SEISMIC RESPONSE OF HIGHWAY BRIDGES WITH DEEP CAISSON FOUNDATIONS EMBEDDED INTO SOFT SOILS

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SUMMARY

This paper presents an analytical result of dynamic responses of soil-pier systems of highway bridges with deep caisson foundations during strong earthquakes. In the analysis, four soil conditions, three pier types, and three pier heights are considered. Totally 36 cases of soil-pier systems are dynamically analyzed. From the analysis, seismic response characteristics of bridge systems with deep caisson foundations are clarified. It is concluded that a bridge pier with the value of Tg/Tp (where Tg:natural period of soil ground, Tp:natural period of bridge pier) of 0.7 to 1.25, might have a high potential to be damaged during strong earthquakes.

INTRODUCTION

Earthquake resistance of a highway bridge can be determined by comparing seismic load induced during an earthquake and strength of the bridge structure and the surrounding soils. In Japan, seismic design of highway bridges with the span length of 200 meters or less has been made with use of either the conventional seismic coefficient method or the modified seismic coefficient method, in which seismic loads are assumed to be static forces obtained by multiplying the dead weight of the bridge by the seismic coefficient obtained by the Specifications of the Japan Road Association (Ref.1).

From the experiences of recent seismic bridge damages, however, it is found that existing highway bridges may sustain heavy damages when the bridge and surrounding soil systems resonate with the induced ground motion during strong earthquakes.

This study intends to obtain a simplified procedure for evaluating seismic vulnerability of existing bridges, and also to establish a more reasonable design criterion against earthquake disturbances.

SOIL CONDITIONS AND BRIDGE CONFIGURATIONS CONSIDERED

Four different soil grounds with natural periods of Tg=0.3, 0.5, 0.7 and 0.9 seconds are taken into consideration, as shown in Table 1. The surface soft soils are assumed to have a constant depth of 20 meters above the firm supporting bedlayer. The depth of the caisson foundation is assumed to be 22 meters with 2 meters embedment into the bedlayer. Three different heights (hp) of pier columns are assumed, namely hp=10, 15 and 20 meters, as shown

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in Table 2 and Fig.1.

The superstructure is assumed to have the span length of 30 meters, the width of 10 meters, and the unit weight (weight of the superstructure, per 1 meter of longitudinal length) of 10 tons. Three different pier types, namely a pier with fixed supports for 3-span continuous girders, a pier with movable supports for the same 3-span continuous girders, and a pier supporting two ends (fixed and movable) of two simple-supported girders, are considered. For each structural type, its structural design is made according to the existing highway bridge design specifications (Refs.1 and 2).

Table - 1 Ground Conditions Considered.

Ground \ G. C.	G. C. 2	G. C. 3	G. C. 4 A	G. C. 4 B	Remarks
Characteristic Value, T_G	0.3 sec	0.5 sec	0.7 sec	0.9 sec	
Angle of Internal Friction, ϕ	30°	20°	5°	0°	
Cohesion, C	5 t/m^2	5 t/m^2	3 t/m^2	3 t/m^2	
Ground Period during Earthquake. T_G^*	0.38 sec	0.62 sec	0.88 sec	1.13 sec	$T_G^* = 1.25\ T_G$
Shear Velocity during Earthquake, $Vs^* = (4H/T_G^*)$	210m/sec	129m/sec	91m/sec	71m/sec	Depth of Surface Layer H = 20 m Unit Weight $\gamma = 1.8\ t/m^3$
Shear Modulus during Earthquake, $Gs^* = (\frac{Vs^{*2}\gamma}{g})$	814 Kg/cm^2	306 Kg/cm^2	152 Kg/cm^2	92 Kg/cm^2	
Shear Velocity for Small Strain, $Vs = (4H/T_G)$	267m/sec	160m/sec	114m/sec	89m/sec	Depth of Surface Layer H = 20 m
Mean N - Value, $N = (Vs / 90)^3$	26	6	2	1	
Deformation Modulus $\alpha Eo = \alpha \cdot 28N$	1456 Kg/cm^2	336 Kg/cm^2	112 Kg/cm^2	56 Kg/cm^2	$\alpha = 2$
Bedrock Layer	Vs = 300 m/sec , αEo = 5000 Kg/cm^2				

Table - 2 Outline of Highway Bridge

	Weight of Superstructure Resting on Pier in Longitudinal Direction (ton・f)	Pier Height hp (m)	Pier Width (m) (in Longitudinal) (m)
Fixed Pier for 3-Span Continuous Girder	900	10	2.2
		15	2.5
		20	3.0
Movable Pier for 3 - Span Continuous Girder	0	10	1.5
		15	1.5
		20	2.0
Pier for Simple Girder	300	10	1.6
		15	1.8
		20	2.2

Totally 36 cases (4 soil conditions x 3 pier types x 3 pier heights) of bridge piers will be designed statically and analyzed dynamically in the longitudinal direction of the bridge axis.

STATIC SEISMIC DESIGN

According to the current Earthquake-Resistant Design Specifications for Highway Bridges in Japan(Ref.1), the design seismic coefficients(k_h) are determined from the following formula:

$$k_h = \beta \cdot \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_0 \quad (1)$$

where, k_0 = standard design seismic coefficient (=0.20),

ν_1 = zone factor (ν_1=0.7 to 1.0, in this analysis ν_1 is assumed to be the highest, ν_1=1.0),

ν_2 = soil conditions factor (ν_2=0.9 to 1.2 depending on soil conditions, in this analysis ν_2=1.0, 1.1 and 1.2 are taken for soils with

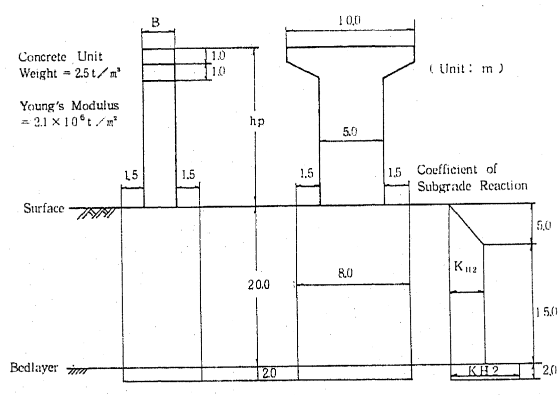

Fig.- 1 Configuration of Bridge Pier.

natural periods of 0.3, 0.5, and 0.7 ∿ 0.9 seconds, respectively),

ν_3 = importance factor (ν_3= 0.8 to 1.0, in the analysis ν_3 is taken as 1.0),

β = dynamic magnification factor (β= 1.0 for the conventional seismic coefficient method, and β=0.5 to 1.25 depending on the natural periods of bridge structures for the modified seismic coefficient method).

Table – 3 Design Horizontal Seismic Coefficient.

	Pier Height hp (m)	Design Horizontal Seismic Coefficient			
		G. C. 2 T_G=0.3 sec	G. C. 3 T_G= 0.5 sec	G. C. 4A T_G =0.7 sec	G. C. 4B T_G =0.9 sec
Fixed Pier for 3-Span C.G.	10.0	0.20	0.28	0.30	0.30
	15.0	0.25	0.28	0.30	0.30
	20.0	0.25	0.28	0.30	0.30
Movable Pier for 3-Span C.G.	10.0	0.20	0.22	0.24	0.24
	15.0	0.20	0.22	0.25	0.30
	20.0	0.20	0.26	0.30	0.30
Pier for Simple G	10.0	0.20	0.22	0.30	0.30
	15.0	0.24	0.28	0.30	0.30
	20.0	0.25	0.28	0.30	0.30

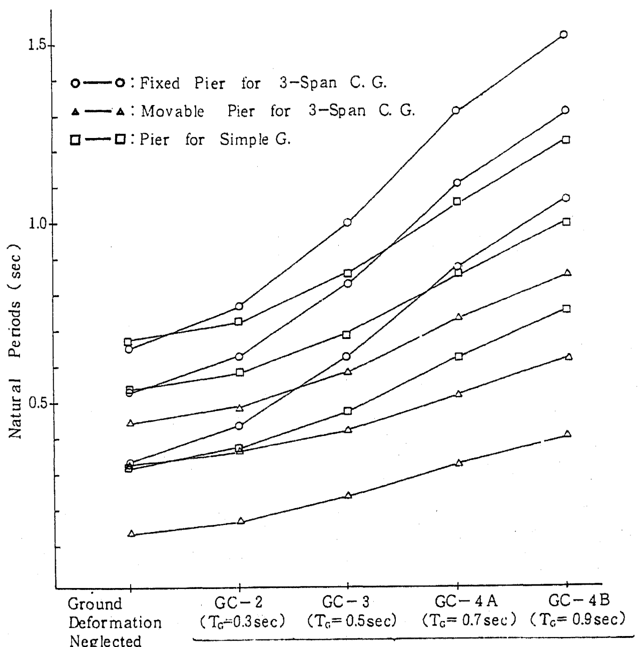

Fig.–2 Natural Periods for Various Piers.

Table-3 tabulates the design horizontal seismic coefficients (k_h) obtained for the 36 cases according to the above equation. Natural periods (Tp*) of various types of bridge piers computed from the formula specified in the Specifications ($Tp^*=2.01\sqrt{\delta}$, δ=displacement at the pier top in meters when subjected to lateral loading of 1-g acceleration), are shown in Fig.2. The extreme left values of the figure indicate natural periods of bridge pier when the deformations of the caisson foundation are neglected, and only the bending deformations of the bridge columns are considered. Bending moments and shear forces acting at the bases of bridge columns will be obtained when these seismic coefficients are applied to respective bridge piers.

MODEL FOR DYNAMIC ANALYSIS

An analytical model which includes the superstructure, the pier column, the caisson foundation, and the surrounding soils is set up for a dynamic analysis, as shown in Fig.3. In the analysis, the bridge column and the surrounding soils are simulated by lumped masses and elastic springs, and the caisson foundation is assumed to be a rigid body. Subgrade reactions for shallow layers where acting earth pressures exceed the bearing capacities of the grounds, are completely neglected. Spring constants between the caisson foundation and the surrounding soils are calculated according to the Design Specifications of Highway Bridge Substructures of the Japan Road Association (Ref.2).

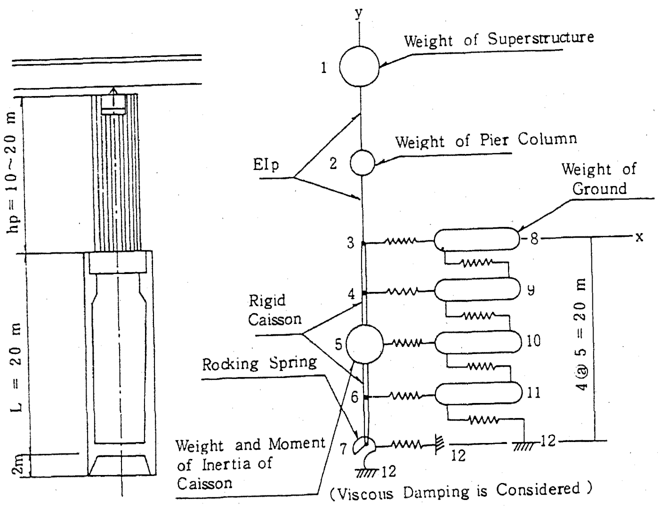

Fig.- 3 Mathematical Model for Dynamic Analysis.

INPUT SEISMIC MOTIONS

As for input seismic motion for the dynamic analysis, average response spectral curves obtained from various records (277-component) triggered on firm grounds in Japan, are employed. The peak accelerations are assumed to be 100 gals at the level of the bedlayer of Fig.1. The average response spectral curves for a linear system are shown by dotted lines in Fig.4, for three values of damping ratios (namely, h=0.05 0.1, and 0.2). In the figure, the values of β in Eq.(1) are also illustrated by a rigid line. It is noted that there is a large discrepancy between the average spectral curves and the β-values for the modified seismic coefficient method.

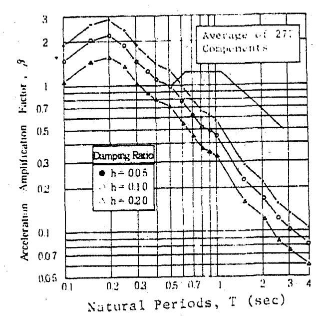

Fig - 4 Input Response Spectra.

RESULTS OF DYNAMIC ANALYSIS

Dynamic analysis was conducted for the 36 cases of bridge-soil systems. In the dynamic analysis, the structural members and soils are assumed to be elastic and psuedo-elastic, respectively, and the damping ratio is assumed to be 20 percent of critical. Fig.5 illustrates peak response accelerations at the pier top analyzed for various cases. From this, it will be pointed out that peak response accelerations are great when the natural periods of bridge piers

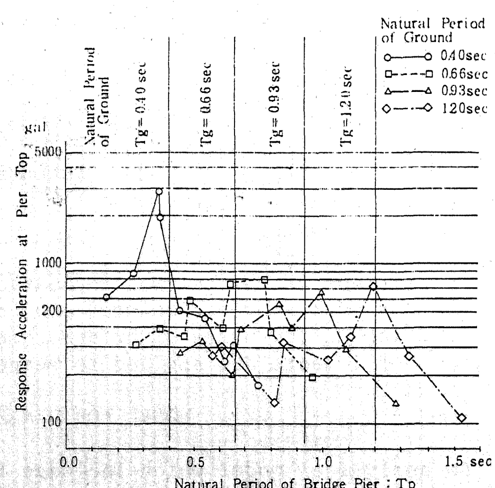

Fig.- 5 Maximum Acceleration at Pier Top.

coincide with those of surface soils and that peak response accelerations are much greater for the case of Tg= 0.40 seconds. Fig.6 also indicates peak response displacements at the pier top. From this it is indicated that large displacements will generate when resonance occurs, and that very large displacements will occur at soft soils.

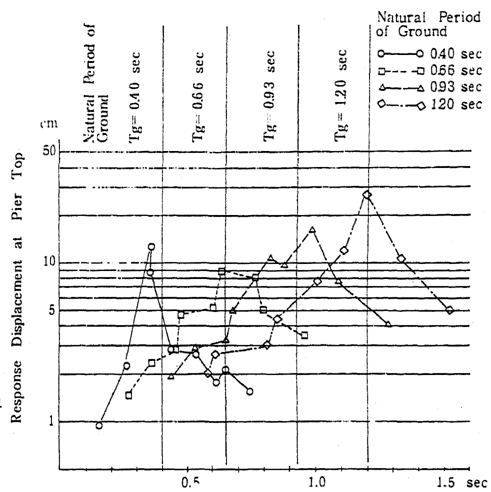

Fig.-6 Maximum Displacement at Pier Top

COMPARISON BETWEEN STATIC DESIGN AND DYNAMIC ANALYSIS

In this section, a comparison will be made between the static design based on the modified seismic coefficient method and the dynamic analysis.

First of all, Fig.7 plots a relation between natural periods of bridge structures (Tp*) obtained from the static design and natural periods (Tp) obtained from the dynamic analysis. It is found from the figure that natural periods of bridges by the simplified formula are very close to those from the dynamic analysis, and the simplified formula specified in the specifications for obtaining natural periods of bridges can be satisfactorily employed if deformation characteristics of soils are carefully considered.

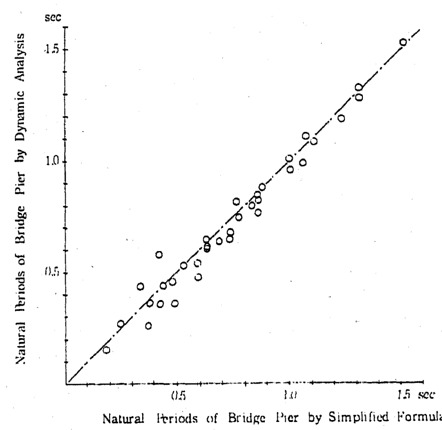

Fig.-7 Natural Periods of Bridge Pier by Simplified Formula Versus Those by Dynamic Analysis.

Figs.8 to 11 indicate relationships between natural periods of bridge piers and ratios of dynamic response values to static design values. The ordinate of Fig.8 is ratios of seismic coefficients obtained by the dynamic analysis to those of the static design. The ratios can be considered as the ratios of dynamic forces actually applied to design static forces. It is seen from the figure that the ratio becomes greater (the maximum is over 10) for the case of a stiff ground with natural period of 0.4 seconds, although the ratio gets smaller (0.4 to 3) for the cases of soft grounds with natural periods of 0.66 to 1.20 seconds.

The ordinate of Fig.9 shows ratios of dynamic response displacements to static design displacements. A relation similar to that of Fig.8 is seen. Figs.10 and 11 illustrate ratios of dynamic response shear forces to static design shear forces, and ratios of dynamic bending moments to static design bending moments, respectively. It is understood from these figures that the ratios are rather great (around 8) for the case of a ground with natural period of 0.4 seconds, and that ratios become rather smaller (0.3 to 2) for cases of grounds with natural periods of 0.66 to 1.20 seconds.

Furthermore, Fig.12 demonstrates a relationship among natural periods of bridge piers, natural periods of grounds, and the ratios of seismic coefficients (dynamic shear coefficients to design seismic coefficients). In the figure, dynamic shear coefficients denote the ratios of dynamic response

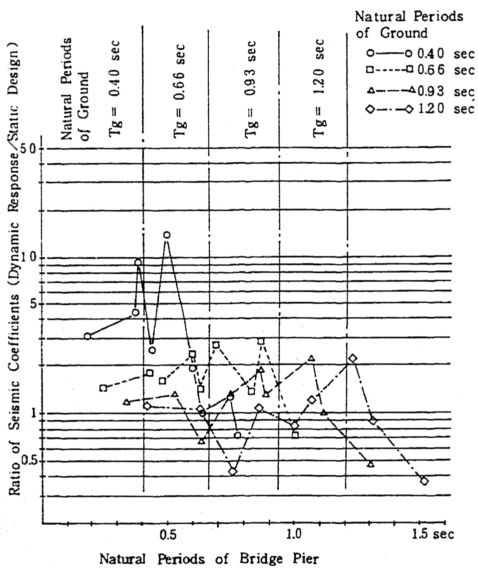

Fig.8 Ratio of Seismic Coefficients
(Dynamic Response to Static Design)

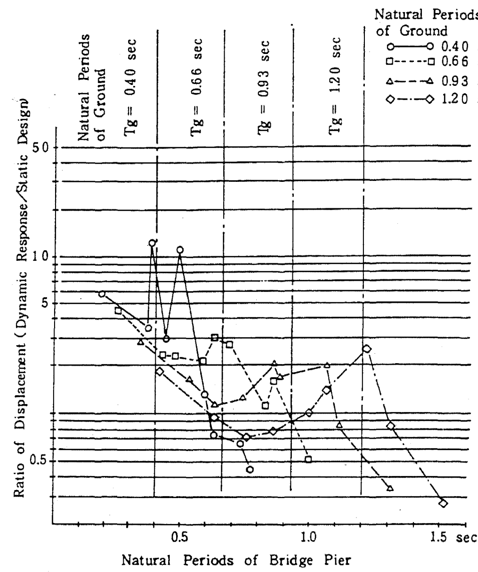

Fig.9 Ratio of Displacement
(Dynamic Response/Static Design)

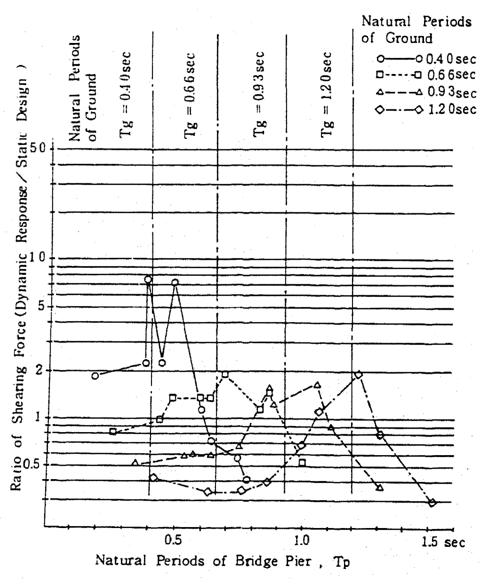

Fig.10 Ratio of Shearing Force
(Dynamic Response / Static Design)

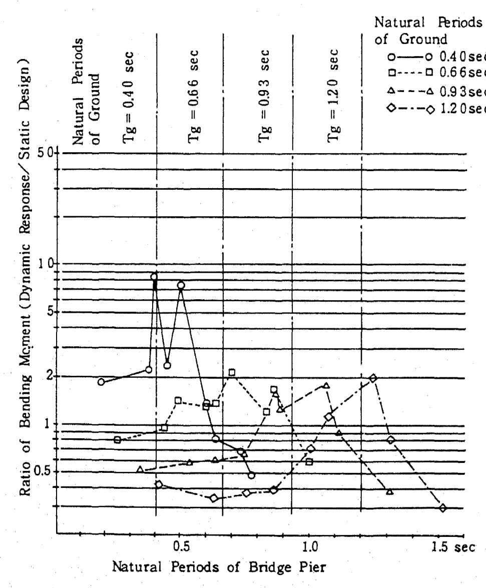

Fig.11 Ratio of Bending Moment
(Dynamic Response / Static Design)

shear forces applied at the bases of pier columns to the total weight of the superstructure and the pier column. The ordinate of Fig.12 can be seen as an index representing amplification factors between dynamic forces actually applied during an earthquake and static design forces. It is understood from the figure that the amplification factor is small for the cases where the natural periods of bridge piers are different from the natural periods of grounds, although the amplification factor is rather great for the cases when the both natural periods become close.

Next, Fig.13 shows a relationship between the ratios of natural periods

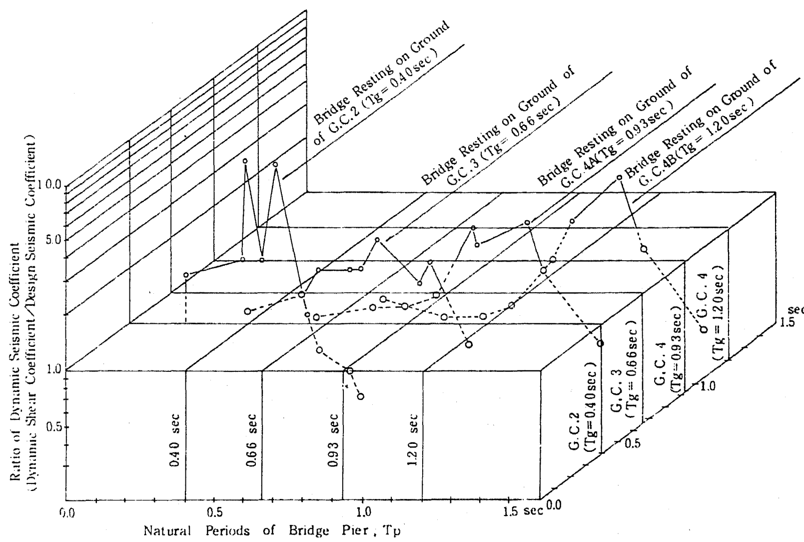

Fig.12 Pier Period Ground Period and Dynamic Magnification Factor (Ground Reaction Reduced)

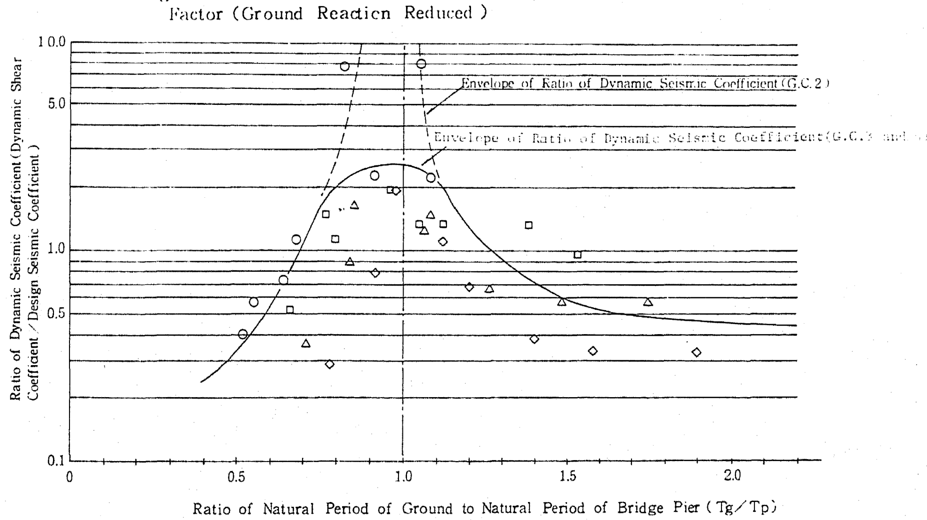

Fig.13 Ratio of Pier Period to Ground Period Versus Dynamic Magnification Factor (Ground Reaction Reduced)

of grounds to natural periods of bridge piers (Tg/Tp) and the ratios of seismic coefficients. In the figure the dotted curve is the envelope (upper limit) of the ratios for the ground natural period of 0.40 seconds, and the solid curve is the envelope (upper limit) for cases of ground natural periods of 0.66 to 1.20 seconds. The ratios become 1 or greater for the cases of Tg/T=0.7 to 1.25. The ratios are 8 or so for a ground with natural period of 0.40 seconds, and are 2 to 3 at most for ground with natural period of 0.66 to 1.20 seconds.

With view of the above results, the ratios of dynamic response values to static design values can be evaluated from the values of Tg/Tp. The values of Tg/Tp can be approximately estimated from T_G^*/Tp^*, where $T_G^*=1.25\ T_G$, $Tp^*=2.01\sqrt{\delta}$, and δ:displacement at pier top (in meter) when subjected to lateral loading of 1-g acceleration.

CONCLUSIONS

Dynamic analyses for nine different bridge pier systems resting on four different soils with natural periods of 0.3, 0.5, 0.7, and 0.9 seconds, are conducted. In the analyses, average response spectra having the peak acceleration of 100 gals at the bedlayer, are employed as input seismic motion to lumped-mass-spring systems which simulate soils and bridge structures. From the analyses the following can be derived:

(1) Seismic responses of a bridge pier become great when the natural period of the surrounding ground (Tg) approximately coincides with that of the bridge pier (Tp).

(2) For a range of Tg/Tp=0.7 to 1.25, the dynamic magnification factor defined as the ratio of dynamic shear coefficients to design seismic coefficient, becomes great than unity. For soft ground conditions, the factor becomes 2 to 3 when Tg/Tp=1.0. The factor gets larger up to 10, when Tg/Tp=1.0 for hard ground conditions.

(3) The natural period of the ground (Tg) determined from the dynamic analysis approximately coincides with the characteristic value of the ground in earthquake time ($T_G^*=1.25\ T_G$, where $T_G=4H/Vs$). The natural period of the bridge pier system (Tp) obtained from the dynamic analysis is approximately equal to the natural period of the bridge pier (Tp*). From this fact, Tg/Tp can be approximately estimated by T_G^*/Tp^*. It will be concluded that a bridge pier with the value of Tg/Tp or T_G^*/Tp^* of 0.75 to 1.25 might have a high potential to be damaged during a strong earthquake.

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