RESPONSE ANALYSIS OF STEEL PIPE SHEET PILE FOUNDATION BY THREE SIMPLE TYPES OF MODELS

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1. INTRODUCTION

SPSP foundation is a combination of steel pipe piles which are connected by two interlockings. The interlockings will be filled concrete into, the heads of piles are connected together rigidly by constructing work of footing. Therefore, the foundation structure has a high bearing capacity in vertical and horizontal directions so it is widely applied in bridge structures which are constructed under large water depth and soft soil condition.

Many studies on SPSP foundation have been carried out in past. The effects of Soil-Structure Interaction (SSI), Soil-Foundation-Structure interaction(SFSI) and nonlinearity of materials of this foundation are adopted. Therefore, in this research three simple types of models were conducted to show the effect of SPSP foundation on pier foundation system by analyzing the differences among their responses. Moreover, difference between response of three models and the result of original design was shown in this research. The response power spectrum method is mainly discussed here and the FEM TDAP software that is available for the diversified models.

2. STRUCTURAL FOUNDATION CHARACTERISTIC

2.1. Actual structure and ground

The proposed structural system is a pier supported by steel pipe sheet pile. The height of pier is 13m, sectional dimension is 2.5mx7.5m, strength of pier concrete is 30 MPa. The SPSP foundation is circular shape with 12 m in diameter and is a combination of steel pipe piles that have a diameter 1.0m, a thickness is 0.012m, type of material SKY400, are connected by interlockings that have diameter 0.248m and thickness of 0.012m, type of material SKK400, as shown in **Fig.1.** The surface ground consists of soft layers.

The ground includes four layers : the first layer is clay with average SPT value N value of 2. E_o=56kgf/cm^2; the second layer is also clay, N=3, E_o=84kgf/cm^2 ; the third layer is sand, N=20, E_o=560kgf/cm^2 ; the final layer is also sand, N=50, E_o=1400kgf/cm^2, as shown in **Fig.1**.

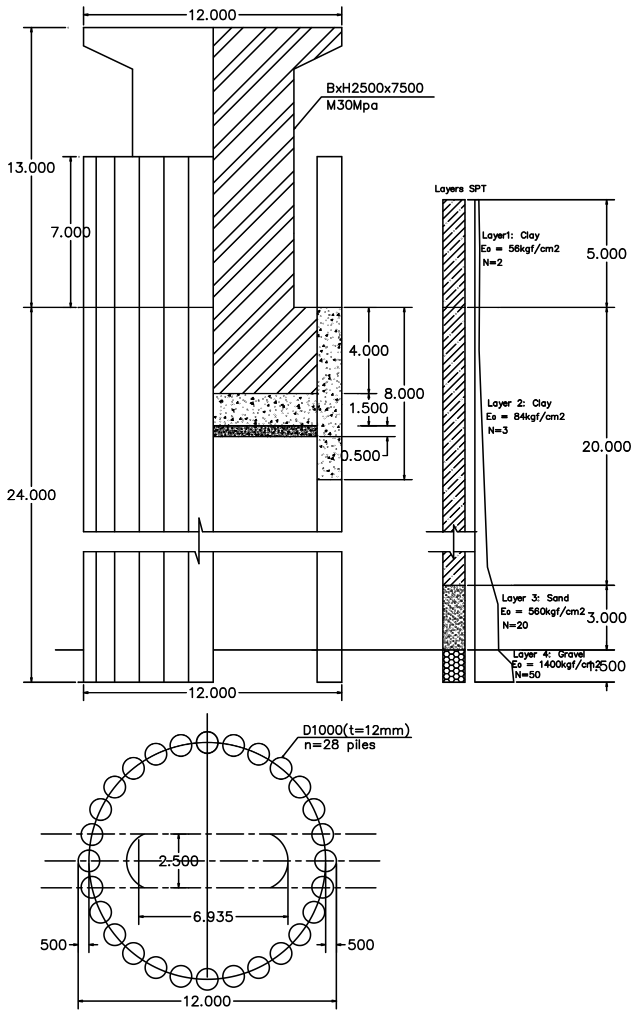

Fig.1 Actual structure and ground condition

2.2. Structural models

This study was carried out on three models by the FEM program consist of spring base model, a beam and spring model and beams and spring model. The pier and SPSP foundation were modeled as beam elements and the soil-structure interactions were considered as springs that were determined from the stiffness of soil and foundation in JRA-2002. The connection between the footing and the foundation was rigid.

The spring base model (SSI): the pier beam was supported by two concentrated springs in the horizontal and vertical direction K_v, K_h, respectively and a rotational spring K_r that were determined from as below:

$$K= F.\delta^{-1} \quad (1)$$

where:
K: the stiffness matrix of spring
δ : the displacement at the bottom of footing
The stiffness of soil surrounding the footing were modeled in the horizontal, K_{fh}, and in the vertical, K_{fv} (shown in **Fig.2**).

Keywords: Steel Pipe Sheet Pile (SPSP), Response Spectrum Analysis, Push-Over Analysis, Soil-Structure Interaction (SSI), Soil-Foundation-Structure Interaction (SFSI).
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A beam and spring model(SFSI) : the pier beam was supported by the SPSP foundation beam that were divided into seven segments in axial direction of the foundation and connected with surrounding soil by seven the couple of concentrated springs having the stiffness is K_{ih} in the horizontal and K_{iv} in the vertical (i: the i^{th} of soil layer) were determined as following formulas :

$$K_{ih}= D.l_r.k_{hi} \tag{2}$$

$$K_{iv}= D.l_r.k_{vi} \tag{3}$$

where: D: the outside diameter of foundation, l_r: the length of the r^{th} segment, k_{hi}: the cofficients of reaction of the i^{th} soil layer in the horizontal and vertical direction that were determined in JRA-2002 as show in **Fig. 3**

Beams and spring model (SFSI): SPSP the foundation beam that were divided into five beams having the same length of the foundation. These piles were connected with ground by seven the couple of concentrated springs (K_{ihj}, K_{ivj})(i is the i^{th} of soil layer and j is the j^{th} of part, $j\in\{1,5\}$) were determined as following formulas :

$$K_{ihj}= L_j.l_r.k_{hi} \tag{4}$$

$$K_{ivj}= L_j.l_r.k_{vi} \tag{5}$$

where: L_j: the length of the j^{th} part as shown in **Fig.4**; l_r: the length of the r^{th} segment; k_{hi}, k_{vi}: the cofficients of reaction of the i^{th} soil layer in the horizontal and vertical direction determined in JRA-2002. Each beam also was jointed with adjacent another by seven couple of concentrated springs, (K_{lhi}, K_{lvi})(i: 1,2 , shown in **Fig.4**) which have the stiffnesses were determined from the relationship between the shear capacity and the displacement of site experiment. In the same cross-section, the stiffness couple of these springs were different:

$$\left(q_j\right)_i = \left(G_j \Delta\nu\right)_i \tag{6}$$

where: q_j : is shear resistance of the j^{th} interlocking; G_j : is shear stiffness of the j^{th} interlocking; $\Delta\nu$: is the displacement of interlocking.

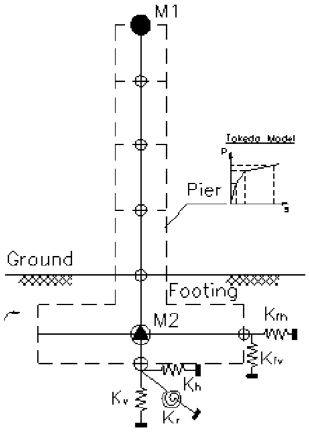

Fig. 2 Spring-Base model(Model

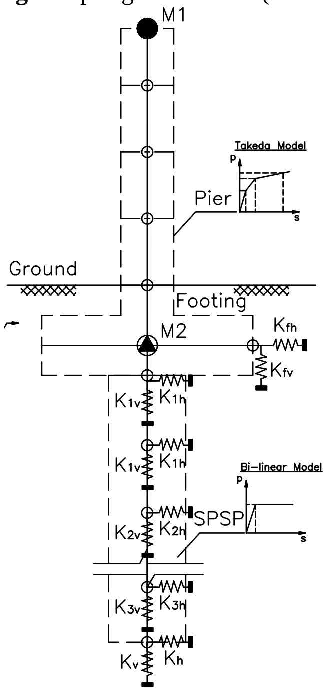

Fig.3 A beam and spring model(Model 2)

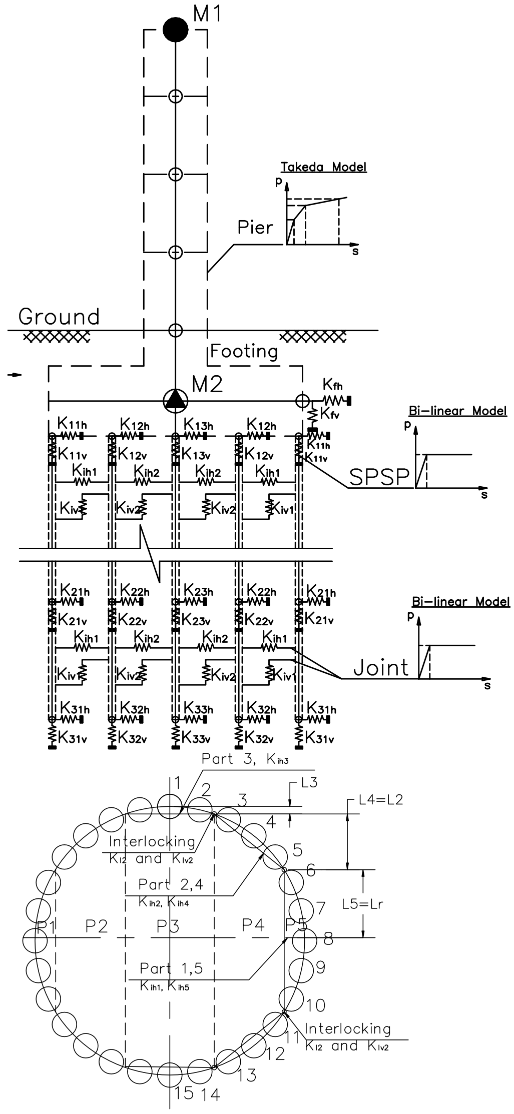

Fig. 4 Beams and Spring model(Model 3)

3. METHODOLOGY

The Response Spectrum analysis and Push-over analysis were carried out to capture the responses of three models under earthquake level 2 on the ground type III following to JRA -2002. Firstly, response spectrum analysis using SRSS method was applied in of linear system to give the response of three models. Secondly, Push-over analysis was conducted to show the difference of structural nonlinearity between the above models. The nonlinear property of steel pipe piles was bi-linear, these piles interact with surrounding soil through bi-linear springs and the interlocking also were modeled as bi-linear springs in push over analysis. The stiffness of elastic springs and bi-linear springs were determined according to stiffness of soil and SPSP foundation and interlocking. Especially, the bending nonlinearity of concrete pier's plastic hinge was assigned following Takeda model.

4. RESULTS AND EVALUATIONS

4.1. Eigen-value analysis

Eigen-value is one of the important steps of response spectrum analysis. The behavior of structural system will be exposed though natural periods and mode damping as shown in **Table 1**. The natural periods in the horizontal X direction were considered.

Table 1 The result of Eigen – value analysis

Mode	Model	Direction	Frequency(Hz)	Period(s)	Mode Damping(%)	Note
First mode	Model 1(SSI)	In the axis X	2.44	0.41	7.85	
	Model 2(SFSI)	In the axis X	2.01	0.498	11.9	
	Model 3(SFSI)	In the axis X	2.38	0.420	10.7	
Second mode	Model 1(SSI)	In the axis X	13.25	0.075	8.58	*Third mode*
or Third mode	Model 2(SFSI)	In the axis X	9.8	0.102	5.81	*Second mode*
	Model 3(SFSI)	In the axis X	7.66	0.131	4.33	*Third mode*

Comparing to the result of spring base model(model 1), the first and the second natural period of a spring and beam model(model 2) increases 20.65%, 28.05% and 9.06% and 42.11% increasing for the beams and spring model(model 3). As for mode damping, increasing of 34 % of model 1 and 27% of model 3 at the first mode and decreasing 33% of model 2 and 49% of model 3 at the second mode, respectively. As this result, both the natural periods and 1st mode damping of model 1 were smaller than these of model 2 and model 3. This was because that model 1(SSI) not considered the foundation mass and soil-foundation interaction.

Table 2 The result of response spectrum analysis

No	Analysis cases		Disp. at the footing (cm)	Disp. at the top (cm)	Moment at the footing (T m)	Acc. at the footing (m/s)	Acc. at the top (m/s)	Difference of Disp. at the footing (A-C) (B-D) (%)	Difference of M at the footing (A-C) (B-D) (%)
A	OD under level 1		**0.45**		**4320**				
B	OD under level 2		**3.85**		**22900**				
	RSA	Model 1	0.54	0.633	4780	1.91	3.47	20	12
C	Under	Model 2	1.15	1.283	5086	2.12	3.78	61	15
	Level 1	Model 3	1.11	1.15	5784	2.55	4.15	59	25
	RSA	Model 1	3.07	7.5	22322	5	15	-25	-3
	Under	Model 2	5.72	11.67	25189	9.38	18.6	33	9
	Level 2	Model 3	5	7.96	25391	11.12	18.1	23	9
D	Push-	Model 1	3.15	8.05	22152	-	-	-22	-8
	Over	Model 2	5.74	12.22	24618	-	-	33	7
	Level 2	Model 3	4.94	8.33	23986	-	-	22	4

Note: RSA: Response Spectrum Analysis
OD : Original Design

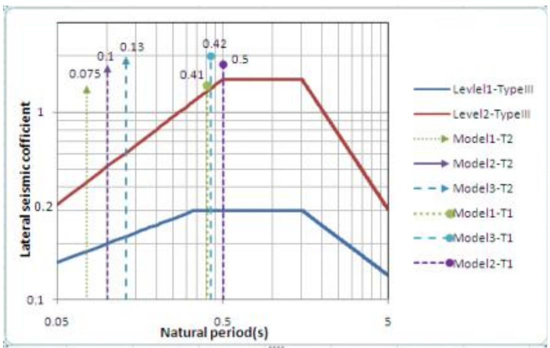

Fig. 5 Relationship between natural periods and Lateral seismic coefficient under earthquake level 1, level 2, type of ground III

4.2. Response Spectrum analysis

Table 2 shows in the result of response spectrum analysis under earthquake level 1 and level 2. Comparing with that of original design, displacement at the footing almost increased 13-61% and 4-25% in case of moment at the footing. The average difference of displacement at the footing under all analysis cases was from 6-17.5% and 6-17.5% and 3.5 -50% in case of the moment.

Under earthquake level 2, the response of model 1 is minimum and that of model 3 is maximum. The cause of displacement is that both the first and third natural period of model 1 were smallest among three models and their periods is in the range of period from 0-0.5 s so it's lateral seismic coefficients would be smallest as shown in **Fig.5**. As for the case of earthquake level 1, the responses of model 1 are minimum and these of model 3 are maximum because the first natural frequencies of three models is equal while the second natural frequency of model 1 is minimum and that of model 3 is maximum as shown in **Fig.5**.

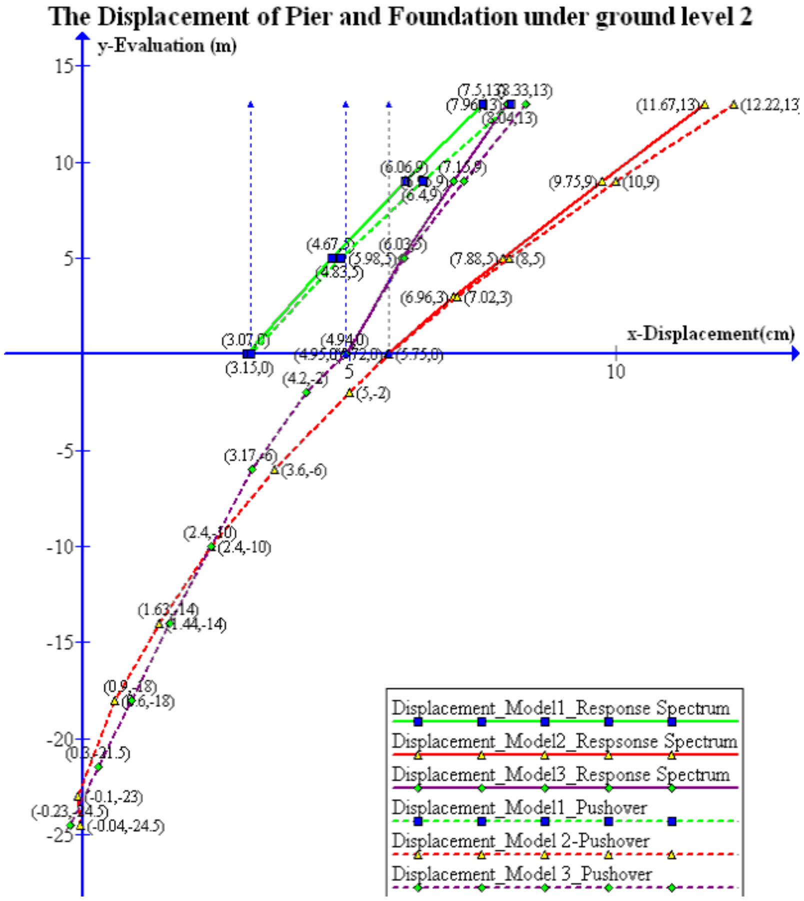

Fig. 6 Displacement of Pier and Foundation under Earthquake level 2

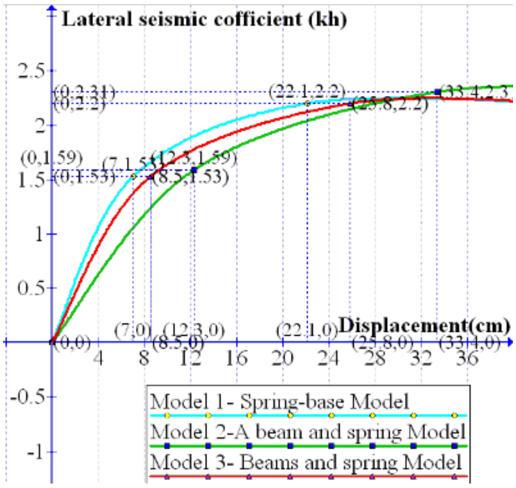

Fig. 7 Relationship between Seismic coefficient and horizontal displacement

The first natural period of model 2 was bigger than that of model 3 but it's the second natural period was smaller so responses of two models were approximately same.

However, if these natural periods of three models are over 1.5s or have a change of earthquake spectrum then the distortion of dynamic response may be reversal or different.

4.3. Static nonlinear analysis (Push-over)

The results of push-over analysis are shown in **Fig.6**, all of the moments of three models under earthquake level 2 decreased 1-5.5% but their displacements at the top of pier increased 5-12.5% while the displacements at the footing almost do not change. The curved lines of displacement in push-over analysis are more curved than these of response spectrum analysis (shown in **Fig.6**). The relationships between seismic coefficient and horizontal displacement at the top of pier of three models are shown in **Fig.7**. As this result, the lateral seismic coefficient of model 1 and model 3 are approximately same but the displacement of model 1 was less than that of model 3. Meanwhile, both lateral seismic coefficient and displacement of model 2 is greater than these of other models.

5. CONCLUSTIONS

There are some main findings as followings:

1. The soil-foundation interaction and type of models of SPSP foundation influenced on the pier foundation system significantly and differently under both the Response Spectrum analysis and Push-over analysis.
2. The response of SSI model is smaller than that of SFSI model under both the Response Spectrum analysis and Push-over analysis. However, depending on the earthquake spectrum, natural period of structural system and the characteristics of soil-foundation-structure interaction, the distortion of the response will be different.
3. The responses of three models in both Response Spectrum Analysis and Push-Over Analysis are almost greater than these of original design.

REFERENCE

1) JRA-2002. Specification for Highway Bridge, part V: Seismic design, Japan Road Association, 2002.
2) JRA-2002. Specification for Highway Bridge, part IV: Substructure, Japan Road Association, 2002.
3) M. IGUCHI, M. KAWASHIMA, T. KASHIMA. Assessment of varying dynamic characteristics of a SFSI system based on earthquake observation, Soil-Foundation-Structure Interaction – Orense, Chouw & Pender (eds), Taylor & Francis Group, London, ISBN 978-0-415-60040-8, pp. 3-10, 2010.
4) W.D. LIAM FINN. Aspects of soil structure interaction, Soil-Foundation-Structure Interaction – Orense, Chouw & Pender (eds), Taylor & Francis Group, London, ISBN 978-0-415-60040-8, pp. 69-75, 2010.