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SEISMIC ANALYSIS SYSTEM OF BRIDGE PIER WITH PILE FOUNDATION IN ARIAKE SOFT CLAY REGION

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ABSTRACT: In our earlier paper which appeared in IALT, it is pointed out that design of bridge pier with big seismic loading needs special attention such as consideration of ground displacement, soil-pile interaction effect etc., when foundation piles penetrate through soft clay layer and ground displacement largely depends on soil shear wave velocity, V_s & strain dependence of G/G_0. The Road Bridge Code in Japan states that shear wave velocity, V_s can be considered 50 m/sec in soft clay having SPT N-value zero. In this study, seismic analysis was carried out considering three cases: Case I using measured V_s value, Case II where $V_s =$ 50 m/sec for all layers and Case III where V_s is calculated from the Railway Bridge Standard in Japan formula for the soft clay layer. Both of Penzien model and single input model analysis were performed. The bridge structure used in the analysis was first designed by Seismic Co-efficient Method and Ductility Design Method. In dynamic analysis, non-linear elasto-plastic material behavior was considered for piles. Linear pile behavior case was also performed. In the former case responses mainly displacement and bending moment were found less compared to linear case. Responses in Case II were found much higher than other two cases and would result very uneconomical design. Penzien model analysis system with non-linear pile material consideration is proposed for analysis of bridge pier with pile foundation in Ariake soft clay region. It is emphasized that V_s and also strain dependence of G/G_0 be precisely measured in the soft clay region because of their big influence in seismic analysis in soft clay region. Difference between responses in Case I and Case III were found small. In the unavailability of measured data, V_s may be calculated by the Railway Bridge Standard formula.

INTRODUCTION

The pathetic lessons from past severe earthquakes urge the engineers to establish proper design guidelines for earthquake resistant design of bridges in soft ground. The great Kobe earthquake of January 17, 1995 with magnitude 7.2 occurred at very short distance from urban area (inland earthquakes) and caused extensive damages to Kobe, Osaka and surrounding regions in Japan. Many highway and railway bridges were completely destroyed.

Saga region is a marine deposit which consists of thick soft clay layer varying from 5 m to about 30 m thickness. This presents a difficult problem for the design of bridge piers or other structures such as multistory buildings with pile foundation penetrating this soft clay and resting on base sand layer. At present, it has been planned to construct high standard overhead-style motorway through this soft ground region. Soft clay layer possesses low shear wave velocity which can result to large ground displacement. In this situation, effect of

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ground displacement must be considered (Aramaki et al. 2001) during analysis thereby soil-pile interaction analysis should be performed. This situation is reflected in the Railway Bridge Standard (Railway Technical Research Institute 1999) revised in October, 1999 where it recommends the use of Penzien model (Joseph Penzien et al. 1964) for soft clay situation, thus ground displacement and soil-pile interaction are accounted for.

In an earlier paper (Mahmudur et al. 2001) it is pointed out that ground displacement input (free field ground displacement calculated separately is inputted in the integrated structural model), soil-pile interaction effect etc. should be considered during seismic analysis (Robert L. Wiegel et al. 1970) of pile-supported bridge piers in soft clay region. Ground displacement largely depends on shear wave velocity, V_s and also on strain dependence of rigidity ratio (G/G_0). *Go* is the shear modulus at small strain range and G is the variable shear modulus at any strain level. Large free field ground displacements are obtained for very low shear wave velocity (Mahmudur et al. 2001). The Road Bridge Code (Japan Road Association 1997) recommends a V_s value of 50 m/sec only in soft clay with SPT N-value equal to zero irrespective of the variation of soil cohesion value with depth. However, the Railway Bridge Standard recommends a formula correlating soil cohesion value with V_s. Usually, a gradual increase of soil cohesion value is obtained with depth and thereby a gradual increase in V_s value can be obtained by the Railway Bridge Standard formula. In this study, three cases of V_s value determination in the top clay layer were considered–

Case I : Measured V_s value

Case II : V_s = 50 m/sec based on the Road Bridge Code

Case III : V_s from The Railway Bridge Standard formula

Non-linear behavior of pile was considered in this study. In the former paper, linear pile behavior only was considered and the analysis was performed on a typical standard bridge structure having concrete deck and girders. The bridge structure considered here was first designed by Seismic Co-efficient Method and Ductility Design Method. The bridge is based on Ariake soft clay region. Steel deck and girders were selected instead of concrete ones. This study aims at establishing proper seismic-resistant design guidelines for pile foundation-superstructure system in Ariake soft clay region. Analysis was carried out by elasto-plastic dynamic analysis (Ray W. Clough et al. 1993) considering ground displacement and the whole foundation-superstructure system as one body. Both Penzien model which considers soil-pile interaction effect and single input model are considered in analysis. For comparison purposes, linear pile behavior case was also performed in this study.

SOIL PROPERTIES AT THE BRIDGE SITE

The bridge structure considered here is designed for Ariake clay which constitutes Saga plain and consists of soft surface clay layer varying 5 m to 30 m in thickness. At the bridge site the clay layer thickness is 19.5 m. Shear wave velocity in the clay layer is low (70 - 100 m/sec). The SPT N-value in the clay layer is zero (Ueda et al. 1998). However, a gradual increase of cohesion value is observed. By correlating cohesion value with V_s, a gradual increase in V_s can be obtained as is recommended by the Railway Bridge Standard. V_s is an important parameter in seismic response analysis. The Road Bridge Code formula for calculating V_s is shown below.

For sand

$$V_s = 80N^{1/3} \qquad (N \le 50) \tag{1}$$

For clay

$$V_s = 100N^{1/3} \qquad (1 \le N < 50) \tag{2}$$

$$V_s = 50 \qquad (N \text{ value zero}) \tag{3}$$

The Railway Bridge Standard formula for clay having SPT N-value less than 2 is given below.

$$V_s = 0.85 \times 120\left(\frac{q_u}{100}\right)^{0.36} \qquad (N < 2) \tag{4}$$

where, V_s = Shear wave velocity in m/sec, q_u = Unconfined compressive strength in kN/m^2.

Other necessary parameters for the analysis, such as vertical spring constant at the pile tip, lateral coefficient of sub-grade reaction, upper limit of sub-grade reaction, lateral yield displacement, equivalent damping of each structural element, etc. were obtained based on the equations proposed in the Road Bridge Code in Japan. Typical values of the above parameters in this model for Case I are 5.07e5 kN/m, 1.09e4 kN/m^3 for V_s = 70 m/sec, 102.47 kN/m^2 at 3.5 m depth, 5.37 mm at 3.5 m depth, 2% for non-linear case respectively. Important soil parameters of Ariake clay used in this study are summarized in Table 1.

Table 1 Soil properties

Description	Layer Thickness (m)	N value	Density (kN/m^3)	Friction angle (degree)	q_u (kPa)	Shear wave velocity, V_s (m/sec)		
						Case I	Case II	Case III
Soft clay	2.5	0	15.0	0.0	58.8	70.00	50.00	84.20
	4.0	0	15.0	0.0		90.00	50.00	84.20
	3.0	0	15.0	0.0		70.00	50.00	84.20
	2.0	0	15.0	0.0	68.6	100.00	50.00	89.10
	2.0	0	15.0	0.0	88.2	100.00	50.00	97.50
	2.0	0	15.0	0.0	107.8	100.00	50.00	104.80
	2.0	0	15.0	0.0	127.4	100.00	50.00	111.30
	2.0	0	15.0	0.0	147.0	100.00	50.00	117.20
Sand	4.0	30	18.0	35.0	0.0	248.58	248.58	248.58
Clay	3.0	24	17.0	0.0	300.0	288.45	288.45	288.45
Clay	4.0	3	15.0	0.0	60.0	144.22	144.22	144.22
Sand	6.0	40	19.0	35.0	0.0	273.60	273.60	273.60

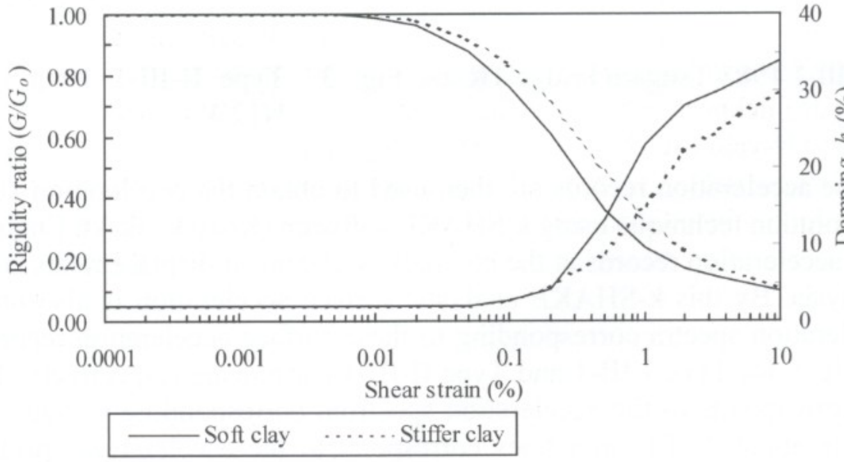

Fig. 1 Strain dependence of rigidity ratio and damping of clay

Strain dependence of rigidity ratio (G/G_0) also has influence on free field soil displacement. During major earthquakes, high strain level occurs in the soil deposit where rigidity ratio decreases rapidly. In the soft clay G/G_0 decreases more sharply than in stiffer clay. Strain dependency of (G/G_0) in Ariake clay region is obtained from Ueda et al. 1998 and shown in Fig. 1.

ACCELERATION RECORDS

Large changes in the earthquake resistant design codes in Japan such as the Road Bridge Code and the Railway Bridge Standard were made after experiencing the devastating Kobe earthquake-1995. Type II ground motion which developed during earthquake with magnitude of about 7-7.2 at very short distance from urban area, in addition to Type I ground motion which developed in the plate-boundary type earthquake with magnitude of about 8 was included in the codes. The codes recommend to use three ground motions for each type. For type I earthquake, "TypeI-III-1 1983 Tsugaru bridge TR", "TypeI-III-2 1983 Tsugaru bridge LG" and "TypeI-III-3 Kushirogawa LG" records were used. For type II earthquake, "TypeII-III-1 1995 Higashi Kobe N12W", "TypeII-III-2 1995 Kobe port island N-S" and "TypeII-III-3 1995 Kobe port island E-W" were used in this study. In the notation of earthquake records, the first index indicates level of earthquake (I or II), next index indicates soil type (III → soft soil) and the last one is a number designation. These records were corrected to fit the Road Bridge Code standard acceleration spectrum by using ARTEQ software (Kozo Keikaku Engineering 2000). ARTEQ software is for making artificial earthquake acceleration fitted to target response spectrum using Fourier transform technique. Modification coefficient for zone, cz = 0.7 for Saga region were used. Ground records TypeI-III-1 (max 398 cm/sec^2) and TypeII-III-1 (max 401 cm/sec^2) thus obtained are shown in Fig. 2 and Fig. 3.

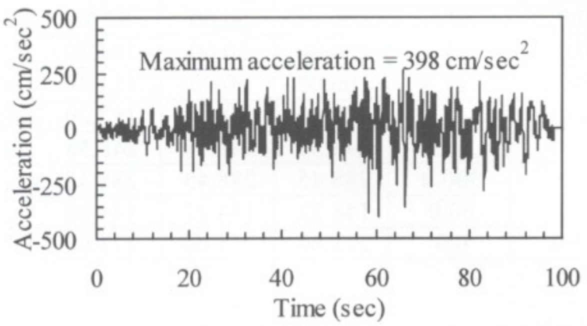

Fig. 2 Type I-III-1 1983 Tsugaru bridge TR

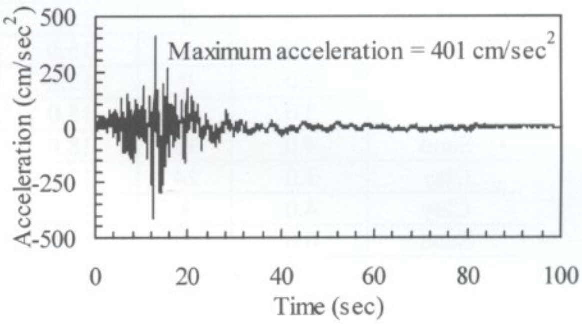

Fig. 3 Type II-III-1 1995 Higashi Kobe N12W

These surface acceleration records are then used to obtain the acceleration at the base bed rock by deconvolution technique using k-SHAKE software (Kozo Keikaku Engineering 2000). Next using the acceleration records at the bed rock, soil deposit displacements are obtained by non-linear analysis. By this k-SHAKE analysis, surface acceleration is also obtained again. Response acceleration spectra corresponding to these surface acceleration records are shown in Fig. 4 and Fig. 5 for Type I-III-1 and Type II-III-1 earthquake respectively. The thick line in the graphs corresponds to the acceleration spectrum corresponding to Saga region where natural period is about 2. The thin lines correspond to the acceleration spectrum fitted to standard spectrum. Dashed line is the standard acceleration spectrum. The surface acceleration time histories after k-SHAKE analysis are also shown in Fig. 6 and Fig. 7 respectively.

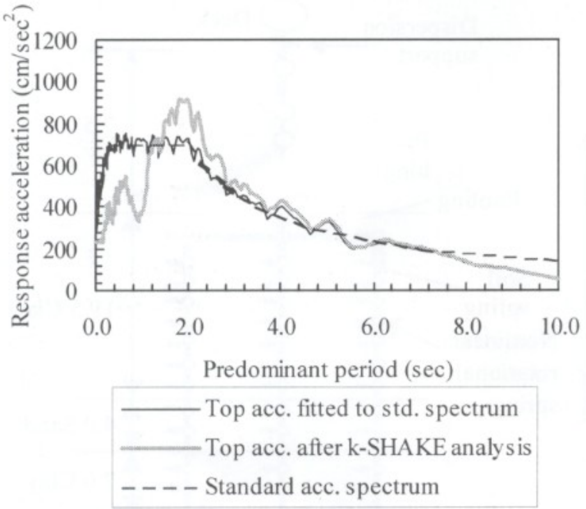

Fig. 4 Acceleration spectra for Type I-III-1

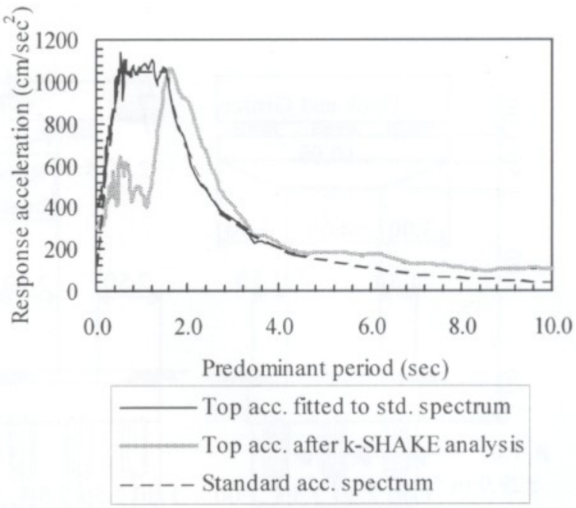

Fig. 5 Acceleration spectra for Type II-III-1

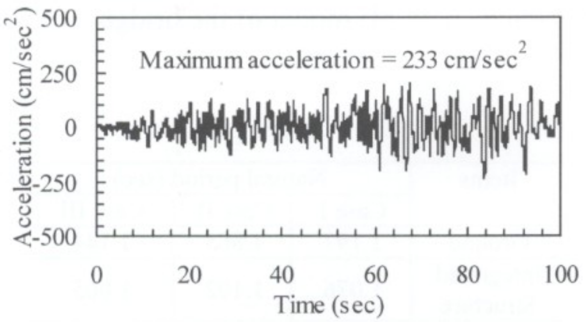

Fig. 6 Type I-III-1 1983 Tsugaru bridge TR top-acc, after k-SHAKE analysis

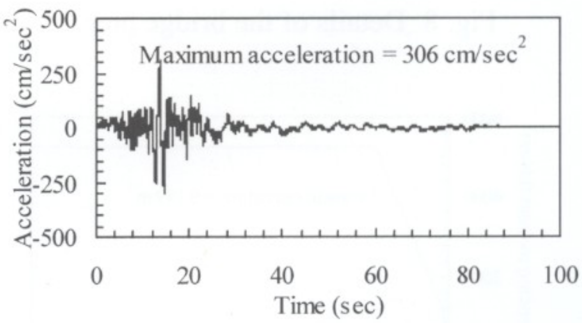

Fig. 7 Type II-III-1 1995 Higashi Kobe N12W top-acc, after k-SHAKE analysis

THE BRIDGE MODEL FOR ANALYSIS

The bridge structure considered for this study was first designed by Ductility Design Method of the Road Bridge Code based on Ariake clay region. Since in soft clay region, structural responses to seismic loading are high, attempt was made to reduce the weight of deck and girders. Instead of concrete, steel deck and girder were chosen. Detailed dimension of the bridge pier and pile foundation are shown in Fig. 8. The pier is reinforced concrete structure 4.0×2.0 m rectangular in plan supported by 9 (3×3), 1.00 m diameter circular RC piles. Corresponding analytical model is prepared and is as shown in Fig. 9 to be used in SESAS program (Imamura et al. 2000). Bending moment-curvature relations for the pile is shown in Fig. 10. SESAS program was developed in Structural System Laboratory, Saga University. It is a finite element program for linear and non-linear structural analysis. It can handle static and elasto-plastic analysis, eigen-value analysis, dynamic elasto-plastic analysis.

It incorporates many material non-linear models such as bi-linear, tri-linear, tri-linear Takeda etc.

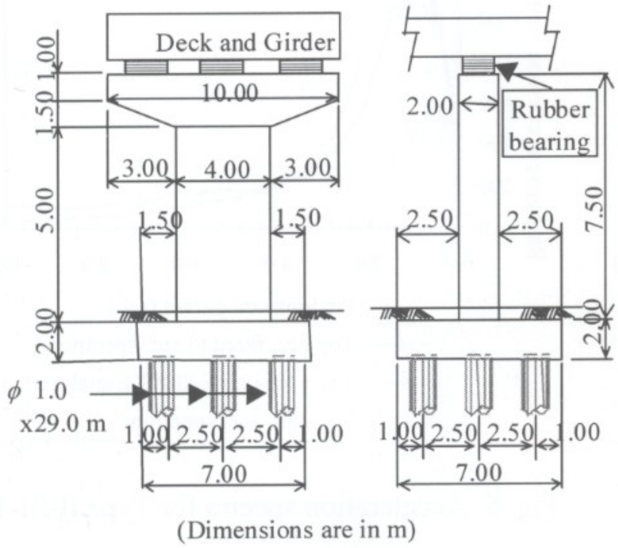

Fig. 8 Details of the bridge pier

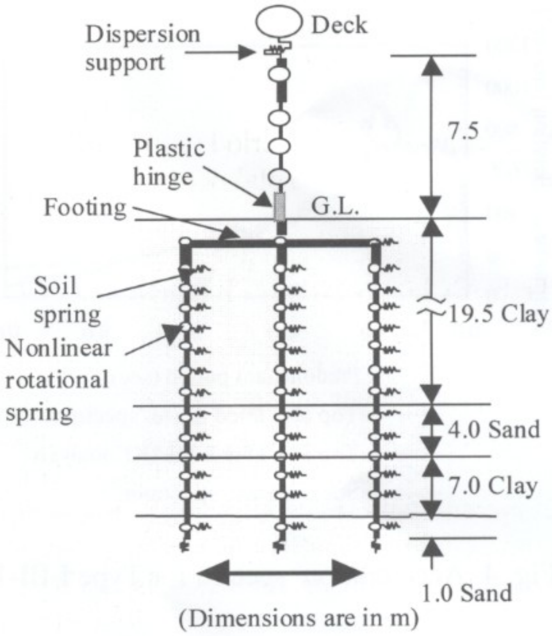

Fig. 9 FE model of the bridge

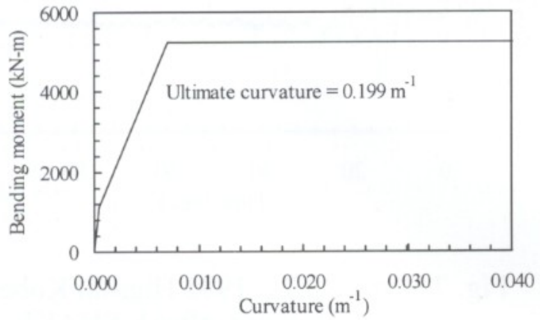

Fig. 10 Bending moment-curvature relation of pile at top end portion

Table 2 Natural periods

Items	Natural period (sec)		
	Case I	Case II	Case III
Ground	1.197	1.865	1.144
Integrated Structure	1.076	1.102	1.065

This program is very suitable to carry out dynamic analysis which the Railway Bridge Standard recommends i.e Penzien model analysis and for general purpose structural analysis.

Lumped mass model was used for piles and the pier. Each pile was divided into 16 elements. Concentrated mass at the top accounts for the total weight of bridge deck, girders etc. The concentrated elasto-plastic bilinear lateral soil springs (Fig. 9) at each nodal position were adopted in the dynamic analysis. Shear (vertical) springs are not included. Following the Road Bridge Code, plastic hinge was introduced at the bottom level of bridge pier. Tri-linear Takeda model (Takeda et al. 1970) was used in the analysis for this plastic hinge. Takeda model is suitable to account for non-linear behavior of RC elements. Calculation of soil spring parameters, plastic hinge parameters were carried out based on the Road Bridge Code.

Natural period of the ground and the integrated structural model are shown in Table 2. Natural period of the ground was calculated by the Road Bridge Code equation as shown in Eq. (5) for small strain level. At high strain level as occurs in major earthquake, shear modulus, G is reduced and natural period of the ground is increased. Natural period of the integrated structural model was obtained by SESAS analysis of the structure.

$$T_G = 4\sum_{i=1}^{n} \frac{H_i}{V_{si}} \tag{5}$$

where, T_G = Natural period of ground, H_i = Thickness of *ith* layer, V_{si} = Shear wave velocity of *ith* layer and n = Total no. of layer.

THE ANALYSIS PROCEDURE

At first, to check the effect of strain dependence of G/G_o and damping, a simple ground model consisting of one 19.5 m clay (with V_s = 80 m/sec) layer resting on base sand was analyzed by k-SHAKE software. Four nos. of analysis: No. 1 using pattern 1 (soft clay, Fig 1) strain curve and pattern 1 (soft clay) damping, No. 2 using pattern 1 (soft clay, Fig. 1) strain curve and pattern 2 (stiffer clay, Fig. 1) damping, No. 3 using pattern 2 (stiffer clay, Fig 1) strain curve and pattern 1 damping, No. 4 using pattern 2 strain curve and pattern 2 damping were performed. In this analysis, for Type I earthquake, "Kushirogawa LG" and for Type II earthquake "Higashi Kobe N12W" earthquake records were used.

For seismic analysis of structures in soft clay region, it is necessary to consider soil-pile interaction effect since large ground displacement occurs in such situation. In this study, analysis was carried out by both Penzien model and single input model concept. In Penzien model analysis pile-soil interaction effect is taken into account by the consideration of ground displacement input. On the other hand, the Road Bridge Code considers only single input model where the free-field ground is considered fixed. The models are described in the articles below.

In the first phase, bending moment rotation behavior of pile in both models was considered linear which are mentioned by the codes. In the second phase, non-linear elasto-plastic pile material behavior was considered. Tri-linear Takeda model was used for non-linear bending moment-rotation behavior of piles. These rotational spring elements were introduced at the nodes of piles.

Penzien Model Analysis

The Railway Bridge Standard states that in complicated ground condition and structure the response is to be calculated based on the whole pile-soil-superstructure system as one body with due consideration to soil-pile interaction effect. Penzien model (Fig. 11) which considers pile-soil interaction effect, is recommended for analysis. In this model, the analysis is separated into two parts; the determination of the dynamic response of the clay medium alone when excited through its lower boundary by prescribed horizontal acceleration and the analysis of the whole system with ground displacement input obtained from the first part.

In the integrated structural model, calculated time histories of free-field displacements are inputted at the far ends of the elasto-plastic soil springs (Fig. 11) and dynamic response analysis of the whole system is performed by SESAS program.

Single Input Model Analysis

In the single input model, it is assumed that the far ends of the elasto-plastic soil springs are fixed. Thus ground displacements are not considered. The prescribed acceleration such as

Type I and Type II earthquakes is input at the bottom level of the piles and the integrated model is analyzed by SESAS. The single input model is shown in Fig. 12. Concentrated elasto-plastic soil springs are used in both of the models.

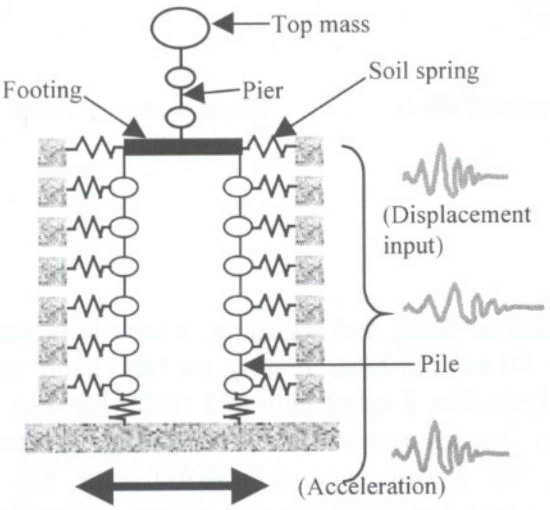

Fig. 11 Penzien model

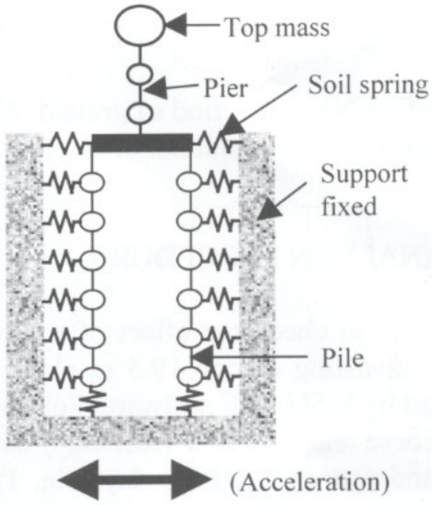

Fig. 12 Single input model

RESULTS AND DISCUSSION

Maximum free-field surface displacement obtained by k-SHAKE analysis of the simple ground model are summarized in Table 3. It is observed from Table 3 that strain dependence of (G/G_o) has large influence on the model while damping has smaller influence.

Table 3 Maximum free-field displacement of simple ground model

Earthquake type	Maximum displacement (cm)			
	Analysis No. 1	Analysis No. 2	Analysis No. 3	Analysis No. 4
Type I	21.86	19.37	11.70	11.63
Type II	31.01	30.92	23.94	24.35

For the bridge model, both Penzien model which considers soil-pile interaction effect and single input model analysis were performed. Shear wave velocity calculation of the soft clay layer was considered in three ways as explained before.

In Penzien model analysis, free field ground displacements are obtained first by k-SHAKE analysis which are shown in Figs. 13, 14 and 15 below for three cases. It is observed that in Case II, soil displacements are large compared to other two cases because of very low V_s value. However, between Case I and Case III, difference in soil displacements are small. In Case II, at about 18.0 m depth a sharp change of displacement occurs. Change of V_s value at this depth is also big compared to other cases.

SESAS program developed in Saga University Structural Analysis Laboratory is then used to carry out Penzien model and single input model analysis by considering non-linear elasto-plastic pile material behavior. Linear pile behavior case was also performed. Maximum bending moment occurred in the pile by SESAS analysis for all cases of analysis are summarized in Table 4. Distribution of maximum bending moments throughout the pile are presented in Figs. 16, 17 and 18 for Type II-III-1 seismic record. Pile capacity is shown in Table 5. Maximum pile bending moment (1636 kN-m) by Seismic Co-efficient Method was much below yield moment (5250 kN-m). In this method design was controlled by pile displacement limitation of 15 mm. In Penzien model analysis, maximum pile bending moment reached yield limit in Case I and Case II and approached to yield limit in Case III. However, in single input model analysis, maximum bending moment (Table 4) was also much below yield limit. Penzien model analysis should be adopted in seismic analysis of bridge pier with pile foundation in soft clay region from safety point of view.

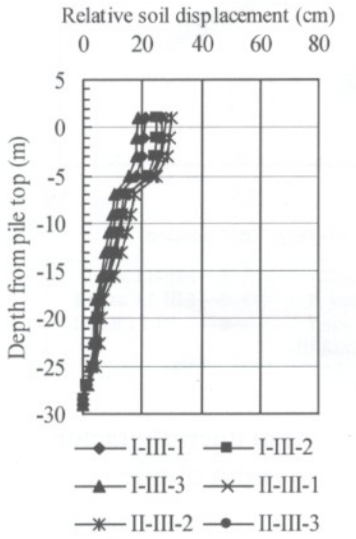

Fig.13 Relative soil displacement-Case I

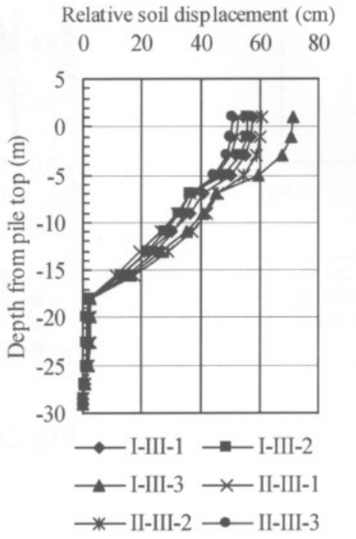

Fig. 14 Relative soil displacement – Case II

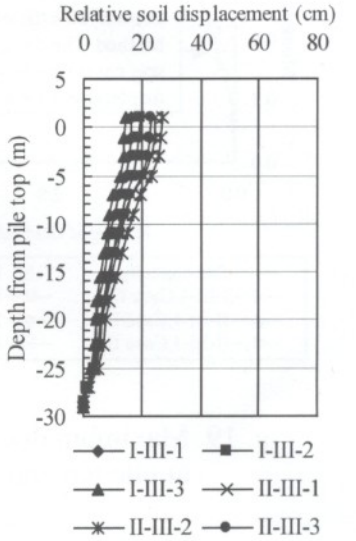

Fig. 15 Relative soil displacement – Case III

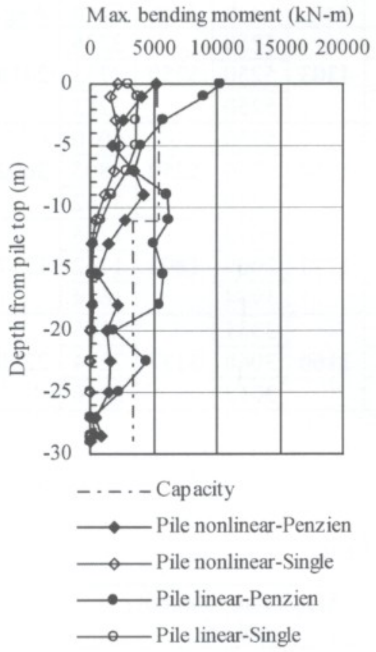

Fig. 16 Max. bending moment in pile in Case I for Type II-III-1 record

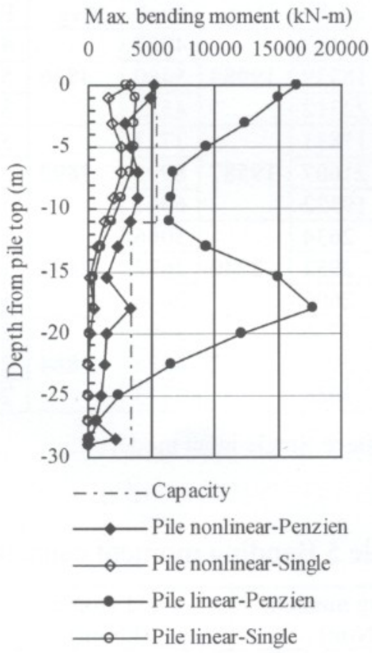

Fig. 17 Max. bending moment in pile in Case II for Type II-III-1 record

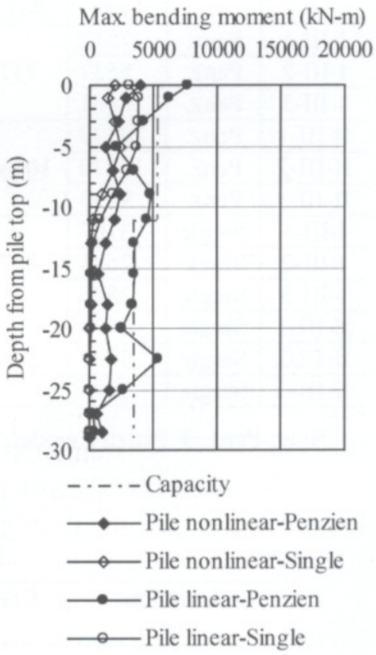

Fig. 18 Max. bending moment in pile in Case III for Type II-III-1 record

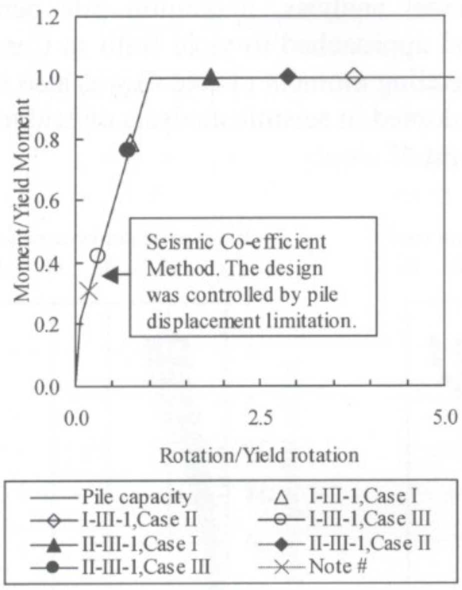

Fig. 19 Maximum ductility factor at pile top end

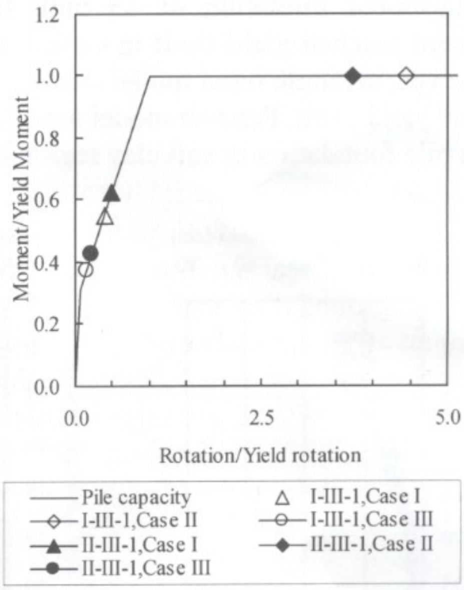

Fig. 20 Maximum ductility factor at 18 m depth of pile

Table 4 Maximum bending moment in the pile

Seismic record	Model	Pile max bending moment (kN-m)											
		Pile linear						Pile nonlinear					
		Case I		Case II		Case III		Case I		Case II		Case III	
		Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.
I-III-1	Penz.	7512		19117		4727		4131		5250		2226	
I-III-2	Penz.	8537	7570	18539	19989	5460	4836	5182	4303	5250	5250	2733	2414
I-III-3	Penz.	6661		22312		4322		3595		5250		2283	
II-III-1	Penz.	10339		17831		7723		5250		5250		4010	
II-III-2	Penz.	10870	10367	21607	19587	8993	7892	5250	5250	5250	5250	3805	3692
II-III-3	Penz.	9893		19322		6961		5250		5250		3262	
I-III-1	Single	3107		2634		3086		1864		1675		1775	
I-III-2	Single	2688	2872	2982	2740	2692	2809	1816	1773	2069	1893	1762	1717
I-III-3	Single	2820		2605		2648		1640		1934		1614	
II-III-1	Single	3708		3706		3904		2282		3331		2385	
II-III-2	Single	3822	3742	3923	3772	4034	3884	2116	2160	3068	3137	2224	2228
II-III-3	Single	3697		3686		3713		2083		3012		2076	

Note:-Penz.→ Penzien model, Single→ Single input model

Table 5 Bending moment capacity of pile

Location	Cracking moment (kN-m)	Yield moment (kN-m)	Maximum moment (kN-m)
At pile top end	1130	5250	1636 (*)
Bottom portion	1058	3394	-

Note - * By Seismic Co-efficient Method. Design is controlled by pile displacement limitation of 15 mm.

Maximum ductility factor (Rotation/Yield rotation) by Penzien model in all three cases are shown in Fig. 19 for pile top end and in Fig. 20 for 18 m depth from pile top. It is noted that for Case II, maximum bending moment occurred at 18 m depth of pile where soil displacement (Fig. 14) changes very sharply. It is found that in Case II where shear wave velocity, V_s is considered constant 50 m/sec for soft clay region with SPT N-value zero, pile bending moments are very high compared to Case I and Case III. Differences in response between Case I and Case III were found small. V_s is a very important parameter in analysis by Penzien model. For low V_s value, rigidity ratio (G/G_o) decreases sharply in the high soil strain level and also shear modulus, G decreases parabolically with V_s ($G = \rho V_s^2$). These cause large free field ground displacement (Fig. 13, 14 & 15). In Penzien model where soil displacements are input at the far ends of the soil spring, pile bending moments and displacements increase sharply with decrease of V_s value (Mahmudur et al. 2001). Maximum ductility factor occurred in Case II is 4.97. Highly uneconomical design would be produced by Case II. Generally an increase in soil cohesion value is observed at higher depth (Table 1). By relating this with V_s, increase in V_s with depth can be obtained as in Case III. The formula by the Railway Bridge Standard is used in this case. Here, it is emphasized that V_s and also strain dependence of G/G_0 should be precisely measured in the soft clay region. In this study, measured values of V_s and strain dependence of G/G_o were taken from Ueda et al. 1998. In the unavailability of measured data, V_s is proposed to be calculated by the Railway Bridge Standard formula.

Table 6 Maximum displacement at the deck level

Seismic record	Maximum displacement (cm) at deck level											
	Penzien model						Single input model					
	Case I		Case II		Case III		Case I		Case II		Case III	
	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.
I-III-1	30.95		76.37		25.88		23.08		22.25		22.32	
I-III-2	40.05	33.23	79.69	87.44	34.36	28.68	22.73	22.66	26.51	24.71	21.74	21.68
I-III-3	28.69		106.27		25.80		22.16		25.36		20.97	
II-III-1	52.90		66.82		42.45		34.58		37.04		35.59	
II-III-2	42.96	45.69	83.09	71.77	37.00	38.72	33.96	33.71	36.96	37.16	33.86	34.43
II-III-3	41.21		65.40		36.71		32.58		37.49		33.83	

Table 7 Maximum displacement at the footing level

Seismic record	Maximum displacement (cm) at footing											
	Penzien model						Single input model					
	Case I		Case II		Case III		Case I		Case II		Case III	
	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.
I-III-1	24.46		69.01		17.66		3.41		4.98		3.45	
I-III-2	30.54	25.45	69.31	78.21	21.56	18.32	3.67	3.55	5.89	5.51	3.24	3.37
I-III-3	21.35		96.30		15.74		3.57		5.65		3.43	
II-III-1	35.74		61.07		28.17		6.06		9.87		5.76	
II-III-2	35.36	34.59	69.59	62.68	28.97	27.45	5.16	5.34	8.95	9.44	5.40	5.21
II-III-3	32.66		57.38		25.20		4.79		9.50		4.48	

Maximum displacement occurred in deck level are shown in Table 6 for non-linear pile model. Footing level displacements also are shown in Table 7. Profile of maximum displacement from deck level to pile bottom end are presented in Fig. 21 for Penzien model and in Fig. 22 for single input model. The profiles are for Type II-III-1 seismic record. It is observed that pile displacements in Penzien model are much higher than those in single input model. However, rubber bearing (between deck and girder) displacements are higher in single input model (Figs. 21 and 22). Complete bending moment-time history at pile top end are shown in Fig. 23 for Case I, in Fig. 24 for Case II and in Fig. 25 for Case III. It is observed that pile undergoes some residual displacement due to non-linearity. Figures 26-28 show the complete displacement-time history at deck level for Case I, II and III respectively. The time-history curves are for Type II-III-1 seismic record and non-linear pile behavior consideration. Since in single input model, soil displacement is not considered (thereby soil-pile interaction effect is not considered), responses are found quite smaller in this model than those in Penzien model.

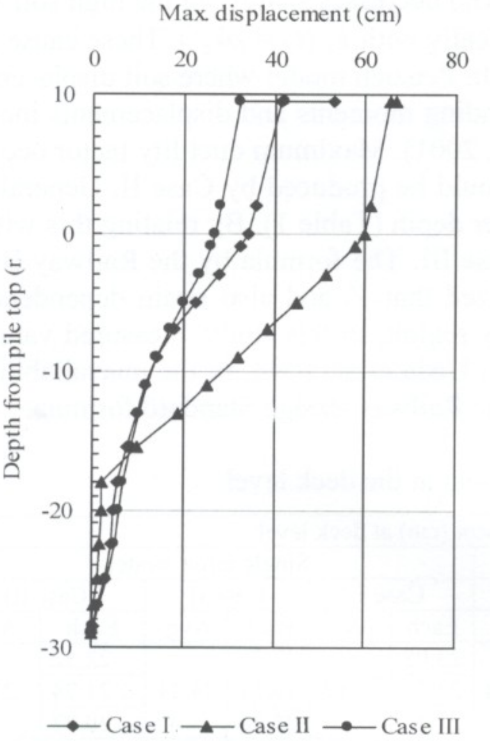

Fig. 21 Maximum displacement profile Penzien model

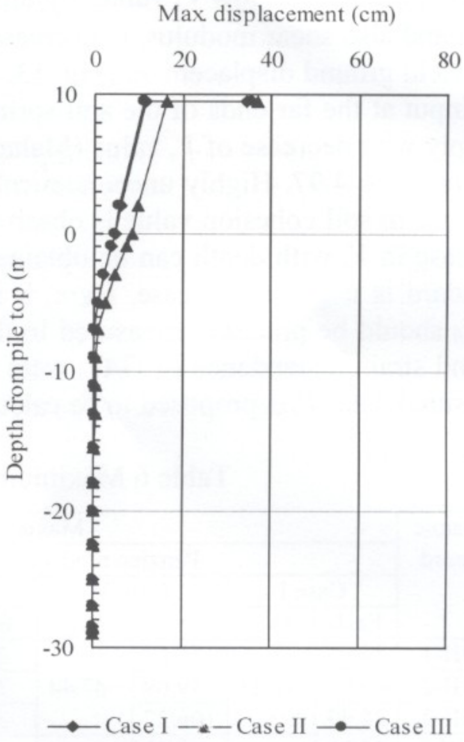

Fig. 22 Maximum displacement profile – single input model

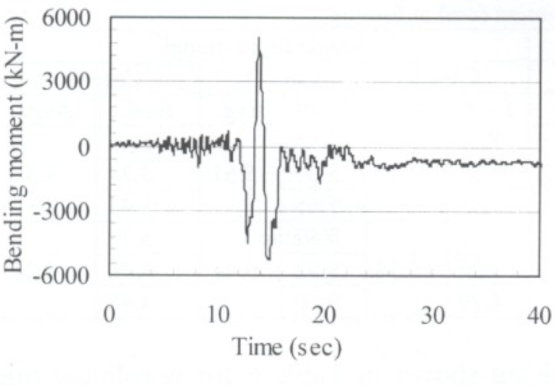

Fig. 23 Bending moment at pile top –Case I Penzien model, Type II-III-1

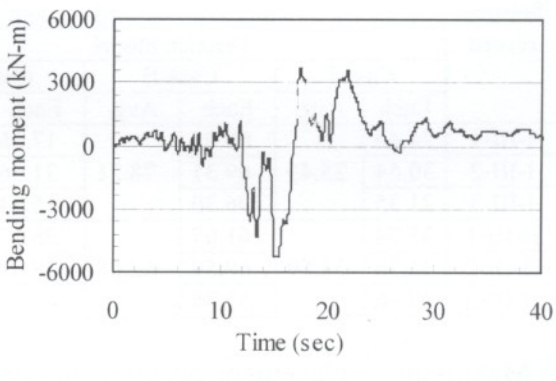

Fig. 24 Bending moment at pile top –Case II Penzien model, Type II-III-1

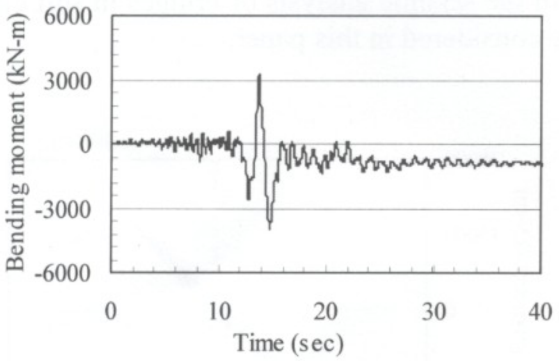

Fig. 25 Bending moment at pile top –Case III Penzien model, Type II-III-1

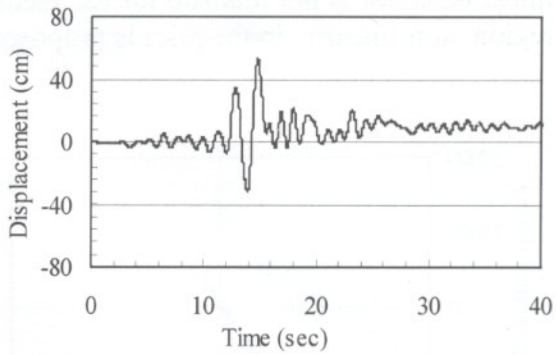

Fig. 26 Deck displacement –Case I, Penzien model, Type II-III-1

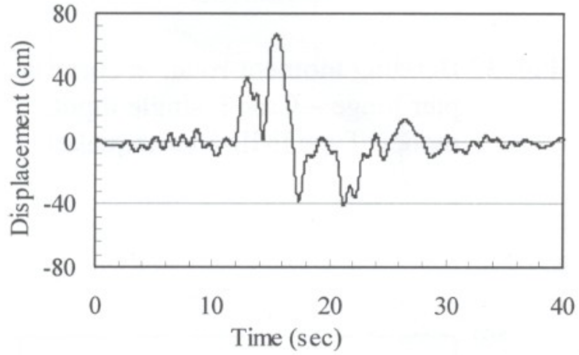

Fig. 27 Deck displacement –Case II, Penzien model, Type II-III-1

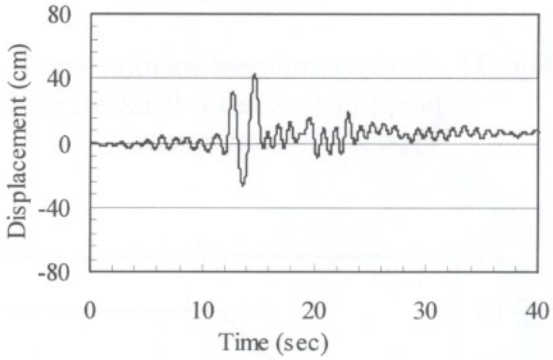

Fig. 28 Deck displacement –Case III, Penzien model, Type II-III-1

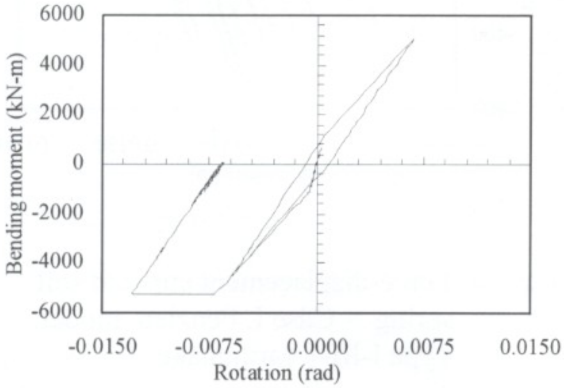

Fig. 29 Bending moment-rotation curve at pile top – Case I, Penzien model, Type II-III-1 earthquake

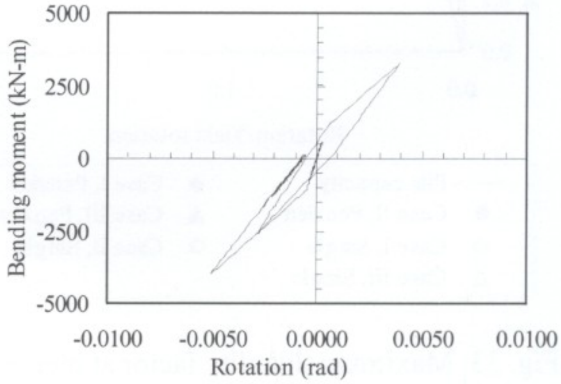

Fig. 30 Bending moment-rotation curve at pile top – Case III, Penzien model, Type II-III-1 earthquake

Bending moment rotation curve at the pile top end for Case I is shown in Fig. 29 and for Case III in Fig. 30 for Type II-III-1 earthquake. The curves show the non-linear histeretic behavior of the pile. From the results presented here, it is observed that pile bending moments are reduced by the consideration of non-linear pile behaviour. After exceeding cracking limit, non-linear histeretic behavior of the elements are considered in the non-linear case which cause histeretic damping and smaller bending moments are happened. Beyond cracking limit, linear behavior is not realistic for RC elements. In the seismic analysis of bridges in soft clay region, non-linearity in the piles is proposed to be considered in this paper.

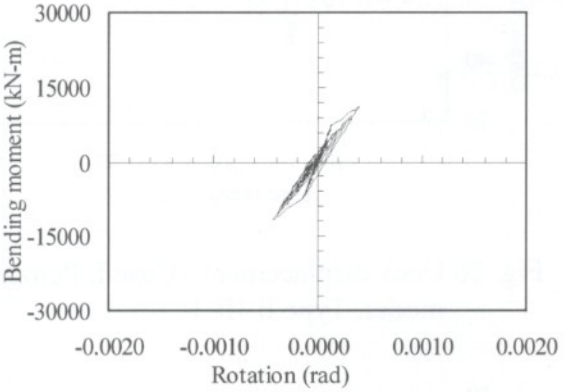

Fig. 31 Bending moment-rotation curve at pier hinge – Case I, Penzien model, Type II-III-1 earthquake

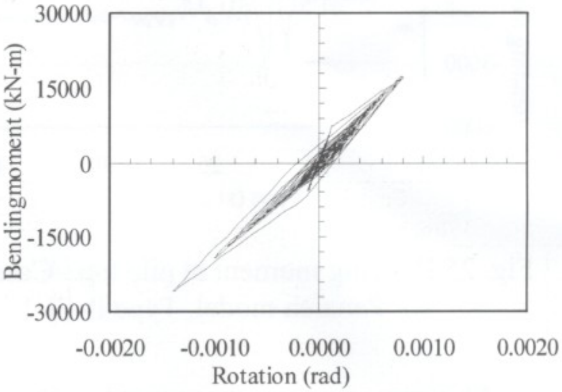

Fig. 32 Bending moment-rotation curve at pier hinge – Case I, single input model, Type II-III-1 earthquake

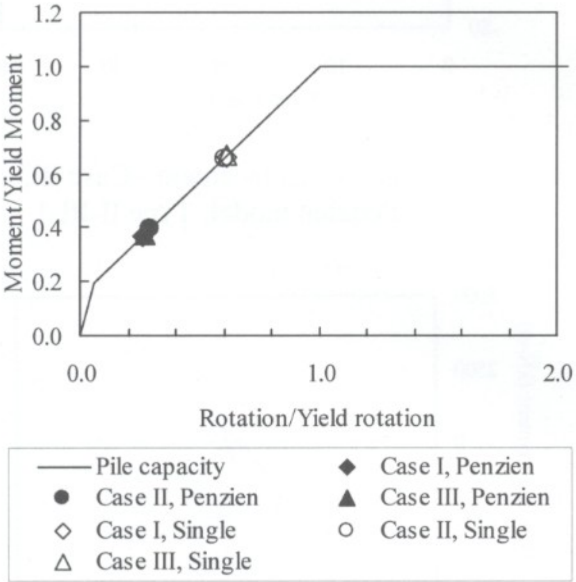

Fig. 33 Maximum ductility factor at pier hinge for Type II-III-2 earthquake

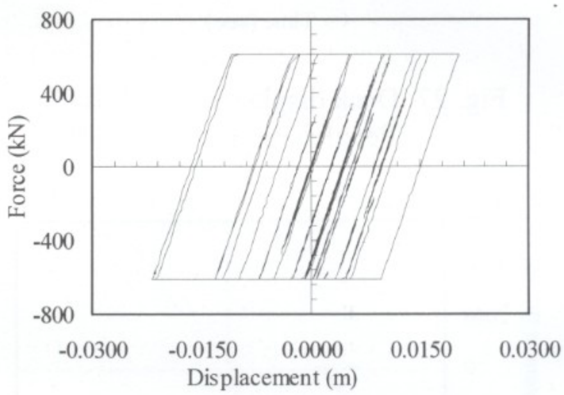

Fig. 34 Force-displacement curve at soil spring – Case I, Penzien model, Type I-III-1 earthquake

Bending moment rotation curve at the pier plastic hinge is shown in Fig. 31 for Penzien model and in Fig. 32 for single input model for Type II-III-1 earthquake. Ductility factor at the pier plastic hinge is also shown in Fig. 33 for Type II-III-2 earthquake. Maximum bending moments at pier plastic hinge for three cases of analysis are summarized in Table 8. In Penzien model, pier bending moments are quite smaller than yield bending moment of the pier which indicates pier capacity could be reduced more.

However, in single input model, pier bending moments are higher (Table 8). In single input model soil is considered fixed, pier base i.e footing level displacement is much smaller compared to Penzien model analysis (Table 7) and relative displacement between pier top and base is higher. In Penzien model pier base is more flexible than single input model case and smaller bending moment in pier happened. Force-displacement curve of soil spring at pile top level is shown in Fig. 34 which shows bilinear histeretic soil behaviour. In three cases of analysis, soil springs up to about 4 m below pile top level yielded.

Table 8 Maximum bending moment at pier plastic hinge

Record	Maximum bending moment (kN-m) at pier plastic hinge											
	Penzien model						Single input model					
	Case I		Case II		Case III		Case I		Case II		Case III	
	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.	Each	Avg.
I-III-1	12341	12989	10111	10533	12508	13740	17682	17243	16348	17996	17438	16799
I-III-2	14093		9637		14600		16577		19453		16227	
I-III-3	12532		11851		14113		17471		18187		16732	
II-III-1	11460	11655	9361	12544	10890	11791	25984	25100	27797	26404	26947	25769
II-III-2	13678		14669		13821		24968		24425		24923	
II-III-3	9826		13601		10661		24347		26990		25436	

Pile response in Type II earthquakes (inland earthquakes included in the codes after Kobe 1995 earthquake) are higher than responses in Type I earthquakes (plate boundary type earthquake). The codes are prudent to include Type II earthquakes in design. However, in Case II where V_s is considered only 50 m/sec for soft clay layer, Type I responses are somewhat higher. For very soft clay region which has longer natural period, this situation may happen. Both Type I and Type II seismic records should be carefully investigated during seismic analysis of bridges with pile foundation in soft clay situation.

Based on the results and discussion presented above, Penzien model analysis with non-linear pile behavior consideration is proposed in this paper for seismic analysis of bridges with pile foundation penetrating thick soft clay layer. It is emphasized that V_s in soft clay be measured by proper field tests. In the unavailability of field test results, the calculation of V_s is recommended to be done by the Railway Bridge Standard formula which correlates soil cohesion value with V_s for SPT N-value less than 2 instead of constant value of 50 m/sec. In Penzien model analysis soil deposit need to be analyzed first for which k-SHAKE for windows software is suitable. On the other hand, for subsequent soil-pile-superstructure integrated analysis with many non-linear elements, SESAS is very suitable.

CONCLUSIONS

This study aims at establishing the proper design guidelines for the analysis of bridge pier supported by pile foundations in Ariake soft clay region with SPT N-value zero. Based on the results and discussion presented above the following concluding remarks can be made–

1) Displacements and pile bending moments by single input model are much less than those by Penzien model which takes into account soil-pile interaction effect by ground displacement consideration. The later model should be adopted for seismic analysis in soft clay region from safety point of view.

2) Non-linear pile behavior should be considered instead of linear pile behaviour. Pile bending moments in the former case are less than those in the later case. Linear pile behavior is not realistic beyond cracking moment of the piles.

3) The Road Bridge Code recommends a V_s value of 50 m/sec for soft clay with SPT N-value zero irrespective of variation of soil cohesion value. Structural responses are very high by this consideration (Case II) compared to the case where measured values of V_s are

considered (Case I). The Railway Bridge Standard correlates V_s value with soil cohesion value for soft clay with SPT N-value less than 2. By this consideration (Case III), responses are much less than the former case and responses are closer to Case I. In this paper, it is emphasized that V_s and also strain dependence of G/G_0 be precisely measured in soft clay region by proper field tests. In the unavailability of measured data, V_s value may be calculated by the Railway Bridge Standard formula where V_s is correlated with soil cohesion value.

4) The bridge structure analyzed in this study was first designed by Seismic Coefficient Method and Ductility Design Method. To reduce the bridge weight, steel structure was chosen for deck and girders instead of concrete ones. In this method, maximum pile bending moments were much less than yield limit. Design was controlled by pile displacement limitation (15 mm). In Penzien model analysis, maximum pile bending moment reached yield limit in Case I and Case II and approached to yield limit in Case III. Maximum ductility factor in Case I was 1.83.

5) For seismic analysis of bridge structures the Road Bridge Code does not consider soil-pile interaction dynamic analysis in soft soil region. But, based on above conclusions we propose Penzien model analysis method with non-linear pile behavior consideration for seismic analysis of all bridge structures with pile foundation in soft clay region such as Ariake soft clay region.

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