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Seismic design of highway bridge foundations with the effects of liquefaction since the 1995 Kobe earthquake

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Abstract

The seismic design of highway bridges has been improved through the lessons learned from earthquake damage and advances in earthquake engineering. The Design Specifications for Highway Bridges, including a volume on seismic design, have been revised three times since the 1995 Kobe earthquake. This report presents the major changes and improvements in the seismic design techniques for highway bridge foundations with the effects of liquefaction and their background since the 1995 Kobe earthquake. Particular emphasis is placed on the liquefaction potential assessment, ductility design of pier and abutment foundations for liquefaction, and seismic design of pier foundations for liquefaction-induced ground flow.

Keywords: Seismic Design, Highway Bridge, Foundation, Liquefaction, Design Specifications for Highway Bridges

1. Introduction

The first seismic design requirements for highway bridges in Japan were included in the Details of Road Structures (Draft), which were issued by the Ministry of Internal Affairs in 1926, three years after the 1923 Kanto earthquake. Since then, the seismic design regulations for highway bridges have repeatedly been revised based on the lessons learned from damaging earthquakes, e.g., the 1964 Niigata earthquake, 1978 Miyagi-ken Oki earthquake, and 1983 Nihon-kai Chubu earthquake, along with the progress of earthquake engineering. The seismic performance of highway bridges was improved by this process, even though the Kobe (Hyogo-ken Nanbu) earthquake on January 17, 1995, caused the worst damage to various structures, including highway bridges, since the 1923 Kanto earthquake. Highway bridges suffered destructive damage, such as the collapse of piers and the unseating of superstructures. The 1995 Kobe earthquake induced extensive soil liquefaction over a wide area, including reclaimed land composed of coarse sand and gravel layers, which caused serious influence on the seismic safety of structures. After this earthquake, the Design Specifications for Highway Bridges were extensively revised in 1996 (Japan Road Association, 1996, Unjoh and Terayama, 1998). The Design Specifications for Highway Bridges have been revised twice more since that revision in 1996.

In the 1996 Design Specifications for Highway Bridges, intensive earthquake motion with a short distance from an inland fault such as that generated by the 1995 Kobe earthquake was designated as a design earthquake motion called the Type II Earthquake Motion. With this, the design earthquake motions in the Design Specifications for Highway Bridges were organized into two levels, i.e., Level 1 and 2 Earthquake Motions, with three different earthquake motions. Note that the Type I Earthquake Motion representing earthquake motion generated by a large interplate earthquake, which is one of the Level 2 Earthquake Motions, was already employed in the 1990 Design Specifications for Highway Bridges. Ductility design was widely applied to bridge piers, foundations, bearing supports, and unseating prevention systems in the 1996 edition of the Design Specifications. The liquefaction potential assessment method was reviewed and revised for the Level 2 Earthquake Motion. Moreover, the seismic design of pier foundations for liquefaction-induced ground flow was newly prescribed.

Later, the Design Specifications for Highway Bridges were revised in 2002 (Japan Road Association, 2002, 2003). In this revision, emphasis was placed on the introduction of a performance-based design concept. For this, the principal requirements for the seismic performance of highway bridges, the design earthquake motions, and the verification of the seismic performance were explicitly specified. The verification methods for the seismic performance were reorganized into static analysis and dynamic analysis methods, and the selection of these two methods was based on the structural properties of highway bridges. A method for evaluating the seismic active earth pressure for the Level 2 Earthquake Motion, which is based on the modified Mononobe-Okabe method, was introduced. This was further applied to newly prescribe a verifying method for the seismic performance of abutment foundations for liquefaction during the Level 2 Earthquake Motion.

Most recently, the Design Specifications for Highway Bridges were revised in 2012 (Japan Road Association, 2012), just one year after the Great East Japan earthquake of 2011. In this earthquake, highway bridges suffered destructive damage, such as the washing away of superstructures as a result of the massive tsunami. The performance of highway bridges to ground motion differed according to the design years. Highway bridges designed by older specifications such as those earlier than the 1980 Design Specifications suffered damage similar to that observed in previous earthquakes. In contrast, those designed by newer specifications such as the 1990 Design Specifications or later performed well under the strong motions developed by the earthquake, and this contributed to the prompt relief from the earthquake and emergency response. In the revision of 2012, stress was placed on the importance of maintenance from the design stage, and the provisions were enhanced. The Type I Earthquake Motion was reviewed according to the results of recent research on large interplate earthquakes such as the Tokai, Tohnankai, and Nankai earthquakes. The provisions for unseating prevention systems were revised. Design considerations related to the connection of a bridge abutment and the earth structure behind it were introduced.

This report presents the major changes and improvements in the seismic design techniques for highway bridge foundations and their background since the 1995 Kobe earthquake, in relation to liquefaction potential assessment, ductility design of pier and abutment foundations for liquefaction, and seismic design of pier foundations for liquefaction-induced ground flow. Note that particular emphasis is laid on describing innovative design practice, and an overall review of previous studies is beyond the scope of this report. The same terms and symbols used in the Design Specifications for Highway Bridges are employed in this report to make it easier to understand the design practice.

2. Liquefaction potential assessment

2.1 Soil layers to be assessed

The 1995 Kobe earthquake caused liquefaction even at coarse sand and gravel layers that had been regarded as invulnerable to liquefaction, and the design practice changed to include both sandy and gravelly layers in the soil layers that require liquefaction potential assessment. The 1996 Design Specifications for Highway Bridges designated that the liquefaction potential should be assessed if an alluvial saturated granular soil layer meets the following three conditions.

1) Saturated soil layer located within 20m from the ground surface in which the groundwater level is less than or equal to 10m deep.
2) Soil layer with the fine particle content ratio FC equal to 35% or less, or the plasticity index I_p equal to 15 or less.
3) Soil layer with the mean grain size D_{50} equal to 10mm or less and the 10% grain size D_{10} equal to 1mm or less.

2.2 Estimation of liquefaction potential

The liquefaction potential is estimated by the liquefaction resistance factor F_L, where a soil layer with F_L of 1.0 or less is judged to be liable to liquefy during an earthquake. The liquefaction resistance factor F_L is defined as

$$F_L = R/L \tag{1}$$

where
F_L: liquefaction resistance factor
R: dynamic shear strength ratio
L: shear stress ratio during an earthquake.

The dynamic shear strength ratio R is practically modeled as

$$R = c_W R_L \tag{2}$$

where c_W is the corrective coefficient for earthquake motion characteristics, and R_L is the cyclic triaxial strength ratio. c_W originally represented a correction coefficient accounting for the difference between the random earthquake loading and the sinusoidal loading normally used in the triaxial test, and was improved to consider different cyclic characteristics of earthquake motions, as discussed in a later section.

The shear stress ratio during an earthquake L may be expressed by the following equation, which is essentially the same as the one proposed by Seed and Idriss (1971):

$$L = \gamma_d k_{hg} \sigma_v / \sigma'_v \tag{3}$$

where γ_d is a reduction factor for the shear stress ratio during an earthquake with depth, k_{hg} is the lateral seismic force coefficient on the ground, and σ_v and σ'_v represent the total and effective overburden pressures (kN/m^2), respectively. A general description of the shear stress reduction factor can be found in the literature (e.g., Idriss and Boulanger, 2004). γ_d employed in the Design Specifications for Highway Bridges is based on the results of a number of earthquake response analyses of various ground layers (Japan Road Association, 1980).

The cyclic triaxial strength ratio, which is defined as the ratio of the cyclic shear stress required to cause a 5% double amplitude axial strain in 20 cycles of loading to the confining pressure, was estimated by laboratory tests with undisturbed samples using the in situ freezing method (Yokoyama et al., 1997). The samples were formed into cylindrical specimens that were 10cm long and 5cm in diameter for sand, and 60cm long and 30cm in diameter for gravel, respectively. The specimens were set in a triaxial cell and undrained cyclic triaxial tests were performed. Isotropic consolidation was performed under the in situ effective overburden pressure. A sinusoidal load with a frequency of 0.1 Hz was applied to the specimens.

The cyclic triaxial strength ratio was primarily related to the N-values of various soils and was proposed for design practice. Fig. 1 shows the relationship between the cyclic triaxial strength ratio R_L of clean sands and the modified N-value N_1, where N_1 is an N-value modified for the effective overburden pressure of 100kN/m^2. R_L of alluvial sands significantly increases as N_1 exceeds 20 to 25, and the sand with N_1 greater than 30 is unlikely to liquefy even under severe cyclic stresses. The effects of the fine particle content ratio FC on the cyclic triaxial strength ratio R_L were studied, and R_L was finally proposed for sandy soil as (Yokoyama et al., 1997)

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (14 \le N_a) \end{cases} \tag{4}$$

$$N_a = c_1 N_1 + c_2 \tag{5}$$

$$N_1 = 170N/(\sigma'_v + 70) \tag{6}$$

$$c_1 = \begin{cases} 1 & (0\% \leq FC < 10\%) \\ (FC + 40)/50 & (10\% \leq FC < 60\%) \\ FC/20 - 1 & (60\% \leq FC) \end{cases} \tag{7}$$

$$c_2 = \begin{cases} 0 & (0\% \leq FC < 10\%) \\ (FC - 10)/18 & (10\% \leq FC) \end{cases} \tag{8}$$

where

N_a: modified N-value with the fine particle content

N_1: N-value modified for the effective overburden pressure of 100kN/m^2

c_1, c_2: correction factor of N-value by the fine particle content

FC: fine particle content ratio (%).

For gravelly soil, the N-value is modified by the mean grain size D_{50} (mm), with Eq. (9), and Eq. (4) is also used to express the cyclic triaxial strength ratio (Yokoyama et al., 1997):

$$N_a = \{1 - 0.36 \log_{10}(D_{50}/2)\} N_1 \tag{9}$$

It is well-known that the occurrence of liquefaction and the degree of the liquefaction depend not only on the amplitude of the earthquake motion but also on its cyclic characteristics. It was stipulated in the 1996 Design Specifications for Highway Bridges that an assessment of the liquefaction potential should be performed for the Level 2 Earthquake Motion, i.e., Type I Earthquake Motion and Type II Earthquake Motion. The Type I and Type II Earthquake Motions have different cyclic characteristics, and the coefficient c_W in Eq. (2) was introduced to incorporate these effects into the liquefaction potential assessment. The modified accumulative damage concept developed by Tatsuoka et al. (1986) was employed to propose this coefficient, and a total of 130 ground motion records from the past eight earthquakes were analyzed. Fig. 2 shows the analytical results for those records (Yokoyama et al., 1997). As seen from this figure, the results for c_W can be divided into two groups, and the following formulae were deduced.

For Type I Earthquake Motion,

$$c_W = 1.0 \tag{10}$$

For Type II Earthquake Motion,

$$c_W = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases} \tag{11}$$

3. Ductility design of pier foundation for liquefaction

3.1 Soil constants of liquefiable layer

The strength and bearing capacity of soil decrease when it liquefies. Based on this, for the seismic design of highway bridges, the soil constants of a sandy soil layer that is judged to be liable to liquefy are reduced according to the value of the liquefaction resistance factor F_L. These reduced constants are calculated by multiplying the coefficient D_E in Table 1 by the soil constants estimated on the assumption that the soil layer does not liquefy. Here, the soil constants reduced by multiplying D_E are the subgrade reaction coefficient, the upper limit value of the ground reaction, and the maximum skin friction force.

The coefficient D_E is based on the results of shake table tests and analyses of bridge foundations damaged by earthquakes. The values of D_E are prescribed to be different above and below a depth of 10m, because the ground vibration decreases as the depth increases, and the cases in which soil layers totally liquefied below a depth of 10m are limited. The values of D_E also depend on the dynamic shear strength ratio R of soil. This is because the reduction in the soil properties is smaller for a larger R.

3.2 Ductility design of pier foundation for liquefaction

The ductility design of a pier foundation was first prescribed in the 1996 Design Specifications for Highway Bridges. Because a bridge foundation is built in the ground, it is more difficult to identify the damage to the foundation compared to a pier, and it is also difficult to repair the foundation. For these reasons, in general, a bridge foundation should be designed so that it has greater dynamic strength than a bridge pier, along with sufficient deformation capacity. However, it is not always practically feasible to ensure that the strength of the foundation is greater than that of a pier in two cases in particular. The first one is when the pier has a sufficiently large ultimate horizontal capacity such as in the case of designing a foundation for a wall-type pier in the transverse direction. The second one is when a sandy layer liquefies and the strength of a foundation as a whole decreases with a decrease in the strength and bearing capacity of the ground. For these two cases, it is practically acceptable to allow a foundation to enter into a plastic zone, which will prevent excessive damage to the foundation and allow it to absorb earthquake motion energy.

When the primary plastic hinge is generated at a foundation, it is necessary to compute the response ductility factor and response displacement of the foundation and confirm that these are less than the allowable values. The deformation capacity of a foundation is verified by confirming that its response ductility factor is equal to or less than the allowable value, for which the principle of energy conservation by Veletsos and Newmark (1960) is employed, as schematically shown in Fig 3. This principle assumes that the input energies of an elasto-plastic response and elastic response are the same when a structure is subjected to an earthquake motion. Note that μ_{Fr} and μ_{FL} in Fig. 3 denote the response ductility factor and the allowable ductility factor, respectively. The allowable ductility factor for a pile foundation is determined to be 4 based on model experiments. Fig. 4 shows an analytical model of a pile foundation for the ductility design. It is assumed in this model that a rigid footing is supported by piles, which are further supported by the surrounding soils, where both the pile bodies and surrounding soils have nonlinear properties. It is verified that the response displacement of the foundation does not deteriorate the seismic performance of the bridge system as a whole. The maximum displacement and rotational angle at the top of the foundation were set as 40cm and 0.025 rad, respectively, in the 1996 Design Specifications for Highway Bridges.

4. Ductility design of abutment foundation for liquefaction

4.1 Characteristics of abutment damage

As described in the previous section, the ductility design of a pier foundation was introduced in the 1996 Design Specifications for Highway Bridges, though the ductility design of an abutment foundation was not stipulated at that time for the following reasons. First, a practical method to verify the seismic performance of an abutment foundation for the Level 2 Earthquake Motion, including an evaluation of the seismic active earth pressure at high seismic loads, was not established. Second, a bridge abutment is a structure that resists the earth pressure exerted by the backfill soil. Thus, it tends to move to the center of a bridge during an earthquake, which would not cause the unseating of the superstructure. Later, Koseki et al. (1998) proposed an evaluation method for the active earth pressure at high seismic loads, and an intensive study on earthquake damage and the seismic behavior of abutment foundations was conducted (Shirato et al., 2002), which made it possible to develop a ductility design method for an abutment foundation. Thus, the ductility design of an abutment foundation for the Level 2 Earthquake Motion was added to the 2002 Design Specifications for Highway Bridges, limited to a case where the ground is judged to be liquefiable.

Shirato et al. (2002) studied the relationship between the degree of damage to abutments and the effects of liquefaction using the damage records of the highway bridges in the previous earthquakes. Their study employed the equivalent thickness of the liquefiable soil layers H_E, which was originally proposed for the seismic inspection of road facilities (Ministry of Construction, 1991):

$$H_E = H_1^* + H_2^* \tag{12}$$

$$H_1^* = 1.5H_{FL1} + 1.0H_{FL2} + 0.5H_{FL3}\ (0m \le z \le 10m) \tag{13}$$

$$H_2^* = 1.0H_{FL1} + 0.5H_{FL2}\ (10m < z \le 20m) \tag{14}$$

where
H_E: equivalent thickness of the liquefiable soil layers (m)
z: depth from the ground surface (m)
H_1^*: equivalent thickness of the liquefiable soil layers for $0m \le z \le 10m$ (m)
H_2^*: equivalent thickness of the liquefiable soil layers for $10m < z \le 20m$ (m)
H_{FL1}: sum of the thicknesses of soil layers with $F_L \le 0.6$ (m)
H_{FL2}: sum of the thicknesses of soil layers with $0.6 < F_L \le 0.8$ (m)

H_{FL3}: sum of the thicknesses of soil layers with $0.8 < F_L \leq 1.0$ (m).

Note that H_E is the sum of the weighted thicknesses of the liquefiable soil layers, in which the thickness of a shallow liquefiable layer and that with a small liquefaction resistance factor F_L are greatly weighted. The damage ranks for abutments were classified into four groups, as shown in Table 2. Based on an investigation of 14 highway bridge abutments damaged in the previous earthquakes, the damage rank has a positive correlation with the equivalent thickness of the liquefiable soil layers, which implies that liquefaction has large influence on the seismic performance of the abutment and its foundation. This stimulated the introduction of seismic design of an abutment foundation for liquefaction.

4.2 Seismic loads and verification of seismic performance

In the ductility design method, the effects of an earthquake are modeled as static loads, i.e., the seismic active earth pressure acting on the backfill soil, the inertia forces acting on the backfill soil, abutment wall and footing, and the horizontal reaction of the bearing support, as shown in Fig. 5. Because the seismic behavior of an abutment and abutment foundation is mainly dominated by the vibration of the backfill soil, the lateral seismic force coefficient used for calculating the inertia force and seismic earth pressure of the abutment may be obtained by

$$k_{hA} = c_A k_{hg} \tag{15}$$

where
k_{hA}: lateral seismic force coefficient for verification of abutment foundation
c_A: modification factor for lateral seismic force coefficient of abutment foundation
k_{hg}: lateral seismic force coefficient on the ground.

c_A is a modification factor used to evaluate the acceleration of the backfill soil from the acceleration on the ground surface. The response acceleration of the backfill soil may increase or decrease from the acceleration of the surrounding ground. Furthermore, the response acceleration of the backfill soil decreases when it liquefies, while the settlement of the backfill soil induced by liquefaction affects the abutment stability. As described above, large uncertainties exist, and for simplicity, c_A was assumed to be 1.0.

A seismic active earth pressure coefficient for the Level 1 Earthquake Motion was evaluated using the Mononobe-Okabe method in the 1996 Design Specifications for Highway Bridges. This coefficient yields an excessively large sliding zone if it is applied to the Level 2 Earthquake Motion, whereas the modified Mononobe-Okabe method proposed by Koseki et al. (1998) can be applied to the Level 2 Earthquake Motion. The effectiveness of the modified Mononobe-Okabe method was verified by a comparison with the results of model experiments for the Level 2 Earthquake Motion. It also consistently reproduced the sliding angles that appeared behind structures resisting earth pressure during the 1995 Kobe earthquake. In the

2002 Design Specifications for Highway Bridges, the modified Mononobe-Okabe method was further simplified to be applicable to both Level 1 and 2 Earthquake Motions. Note that general conditions for a highway bridge abutment, such as the backfill soil material, construction conditions, and abutment shape were assumed during this process.

The seismic behavior of the abutment and abutment foundation is dominated more by the response of the backfill soil than by the vibration of the abutment, because an abutment is a relatively rigid structure. Consequently, the response displacement of the abutment foundation during an earthquake tends to accumulate in one direction, showing hysteretic characteristics. Despite the uncertainties which prevent a precise estimation of the dynamic nonlinear response of an abutment on which seismic earth pressure acts as a biased varying load, the seismic active earth pressure may increase in proportion to the acceleration of the backfill soil. Assuming that the peak response of the abutment foundation is induced by the peak ground acceleration, the response ductility factor and response displacement of the abutment foundation can be calculated by using the principle of energy conservation, similar to the case of a reinforced concrete pier subject to a biased bending moment and a pier foundation.

The response ductility factors were computed for the 14 highway bridge abutments damaged in previous earthquakes (Shirato et al., 2002). Fig. 6 shows the relationship between the response ductility factor and damage rank. Note that two abutment foundations did not yield, and their response ductility factors are assumed to be equal to 1 in Fig. 6. As seen from this figure, an abutment can avoid severe damage (categorized as rank 4) when the foundation is designed to have a response ductility factor of less than or equal to 3. Thus, an allowable ductility factor of 3 was established for an abutment foundation in the 2002 Design Specifications for Highway Bridges. An allowable ductility factor of 4 was specified for a pier foundation in the 1995 Design Specifications for Highway Bridges, and the allowable ductility factor for an abutment foundation was set smaller than that for a pier foundation. This is attributed to the structural difference between a pier foundation and abutment foundation. Because of the existence of backfill soil, an abutment foundation has limited reparability and large uncertainties in computing the nonlinear response displacement using the principle of energy conservation under a biased earth pressure.

5. Seismic design of pier foundation for liquefaction-induced ground flow

5.1 Estimation of ground flow force by back analysis

The 1995 Kobe earthquake caused extensive soil liquefaction over a wide area of offshore reclaimed lands and natural deposits. Moreover, near the water's edge, liquefaction induced ground flow with the movement of quaywalls or seawalls. Hamada et al. (1995) revealed using aerial photogrammetry that the maximum residual displacement caused by ground flow reached 3-4m. Liquefaction and its associated ground flow exerted serious influence on various engineered structures. Although highway bridges did not suffer fatal damage as a result of liquefaction, liquefaction-induced ground flow caused large deformations in bridge foundations.

The force acting on a bridge foundation due to the liquefaction-induced ground flow was estimated by a back analysis of the bridges that suffered residual horizontal displacements (Tamura, 2004). The result for a bridge pier located at the north edge of Rokko Island is presented herein. This pier was a two-story steel rigid frame pier and was supported by cast-in-place concrete piles 1.5m in diameter. The soils were composed of sandy artificial fill, alluvial clay, and alternating layers of diluvial sand and clay. The residual horizontal displacement of this pier was 0.9m. The groundwater level at the site was 3.3m below the ground surface, and the liquefaction was judged to occur in the sandy artificial fill below the groundwater level. Fig. 7 shows an overview of the analyzed foundation and the distribution of the applied force in the following analysis (Tamura, 2004).

In the estimation of the force applied to a bridge foundation due to the liquefaction-induced ground flow, it was assumed that the surface non-liquefied layer was conveyed by the liquefied layer spreading underneath, with both layers exerting force on the bridge foundation. In the non-liquefied layer, a force equivalent to the passive earth pressure was assumed to act on the bridge foundation. The liquefied layer was considered to move fluidly around the structure, and a force corresponding to a certain portion of the overburden pressure was assumed to act on the bridge foundation in the liquefied layer. This portion was estimated by back analysis of bridge piers with residual displacements. Note that the inertia force of the structure was ignored in the analysis, because the liquefaction-induced ground flow may take place after the principal ground motion ends. The pressure from the soil on a bridge foundation depends on various factors, including the displacement of the foundation relative to the soil. However, these effects were ignored for simplicity.

In the analysis, a bridge foundation was modeled as already shown in Fig. 4. A rigid footing is supported by piles, and the piles are supported by soil springs. This assumption allows for the nonlinear features of the pile bodies and soils. In addition, the soil resistance was ignored for the non-liquefied and liquefied layers that were considered to move when ground flow occurred. The width of the applied ground flow force was set as the width of the structure for a pier and footing, and as the projected width between the end piles for the pile bodies.

Because some portion of the overburden pressure of the liquefied layer acted on a bridge foundation as a lateral force, this portion was estimated by considering that the obtained pile top displacement was eventually identical to the observed residual displacement. The resultant relationship between the lateral force and pile top displacement is shown in Fig. 8. Similar analyses were conducted for the four bridge piers on the Route 5 of the Hanshin Expressway, and the ratio of the force applied in the liquefied layer to the overburden pressure was estimated to be approximately 0.3.

5.2 Design loads and verification of seismic performance

Based on the results of the above-mentioned analysis and a series of shake table tests (Tamura and Azuma, 1997), the seismic design of bridge foundations for liquefaction-induced ground flow was incorporated into the 1996 Design Specifications for Highway Bridges for cases where liquefaction-induced ground flow that may affect the seismic performance of a bridge is likely to occur. Generally, a case in which ground flow that may affect the seismic performance of a bridge is likely to occur is that the ground is judged to be liquefiable and is exposed to biased earth pressure, e.g., the ground behind a seawall.

The effect of the liquefaction-induced ground flow is modeled as the static force acting on a structure. It is assumed in this method that the surface non-liquefiable layer is underlain by a liquefiable layer, and forces equivalent to the passive earth pressure and 30% of the overburden pressure are applied to the structure in the surface non-liquefiable layer and liquefiable layer, respectively, as shown in Fig. 9. Because the magnitude of ground flow decreases with an increase in the distance from the water's edge, modification by distance is incorporated in the estimation of the ground flow force. Modification by the degree of liquefaction is also established.

The seismic performance of a bridge foundation is verified by confirming that the displacement at the top of the foundation caused by ground flow does not exceed an allowable value. The allowable displacement of a foundation may be taken as two times the yield displacement of the foundation. This is because uncertainties still remain about how to accurately evaluate the loads acting on bridge foundations, and the displacement of the foundation may increase considerably with a small amount of added load after it reaches two times the yield displacement.

6. Concluding remarks

This report presented the progress in the seismic design of highway bridge foundations with the effects of liquefaction and its background since the 1995 Kobe earthquake, in relation to the liquefaction potential assessment, the ductility design of pier and abutment foundations for liquefaction, and seismic design of pier foundations for liquefaction-induced ground flow. In order to enhance the seismic performance of a highway bridge as an entire system, improvement in the seismic design of foundations is indispensable since large uncertainties still remain in the seismic behavior of foundations with the surrounding soils. In addition, liquefaction and its induced ground flow exert a critical influence on the seismic performance of foundations. Further progress in the research in this area is expected to improve the seismic performance of highway bridges.

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Table 1 Coefficient D_E to be multiplied to soil constants for Level 2 Earthquake Motion

Range of F_L	Depth from ground surface x(m)	Dynamic shear strength ratio R	
		$R \leq 0.3$	$0.3 < R$
$F_L \leq 1/3$	$0 \leq x \leq 10$	0	1/6
	$10 < x \leq 20$	1/3	1/3
$1/3 < F_L \leq 2/3$	$0 \leq x \leq 10$	1/3	2/3
	$10 < x \leq 20$	2/3	2/3
$2/3 < F_L \leq 1$	$0 \leq x \leq 10$	2/3	1
	$10 < x \leq 20$	1	1

Table 2 Classification of abutment damage rank

Damage rank	1	2	3	4
Damage degree	Minor	Medium to major		Severe
Serviceability	Fully operational	Operational with restrictions, e.g., weight and/or velocity of vehicles	Temporarily no operation	No operation
Reparability	Easy	Possible with minor repair works	Possible with major repair works	Impossible (Reconstruction)
Typical damage	Shortening of spacing of expansion joint Cracks of parapet wall	Minor settlement of backfill soil Cracks of structural members	Horizontal movement or rotation of abutment Major settlement of backfill soil Collapse of parapet wall	Excessive horizontal movement or rotation of abutment Collapse of structural members

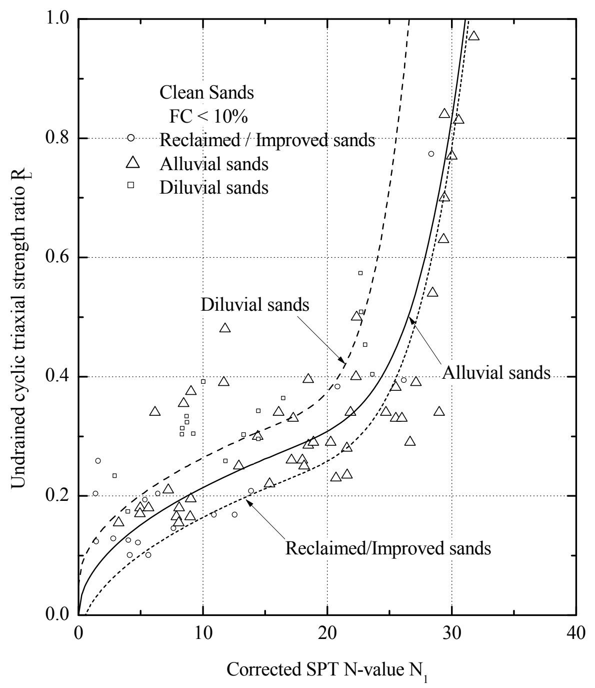

Fig. 1 Relationship between cyclic triaxial strength ratio and modified N-value for clean sands (Yokoyama et al., 1997)

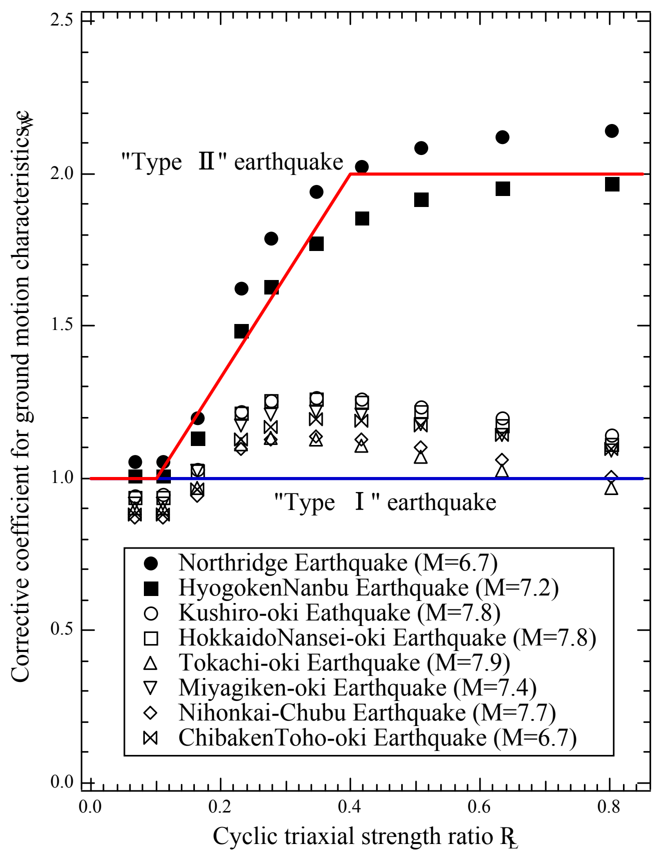

Fig.2 Corrective coefficient for ground motion characteristics (Yokoyama et al., 1997)

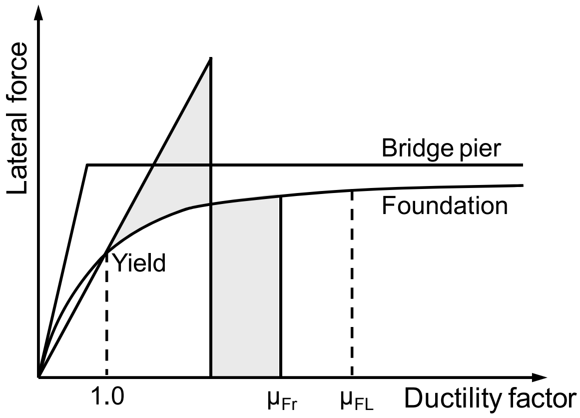

Fig.3 Calculation of response ductility factor based on the principle of energy conservation

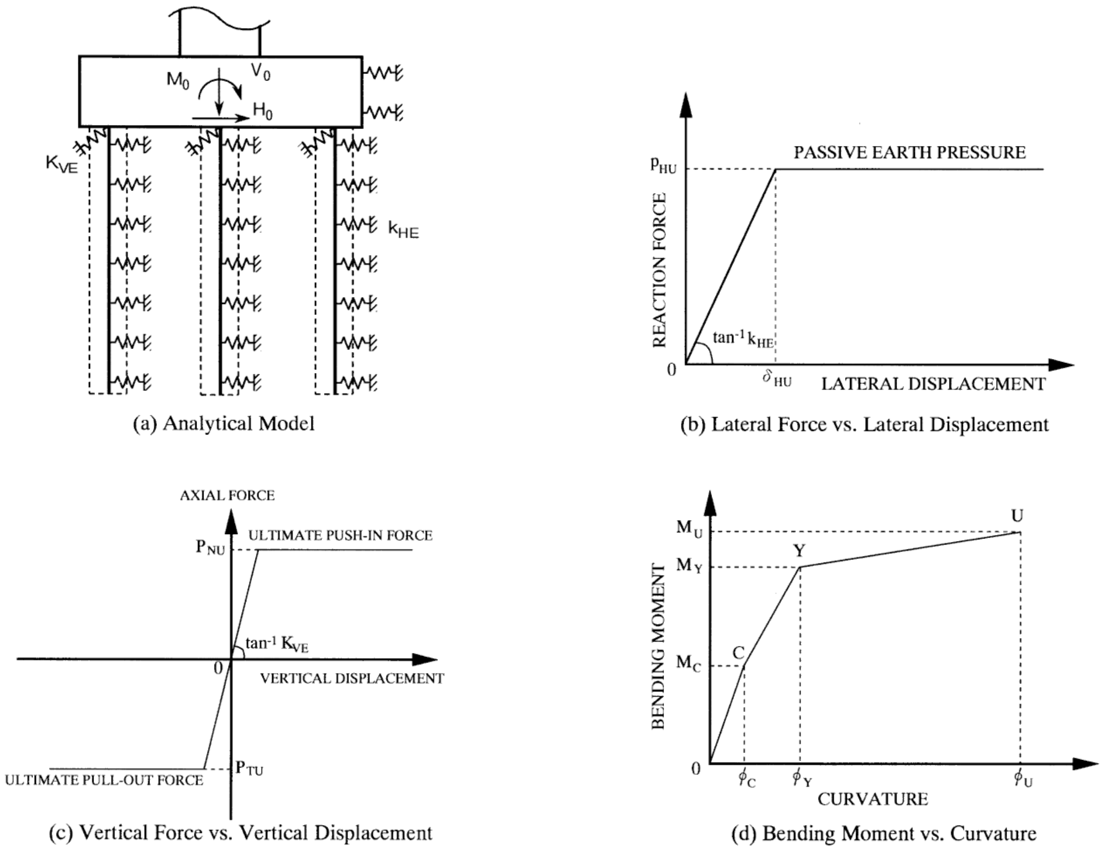

Fig. 4 Idealization of pile foundation for analysis

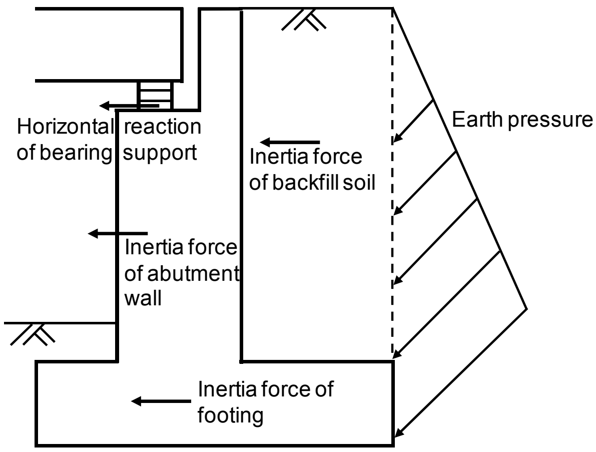

Fig. 5 Loading state assumed in verification of abutment foundation (Vertical forces except earth pressure are omitted for simplicity)

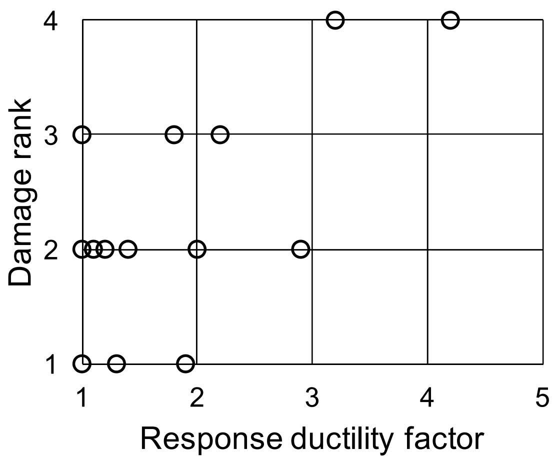

Fig. 6 Relationship between response ductility factor and damage rank

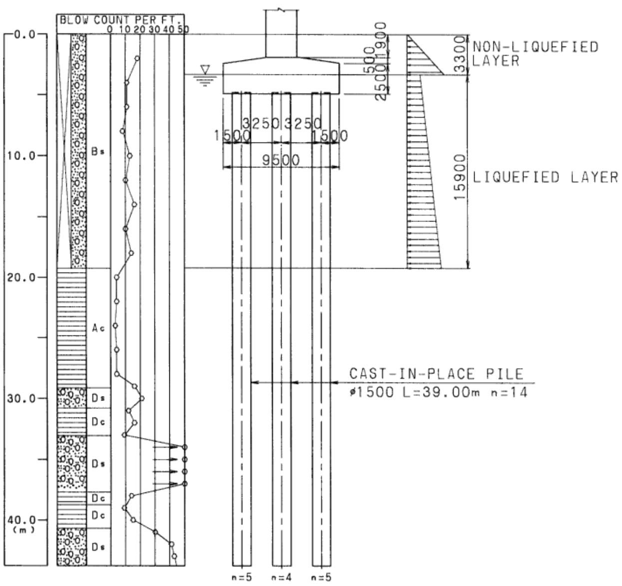

Fig. 7 Overview of analyzed foundation and applied force (Tamura, 2004)

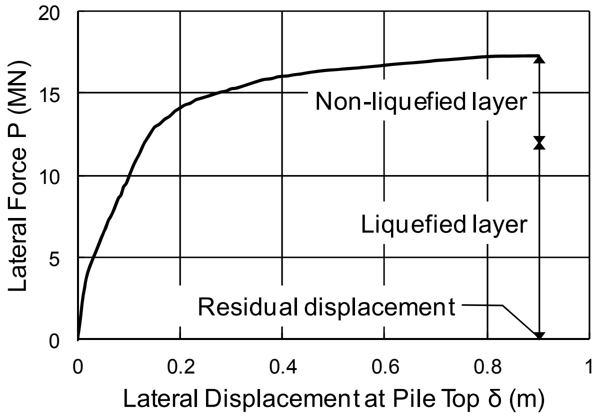

Fig. 8 Relationship between lateral force and displacement at pile top

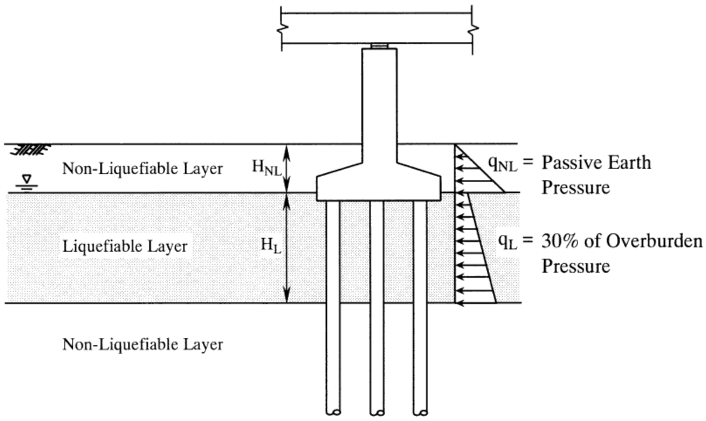

Fig. 9 Idealization of ground flow force for seismic design of bridge pier foundation