Shake Table Study of Soil Structure Interaction Effects in Surface and Embedded Foundations

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SUMMARY:
Dynamic Soil Structure- Interaction (SSI) is a collection of phenomena in the response of structures caused by the flexibility of the foundation soils, as well as in the free field response of soils media caused by the presence of structures. The effects of this phenomenon in dynamic behavior of building structures can be changed by embedment of foundation. The attempt of this study is to evaluate the seismic response characteristics of surface and embedded model buildings using experimental tests on the shaking table and finite element analyses. For this purpose, four scaled models of steel building structures with 5, 10, 15, and 20 stories have been designed, constructed, and studied as common representative buildings in urban areas. Both soft and relatively soft soil media with geometric scale model of 1/100 have been designed and considered in this study. These models subjected to earthquake records of El Centro, USA (1940), and Tabas, Iran (1981) using International Institute of Earthquake Engineering and Seismology (IIEES) shaking table. Different parameters studied in this research includes: building aspect ratio, shear wave velocity, frequency content, damping ratio, and acceleration of structural models. Also, the results of finite element analyses of soil-structure system have been compared with shake table results. It can be concluded that SSI effects reduces by increasing of the foundation embedment.

Keywords: Soil-Structure Interaction (SSI), Building structures, Shake table, embedded foundations, free field response.

1. INTRODUCTION

Determine the stress and displacement of structures under dynamic loads such as earthquake is the most important issues in dynamic structure. But in general, structures are in action with the surrounding soil, and therefore incoming loads to the surrounding soil of the structure, should be considered during the earthquake stimulations. Compared with the Structure, soil has an unlimited scope that wave propagation conditions should be considered in dynamic model. In recent years, many works have been done on dynamic soil structure interaction (SSI) for different types of structures, especially for heavy and massive structures, such as nuclear power plants, dams, coastal platforms, bridges and tall structures on the soft soil, which dynamic soil–structure interaction is very important.[Getmiri B. and Haeri S.M. *,1996].* The reduction of structural response results from the scattering of the incident waves from the foundation, and from radiation of the structural vibration energy into the soil. When the soil surrounding the foundation experiences small to moderate levels of nonlinear response, the SSI lead to significant absorption of the incident wave energy, thus reducing the available energy to excite the structure. An important challenge for future seismic design is to quantify this loss and exploit it in design of soil-structure systems [Mihailo D., et al, 2001].

There are limited criteria in building code requirements for investigation of soil–structure interaction effects. For example, we can refer to the NEHRP provisions for investigation of the SSI effects in seismic structural responses. But these rules didn't provide specific requirements for buried foundation. Therefore, a comparison of soil–structure interaction effects between building code requirements and shake table study is important for developing building code requirements. Complete information about background and different methods of soil–structure interaction analysis has can be find in references [Mihailo D., et al, 2001, Hosseinzadeh N., 2002, and Bazyar, M. H., Khani, M., 2007].

In the present study, the authors attempt to evaluate the seismic response characteristics of surface and embedded model buildings using experimental tests on the shaking table. Shaking table tests and finite element analyses of four steel building models with 5, 10, 15, and 20 stories have been studied in this paper for accounting the soil–structure interaction effects in the case of surface and embedded buildings.

2. INERTIAL AND KINEMATIC INTERACTION

Two physical phenomena that comprise the soil–structure interaction mechanisms are: Inertial interaction and kinematic interaction. The inertial interaction is due to structural vibrations gives rise the horizontal and rocking motion of the foundation relative to the free field. Frequency dependant of foundation impedance functions describes the flexibility of the foundation support as well as the damping associated with foundation soil interaction.

The kinematic interaction is deviation of stiff foundation motions as a result of ground motion incoherence, wave inclination, or foundation embedment. These effects are described by a frequency dependent transfer function relating the free-field motion to the motion that would occur on the base slab if the slab and structure were mass less. An explanation of the kinematic interaction due to translational excitation of a rigid mass less foundation slab so called "*τ factor*". This factor is defined as the ratio of the amplitudes of the harmonics in the rigid-base translational motion to the corresponding free field amplitudes.

A system commonly employed in simple field analyses of inertial interaction is shown in Figure 1, consists of a single-degree-of-freedom structure of height h, mass m stiffness k and damping c on a flexible foundation medium. The base flexibility including translation (u_f) and rotation (θ) is represented by complex stiffness $\overline{K}_y$ and $\overline{K}_\theta$. The real static stiffness K_u and K_θ of a rigid disk on a half space defined by:

$$K_u = 32\frac{1-\mu}{7-8\mu}Gr_{\circ} \tag{1}$$

$$K_\theta = 8\frac{1}{3(1-\mu)}Gr_{\circ}^{3} \tag{2}$$

Where G is the soil dynamic shear modulus, μ is the soil Poisson ratio, and $r_\circ$ is the foundation radii corresponding translation or rotation deformation modes to match the area or moment of inertia of the actual foundation.

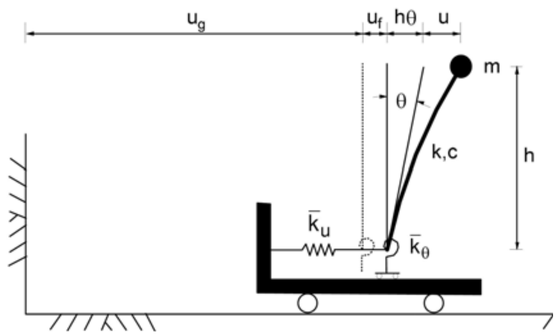

Figure 1. Experimental model of SSI system [Stewart J., P., et al 1998]

The flexible base parameters including effective period $\widetilde{T}$ and effective damping $\widetilde{\beta}$ are evaluated as follows:

$$\widetilde{T} = T\sqrt{1+\frac{\overline{K}}{K_u}(1+\frac{K_u \overline{h}^2}{K_\theta})} \quad (3)$$

$$\widetilde{\beta} = \beta_\circ + \frac{0.05}{\left(\frac{\widetilde{T}}{T}\right)^3} \quad (4)$$

Where $T = k/m$ is the fixed base period, β is the fixed base damping ratio, and $\beta_\circ$ is foundation damping factor. Simplified soil–structure interaction provisions are included in NEHRP-2003 code based on the above equations [*NEHRP*, 2004].

The finite element formulation in dynamic soil and foundation problems implies a step further in the approximations for the definition of the soil media. It requires a discretization and a finite element definition of a determinate soil volume. This discretization alone would trap the energy of the system and distort its dynamic characteristics. To avoid this problem, the finite element formulation is often coupled with a transmitting boundary formulation. The resulting formulation is usually referred as Dynamic Finite Element Method. The transmitting boundary simulates the wave propagation into the exterior semi-infinite media and expresses the far field in terms of a free field behavior (isolated from the interaction with any other mechanical system). The transmitting boundary was then attached to a simpler discretization: a vertical column of quadrilateral elements for 2-D configurations and a vertical column of cylindrical elements for 3-D configurations. The mechanical formulation of this discrete region was expressed in terms of finite element approximations. This formulation was repeated on every common soil-foundation node to compute the stiffness matrix of the whole layered media [M. I. Julio Abraham Garcia., 2002]

3. EXPERIMENTAL APPROACH

Dimensional analyses are the framework for the scale model similitude in this test program. Three principle test conditions established for scaling parameters are as follows:

1. Testing is conducted in a 1-g environment, which defines model and prototype accelerations to be equal.
2. A model soil with similar density to the prototype soil is desired, which fixes another component of the scaling relations.
3. The test medium is primarily composed of saturated clay, whose undrained stress-strain response is independent of confined pressure thereby simplifying the constitutive scaling requirements.

Four structural models of 5, 10, 15 and 20 stories high and two relatively soft soil models were designed for the laboratory tests. The foundation system of structural models was considered as square rigid mats. In all building models, the height of each storey is 3 *cm* and the dimension of square rigid surface mats is 20cm$\times$20 *cm*. A geometrical scaling of 1/100 is considered for both soil and structure models. A special cylindrical flexible-wall container was designed and constructed to support the soil model with special emphasis on easy connection to the shake table. This container also provided sufficient environment to allow for the elastic half space of the soil. The diameter of ground specimen is 120 cm and the thickness of homogeneous single soil layer from the base rock is 60 cm. General view of structural models, single soil–structure interaction models and very low mass accelerometers for vibration recordings are shown in Figure 2 and Figure 3 respectively [Hosseinzadeh N., 2002].

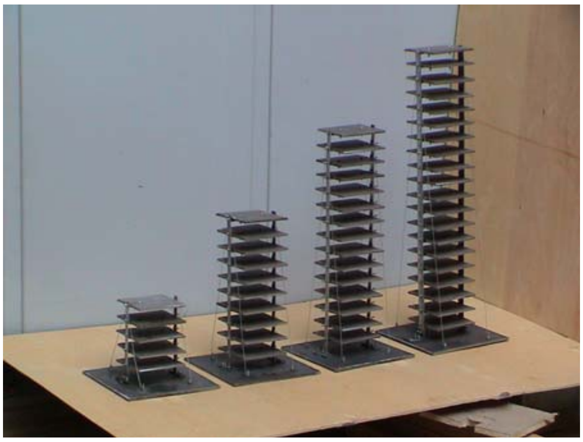
Figure 2. Structural Models

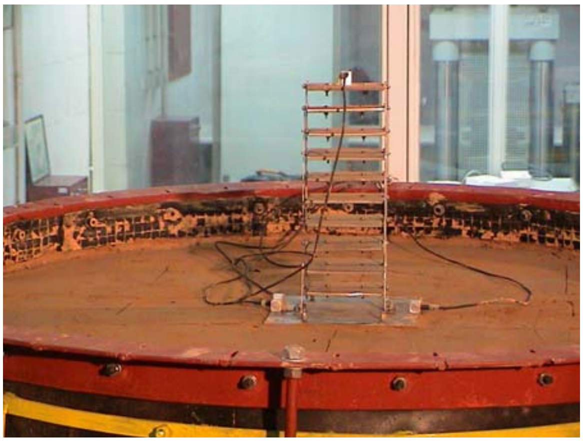
Figure 3. Single Structure on the soil

Horizontal component of scaled motions of Elcentro1940 (USA) and Tabas1981 (Iran) earthquakes with different Peak Ground Accelerations (PGA) was used as the inputs for the shaking table. Experimental tests have been carried out on the International Institute of Earthquake Engineering and Seismology (IIEES) one-component shaking table in Iran [Hosseinzadeh N., 2002]. The dimension of this table is 120 cm × 140 cm and the capacity of hydraulic jack is 50 KN.

The complete shaking table test program including four steps is summarized as follows. These steps repeated in two phase for two soil types III and II as defined in Iranian seismic code (standard IS2800) [BHRC, 1998].

1. Fixed base structural models.
2. Free field response of soil models.
3. soil–structure interaction tests for surface structure models.
4. soil–structure interaction test for embedded structure models.

Before the main test program, some preliminary tests considered to evaluate the shaking table performance. A typical test series for an individual model consisted of a hammer blow test, a sine sweep test, the Elcentro, and the Tabas earthquake ground motions, another sine sweep test, and a final hammer blow test.

4. TEST RESULT

A comparison between command signals and the shaking table shows that the table response match to the command signal is good and repeatable. So the results obtained from several model tests are comparable. Some important of free field response test results are as follows. These results are presented in the real scale.

Based on the soil dynamic laboratory tests and shaking table tests, the soil properties determined at small amplitude strains. In accordance to ASTM standard the soil type classified as Silty clay (CL-ML). The results of shear wave velocity (V_s), resonant frequency (f_n), and damping ratio (D) in small excitations are summarized in table 1. These frequencies are very close to the values obtained from the following analytical equation determined for a homogeneous soil layer of thickness H_s underlain by a rock or rocklike material [Das B. M., 1993].

$$\omega_n = \frac{(2n-1)\pi}{2H_s} V_s \tag{5}$$

Where n is the nth mode and V_s is the shear wave velocity that can be determined from shear modulus (G) and density of soil (ρ) as follows:

$$V_s = \sqrt{\frac{G}{\rho}} \tag{6}$$

Table 1. Frequency content and damping ratio of soil model in small excitations

Soil type	Vs (m/s)	Test freq. (f_n) (Hz)	Anal freq. (eq. 5) (Hz)	Damping (D) (%)
III	310	1.3	1.33	4.2
II	430	1.8	1.79	4.1

5. NUMERICAL METHOD

Due to the complexity of full scale finite element models it is helpful to perform preliminary tests on simplified models in order to verify the adequacy of the time and mesh discretization with respect to the input motion. It also provides good insight in the performance of the nonlinear material model. Therefore, a series of tests on a one-dimensional soil column have been proposed as follows:

- Static pushover test on nonlinear soil column to obtain the nonlinear behavior of the material model.
- Dynamic test of elastic soil column by applying an earthquake motion to the elastic soil column.
- Dynamic test of nonlinear soil column for investigation of the stability and the accuracy of the numerical method. It will be examined how propagation through an elastic-plastic material will change the frequency content of the motion [Jeremic B., et al, 2008].

5.1 Model Description

The material properties of the soil are given in Table 2. The discretization parameters, the maximum grid spacing Δh and the time step Δt are determined as following:

$$\Delta h \le \frac{V_s}{10\, f_{\max}} = \frac{310}{10 \times 10} = 3.1m \tag{7}$$

For the following analysis $\Delta h = 3m$ is selected. Therefore, the maximum time step is:

$$\Delta t \le \frac{\Delta_h}{V_s} = \frac{3}{310} = 0.0096\,s \tag{8}$$

Taking into account a further reduction of the time step about 60% for nonlinear material models $\Delta t = 0.0055s$ is chosen.

Table 2. The material properties of the soil

Friction angle (φ)	41.2°
Undrained shear strength (C_u)	10 KPa
Mass density (ρ)	1900 Kg/m^3
Shear wave velocity (V_s)	310 m/s

5.2 Static Pushover Test on Elastic-Plastic Soil Column

For the static pushover test, an elastic perfectly plastic Von-Mises material model is used. After applying self weight a horizontal load of 3.4 KN is applied to a surface node. The predicted shear strength of the first element that is expected to fail, the one at the surface, is:

$$\tau_f = C_u + z \times \rho \times g \times tan(\varphi) = 10 + 0.5 \times 1.9 \times 9.81 \times tan(41.2°) = 18.16\ KPa \quad (9)$$

Where z is the depth of the center of the first element, τ_f is shear strength, C_u is the undrained shear strength, ρ is the Mass density and φ is the friction angle.

Figure 4 shows the shear stress - shear strain of soil for the first element obtained from pushover analysis. Maximum analytical stress is 18.61 KPa which is 2.5% greater than the theoretical value. The initial slop in diagram (linear range) obtained from the soil shear modulus, G = 182585 KPa, is slightly lower than the value obtained from test results (G = 182590 KPa). Therefore, soil behavior of the finite element model is almost similar to the existing theoretical behavior.

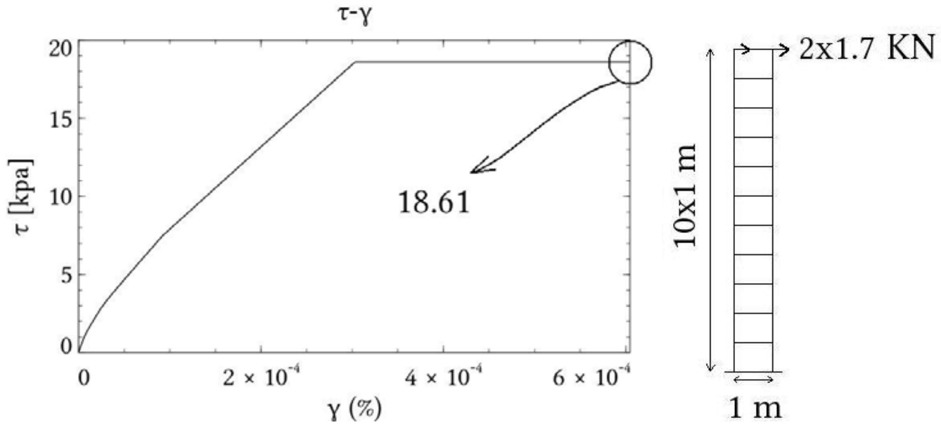

Figure 4. Shear stress-strain obtained from pushover analyses

5.3 Dynamic Test on Elastic Soil Column

The transfer function of a soil deposit describes the amplification between the frequencies of the motion at the base and at the soil surface and can be determined from the following equation [Jeremic B., et al, 2008]:

$$TF(\omega) = \frac{1}{\cos(\omega H \sqrt{\frac{\rho}{G + i\omega\eta}})} \quad (10)$$

Where H is the thickness of the soil deposit above the bedrock, ω is the circular frequency, and η is the damping coefficient. Figure 5 shows a comparison between the closed form solution and the numerical transfer functions obtained from the finite element analysis. Rayleigh damping is used to obtain the damping matrix. The analysis performed using stiffness proportional Rayleigh damping of β = 0.001 and β = 0.01 without mass proportional damping (α = 0).

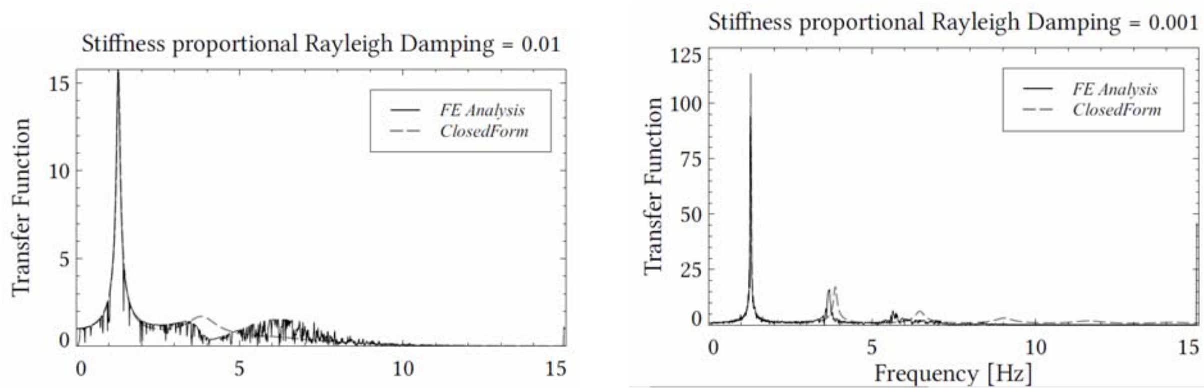

Figure 5. Transfer function of finite element and closed form solutions

5.4 Dynamic Test on Elastic-Plastic Soil Column

In the next step, Von-Mises elastic-plastic material model has been selected. The analysis were performed using time steps of dt = 0.0055 s and dt = 0.0028 s. Figure 6 shows the output acceleration and response spectra of free field surface. As shown in this Figure, there is acceptable agreement between two analyses. Therefore, dt = 0.0055s is selected for further analyses.

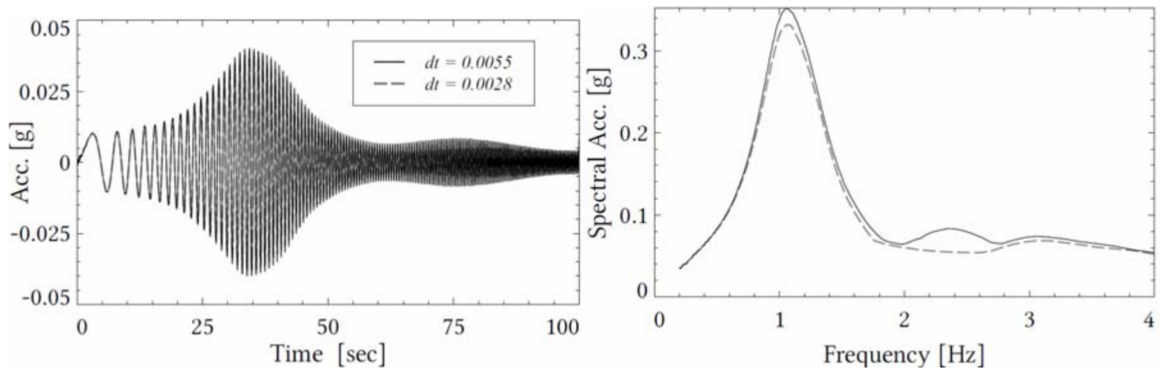

Figure 6. Acceleration responses for free field analyses with dt = 0:0055 *and dt*= 0:0028s

6. COMPARISON BETWEEN ANALYSIS AND TEST RESULT

For analytical models OpenSees software is used. Investigations indicate very good agreement between analysis and experimental results of the free field and structural models top responses. For example, Figure 7 shows a comparison between acceleration output from the analysis and testing soil model. Also, a review between test and analysis results of structural models in high amplitude input motions (PGA ≅ 0.3g) are shown in Tables 3 and Table 4. The results of these tables indicate the important effects of SSI in reducing the frequencies and increasing the damping ratios of structural models in comparison with fixed base models.

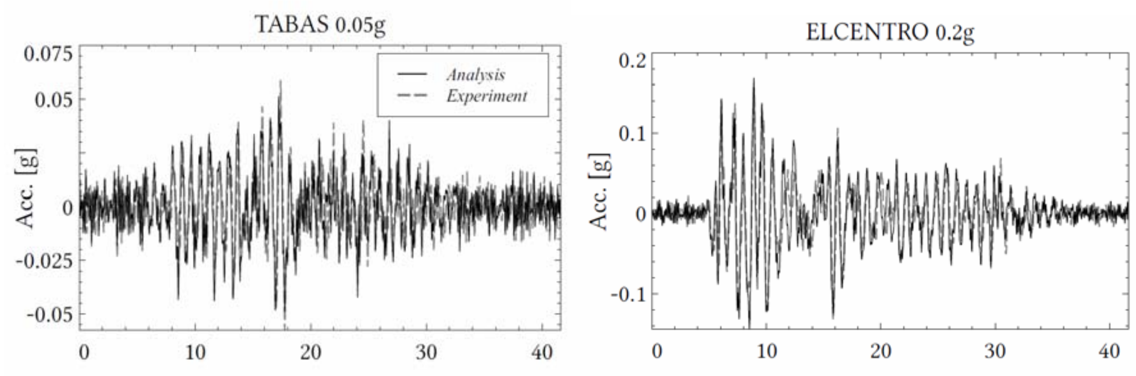

Figure 7. Comparison of analytical and experimental results

Table 3. Comparison between experimental and analysis frequencies for soil type III (Hz)

structural model	Fixed		with SSI(surface)		with SSI (embedded)	
	experiment	analysis	experiment	analysis	experiment	analysis
5 story	1.55	1.55	1.54	1.54	-	1.54
10 story	0.8	0.82	0.68	0.792	-	0.799
15 story	0.54	0.533	0.50	0.51	0.518	0.518
20 story	0.374	0.377	0.355	0.355	0.363	0.363

Table 4. Comparison between experimental and analysis damping ratio for soil type III (%)

structural model	Fixed		with SSI(surface)		with SSI (embedded)	
	experiment	analysis	experiment	analysis	experiment	analysis
5 story	0.43	0.66	0.93	1.18	-	1.12
10 story	7.03	6	2.9	13	-	12.5
15 story	1.45	1.98	2.26	2.7	1.47	2.77
20 story	1.57	1.82	2.58	2.47	2.42	2.35

7. INVESTIGATION OF CODE REQUIREMENTS

In this section, a comparison between test and analyses results of this study has been made with NEHRP and ATC code requirements. Important soil–structure interaction effects in these codes summarized in the effective period and the effective damping ratio of the dominant first mode of vibration.

7.1 Comparison of the Effective Period

The effective period ($\tilde{T}$) base on the NEHRP requirements is determined from equation 3. In this equation K_u and K_θ are horizontal and rocking stiffnesses respectively. Variation of effective period with respect to the ratio of $\bar{h}/r$ is shown in Figure 8 to Figure 10 for soil types III and IV. As shown in these Figures, the effect of horizontal and rocking mode are similar in the low values of this ratio. However, by increasing of the $\bar{h}/r$ ratios, the rocking mode will be dominant. Comparison of these results with NEHRP recommendations indicates good agreement and similar trends. However, NEHRP recommendation located in the lower bound for determination of effective periods especially in the higher range of $\bar{h}/r$ ratios.

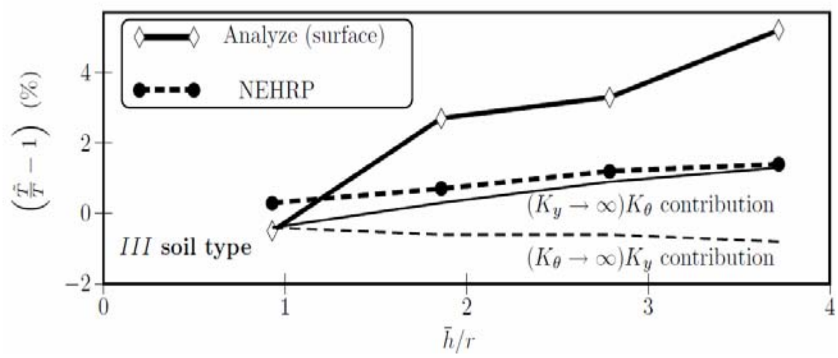

Figure 8. Variation of fundamental periods with $\bar{h}/r$ ratios in surface foundations (soil type *III*)

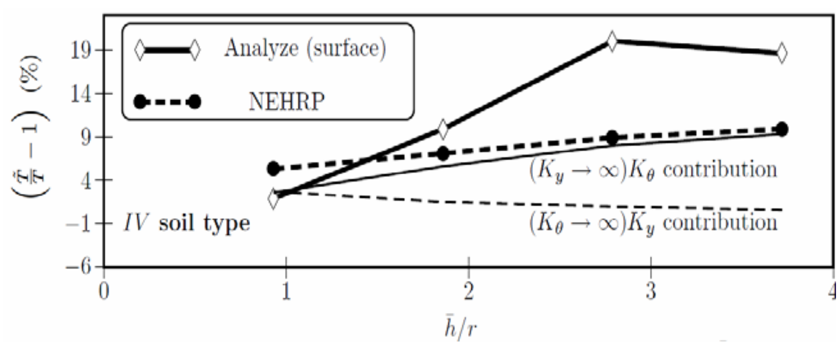

Figure 9. Variation of fundamental periods with $\bar{h}/r$ ratios in surface foundations (soil type *IV)*

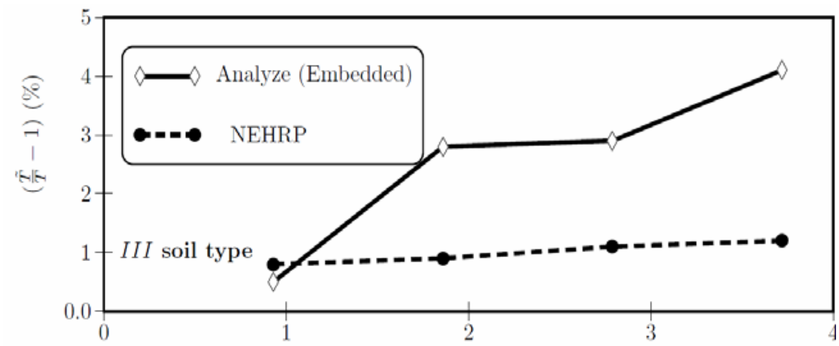

Figure 10. Variation of fundamental periods with $\bar{h}/r$ ratios in embedded foundations

7.2 Comparison of the Effective Damping

The effective damping ratio ($\tilde{\beta}$) base on the NEHRP requirements is determined from equation 4. In this equation β is the fixed base damping ratio, and $\beta_{\circ}$ is foundation damping factor. Variation of effective damping ratios with respect to the ratio of $\bar{h}/r$ is shown in Figure 11 for soil types III. As shown in these figure, by increasing the $\bar{h}/r$ ratios, the effective damping ratio is decreases. Comparison of these results with NEHRP recommendations indicates good agreement. However, NEHRP recommendation located in upper bound for determination of effective damping especially in the higher range of $\bar{h}/r$ ratios.

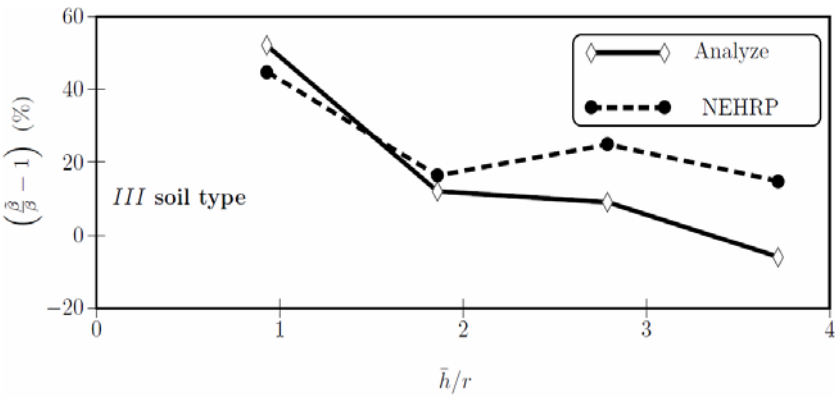

Figure 11. Variations of structural damping with $\bar{h}/r$ (soil type *III*)

8. CONCLUSIONS

In this paper, four structural models 5, 10, 15 and 20 floors as common representative of real buildings in urban area (such as Tehran) designed and constructed for experimental study of SSI (Soil–Structure Interaction) effects on the shake table. Two soil type III (*Vs* = 310 *m/sec)* and II (*Vs* = 430 *m/sec)* have been considered in this study based on the standard IR2800. Also, an additional soil type IV (*Vs* = 150 *m/sec)* has been selected for the analytical study. Based on the experimental-analytical study performed in this paper the following conclusions have been obtained:

1. A good agreement between experimental and finite element numerical modeling results has been observed. This agreement is very good especially for free field responses. Generally good agreement between experimental–analytical responses has been observed in fundamental mode response of soil and structure system.

2. Comparison of Experimental–Analytical model results with NEHRP recommendations indicates that by increasing the building aspect ratio ($\bar{h}/r$), the effective periods of the building increases, but these variations in experimental–analytical models is higher than the code requirements.

3. The fundamental period of SSI models in embedded foundations is lower than in the surface foundations. This means that in the case of embedded foundation, the effects of SSI are lower than the surface foundations.

4. The damping ratio in the case of embedded foundations is lower than the surface foundations. However, the embedment effects in low rise buildings are negligible.

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