DYNAMIC RESPONSE ANALYSIS OF STEEL PIPE SHEET PILE FOUNDATION CONDUCTED ON THREE SIMPLE MODELS

Waseda University Student member ○ Nguyen Thanh Trung
Waseda University Fellow Osamu Kiyomiya
Waseda University Member Tongxiang An

1. INTRODUCTION

SPSP foundation is a combination of steel pipe piles which are connected by interlocks. The interlock inside is filled by concrete, and the heads of the piles are rigidly connected by top slab - footing. The SPSP foundation structure has a high bearing capacity in vertical and horizontal directions, so it is widely used as the foundations of large bridges especially which are constructed under large water depth or/and soft soil condition. In order to investigate the effects of Soil - Structure Interaction (SSI), Soil - Foundation - Structure Interaction (SFSI) and the nonlinearity of the pier column and the interlocks of the pipes, the dynamic response analysis was conducted on three calculation models in this study. Dynamic analysis was carried out by time history direct integration method on TDAP III software.

2. PROTOTYPE BRIDGE PIER FOUNDATION

A typical bridge pier with steel pipe sheet pile foundation was used in this work. The height of the pier is 13 m, the sectional dimension of the pier column is 2.5 m x 7.5 m. The pier column is constructed of RC, and the strength of the concrete is 30 MPa. The SPSP foundation has a circular shape in plan with a diameter of 12 m. The steel pipe has a diameter of 1.0 m and a thickness of 0.012 m. The interlock has a diameter of 0.248 m and a thickness of 0.012 m. The material of the steel is SKK400.

The surface ground with 26m thickness consists of four layers: the first layer is a clay layer with an average SPT value of 2 and its deformation modulus is E_o=5.6 MN/m^2; the second layer is also a clay layer, its N is 3 and E_o is 8.4 MN/m^2; the third layer is a dense sand layer, its N is 20 and E_o is 56 MN/m^2; the bearing layer is a sand gravel layer with an N of 50 and an E_o=140 MN/m^2.

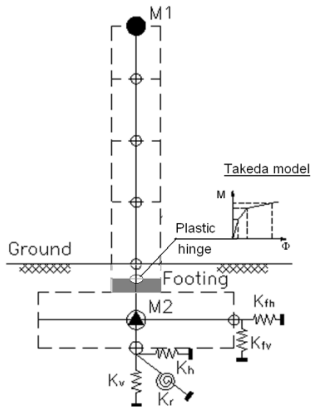

Fig.1 Concentrated spring model (Model 1)

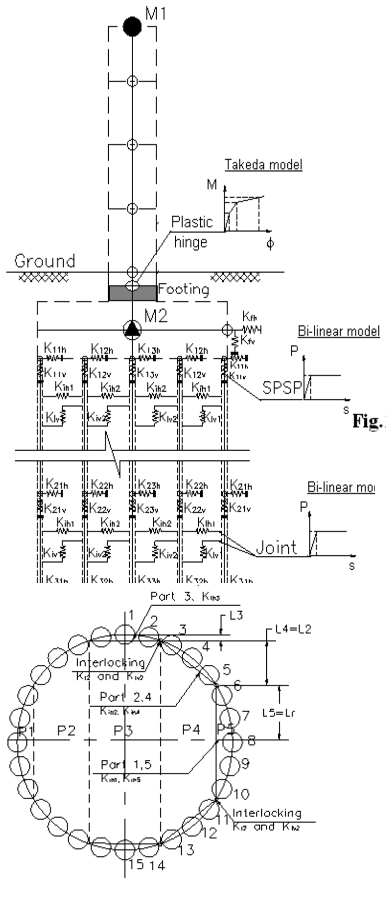

Fig.3 Frame and spring model (Model 3)

3. CALAULATION MODELS

The analysis was carried out on three models in this work. The pier and SPSP foundation were modeled as beam elements and the soil - structure interactions were considered as springs whose stiffness was determined from the stiffness of soil and foundation according to JRA-2002.

Concentrated spring model (SSI): the SPSP foundation was modeled as three linear concentrated springs K_v, K_h and K_r in horizontal, vertical and rotational direction, respectively and the pier column and top slab was supported by these three springs as shown in **Fig.1**.

One beam and spring model (SFSI): the SPSP foundation was modeled as one beam supported by springs that represented the function of the soil, and the pier column and top slab was supported by this beam. The foundation beam was divided into seven segments in its axial direction. The surrounding soil was represented by seven couple of concentrated springs: K_{ih} in horizontal and K_{iv} in vertical direction (i: the ith soil layer) as show in **Fig. 2.**

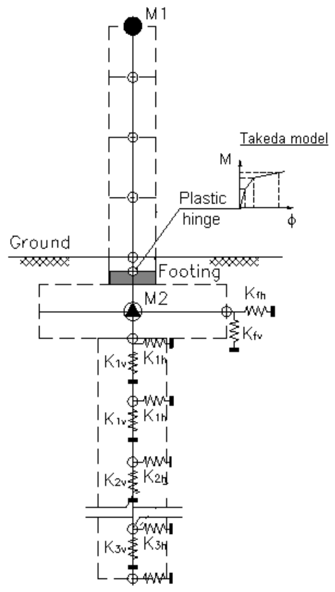

Fig.2 One beam and spring model (Model 2)

Frame and spring model (SFSI): the SPSP was divided into five blocks in plan, and each block has the same width in diameter direction and was represented by a beam at its center. These beams were supported by seven couple of concentrated springs (K_{ihj}, K_{ivj}) (i: the ith soil layer, j: the jth beam, j∈{1,5}). The beam was also connected with adjacent one by seven couple of concentrated springs, (K_{lhi}, K_{lvi}) (i: 1,2 , shown in **Fig.3**).

4. METHODLOGY AND INPUT GROUND MOTION

The nonlinear dynamic analysis was conducted by time history direct integration method on three models considering the material nonlinearity and Rayleigh's damping was used in this work, the damping ratio was 0.03 and 0.15 of structure and soil springs, respectively.

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Address: 169-8555, Tokyo, Shinjuku, Okubo, 3-4-1, Civil & Environmental Engineering, Waseda University. TEL: 03-5286-3852.
Email:nguyenthanhtrung@ruri.waseda.jp

The selected ground motion time histories had a mean acceleration response spectrum comparing with the design response spectra proposed by JRA 2002 for level 1 and Level 2 of type 2 of ground type III. The first record was named 1983 Tsugaru Ohhashi with the peak acceleration of 141Gal and the duration time of 50 s, and the second one was named 1995 Kobe Port with the peak acceleration of 619 Gal and the duration time of 50s as shown in **Fig. 4**.

5. RESULTS AND EVALUATIONS

The natural periods in horizontal direction (X) were calculated. Fundamental periods and mode dampings were shown in **Table 1**.

Table 1 The result of eigen - value analysis

Mode	Model	Frequency (Hz)	Period (s)	Mode Damping(%)
First mode	Model 1(SSI)	1.81	0.552	9.36
	Model 2(SFSI)	1.76	0.568	7.77
	Model 3(SFSI)	1.71	0.585	6.30
Second mode	Model 1(SSI)	6.17	0.162	12.36
	Model 2(SFSI)	6.04	0.166	8.86
	Model 3(SFSI)	4.75	0.211	9.32

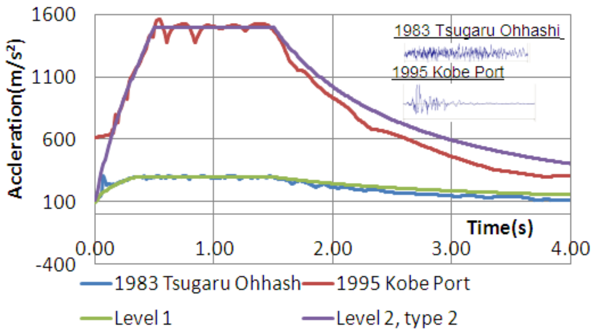

Fig.4 Response spectra acceleration of input ground motion

Both the first natural period and the second natural period of model 1 were the shortest. It is thought that the foundation mass affected the other two models. Meanwhile, the natural periods of model 3 were longer than that of model 2. Regard as mode damping, SSI model makes an increase of damping ratios, so the damping ratios of model 1 were the biggest.

The response displacement and the moment at the bottom of the pier were shown in **Table 2**.

Table 2 The response of nonlinear dynamic analysis

No	Analysis cases		Disp. at the footing (cm)	Disp. at the top (cm)	Moment at the bottom of pier (kN·m)	Difference of Displacement, Moment at the footing: Disp. (%)	Moment (%)
A	OD under level 1		0.70	-	44500		
B	OD under level 2		1.25	-	103500		
C	NDA 1983 Tsugaru Ohhashi	Model 1	0.51	2.31	38700	27	13
		Model 2	0.71	2.84	36500	-1	18
		Model 3	0.87	3.06	35200	-24	21
D	NDA 1995 Kobe Port	Model 1	1.90	13.2	117640	-52	-14
		Model 2	2.64	16.4	106920	-111	-3
		Model 3	3.41	16.9	104170	-173	-2

Note: NDA: Nonlinear Dynamic Analysis; OD: Original Design;

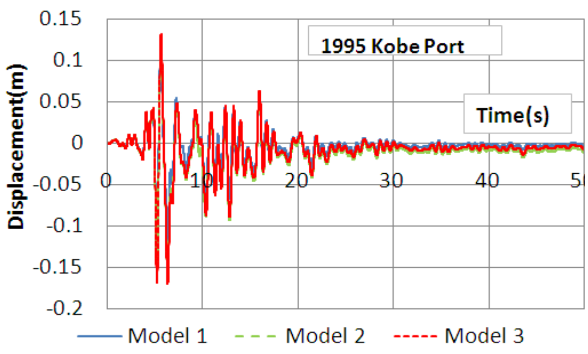

Fig.5 Displacement response at the top of pier

The moments at the bottom of the pier calculated on the three models by the 1983 Tsugaru Ohhashi (level 1) were smaller than that of the original design (static analysis) by 13–21%. As to the responses calculated by the 1995 Kobe Port (level2), both the moment at the pier bottom and the displacement were bigger than that of the original design (static analysis), the moment was bigger by 2–14% and the displace at the top of footing was bigger by 52–173%. Among the three models, the model 1 calculated the biggest moment but the smallest displacement. The maximum response ductility ratios of the pier column of the three models, μ calculated following JRA-2002 were 2.45 of model 1 (SSI), 3.28 of model 2 (SFSI) and 3.19 of model 3 (SFSI) as shown in **Fig.6**.

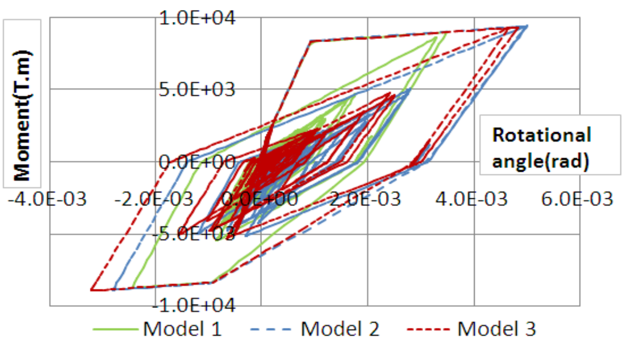

Fig.6 M-θ hysteretic loops of the plastic hinge

6. CONCLUSIONS

The main findings are as following:

1. SSI model makes an increasing of mode damping ratios; 2. Dynamic analysis calculated a bigger response by the 1995 Kobe Port comparing with the static analysis, among the three models, the model 1 calculated the biggest moment but the smallest displacement; 3. Comparing with SSI model, SFSI model of SPSP foundation decreased the ductility demands of structure. Model 1 cannot always provide a safe design.

REFERENCE

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2) NGUYEN Thanh Trung and Osamu KIYOMIYA : Response analysis of steel pipe sheet pile foundation by three simple models. Proceeding of the 39 Conference of JSCE, Kanto Branch, March, 2011.