ECCOMAS Congress 2016
VII European Congress on Computational Methods in Applied Sciences and Engineering
M. Papadrakakis, V. Papadopoulos, G. Stefanou, V. Plevris (eds.)
Crete Island, Greece, 5–10 June 2016

EFFECT OF HIGHER MODES ON STRUCTURAL RESPONSE WITH NONLINEAR SOIL-FOUNDATION-STRUCTURE INTERACTION

X. Qin[1], N. Chouw[2], and T. Larkin[3]

The University of Auckland
Dept. of Civil and Environmental Engineering
Auckland Mail Centre, Private Bag 92019, Auckland, New Zealand
e-mail: [1]xqin009@aucklanduni.ac.nz; [2]n.chouw@auckland.ac.nz; [3]t.larkin@auckland.ac.nz

Keywords: Nonlinear SFSI, higher modes, macro element, seismic response of structure.

Abstract. *To calculate the response of a multi-storey structure (MDOF model) with structure-foundation-soil interaction (SFSI), for simplicity the structure is often assumed as a SDOF model corresponding to the fundamental mode. However, depending on the distribution of mass and stiffness of the structure, the contribution of the fundamental mode may vary. The contribution of the higher modes to the response of the structure could become more significant. This study reveals the effect of higher modes on the response of structure including SFSI. The response of a multi-storey structure and the response associate with the fundamental mode of the structure are compared. To calculate the response of structures with SFSI, a macro element model is used to simulate the plastic soil deformation. The consequence of using a SDOF model for the structural response including nonlinear SFSI is discussed.*

1 INTRODUCTION

For structures with a shallow footing, seismic forces induced by a ground excitation may cause a moment at the footing to exceed the elastic limit of the foundation soil. Consequently, soil bearing failure may occur. It is commonly accepted that nonlinear soil behaviour should be avoided in seismic design by providing more than sufficient foundation bearing strength. However, it has been observed that although the footing has been properly designed, the structure can still experience plastic deformation of foundation soil during earthquake [1-6]. Examples can be found in almost all major earthquakes, e.g. Valdivia Earthquake 1960 [2], Michoacan Earthquake 1985 [3], Kobe Earthquake in 1995 [4], Izmit Earthquake 1999 [5, 6], and the 2010 Darfield and 2011 Christchurch earthquakes [7]. Study of the response of structure with nonlinear SFSI is thus relevant.

Observations of buildings following major earthquakes e.g. Housner [2] and Gazetas et al. [5] showed that structures experiencing minor nonlinear SFSI, in fact, performed better than expected. These observations triggered a number of analytical and numerical investigations. Yim and Chopra [8] developed one of the first SFSI models using a Winkler foundation. The flexibility and damping of the supporting soil were represented by independent spring-damper elements distributed over the width of the foundation. Psycharis and Jennings [9] extended the study by simplifying the Winkler foundation model using a two-spring foundation model. Psycharis [10] used the Winkler foundation model to conduct a parametric study. The influence of damping ratio, aspect ratio and stiffness ratio between the structure and the supporting spring on the structural response was investigated. Based on the results obtained from these analytical models [8-10], a series of response spectra were proposed to determine the seismic response of structures on flexible supporting ground. Recently, numerical methods were also developed to incorporate the nonlinearity of soil. The response of a structure with SFSI calculated using finite-element approach was discussed by Wolf [11]. Nova and Montrasio [12] proposed an equation to calculate the movement of the footing during soil plastic deformation. This equation was incorporated in a macro element model so that the response of structure with nonlinear SFSI can be calculated [13]. Chouw and Hao [14] also numerically studied and showed that SFSI can affect the pounding force between adjacent bridge structures.

For simplicity, most numerical studies of structures with SFSI use a single-degree-of-freedom (SDOF) system to represents the structure. The contribution of higher modes to the total response of the structure with SFSI was neglected. Depending on the distribution of mass and stiffness of the structure, the contribution of the higher modes to the response of the structure could be significant. This study reveals the effect of higher modes on the response of a structure with SFSI. The response of a two storey structure with SFSI was numerically calculated by representing the structure using a SDOF system and a 2-degree-of-freedom (2DOF) system. To include the effect of nonlinear SFSI, a macro element model is used.

2 NUMERICAL SIMULATION

2.1 Higher mode of structure

The structure used for this study was a two storey steel building. The structure had a storey height of 3 m and floor area of 25 m^2. For the 1^{st} floor and the roof, the seismic mass was 42000 kg and 21000 kg, respectively. The columns of the structure were 250UC72.9 steel section. The footing mass (m_o) was 42000 kg. The beams and foundation were assumed rigid. The structure can be described using a 2DOF system. Each DOF represented the horizontal movement at the floor level. The lateral bending stiffness of each floor can be calculated using the total bending stiffness of the columns.

The structure can also be represented using a SDOF system. An equivalent base shear method was used to obtained the effective mass (m^*) and height (h^*). The procedure is described e.g. by Chopra [15]. The effective mass and height of the SDOF system was determined to be 61000 kg and 4.3 m, respectively. The effective mass represented 96.8% of the total mass of the structure. The fundamental frequency of the SDOF system was defined to be the same as that of the two storey structure. Figure 1 shows the parameters of the 2DOF system and the SDOF system. The footing mass and size of the SDOF system were identical to that of the 2DOF system.

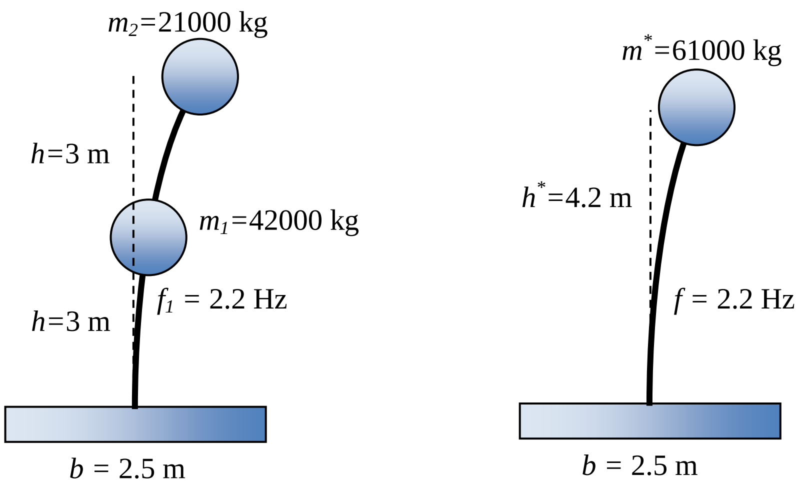
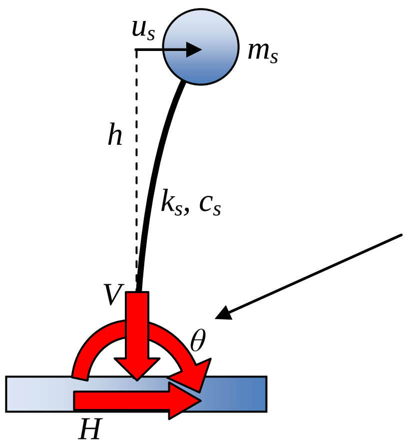


Figure 1: The 2DOF system and the equivalent SDOF system.

2.2 Macro element model

The response of a structure with nonlinear SFSI was calculated with the help of a macro element model. The concept of the model is to simplify the response of a foundation-soil system using three degrees of freedom. They are the vertical, horizontal displacements and rotation of the center of the foundation. With these additional DOFs at the footing location, the effect of footing response under the deformation of foundation soil can be included. Figure 2 shows the macro element model. The elastic stiffness of the 3DOF footing system can be calculated using the formulation described in Chouw and Hao [14].



$$\mathbf{m_F} = \begin{bmatrix} m_s + m_o & m_s h & 0 \\ m_s h & m_s h^2 + J_o & 0 \\ 0 & 0 & m_s + m_o \end{bmatrix}$$

$$\mathbf{k_F} = \begin{bmatrix} k_H & & \\ & k_\theta & \\ & & k_V \end{bmatrix} \quad \mathbf{c_F} = \begin{bmatrix} c_H & & \\ & c_\theta & \\ & & c_V \end{bmatrix}$$

$$\mathrm{u_F} = \begin{bmatrix} H & \theta & V \end{bmatrix}^T$$

Figure 2: The macro element model.

2.3 Nonlinearity of the foundation system

The nonlinearity of the structure-foundation-soil system is described at the 3DOF of the foundation system. The nonlinearity will be initiated when a bearing failure of soil is intitiated. To determine the initiation of the bearing failure, a bearing strength surface was considered. Nova and Montrasio [12] have proposed an equation for determining the initiation of a bearing failure of a strip footing on sand (Eq. (2)). The equation was developed and improved though a number of experimental and numerical studies (e.g. [16]).

$$f(\mathbf{F}) = \left(\frac{H}{\mu V_{\max}}\right)^2 + \left(\frac{M}{\psi B V_{\max}}\right)^2 - \left(\frac{V}{V_{\max}}\right)^2 \left(1 - \frac{V}{V_{\max}}\right)^{2\xi} \tag{2}$$

where B is the width of the foundation; V_{max} is the ultimate bearing capacity of foundation soil under vertical centered load; ψ, μ and ξ are the parameters of the bearing strength surface and suggested to be 0.43, 0.9 and 0.95, respectively [16]; **F** is a vector which contains the horizontal, vertical actions and moment at the foundation (H, V and M in Figure 2).

With **F** calculated using the product of the stiffness matrix ($\mathbf{k_F}$) and the displacement vector ($\mathbf{u_F}$) of the foundation. Eq. (2) can be evaluated. If $f(\mathbf{F}) < 0$, the combined action of a shallow foundation is below the bearing capacity of foundation soil. Bearing failure of the foundation soil does not occur. In contrast, a value of $f(\mathbf{F}) = 0$ indicates that the foundation action will cause bearing failure. Theoretically, it is impossible to have a value of $f(\mathbf{F}) > 0$, since the soil cannot sustain any actions greater than the bearing capacity.

Once the indication of bearing failure was confirmed, a non-associated flow rule (Eq. (3)) was applied to calculate the unrecoverable displacement and rotation of the foundation due to soil plastic deformation.

$$g(F) = \lambda^2 \left(\frac{H}{\mu V_{\max}}\right)^2 + \chi^2 \left(\frac{M}{\psi B V_{\max}}\right)^2 + \left(\frac{V}{V_{\max}}\right)^2 - 1 \tag{3}$$

where B is the width of the foundation; λ and χ are two non-dimensional parameters, and V_{max} is the maximum bearing capacity of soil under vertical loading.

This flow rule was originally developed by conducting a large scale shake table test to quantify the non-dimensional parameters of Eq. (3) [13, 16 and 17]. For sand, the values of λ and χ were suggested to be 2.5 and 3, respectively. In this study, the supporting soil was defined to be sand and had a shear wave velocity of 400 m/s. The Poisson's ratio and density were assumed to be 0.33 and of 1560 kg/m^3, respectively. It is assumed that the structure behaves elastically during the earthquake loading.

2.4 Earthquake excitation

The ground excitation for the nonlinear time history analysis was simulated based on a Japanese design spectrum [18]. In this study, medium soil category was considered. This is because in general, loose soil is of no practical interest for shallow foundation design. An alternative foundation scheme, such as foundation through piles, is usually preferred. On the other hand with dense or very-dense soil condition significant settlements are not likely to occur [19]. The applied ground acceleration is shown in Figure 3.

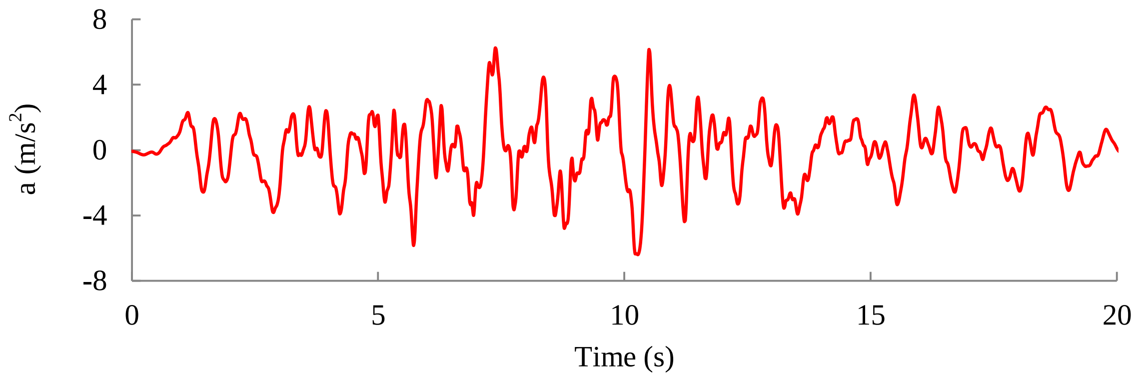


Figure 3: Ground acceleration.

3 RESPONSE OF STRUCTURE WITH SFSI

3.1 SFSI with elastic soil

Figure 4 shows the horizontal displacement (u) at the top of the structure relative to the ground. The structure was represented using the SDOF (dashed line) and 2DOF (solid line) system, respectively. Considering elastic soil, the horizontal displacement calculated using the SDOF system was larger than that using the 2DOF system. Using the SDOF system, the maximum top horizontal displacement was 66.4 mm. When considering the 2DOF system, the maximum top horizontal displacement was 59.1 mm. The maximum horizontal displacement of structure estimated using the SDOF system was 12.4% larger than that calculated using 2DOF system. It is shown that without considering the higher modes, horizontal displacement of structure can be overestimated.

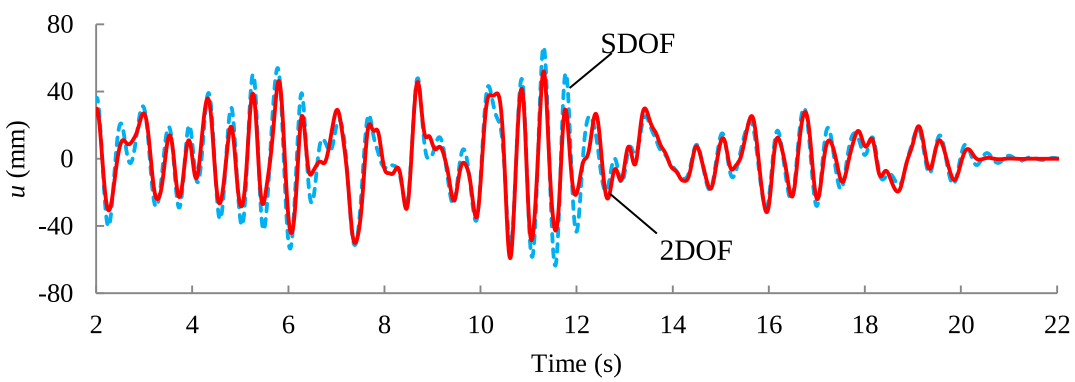


Figure 4: Influence of MDOF system on the structural response with linear SFSI

3.2 Response of structure with nonlinear SFSI

Figure 5 shows the horizontal relative displacement (u) at the top of the structure with non-linear SFSI. While the dashed line represents the result calculated using the SDOF, the solid line illustrates that obtained considering the 2DOF system. A residual horizontal displacement was found at the end of the excitation due to soil plastic deformation. When using the SDOF and 2DOF system, the residual horizontal displacement was 4.9 mm and 1.4 mm, respectively. Including the structural higher mode, the residual displacement of the structure was smaller. The maximum horizontal displacement at the top of the 2DOF system was also smaller than that of the SDOF system. The maximum horizontal relative displacement considering the SDOF and the 2DOF systems was 62.6 mm and 57.5 mm, respectively.

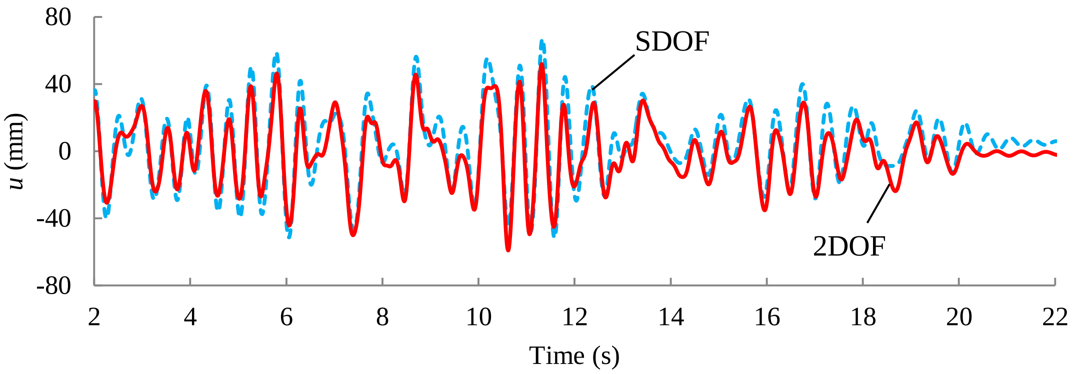


Figure 5: Influence of MDOF system on the structural response with nonlinear SFSI

Using the SDOF and 2DOF systems, the calculated footing rotation (θ) of the structure was also different. As shown in Figure 6, the maximum footing rotation calculated using the SDOF system was 0.17^o. Using the 2DOF system, the maximum footing rotation was -0.06^o. At the end of the excitation, the residual rotation of the footing was 0.06^o and -0.02^o for the case of the SDOF and 2DOF, respectively.

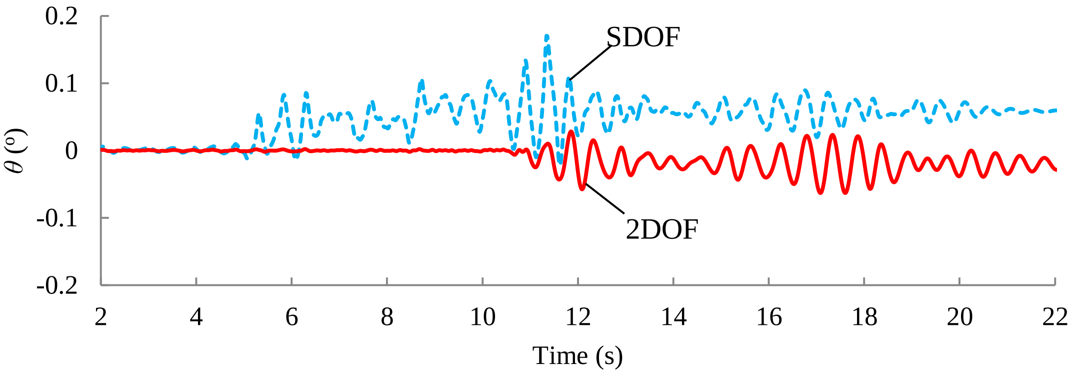


Figure 6: Effect of nonlinear SFSI and 2DOF system on footing rotation θ

4 CONCLUSIONS

This study focused on the effect of a higher mode on the response of a structure with SFSI. The response of a two storey structure was numerically calculated by representing the structure using a SDOF system and a 2DOF system. The effect of foundation soil deformation was included using a macro element model. Both elastic and nonlinear SFSI were considered. The mass of the SDOF system represented 96.8% of the total mass of the structure.

- Despite the mass of the SDOF system represents a large portion of the structural mass, the responses of the structure with SFSI calculated using the SDOF and the 2DOF system were different.
- With an elastic soil , the horizontal displacement of the structure calculated without including the higher mode was larger.
- Considering nonlinear soil, the maximum and residual horizontal displacement of the structure calculated using the SDOF system was also larger than the corresponding 2DOF system.
- Without considering the higher mode, the maximum and residual footing rotation of structures during earthquake can be overestimated.

ACKNOWLEDGEMENT

The authors are grateful for the support of the Ministry of Business, Innovation and Employment through the Natural Hazards Research Platform under the Award 3708936.

REFERENCES

[1] N. Chouw and H. Hao. Pounding damage to structures in the 2011 Christchurch earthquake. *International journal of Protective Structures*, **3**(2): 123-139. 2011.

[2] G.W. Housner. The behavior of inverted pendulum structures during earthquakes. *Bulletin of the Seismological Society of America*, **53**(2): 403-417. 1963.

[3] M. J. Mendoza and G. Auvinet. The Mexico earthquake of September 19, 1985 Behaviour of building foundations in Mexico City. *Earthquake Spectra*, **4**(4), 835-852, 1988.

[4] N. Chouw. Effect of the earthquake on 17th of January 1995 on Kobe, D-A-CH meeting of the German, Austrian and Swiss Society for Earthquake Engineering and Structural Dynamics, Technical University of Graz, Austria, 135-169

[5] R. Ulusay, O, Aydan and M. Hamasa. The behaviour of structures built on active fault zones: Example from the recent earthquakes of Turkey. *Structural Engineering and Earthquake Engineering*, **19**(2), 149-167, 2002.

[6] G. Gazetas, M. Apostolou and J. Anastasopoulos. Seismic uplifting of foundations on soft soil, with examples from Adapazari (Izmit 1999 earthquake), in *Foundations: Innovations, observations, design and practice*. London: Thomas Telford, 2003.

[7] L. Wotherspoon, A. Bradshaw, R. Green, C. Wood, A. Palermo, M. Cubrinovski and B. Bradley. Performance of bridges during the 2010 Darfield and 2011 Christchurch earthquakes. *Seismological Research Letters*, **82**(6), 950-964,2011.

[8] C. S. Yim and A. K. Chopra. Earthquake response of structures with partial uplift on winkler foundation. *Earthquake Engineering and Structural Dynamics*. **12**: 263-281. 1984.

[9] I. N. Psycharis and P. C. Jennings. Rocking of slender rigid bodies allowed to uplift. *Earthquake Engineering and Structural Dynamics*, **11**(1), 57-76, 1983.

[10] I. N. Psycharis. Effect of base uplift on dynamic response of SDOF structures. ASCE *Journal of Structural Engineering*, **117**(3), 733-754, 1991.

[11] J. P. Wolf. *Soil-Structure-Interaction Analysis in Time Domain*. Prentice Hall, Englewood Cliffs (NJ), 1988.

[12] R. Nova and L. Montrasio. Settlements of shallow foundations on sand. *Géotechnique,* **41**(2), 243-256, 1991.

[13] C. Cremer, A. Pecker and L. Davenne. Cyclic macro-element for soil-structure interaction: material and geometrical non-linearities. *International Journal for Numerical and Analytical Methods in Geomechanics*, **25**(13), 1257-1284,2001.

[14] N. Chouw and H. Hao. Significance of SSI and nonuniform near-fault ground motions in bridge response I: Effect on response with conventional expansion joint. *Engineering Structures*, **30**(1), 141-153, 2008.

[15] A. K. Chopra. *Dynamics of structures, 3rd Edition*. New Jersey: Prentice Hall, 2007.

[16] R. Paolucci, M. Shirato and M.T. Yilmaz. Seismic behaviour of shallow foundations: Shaking table experiments vs numerical modeling. *Earthquake Engineering and Structural Dynamics*, **37**(4), 577-595, 2008.

[17] P. Negro, R. Paolucci, S. Pedretti and E. Faccioli. Large-scale soil-structure interaction experiments on sand under cyclic loading. *Proceedings of the 12th World Conference on Earthquake Engineering*, Paper 1191, 2000.

[18] Japan Society of Civil Engineering (JSCE). *Earthquake resistant design code in Japan*, Tokyo, Maruzen, 2000.

[19] R. Paolucci. Simplified evaluation of earthquake-induced permanent displacements of shallow foundations. *Journal of Earthquake Engineering*, **1**(03), 563-579, 1997.