SEISMIC PERFORMANCE ASSESSMENT METHOD FOR EXISTING HIGHWAY BRIDGE FOUNDATIONS

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Abstract

This paper presents a simplified method of assessing the seismic performance of existing Japanese highway bridge foundations. The seismic performance of existing bridge foundations of various types was assessed by classification into five damage levels on the basis of damage experienced in past earthquakes and numerical analysis methods. Using these results, a flow chart is proposed to assess the seismic performance of bridge foundations according to the damage levels by using existing fundamental bridge data, geological data, and inspection data. This flow chart will be used to prioritize seismic retrofitting of bridge foundations.

Introduction

To prevent severe damage to bridges, it is best to upgrade the seismic performance of existing bridges to satisfy current seismic design specifications. However, it is not possible to upgrade all existing bridges in the short term owing to budget restriction. Therefore, existing bridges are upgraded according to priority.

Seismic retrofitting of bridges is prioritized by considering damage experienced in past earthquakes, structural characteristics, applied design specification, importance of the route, and other factors. For example, road bridges that crossed over roads or railways were given priority for retrofitting after the 1995 Hyogo-ken Nambu Earthquake in Japan.

In March 2005, the MLIT formulated a three-year program for seismic retrofitting of bridges on emergency routes. The purpose of this program is to upgrade road bridges on emergency routes within three years to minimize damage and maintain emergency vehicle traffic even if a Hyogo-ken Nambu-level earthquake occurs. Because of time and budget restrictions, this program concentrated on seismic retrofits of only specific parts of bridges, upgraded only by retrofitting a limited portion of their piers and setting up unseating prevention systems. For example, RC piers were basically retrofitted only at the cross section of the cut-off longitudinal bars to prevent severe shear failure.

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The seismic performance of bridges retrofitted under this program was slightly lower than that of those designed by the current design specification, so it is necessary to consider how their seismic performance would need to be re-upgraded in the next seismic retrofit program.

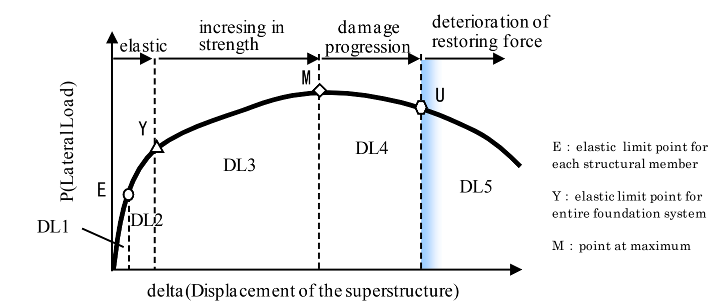
Bridge foundations are one of the key structural members for re-upgrading, although it is not clear what type of foundation should be given priority for retrofitting. Therefore, it is important to clarify the order of priority for strategic upgrading of the seismic performance of existing bridge foundations.

In light of these considerations, we propose a draft of a seismic performance assessment method for existing Japanese highway bridge foundations.

Bridge foundation damage level classification

Damage pattern in existing bridge foundations were classified in terms of safety, serviceability, and short-term repairability of foundation after an earthquake. Short-term repairability is the potential of a bridge to reopen to the traffic quickly despite damage to its foundation. Figure 1 shows the typical relationship among lateral load, lateral displacement, and damage to an existing bridge foundation. Figure 2 shows examples of damage patterns in pile foundations.

P-delta curve



Damage Level (DL)	DL1	DL2	DL3	DL4	DL5
	low				high
Restoring force	(elastic)	(elastic)	enough	deteriorate	loss
State of foundation	elastic for all structural members	elastic for whole foundation system Yiled	Maximum strength	Ultimate	excessive residual displacement
Serviciability after an earthquake	Passable	Passable	Passable under traffic control	Passable only emergency vehicles after emergency repair work	Impassable
Short-term repairability of bridge	No repair work	No repair work	Repair work is seldom needed	Capable	-
Long-term repairability of foundation	No repair work	No repair work	Repair work is seldom needed or needs repair work	Needs to repair work	-

Figure 1 Relationships among lateral load, lateral displacement and level of damage to bridge foundations

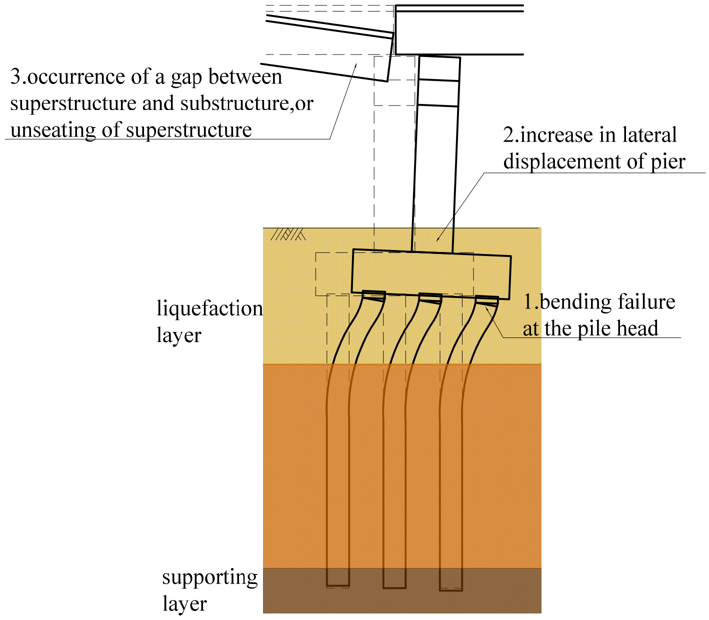


Figure2 Examples of damage patterns of pile foundations

The types of failure in bridge foundations can be classified as follow:

Type 1: bending failure of foundation
Type 2: shear failure of a foundation after bending yielding
Type 3: shear failure of a foundation
Type 4: excessive sinkage or motion of a foundation due to liquefaction of the supporting layer, or motion of the embankment surrounding the foundation due to liquefaction.

Figure 3 and 4 show the relationship between lateral load and lateral displacement of the superstructure for Type 2 and 3, respectively. Figure 5 illustrates Type 4 damage.

In general, damage to spread foundations, steel pipe piles, cast-in-place concrete piles are classified as Type 1; damage to RC pile, PC pile, PHC pile, and caisson foundations are classified as Type 2, and that to a single row pile bent bridge is classified as Type 3. Timber piles could be considered as two cases depending on the pile-footing connection. If the timber pile completely connects to the footing, it could be considered a pile foundation; if not, it could be considered a spread foundation.

Damage levels (DL) in this study are defined as follows.

Damage level 1(DL1):

In this state, all structural members of the foundation and the geotechnical strength remain within the elastic limit ("E" in Figure 1). Traffic can be permitted after an earthquake without repair work.

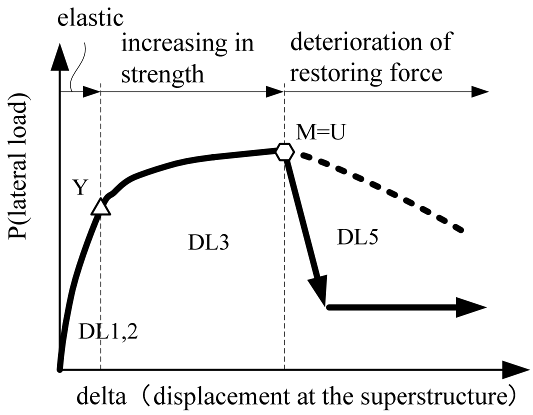


Figure 3 P-delta relationship for Type 2 damage

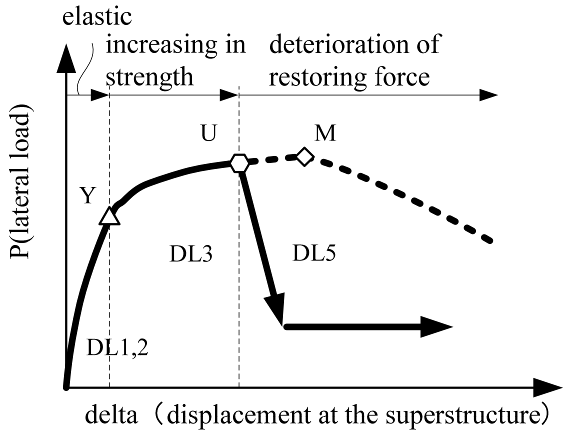
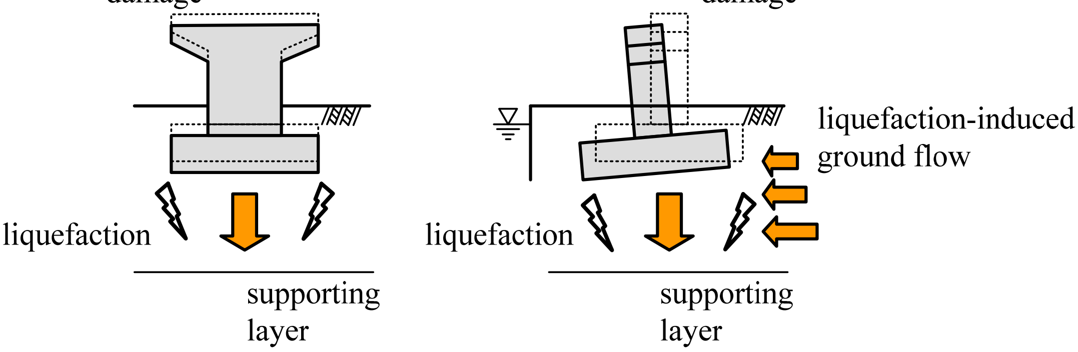


Figure 4 P-delta relationship for Type 3 damage



(1) Excessive sinkage by liquefaction

(2) Excessive sinkage and motion due to liquefaction-induced ground flow

Figure 5 Damage pattern for Type 4 damage

Damage level 2 (DL2):

In this state, the response behavior of the entire bridge foundation system remains within the elastic limit (“Y” in Figure 1). Some structural members or the geotechnical strength might exceed the elastic limit. Traffic can be permitted after an earthquake with no repair work.

Damage level 3 (DL3):

In this state, the maximum lateral response displacement of the foundation does not exceed the displacement at the peak strength of the foundation (“M” in Figure 1). Damage to the foundation is limited, and the road is passable to emergency vehicles. Others vehicles can use the road under traffic controls such as maximum weight or speed limitations.

Damage level 4 (DL4):

In this state, the maximum lateral response displacement exceeds the lateral displacement at the peak strength of the foundation, although it does not exceed the ultimate displacement (“U” in Figure 1). The ultimate point is defined as that at which the strength of foundation clearly deteriorates. The capacity for ductility is expected to remain, possibly causing residual displacement of the foundation and superstructure after the earthquake. Emergency vehicles could be permitted after emergency repair work or treatment.

Damage level 5 (DL5):

In this state, the maximum lateral response displacement exceeds the ultimate displacement and the bridge suffers from severe damage, such as collapse of the superstructure or the substructure. The road is impassable to all traffic.

The relationship between damage level and the priority of seismic retrofitting of bridge foundations is shown in Figure 6.

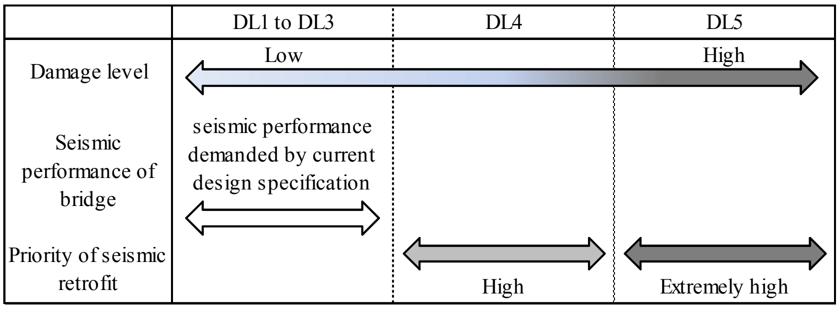


Figure 6 Damage level and priority of seismic retrofit of bridge foundations

<u>Earlier damage and changes in design specifications for bridge foundations</u>

In the 1923 Kanto Earthquake [JMA magnitude (M_{JMA}) =7.9] and the 1948 Fukui Earthquake (M_{JMA} =7.1), some bridges were severely damage because of damage to their foundations. One of the main reasons for foundation damage was assumed to be that a lack of bearing capacity in the foundation induced movement or tilting in the substructure.

It has been difficult to construct a pile longer than 15m because the work of digging and piling was done mainly by human power before the 1950s. In 1964, a substructure design guideline for pile foundation design was published; it specified that a pile foundation had to be designed not as a friction pile but as a bearing pile in principle. Therefore, bridge foundations constructed before the 1960s may include bearing piles that did not reach the fine supporting layer or friction piles that had an unsatisfactory bearing capacity.

In the 1964 Niigata Earthquake (M_{JMA} =7.5), some pile bent bridges and unsatisfactorily supported pile foundation bridges were severely damaged due to large earthquake ground motion, liquefaction, or liquefaction-induced ground flow (Photo 1). The lesson learned from this experience, “Seismic Design Guideline for Highway Bridge” was published in 1971 (hereafter called the 1971 specification); it specified a design method for dealing with liquefaction.

Photo1 Damage of pile bent pier bridge the 1964 Niigata Earthquake

In the 1995 Hyogo-ken Nambu Earthquake (M_{JMA}=7.3), many road bridges were severely damaged. Damage to bridge foundation systems were minor compared with those to piers or bearing shoes, although minor cracks were observed in some foundations, and residual displacement sometimes occurred due to liquefaction-induced ground flow near the waterfront. One reason is that allowable stress values and structural detail specifications, such as the minimum reinforcement volume of hoops at the pile head, were modified or introduced to require high ductility and shear capacities of structural members in the design specification of 1980 (hereafter called the 1980 specification). However, some architectural foundations suffered damage to their precast concrete piles. Precast concrete piles had been used mainly as architectural foundations and had shown shear failure or pile-failure-induced tilting of buildings in past earthquakes. This is due to inadequate ductility and shear capacities of the piles. After the 1995 earthquake, the maximum spiral reinforcement spacing of precast concrete piles in the plastic hinge region was specified to be within 100mm in the seismic design specification for road bridges published in 1996 (hereafter called the 1996 specification).

Seismic performance assessment of existing Japanese bridge foundations by pushover analysis

To assess the seismic performance of existing Japanese road bridge foundations, pushover analyses were conducted by the method specified for current seismic road bridge design. A total of 28 bridge foundations constructed before the 1995 Hyogo-ken Nambu Earthquake were analyzed (Table 1). The bridges were divided into three groups depending on the applied design specification: those designed before the 1971 specification, using the 1971 specification, and after the 1980 specification. New and modified structural details for upgrading structural members were specified in the 1980 specification in addition to the introduced liquefaction design in the 1971 specification.

Moreover, for cast-in-place concrete piles, pushover analyses were performed for 22 virtual bridge foundations that satisfied the specification at that time to analyze the relationship between the effect of liquefaction and the applied design specification (Table 2). Liquefaction-induced soil resistance reduction coefficients (hereafter called D_E values) defined four cases.

Table 1 Analytical cases for existing bridge foundations

Foundation type	Applied design specification		
	Before 1971	1971	1980
Spread foundation	1	3	1
Steel pipe pile	4	2	1
Cast-in-place concrete pile	3	1	1
RC pile	3	-	-
PC pile	1	-	-
Caisson foundation	4	1	-
Timber pile	1	-	-
Single-row-type pile bent pier	1	-	-

Table 2 Analytical cases for cast-in-place piles when the liquefaction-induced soil resistance reduction coefficient (D_E value) was assumed as the parameter

D_E value	Ground type	Applied design specification		
		Before 1971	1971	1980
0	Type 2	1	2	1
	Type 3	-	1	1
1/6	Type 2	2	-	-
	Type 3	2	-	-
1/3	Type 2	3	-	2
	Type 3	2	-	1
2/3	Type 2	2	-	-
	Type 3	2	-	-

*Ground type:
Type 2: T_G (sec) is between 0.2 sec and 0.6 sec
Type 3: T_G exceeds 0.6 sec
$T_{G:}$ Characteristic value of the ground. Originally referred to the fundamental natural period of the surface layer ground in a slight strain amplitude region.

The results of the analyses and seismic performance assessment of foundations using the damage level are as follows.

(1) Spread foundations

The analytical results showed that most spread foundations had good ductility performance, as illustrated by the lateral load- lateral displacement relationship (the

P-delta relationship) shown in Figure 1.

Spread foundations were assessed at DL1 or DL2 for Level 1 earthquake ground motion (hereafter called an L1 earthquake) and at DL3 for Level 2 earthquake ground motion (hereafter called an L2 earthquake).

However, damage to spread foundations is also affected by the condition of the supporting layer. This effect could not be predicted by pushover analysis, so the supporting layer condition was introduced as a new index for seismic performance evaluation. By using this index, damage could be assessed at DL5 in case when the supporting layer is liquefied.

(2) Steel pipe piles

The analytical results showed that most steel pipe piles also had good ductility performance, as illustrated by the P-delta relationship shown in Figure 1.

Most steel pipe piles were assessed at DL1 for an L1 earthquake and an interplate-type L2 earthquake and at DL3 for an inland near-field-type L2 earthquake. However, foundations designed before the 1971 specifications were assessed at DL5 for an inland near-field-type L2 earthquake if D_E was zero in the liquefaction layer. This is because the maximum response displacements of foundations, particularly those designed before the 1971 specifications, were found to be very large.

The specification for steel pipe piles was modified in 1990 to add concrete fill at the pile head. However, only hollow steel pipe piles were analyzed in this study. Additionally, there were not enough data to assess the allowable ductility ratio for hollow steel pipe piles, so this ratio pile was assumed to be half that of steel pipe piles filled with concrete. Hence, the allowable ductility ratios were assumed to be 2 for DL3 and 4 for DL4.

(3) Cast-in-place piles

The analytical results showed that most cast-in-place piles also had good ductility performance, as illustrated by the P-delta relationship shown in Figure 1.

Most cast-in-place piles were assessed at DL1 for an L1 earthquake and an interplate-type L2 earthquake and at DL3 or DL4 for an inland near-field-type L2 earthquake. The foundations were assessed at DL5 for an inland near-field-type L2 earthquake if D_E was zero and the foundation was constructed before 1971.

The specifications for structural details such as the minimum reinforcement volume of hoops at the pile head were modified according to the applied design specifications so that the upper limit of the ductility ratio corresponded to the DLs shown in Table 3.

The analytical results for cast-in-place piles designed before the 1980 specification showed that most cast-in-place piles showed shear failure in an inland near-field-type L2 earthquake. This is because the allowable stress on the concrete was reduced and the minimum reinforcement volume of hoops at the pile was introduced by the 1980 specification.

Table 3 Allowable ductility ratio for each damage level

Foundation type			Allowable ductility ratio for each damage level		
			DL2	DL3	DL4
Spread foundation			1	4	8
Pile foundation	Steel pipe pile	Before 1990 spec.	1	2	4
		1990 spec.	1	4	8
	Cast-in-place concrete pile	Before 1971 spec.	1	2	4
		1971 spec	1	3	6
		After 1980 spec.	1	4	8
	Precast concrete pile	Before 1996 spec.	1	2	4
		1996 spec.	1	4	8
	Timber pile		1	———	———
	Pile bent pier	Single row direction (steel pipe pile)	1	4	8
		Single row direction (except for steel pipe pile)	1	$\mu_a = 1+\frac{\delta_u - \delta_y}{\alpha\cdot\delta_y}$ (α=1.8)	$\mu_a = 1+\frac{\delta_u - \delta_y}{\alpha\cdot\delta_y}$ (α=1.0)
		Two or more row direction	Applied values for each pile type foundation		
Caisson foundation or Cast-in-situ diaphragm wall foundation ($M_c<M_y<M_u$)			1	$\mu_a = 1+\frac{\delta_u - \delta_y}{\alpha\cdot\delta_y}$ (α=1.8)	$\mu_a = 1+\frac{\delta_u - \delta_y}{\alpha\cdot\delta_y}$ (α=1.0)
Steel pipe sheet pile foundation			Applied values for steel pipe pile foundation		

*spec.: Applied design specification

Table 2 shows the analytical results when the D_E value for an L2 earthquake was assumed as the parameter. Here, the liquefaction layer was set from the present ground surface to a 10m depth.

Bridge foundations designed before the 1971 specification were assessed at DL5 when D_E was zero or 1/6 and at DL4 or lower when D_E was 1/3 or 2/3.

A design method considering liquefaction was specified in 1971, so the seismic performance of foundations designed after the 1971 specifications was better than that of foundations designed before 1971. However, some foundations designed between the 1971 and 1980 specifications were assessed at DL5 for an inland near-field-type L2 earthquake when D_E value was small. These analyses assumed that foundations failed because of bending failure, although the actual failure mechanism was shear failure. On the other hand, according to past experience with earthquakes, in no case did shear failure at the pile head cause severe damage. Therefore, the DL of bridge foundations designed between the 1971 and 1980 specifications was assumed to be DL4. Bridge foundations designed after the 1980 specifications were assessed at DL3 or lower.

(4) RC piles

The analytical results showed that most RC piles had poor ductility performance, as illustrated by the P-delta relationship shown in Figure 3. This is because that bending strength capacity of RC pile is generally poor.

Most RC piles were assessed at DL1 for an L1 earthquake and at DL5 for an L2 earthquake.

(5) PC piles

The analytical results showed that most PC piles had inadequate ductility performance, as illustrated by the P-delta relationship shown in Figure 3. This is because the shear strength capacity of PC piles is generally poor, although the bending strength capacity is not always poor.

Most PC piles were assessed at DL1 for an L1 earthquake and an interplate-type L2 earthquake and at DL3 for an inland near-field- type L2 earthquake. PC piles constructed on the liquefaction sites were assessed at DL5 for inland near-field type L2 earthquake. Shear-failure-type PC piles were assessed at DL5 for an L2 earthquake.

(6) Caisson foundations

The analytical results showed that most caisson foundations had poor ductility performance like, as illustrated by the P-delta relationship shown in Figure 3.

Most caisson foundations were assessed at DL1 for an L1 earthquake. In an L2 earthquake, damage levels were assessed at DL5 and DL3 for caisson foundations designed before and after the 1971 specifications, respectively.

Some shear-failure-type caisson foundations were assessed at DL5 for an L1 earthquake. Caisson foundations designed before the 1996 specification had very low longitudinal bars installed. Therefore, the cracking bending moment is sometimes higher than the ultimate bending moment if the caisson foundation is assumed to be a beam member. However, cyclic loading tests for caisson foundations by the PWRI showed that locking behavior was predominant after a crack occurred in the caisson body, and caisson foundations embedded in the ground retained their vertical loading support capacity because of passive subgrade resistance around the caisson. Consequently, bending-failure-type caisson foundations are assessed at DL4 or lower. It is not easy to assess the seismic performance of shear-failure-type caisson foundation owing to lack of knowledge, so their seismic performance must be assessed by another method.

(7) Timber piles

The analytical results showed that most timber piles had poor ductility performance, as illustrated by the P-delta relationship shown in Figure 3.

Most timber piles were assessed at DL1 for an L1 earthquake and at DL5 for an L2 earthquake.

The ductility characteristics of timber piles, which are arranged in high density and are unconnected to the footing, are assumed to be similar to those of spread foundations.

(8) Single-row-type pile bent pier

The analytical results showed that most pile bent piers with single row had poor ductility performance, as illustrated by the P-delta relationship shown in Figure 4.

Most pile bent piers were assessed at DL5 for an L2 earthquake whether the piles were made of steel or precast concrete members.

Pile bent piers were generally constructed before 1980, so most steel-pipe-type pile bent piers were not filled with concrete at the pile head. Therefore, the allowable ductility ratios were assumed to be 2 for DL3 and 4 for DL4, like those of hollow steel pipe piles.

(9) Other foundations

The seismic performance of steel pipe sheet piles was assumed to be similar to that of steel pipe pile and caisson foundations. Therefore, the damage level of steel pipe sheet piles was assumed to be DL3 for an L2 earthquake.

Using only fundamental bridge data, it is difficult to assess the seismic performance of other types of foundations, such as cast-in-situ diaphragm wall foundations, PC-well foundations, pile bent piers with two or more rows, foundations constructed near slopes (which will move because of slope failure), and foundations located in the ground where liquefaction-induced ground flow is possible. Therefore, the seismic performance of these types of foundation must be assessed by another method.

Proposal for a simplified seismic performance assessment method for existing Japanese highway bridges

To advance the strategic seismic retrofitting of existing road bridge foundations, it is important to estimate a round number of foundations that should be retrofitted first. A flow chart for simple assessment of the seismic performance of Japanese highway bridge foundations is proposed on the basis of the information above. The proposed flow chart is shown in Figure 7. This flow chart was made by using existing data, such as the plan for the bridge, geotechnical data, and inspection data for disaster prevention that was gathered in 1996. Therefore, it is easy to estimate the seismic performance of bridge foundations. However, some foundations require additional investigation owing to lack of data. For example, approximately one-fifth of the total Japanese highway bridge foundations are of unknown type, therefore, field work will be needed to investigate their performance.

Conclusions

The proposed seismic performance assessment method is useful for prioritizing the seismic retrofitting of Japanese highway bridge foundations. However, it is necessary to individually assess the seismic performance of cast-in-situ diaphragm wall foundations, PC-well foundations, foundations located in the ground where with liquefaction-induced ground flow is possible or in weathering slope or landslide areas, and so on.

As a next step, we need to establish performance verification methods for the new seismic retrofit techniques that will be proposed by construction companies and others.

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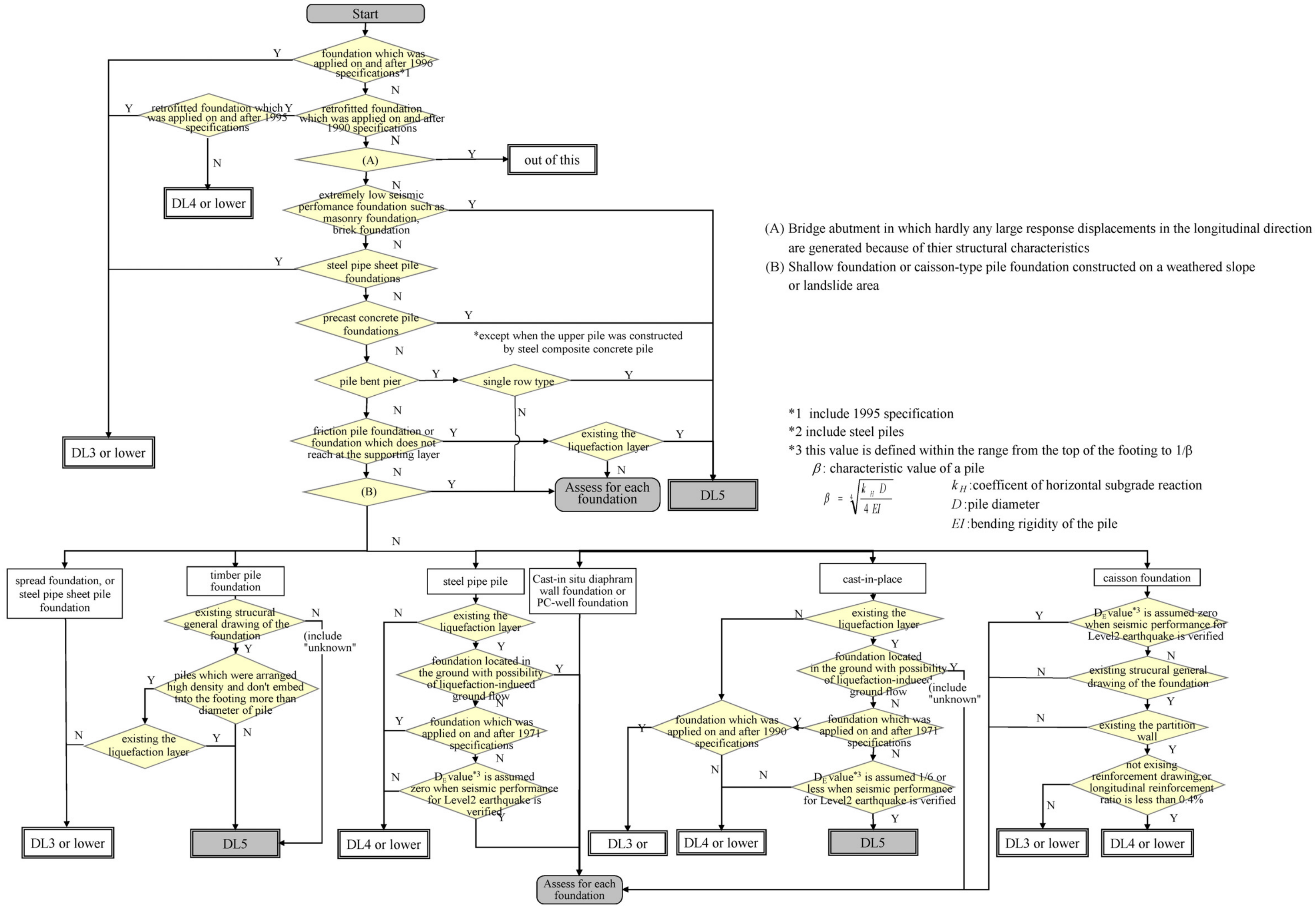

(A) Bridge abutment in which hardly any large response displacements in the longitudinal direction are generated because of thier structural characteristics

(B) Shallow foundation or caisson-type pile foundation constructed on a weathered slope or landslide area

*1 include 1995 specification

*2 include steel piles

*3 this value is defined within the range from the top of the footing to $1/\beta$

β: characteristic value of a pile

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}}$$

k_H: coefficent of horizontal subgrade reaction

D: pile diameter

EI: bending rigidity of the pile

Figure 7 Flow chart of seismic performance assessment for Japanese highway bridge foundation for an L2 earthquake.