$) for $T < 0.29$	$k_{hc0}=1.0$ for $0.29 \leqq T \leqq 2.0$	$k_{hc0}=1.59T^{-2/3}$ for $T > 2.0$

(b) Type-II Ground Motions

Soil Condition	Lateral Force Coefficient k_{hc0}		
Group I (stiff)	$k_{hc0}=4.46T^{2/3}$ for $T \leqq 0.3$	$k_{hc0}=2.00$ for $0.3 \leqq T \leqq 0.7$	$k_{hc0}=1.24T^{-4/3}$ for $T > 0.7$
Group II (moderate)	$k_{hc0}=3.22T^{2/3}$ for $T < 0.4$	$k_{hc0}=1.75$ for $0.4 \leqq T \leqq 1.2$	$k_{hc0}=2.23T^{-4/3}$ for $T > 1.2$
Group III (soft)	$k_{hc0}=2.38T^{2/3}$ for $T < 0.5$	$k_{hc0}=1.50$ for $0.5 \leqq T \leqq 1.5$	$k_{hc0}=2.57T^{-4/3}$ for $T > 1.5$

Table 3 Safety Factor α in Eq.(9)

Type of Bridges	Type-I Ground Motions	Type-II Ground Motions
Type-B	3.0	1.5
Type-A	2.4	1.2

Table 4 Modification Factor on Scale Effect for Shear Capacity Shared by Concrete

Effective Width of Section d (m)	Coefficient c_e
$d \leqq 1$	1.0
d=3	0.7
d=5	0.6
$d \geqq 10$	0.5

Table 5 Standard Acceleration Response Spectra

(a) Type-I Response Spectra S_{I0}

Soil Condition	Response Acceleration S_{I0} (gal=cm/sec^2)		
Group I	$S_{I0}=700$ for $T_i \leqq 1.4$		$S_{I0}=980/T_i$ for $T_i > 1.4$
Group II	$S_{I0}=1{,}505T_i^{1/3}$ ($S_{I0} \geqq 700$) for $T_i < 0.18$	$S_{I0}=850$ for $0.18 \leqq T_i \leqq 1.6$	$S_{I0}=1{,}360/T_i$ for $T_i > 1.6$
Group III	$S_{I0}=1{,}511T_i^{1/3}$ ($S_{I0} \geqq 700$) for $T_i < 0.29$	$S_{I0}=1{,}000$ for $0.29 \leqq T_i \leqq 2.0$	$S_{I0}=2{,}000/T_i$ for $T_i > 2.0$

(b) Type-II Response Spectra S_{II0}

Soil Condition	Response Acceleration S_{II0} (gal=cm/sec^2)		
Group I	$S_{II0}=4{,}463T_i^{2/3}$ for $T_i \leqq 0.3$	$S_{II0}=2{,}000$ for $0.3 \leqq T_i \leqq 0.7$	$S_{II0}=1{,}104/T_i^{5/3}$ for $T_i > 0.7$
Group II	$S_{II0}=3{,}224T_i^{2/3}$ for $T_i < 0.4$	$S_{II0}=1{,}750$ for $0.4 \leqq T_i \leqq 1.2$	$S_{II0}=2{,}371/T_i^{5/3}$ for $T_i > 1.2$
Group III	$S_{II0}=2{,}381T_i^{2/3}$ for $T_i < 0.5$	$S_{II0}=1{,}500$ for $0.5 \leqq T_i \leqq 1.5$	$S_{II0}=2{,}948/T_i^{5/3}$ for $T_i > 1.5$

Table 6 Recommended Damping Ratios for Major Structural Components

Structural Components	Elastic Response		Nonlinear Response	
	Steel	Concrete	Steel	Concrete
Superstructure	0.02 ~ 0.03	0.03 ~ 0.05	0.02 ~ 0.03	0.03 ~ 0.05
Rubber Bearings	0.02		0.02	
Menshin Bearings	Equivalent Damping Ratio by Eq.(26)		Equivalent Damping Ratio by Eq.(26)	
Substructures	0.03 ~ 0.05	0.05 ~ 0.1	0.1 ~ 0.2	0.12 ~ 0.2
Foundations	0.1 ~ 0.3		0.2 ~ 0.4	

Table 7 Modification Coefficient for Energy Dissipation Capability

Damping Ratio for 1st Mode h	Coefficient c_E
$h < 0.1$	1.0
$0.1 \leqq h < 0.12$	0.9
$0.12 \leqq h < 0.15$	0.8
$h \geqq 0.15$	0.7

Table 8 Reduction Coefficient D_E for Soil Constants due to Soil Liquefaction

Range of F_L	Depth from the Present Ground Surface x (m)	Dynamic Shear Strength Ratio R	
		$R \leqq 0.3$	$0.3<R$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/6
	$10<x \leqq 20$	1/3	1/3
$1/3<F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10<x \leqq 20$	2/3	2/3
$2/3<F_L \leqq 1$	$0 \leqq x \leqq 10$	2/3	1
	$10<x \leqq 20$	1	1

Table 9 Modification Coefficient for Clearance c_B

$\Delta T/T_1$	c_B
$0 \leqq \Delta T/T_1 < 0.1$	1
$0.1 \leqq \Delta T/T_1 < 0.8$	$\sqrt{2}$
$0.8 \leqq \Delta T/T_1 \leqq 1.0$	1

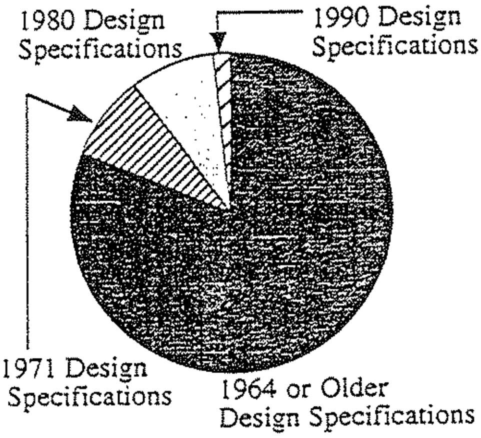


Fig.1 Design Specifications Referred to in Design of Hanshin Expressway

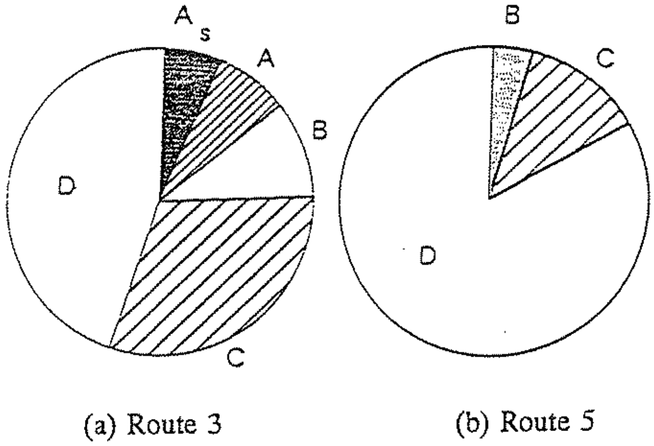


Fig.2 Comparison of Damage Degree between Route 3 and Route 5 (As : Collapse, A : Nearly Collapse, B : Moderate Damage, C : Damage of Secondary Members, D : Minor or No Damage)

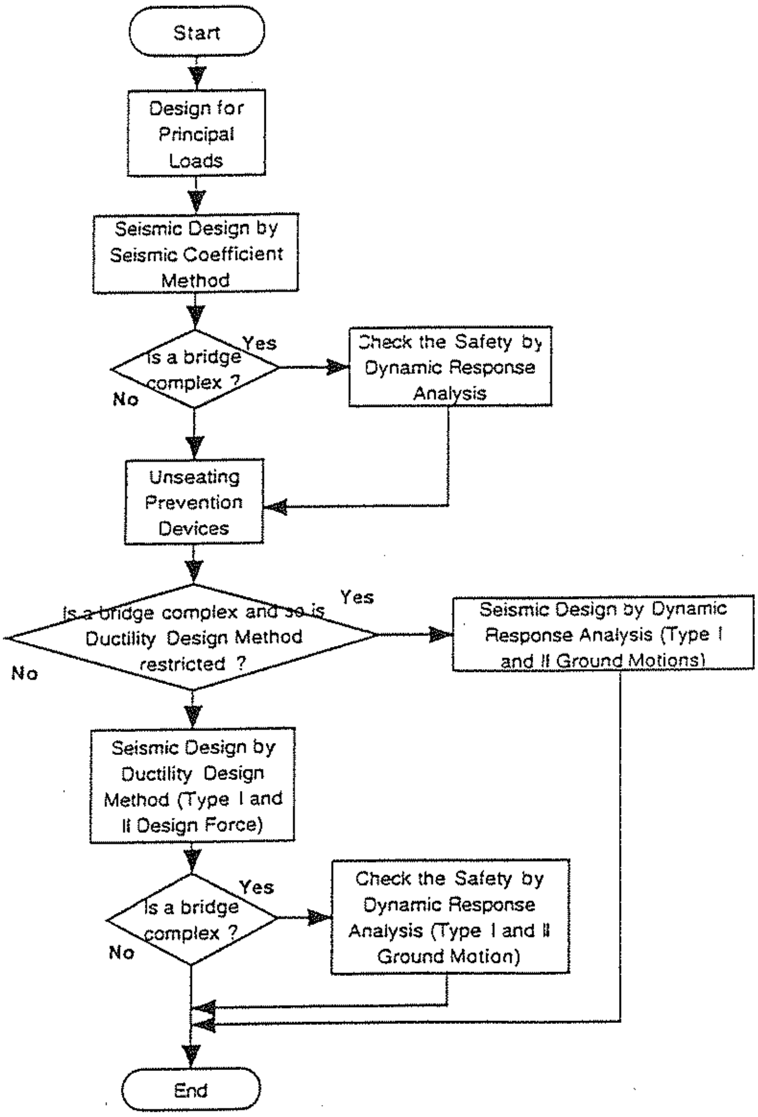
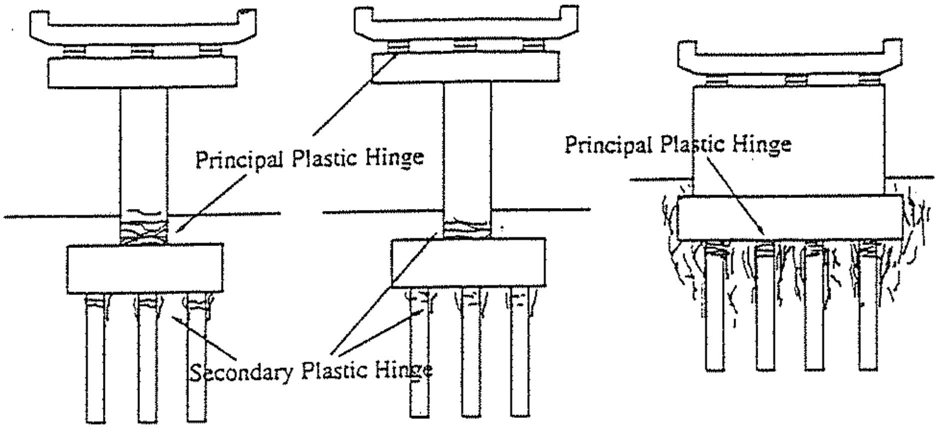


Fig.3 Flowchart of Seismic Design



(a) Conventional Design (b) Menshin Design (c) Bridge Supported by A Wall-type Pier

Fig.4 Location of Primary Plastic Hinge

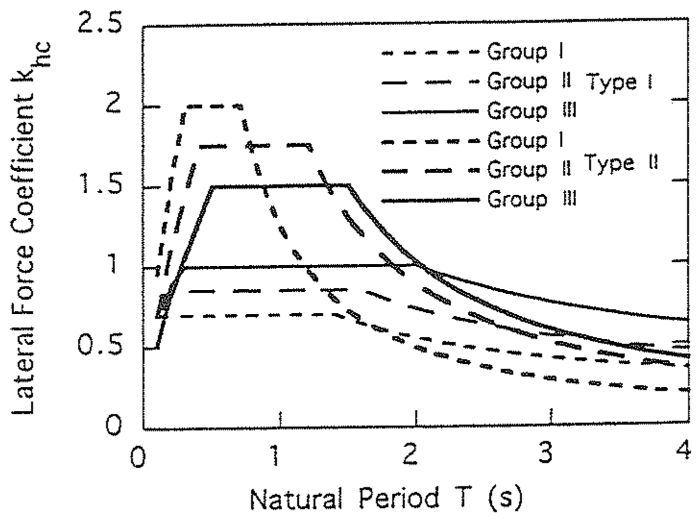
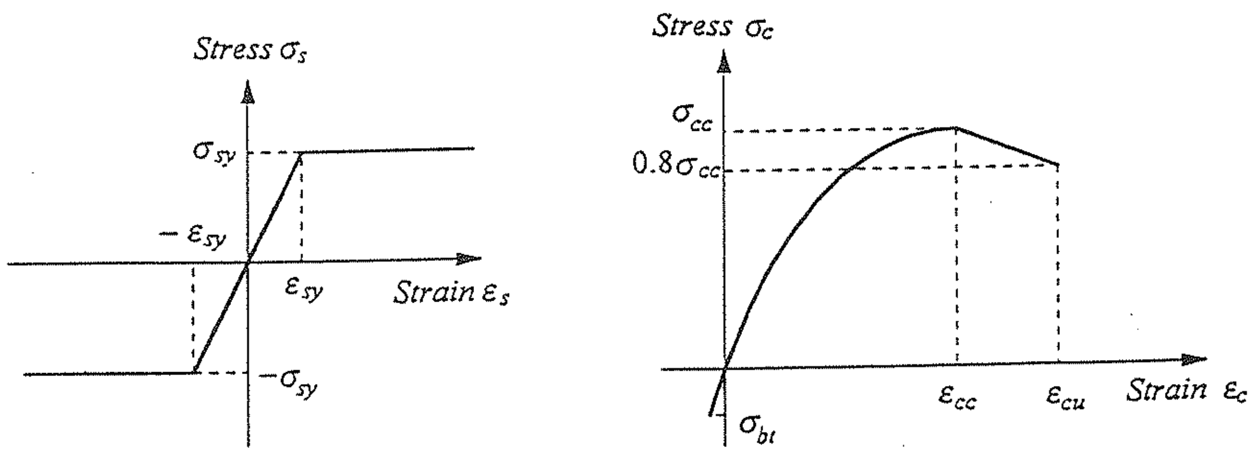
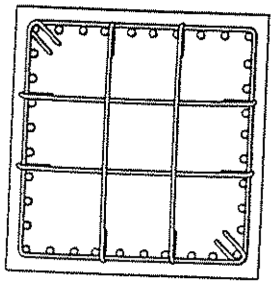


Fig.5 Type I and Type II Lateral Force Coefficients in the Ductility Design Method

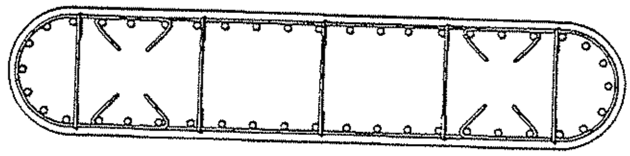


(a) Reinforcing Bars (b) Concrete

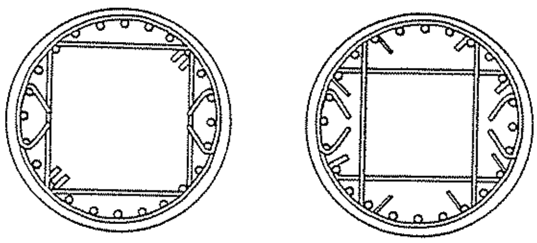
Fig.6 Stress and Strain Relation of Confined Concrete and Reinforcing Bars



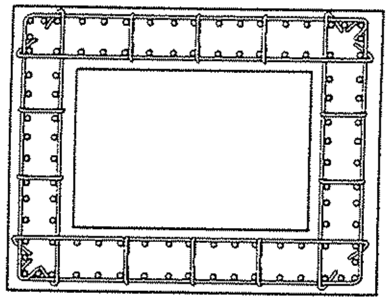
(a) Square Section



(b) Semi-square Section



(c) Circular Section



(d) Hollow Section

Fig.7 Confinement of Core-concrete by Tie Reinforcement

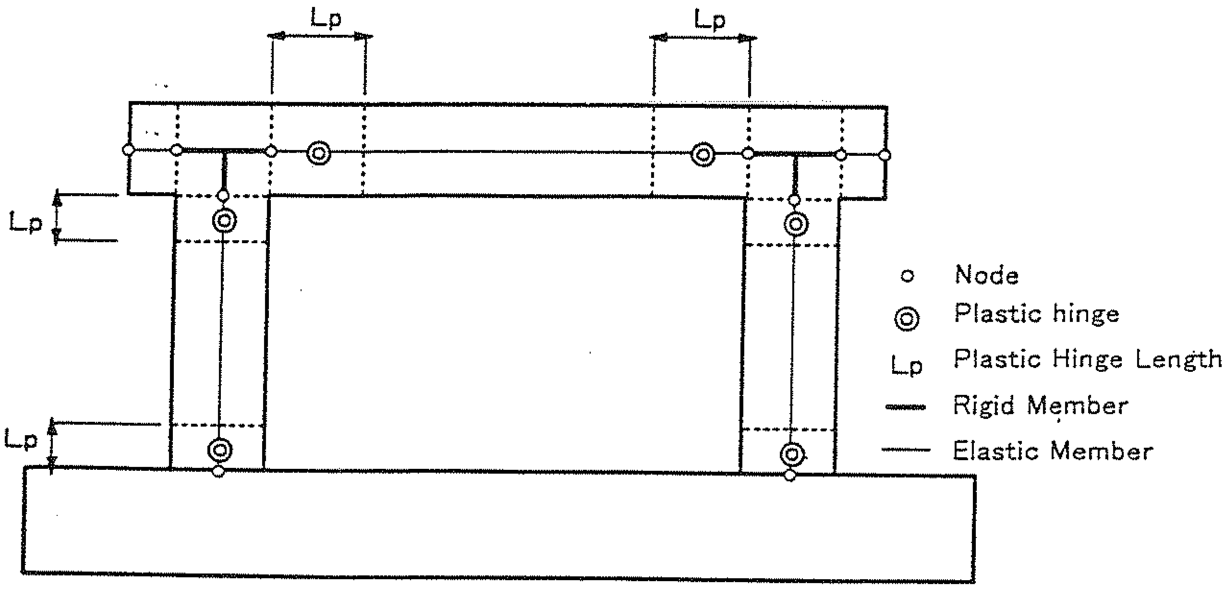
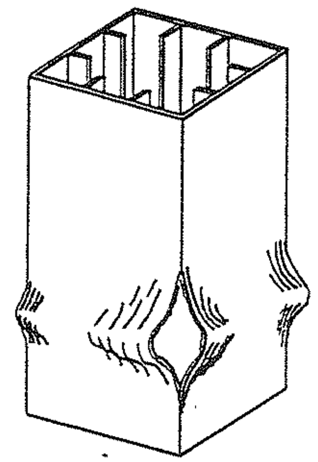
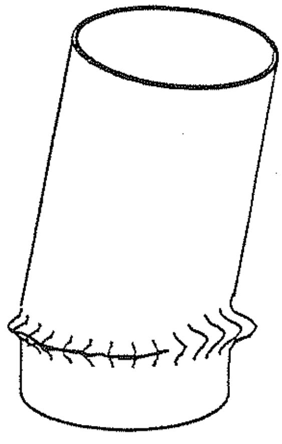


Fig.8 Analytical Idealization of A Two-Column Bent

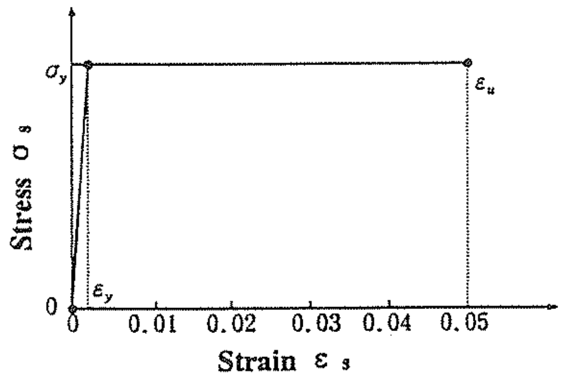


(a) Fracture of Corners

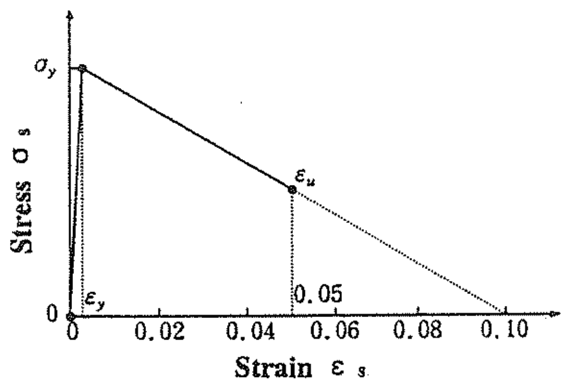


(b) Elephant Knee Buckling

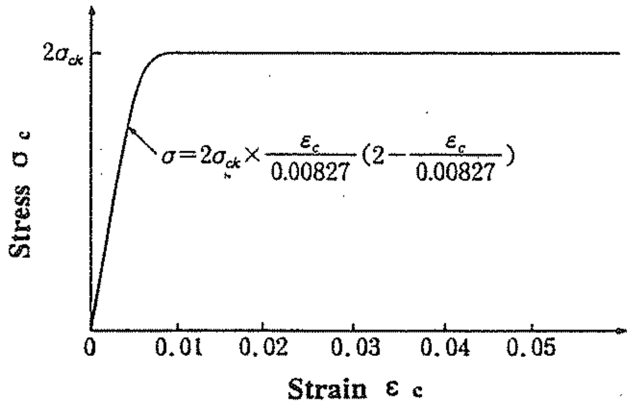
Fig.9 Typical Brittle Failure Modes of Steel Piers



(a) Steel (Tension Side)



(b) Steel (Compression Side)



(c) Concrete

Fig.10 Stress-Strain Relation of Steel and Concrete

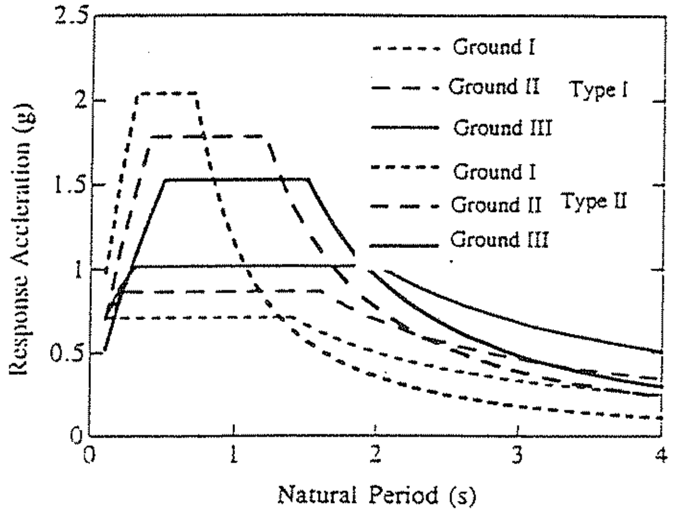
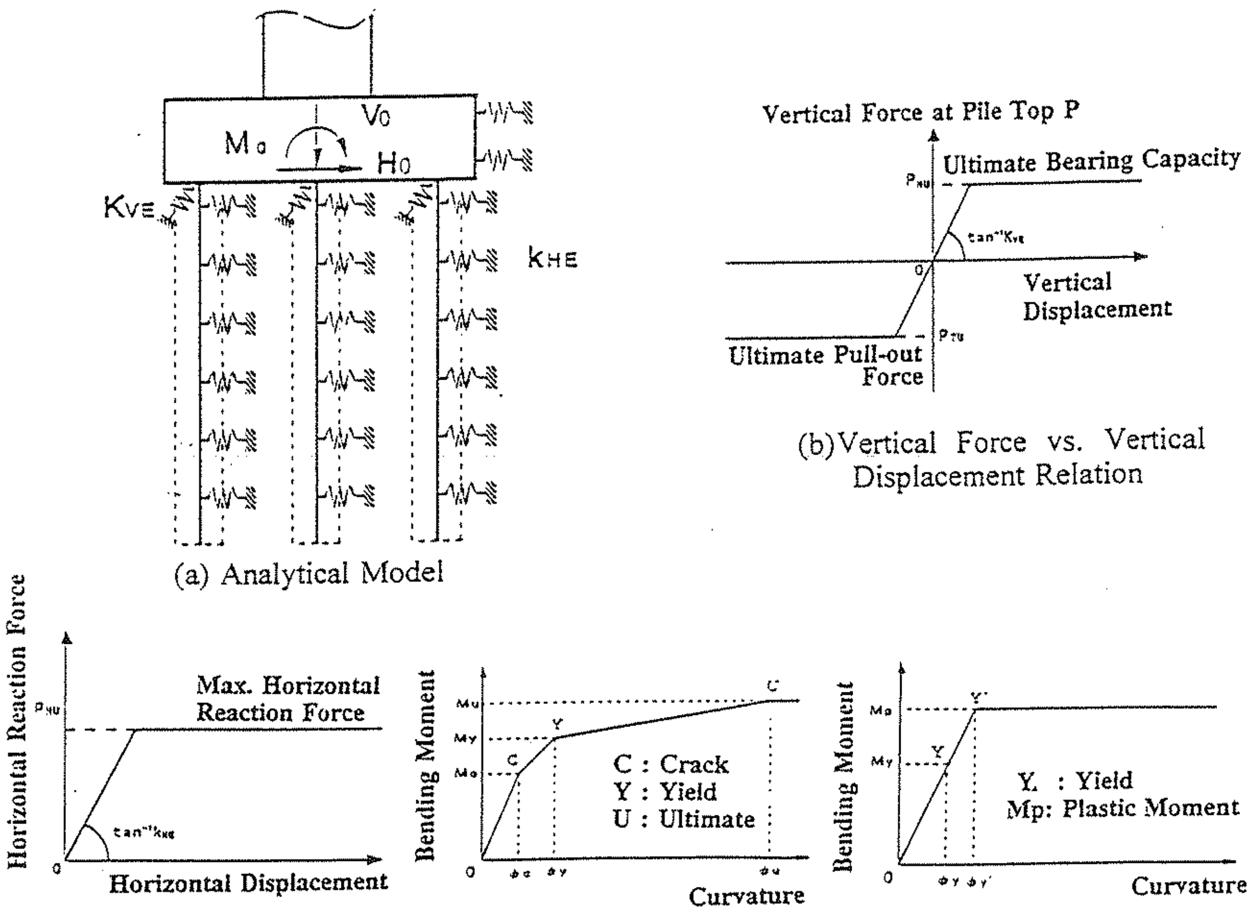


Fig.11 Type I and Type II Standard Acceleration Response Spectra



(c) Horizontal Force vs. Horizontal Displacement Relation

(d) Moment vs. Curvature Relation of Reinforced Concrete Piles

(e) Moment vs. Curvature Relation of Steel Pipe Piles

Fig.12 Idealized Nonlinear Model of A Pile Foundation

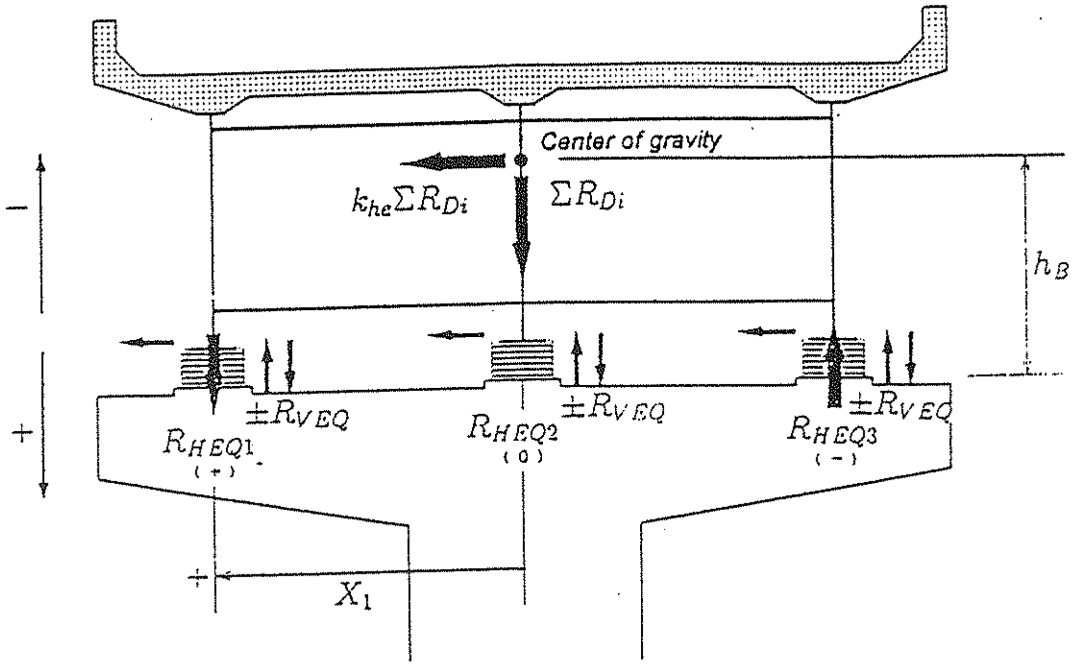


Fig.13 Design Forces for Bearing Supports