NON-LINEAR ANALYSIS OF SOIL-PILE-STRUCTURE INTERACTION UNDER SEISMIC LOADS

Yingcai Han[1] and Shin-Tower Wang[2]

[1] *Fluor Canada Ltd., Calgary AB, Canada   Email: yingcai.han@fluor.com*
[2] *Ensoft, Inc. Austin, Texas, USA   Email: stw@ensoftinc.com*

ABSTRACT :

The seismic response of structures supported on a pile foundation is extremely complex, since the soil behavior is non-linear and liquefaction may occur during earthquakes. The soil-pile-structure interaction becomes extremely important for seismic analysis and design, so that this topic has been studied widely. In this study, an approximate and practical method is described for the seismic analysis. Two commercial software packages are used for considering the nonlinear soil-pile-structure interaction. Stiffness and damping of the pile foundation are generated from a computer program *DYNAN*, and then input into a finite element model by *SAP2000* program. The seismic response of a vacuum tower structure supported on pile foundation is examined in a high seismic zone, including response spectrum analysis and time history analysis. The vacuum tower with weight of 5,600 kN and height of 35 m set on a steel frame. To illustrate the effects of soil-pile-structure interaction on the seismic response of structure, three different base conditions are considered, rigid base, i.e. no deformation of the foundation; linear soil-pile system; and nonlinear soil-pile system. The case of pile foundation with liquefaction of sand layer is discussed. The method and procedure introduced can be applied to the design of tall buildings, bridges, industrial structures and offshore platforms with soil-pile-structure interaction under seismic, blast, sea wave and other dynamic loads.

KEYWORDS: soil dynamics, nonlinear soil, soil-pile-structure interaction, seismic response

1. INTRODUCTION

Many industrial structures supported on pile foundations are constructed in soft soil in seismically active areas. The behavior of such structures can be greatly affected by non-linear soil-pile interaction during strong earthquakes. An evaluation of soil-pile-structure interaction is needed to establish the forces expected to act on the structure and the piles in a seismic event. A simple procedure based upon substructure method is adequate for routine design. The following assumptions are adopted in developing a more detailed method of analysis. The input ground motion is given at the level of pile heads and is not affected by the presence of the piles and their caps. Soil-pile interaction analysis is conducted separately to yield the impedance of the pile foundation. The seismic response is obtained in the time domain using input of earthquake records or in the frequency domain with input of response spectra. This procedure is considered as an efficient technique for solving problem of the nonlinear soil-pile-structure interaction (Han, 2002).

The soil-pile system is simulated by a boundary zone model with non-reflective interface. The model is an approximate but simple and realistic method that accounts for the nonlinearity of a soil-pile system. The validity of the computation method has been verified by dynamic experiments on full-scale pile foundations. The nonlinear features of the pile foundation and the group effects were examined.

In this study, a vacuum tower structure is examined in a seismic zone as a typical industrial structure supported on a pile foundation, including response spectrum analysis and time history analysis. The vacuum tower with diameter of 8.5 m, height of 35 m and weight of 5,600 kN set on a steel frame. There are 25 steel piles in the foundation. To illustrate the effects of soil-pile-structure interaction on seismic response of structure, three different base conditions are considered, rigid base, i.e. no deformation in the foundation: linear soil-pile system; and nonlinear soil-pile system. The case of liquefaction of sand layer is discussed.

2. DYNAMIC BEHAVIOR OF SOIL-PILE SYSTEM

A number of approaches are available to account for dynamic soil-pile interaction but they are usually based on the assumption that the soil behavior is governed by the law of linear elasticity or visco-elasticity and the soil is perfectly bonded to a pile. In practice, however, the bonding between the soil and the pile is rarely perfect and slippage or even separation often occurs in the contact area. Furthermore, the soil region immediately adjacent to the pile can undergo a large degree of straining, which would cause the soil-pile system to behave in a nonlinear manner.

Many efforts have been spent on the numerical analysis with a 3D finite element method (FEM) to model the soil-pile interaction. However, it is too complex, especially for group piles in nonlinear soil. A rigorous approach to the nonlinearity of a soil-pile system is extremely difficult and time consuming.

As an approximate analysis, a procedure is developed using a combination of the analytical solution and the numerical solution rather than using a general FEM. This procedure is considered as an efficient technique for solving the nonlinear soil-pile system (Han, 1997).

The relationship between the foundation vibration and the resistance of the side soil layers was derived using elastic theory by Baranov (1967). Both theoretical and experimental studies have shown that the dynamic

(1980) proposed including a cylindrical annulus of softer soil (an inner weakened zone or so called boundary zone) around the pile in a plane strain analysis. One of the simplifications involved in the original boundary zone concept was that the mass of the inner zone was neglected to avoid wave reflections from the interface between the inner boundary zone and the outer zone. To overcome this problem, Velestsos and Dotson (1988) proposed a scheme that can account for the mass of the boundary zone. Some of the effects of the boundary zone mass were investigated and found that a homogeneous boundary zone with a non-zero mass yields undulation impedance due to wave reflections from the fictitious interface between the two media.

The ideal model for the boundary zone should have properties smoothly approaching those of the outer zone to alleviate wave reflections from the interface. Consequently, Han and Sabin (1995) proposed such a model for the boundary zone with a non-reflective interface.

The impedances of the composite layer are derived from the plane-strain assumption. The outer zone medium is assumed to be homogeneous, isotropic, and viscoelastic, with frequency independent material damping; within the boundary zone, the complex shear modulus, G *(r),* is assumed to vary parabolically, as expressed by the function *f(r).* The variation of G *(r)* is continuous at the boundary, both the function itself and its derivatives, so that no reflective wave can be produced at the interface. The interface is referred to as the "non-reflection boundary".

The properties of the soil medium for each region are defined by the complex-valued modulus

$$G^*(r) = \begin{cases} G_i^* & r = r_o \\ G_o^* \; f(r) & r_o < r < R \\ G_o^* & r > R \end{cases} \tag{1}$$

and

$$\begin{aligned} G_i^* &= G_i\,(1 + i\,2\beta_i) \\ G_o^* &= G_o\,(1 + i\,2\beta_o) \end{aligned} \tag{2}$$

in which G_i and G_o= shear modulus of soil in the boundary zone and outer zone; r_o = radius of pile; R = radius of boundary zone; r = radial distance to an arbitrary point; β_i and β_o = damping ratio for the two zones; and i = root(-l). The parabolic function, $f(r)$, can be expressed as

$$f(r) = 1 - m^2 \left(\frac{r - R}{r_o} \right)^2 \tag{3}$$

and

$$m^2 = \frac{1 - G_i^* / G_o^*}{(t_m / r_o)^2} \tag{4}$$

where t_m = thickness of boundary zone; m = a constant whose value depends on the shear modulus ratio G_i^*/G_o^* and the thickness ratio t_m / r_o as shown above. Obviously, as the modulus ratio equal to one, the soil behavior is linear. The shear modulus in the boundary zone and outer zone is a constant.

If the modulus ratio is less (or larger) than one, the soil behavior is nonlinear. The thickness ratio is assumed to be one. Thus, the modulus ratio is an indicator to show the approximate nonlinear behavior of soil. The case of static loading is different, because *p-y* curves are used to indicate the nonlinear behavior of the soil.

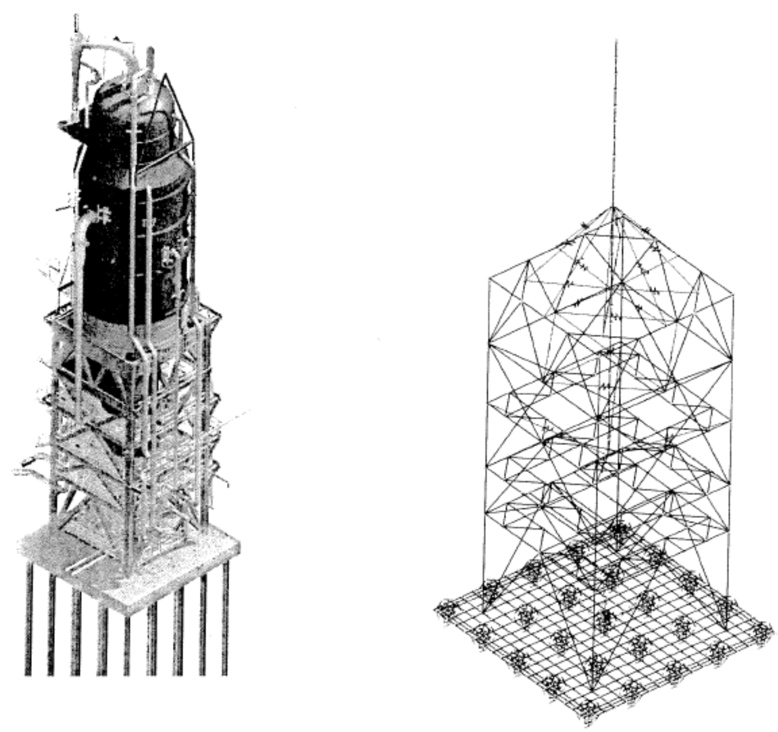
With the impedance of the soil layer, the element stiffness matrix of the soil-pile system can be formed in assembled for different modes of vibration, including three translations and three rotations.

The group effect of piles is accounted for using the method of interaction factors. The static interaction factors are based on Poulos and Davis (1980). The dynamic interaction factors are obtained from the static interaction factors multiplied by a frequency variation. To investigate soil-structure interaction, a series of dynamic experiments have been done on full-scale piles in the field, including single piles and groups (Han & Novak 1988).

The reasonable assumption is made that the caps of piles are rigid. However, in most cases, the superstructures are flexible rather than rigid. By means of a substructure method, the dynamic response of the superstructure is computed using a finite element program, such as SAP2000 (2007), and the stiffness and damping of the foundation can be generated from the DYNAN program (2003).

3. SEISMIC RESPONSE OF VACUUM TOWER STRUCTURE

A vacuum tower structure shown in Fig. 1 (A) was built for a petrochemical plant in a seismically active area of Canada. The details of steel structure are described as follows. Four columns, using WWF 400x243 (400 x 400 mm, weight of 243 kg/m) and height of 20m, are arranged rectangularly with a column center to center spacing of 8.55m. The vacuum vessel is supported directly by a top frame using beam of WWF 1400x358 (1400 x 400 mm, weight of 358 kg/m) on top and beam of W610x155 at bottom. There are three layers of beam beneath the top frame and the main beam is W460x82. The concrete mat foundation is 12 x 12 m and thickness is 1.2 m.



(A) View original model of as-built structure (B) FEM model for seismic analysis
Fig.1 Vacuum tower structure

The vacuum vessel is modeled as an elastic column with the mass distributed uniformly along its height. The steel structure is modeled using frame elements and the mat foundation is modeled using shell elements, as shown in Fig. 1 (B). The thickness of vessel wall is 25.4 mm (one inch). The seismic response of the structure is calculated using the substructure method. The deflection of structure, base shear and overturning moments for different base conditions are investigated.

3.1 Soil conditions and pile foundation

The structure is in a seismically active area, and the range of peak horizontal ground acceleration is equal 0.13 g. At the site, surface soil is soft clay with depth of 2m, followed by a layer of saturate fine sand with depth of 2 m, some clay and sand then bedrock. The depth to bedrock is about 30 m. Soil properties vary with depth and are characterized by the shear wave velocity and unit weight, as shown in Table 1.

Table 1. Soil Properties

Depth (m)	Soil	Unit Weight (kN / m^3)	Shear Wave Velocity (m / s)
0 -2	Soft Clay	18	130
2 - 4	Fine Sand	18	140
4 - 12	Stiff Clay	20	300
12 - 16	Silty Sand	19	240
16 - 20	Silty Clay	18	300
20 - 25	Weathered Shale	18	200
25 - 30	Dense Sand	20	300
Below 30	Bedrock	21.5	370

The piles are steel HP 360 x 108, (346 x 370 mm) and length of 30 m driven to bedrock. Twenty-five piles in a square pattern are fixed to the mat foundation, with spacing of 2.75 m and the spacing ratio is 7.6.

The stiffness and damping of the pile foundation are calculated for different base conditions. In the first case a nonlinear soil-pile system is assumed, and the boundary zone model is used around the piles. The parameters of the weakened zone are selected as: $G_i / G_o = 0.3$, $t_m/r_o = 1.0$, $\beta_i = 2 \times \beta_o$. In the second case, a linear soil-pile system is assumed, the soil layers are homogeneous, and there is no weakened zone.

Table 2. Stiffness and Damping of Pile Foundation (f = 1.0 Hz)

Soil – Pile Interaction	Stiffness			Damping		
	K_x (kN/m)	K_z (kN/m)	K_φ (kN.m/ra)	C_x (kN/m/s)	C_z (kN/m/s)	C_φ (kN.m/rad/s)
Linear Soil	1.283×10^6	3.215×10^6	1.333×10^8	1.244×10^4	1.803×10^4	6.411×10^5
Nonlinear Soil	0.646×10^6	2.877×10^6	1.160×10^8	0.998×10^4	1.005×10^4	3.171×10^5
Liquefaction	0.1799×10^6	2.527×10^6	1.006×10^8	0.749×10^4	0.943×10^4	2.787×10^5

Where, K_x, K_z, and K_φ are stiffness in the horizontal, vertical and rocking directions, and C_x, C_z, and C_φ are damping constants in the same directions.

In the third case, liquefaction is assumed in the saturate fine sand layer, and top layer of soft clay has not yielded. Both stiffness and damping are frequency dependent. Since the fundamental period of structure is closed to 1.0 second, the stiffness and damping are calculated at frequency f = 1.0 Hz, and the results are shown in Table 2. It can be seen that both stiffness and damping in the nonlinear case are lower than the linear case. For example, the horizontal stiffness in nonlinear case is about half of that in linear case. In the case of liquefaction, the values of horizontal stiffness are reduced significantly, and significant damage is possible.

3.2 Time history analysis

A record of horizontal ground acceleration from an earthquake is employed for the time history analysis. The peak value of acceleration is 0.13 g as shown in Fig. 2. The time step is 0.005 second, and duration is 80 second in the earthquake record. To investigate the influence of foundation flexibility on the superstructure, the seismic analysis of the vacuum tower structure is conducted for three different foundation conditions: rigid base, linear and nonlinear soil-pile system. For the case of the rigid base, the stiffness of the foundation is assumed to be infinite with no deformation occurring in the footing. Initial seismic analysis was done in this way forty years ago, when the soil-structure interaction was not considered. For the cases of linear and nonlinear soil-pile system, the values of stiffness and damping shown in Table 2 are used.

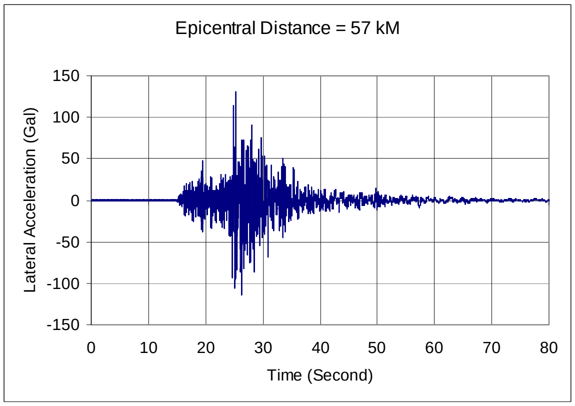


Fig. 2 Horizontal ground acceleration from an earthquake record

The analysis is done using the model of finite element as shown in Fig. 1 (B). The seismic response and natural frequency of structure are different for the three base conditions. The deflection, base shear and overturning moment are shown in Table 3 and the natural periods of structure are shown in Table 4.

From Table 3, it can be seen that the earthquake forces for the fixed base condition are larger than those for the cases of the soil-structure interaction to account for flexible base. The theoretical prediction for a structure fixed on a rigid base without soil-structure interaction does not represent the real seismic response, since the stiffness is overestimated and the damping is underestimated. From Table 4, it can be seen that the structure with a flexible base has longer natural periods than that with fixed base. From the comparison, it can be seen that the maximum values and time histories for the seismic forces and seismic response are different when the foundation is considered as a fixed base or a flexible base. The soil-pile-structure interaction should be considered for the seismic analysis.

Table 3. Maximum Values of Seismic Response and Seismic Forces of Tower Structure

Base Conditions	Amplitude at Top of Tower (mm)	Base Shear (kN)	Overturn Moment (kN-m)
Fixed Base	22.05	807	19,630
Linear Soil	26.30	598	14,980
Nonlinear Soil	26.05	545	14,120

Table 4. Natural Period of Tower Structure (Second)

Model	Shape	Fixed Base	Linear Soil	Nonlinear Soil
1	Lateral X1	0.769	0.967	1.004
2	Lateral Y1	0.767	0.962	0.991
3	Lateral X2	0.184	0.191	0.197
4	Lateral Y2	0.173	0.187	0.190
5	Vertical	0.161	0.181	0.189
6	Torsional	0.122	0.130	0.140

3.3 Response spectrum analysis

An elastic dynamic analysis of a structure utilizes the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

The equivalent lateral seismic force, V, is calculated in accordance with the formula in NBC 2005.

$$V = S\,(T_a)\, M_v\, I_E\, W\, /\, (R_d\, R_o) \qquad (5)$$

and

$$S\,(T_a) = F_v\, S_a\,(T_a) \qquad (6)$$

where, T_a = fundamental lateral period, 0.75 second is calculated for fixed base; S_a (T_a) = ground acceleration. The values of S_a (T_a) are given for different locations. For the location of vacuum tower, S_a (0.75) = 0.13 g. F_v = site coefficient 1.37 is used based on the soil properties. M_v = higher model factor 1.0 is used here. I_E = important factor, 1.0 is used here. R_d, R_o = ductility factor and overstrength factor respectively, 1.5 and 1.3 are used for conventional construction of moment frames and braced frames. The weight of steel frame is 1,963 kN, and total weight (including vessel) is W = 7,563 kN.

The most difficult part of the entire RSA (Response Spectrum Analysis) procedure is calculating the scaling factor. The unscaled RSA base shear is calculated using a finite element program RISA – 3D. Thus, Scale Factor is equal to V/Unscaled RSA base shear. The spectra are normalized using modal participation. In the calculation for scale factor, 15 vibration modes are calculated making the modal participation to be over 90%.

A local response spectrum is used in the analysis. The following values from NBC 2005 are used, considering the location of vacuum tower structure: S_a (0.2) = 0.28 g, S_a (0.5) = 0.17 g, S_a (1.0) = 0.090 g, and S_a (2.0) = 0.053 g. The response spectrum analysis is done for fixed base. The seismic response and seismic forces are calculated, the amplitude at top of tower = 20.9 mm, base shear = 776 kN and overturn moment = 19,936 kN-m. By comparison with the data in Table 3, it is interesting to note that the seismic response and seismic forces generated from the response spectrum analysis are closed to those from the time history analysis at the same base condition.

4. CONCLUSIONS

An examination of the computation results for the seismic response of the vacuum tower structure, supported with different foundation conditions, suggests the following conclusions:

1. The nonlinear behavior of the soil-pile system can be simulated using the model of boundary zone. The validity of the model has been verified by dynamic experiments on full-scale pile foundations for both linear and nonlinear vibrations.

2. The soil – pile interaction is an important factor which affects the stiffness and damping of foundation. The liquefaction of a layer of saturated fine sand can reduce the horizontal stiffness significantly, and further damage is possible.

3. The soil-pile-structure interaction should be considered in a seismic analysis. The theoretical prediction for a structure fixed on a rigid base without the interaction does not represent the real seismic response, since the stiffness is overestimated and the damping is underestimated.

4. The problem of soil-pile-structure interaction is complex in a seismic environment. The approximate and practical method described in this study is workable with the help of two computer programs (DYNAN 2.0 and SAP2000).

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