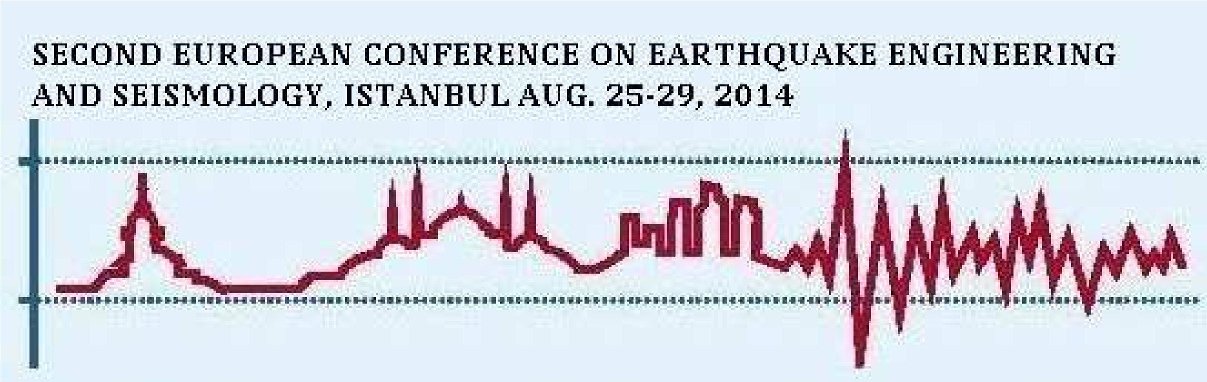

EFFECT OF SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF HISTORICAL MONUMENTAL BUILDING

Alireza MORTEZAEI[1]

ABSTRACT

Due to the presence of large number of masonry monuments in Iran, that is one of the most seismically active countries in the world, seismic vulnerability assessment of this buildings is one of the main concern. In these assessments, structural modeling has a very important role in the evaluation process. When the soil beneath the structural foundation is not “rock”, the structure cannot be modeled with a fixed base, as it is usually made in more practice, but it is necessary to take into account the interaction with the foundation soil. Soil-structure interaction (SSI) play an important role in the assessment of the dynamic behavior of the structures especially masonry ones. In this paper, the interaction between the super-structure and sub-structure (SSI) is investigated by modeling the soil to capture the overall response of such buildings. Firstly, the nonlinear response of masonry monumental building which can be representative of a broad range of such existing structures, is investigated while allowing for flexibility of the soil-foundation system and SSI effects. The results show that, use of flexible base in the analysis can lead to reduction in the structural response and damage consequences in joints. The results of this study also suggest that the compliance of the modelled soil for typical masonry structure have in average beneficial effects in terms of structural demand especially for low-rise element of structures. On the other hand, the governing component of these effects, i.e. rocking of foundation, can result in average to higher deformation of floors which points out to potential P-Δ problems and structural integrity.

Keywords: Soil-structure interaction, Masonry, Historical building, Nonlinear behavior.

INTRODUCTION

The risk of earthquake is unavoidable in every time and its occurrence follows irreparable damage. For this reason, prevention is necessary. Disasters, such as the devastating earthquake in Bam, Iran such a big shock, obliged experts and professionals to think about the ways to reduce earthquake-related building damage, save lives and property during earthquakes and seismic improvements made through monuments.

Seismic rehabilitation means to improve the seismic behavior of existing structures. As it is very important to protect the monuments from incidents and damage, seismic rehabilitation can help to reduce the damage significantly in historical monuments and cultural heritage. Conservation, repair

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and restoration of buildings, collections, and historical sites are essential. Architects and engineers that are responsible for this highly technical and artistic work, must, in addition to the dominance of the nobility and the technical aspects of their career, have cautiously and wisely insight to the past. National monuments are alive and rich in past time messages and witness of ancient traditions that has remained to date. Restoration of the historic fabric of cities and towns is a way of interfering with the historic fabric and old buildings. That means a continuous and conscious effort to prevent erosion and corrosion, increasing the life of historical context of the city is needed.

One of the important issues that affect the actual behavior of historical monumental buildings is soil-structure interaction (SSI). Soil-structure interaction is a typical subject but, nevertheless, still new in its applicative aspects. Various research demonstrated the important effects that SSI has on the seismic response of structures. These studies included two-dimensional and three-dimensional soil-structure models using either the substructure method or the direct approach. A prevailing common conclusion of all studies is that SSI could produce significant effects on the seismic response of structures: both beneficial and detrimental effects were reported. Nevertheless, findings regarding the effects of SSI on the dynamic behavior of historical masonry structures is still very rare if not absent. In this paper, after the introducing SSI phenomena, the effect of soil-structure interaction on an historical masonry chimney is studied analytically.

SOIL-STRUCTURE INTERACTION

When the foundation soil is not a rock, the structure can't be modelled with a fixed base, as it is usually made in current practice, but it is necessary to take into account the interaction with the foundation soil (SSI). Under seismic actions, the interaction generated between the structure and the foundation soil can be divided in two different mechanisms, generally known as kinematic and inertial interaction. Due to the first mechanism, the motion of the foundation is different from the motion of the soil in free field conditions. The second mechanism is originated because the inertial forces of the structure (included the foundation) are transferred to the foundation as shear and bending moments, which in turn will be transferred again to the surrounding soil. Early studies showed that neglecting the SSI is usually considered safe in the structural design. However, recent studies have enhanced cases for which soil-structure interaction, if neglected, can lead to unsafe design (Nakhaei and Ghannad, 2008).

In general, the importance of soil-structure interaction phenomena can be estimated by means of some synthetic parameters (Ciampoli and Pinto, 1995), such as:

1) the relative stiffness between the structure and the foundation soil, $1/\sigma$:

$$\frac{1}{\sigma} = f_{fix}\frac{h}{V_s} \tag{1}$$

where f_{fix} is the frequency of the fixed-base structure, h is the height of the structure, V_s is the shear wave velocity in the foundation soil;

2) the ratio h/r, where r is the radius of the equivalent circular foundation to which can be assimilated a shallow foundation with any shape, provided that it is characterized by a shape coefficient lower than 4:1. It can be intuitively seen that the higher are the parameters l/σ and h/r, more relevant is the SSI effect.

3) a third parameter combining the first two ones, ϕ:

$$\phi = \frac{1}{\sigma}.\left(\frac{h}{r}\right)^{0.25} \tag{2}$$

which can be used for the evaluation of the limit condition, over which the SSI effects are negligible for the dynamic behavior of the structure and its design. Past studies (Veletsos, 1977) indicate that for $\phi \leq 0.125$ the effect of SSI can be neglected.

In order to evaluate the effect induced by SSI on the fundamental vibration period of the structure, it is usually adopted the scheme of a single degree of freedom (SDOF) system, defined by a mass m placed at a height h and connected to the ground by means of concentrated translational and rotational springs, i.e. complex functions known as dynamic impedances. If the foundation structure is stiffer than the foundation soil, the springs can be considered uncoupled each other. According to this approach, the period of the coupled system, T_{comb}, can be evaluated using the formula proposed by Veletsos and Meek (1974):

$$T_{comb} = T_{fix} \cdot \sqrt{1 + \frac{k}{k_u} + \frac{k.h^2}{k_\theta}} \quad (3)$$

where T_{fix} represents the period of the fixed-base structure, k is the stiffness of the fixed-base structure modelled as SDOF, k_u and k_θ are translational and rotational spring stiffness, respectively.

In general, k_u and k_θ depend on the frequency ω of the input and can be expressed multiplying the static stiffness, K_u or K_θ, by a dynamic coefficient, α_u or α_θ, (Gazetas, 1991):

$$k_u = \alpha_u(\omega).K_u \quad (4a)$$

$$k_\theta = \alpha_\theta(\omega).K_\theta \quad (4b)$$

For the estimation of parameters in Eq. 4, Gazetas (1991) provides different analytical expressions and/or charts, depending on the shear stiffness, G, and the Poisson coefficient, ν, of the soil, the geometrical characteristics of the foundation and the frequency, ω, of the input.

General outline of seismic response analyses considering soil-structure interaction effects is depicted in Figure 1. In this paper, first of all, seismic response analyses of subsurface soil models are carried out to obtain the foundation input motions and physical properties of the subsurface soil considering the nonlinearity. Foundation input motions are defined as response waves at the bottom of foundations in the subsurface soil models. When seismic response analyses of models are performed, the obtained foundation input motions are used as input motions for the analyses of buildings. Next, dynamic soil stiffness and damping for models are calculated based on the physical properties of the subsurface soil. Finally, maximum drift angle is computed from the results of seismic response analyses of models by using foundation input motions. On the other hand, the seismic response analyses of the fixed-base model are carried out by using the foundation input motions at the ground surface.

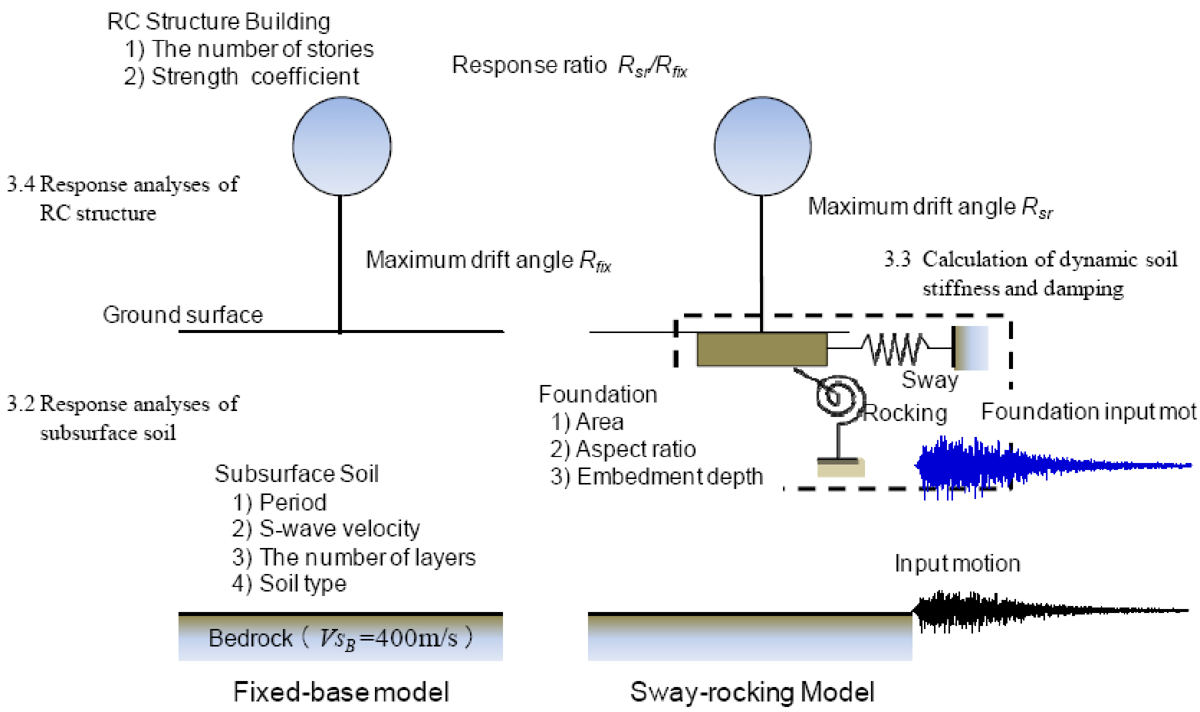

Figure 1. General outline of analysis models

DESCRIPTION OF THE STRUCTURE

Structures with historic value are regional cultural assets worth preserving. Industrial masonry chimneys built at the early of 20th century are common in many Iranian cities. They were built to get rid of smoke and to create the necessary draught for industrial processes. Currently, in many cities these chimneys form a characteristic landscape and are often protected by law as part of the cultural heritage. Figure 2 shows one of the industrial chimneys that can be observed in the city of Semnan (Iran). Existing literature related to the modelling of this type of structure is rather scarce. It is therefore appropriate herein to study these constructions since existing knowledge regarding their behaviour is very limited and, in addition, in many places these chimneys are considered the silent witnesses of the past and they are protected by law as cultural heritage.

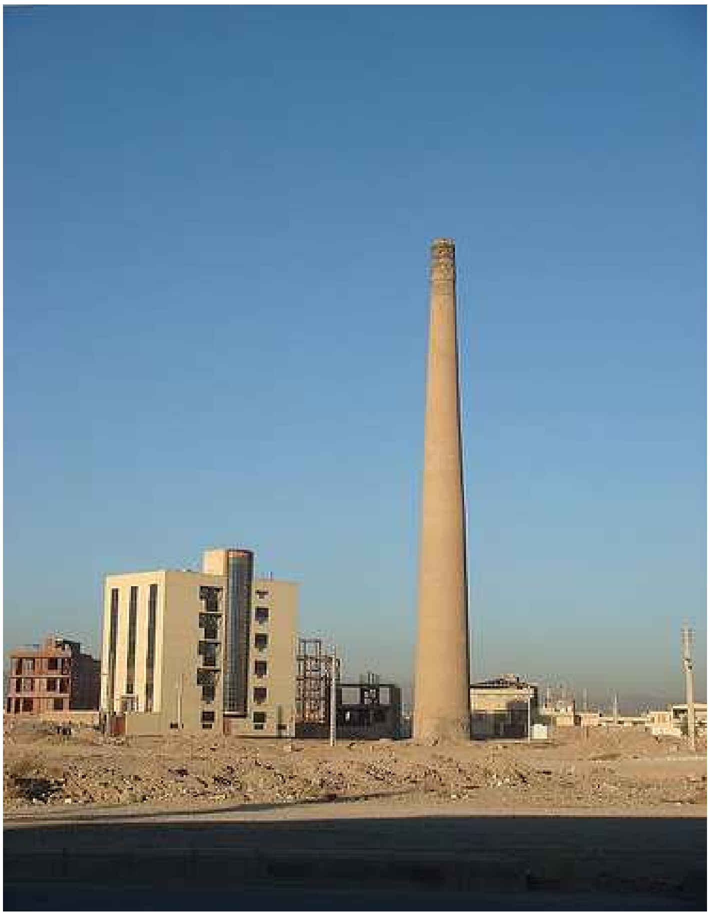

Figure 2. Industrial masonry chimney in Semnan (Iran)

The structure studied in this paper has been made of brick and stands to an elevation of over 25m for discharging combustion smoke and creates the necessary chimney effect in the industrial procedure. Therefore, it comprised of straight and prismatic forms and consists of three parts: base, shaft and crown.

The circular section and dimensions of this chimney are representative of the existing chimneys throughout Semnan and central Iran (Mortezaei and Kheiroddin, 2010). The dimensions of the chimney (in cm) and a cross section at 17m high are given in Figure 3a. The chimney internal diameter and wall thickness are assumed to be linearly varying. It will be assumed that the chimney under study does not exhibit structural damage, cracks or deformations caused by thermal actions. The 3D finite element model developed in order to study the seismic behaviour of the chimney is shown in Figure 3b. For modelling of chimney shaft, 8-node hexahedral solid elements with three degrees of freedom per node and eight integration points are used. Displacements at the base of the chimney have been considered fixed in vertical direction. No soil-structure interaction or base rotations have been taken into account.

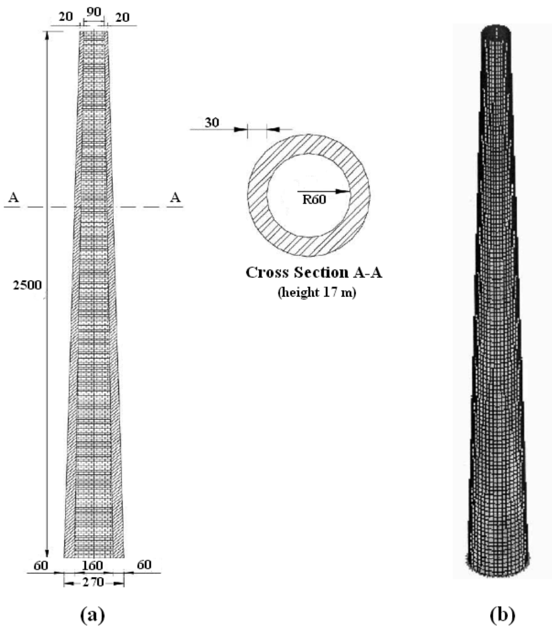

Figure 3. Studied masonry chimney; (a) longitudinal section and cross-section at 17m high, (b) finite element model

NONLINEAR FINITE ELEMENT PROGRAM

A nonlinear finite element analysis program, NONLAMS (NONLinear Analysis of Masonry Structures), from the earlier version of NONLACS2 program developed by Kheyroddin (1996) is used to perform the analysis here. The program employs a layered finite element approach and can be used to predict the nonlinear behavior of any masonry structure. This includes shells, shear walls, or any combination of these structural elements.

In this program, a hypo-elastic model using the principle of equivalent uniaxial strains is used. This nonlinear model is based on the biaxial orthotropic hypo-elastic concrete model of Darwin and Pecknold (1977) and has been modified for application to masonry according to Luciano and Sacco (1997). It uses the principle of equivalent uniaxial strains as a simplification of the complex biaxial material behavior.

The constitutive relation is described for each principal stress direction by means of the uniaxial stress-strain relation and the equivalent uniaxial strains are fictitious strains in the principal stress directions. The principal stresses correspond to the stresses of a fictitious uniaxial state. An important advantage is that the uniaxial stress-strain relationships and other required material characteristics can be derived by means of uniaxial tests. The failure criterion used is based on the assumption that the angle between the bed joints and the first principal stress direction is 45°, since the cracks in shear walls subjected to in-plane seismic loading usually arise under this angle.

The monotonic stress-strain relationship (Figure 4a) consists of three regions. The tension region is defined by a straight line OE ending at the point of the maximum tensile strength E (ε_{cr}). The compression region consists of the increasing range OC, defined by the compression strength C (ε_{max},f'_m), and the decreasing range CD describes the softening effect by a linear behavior up to point D (ε_{cu}).

For analysis of most plane stress problems, masonry is assumed to behave as a stress-induced orthotropic material. In this study, the orthotropic constitutive relationship developed by Bazant et al. (1986) model is used for modeling the masonry using the smeared cracking idealization. The constitutive matrix, D, is given by:

$$D = \frac{1}{(1-\nu^2)}\begin{bmatrix} E_1 & \nu\sqrt{E_1 E_2} & 0 \\ \nu\sqrt{E_1 E_2} & E_2 & 0 \\ 0 & 0 & \frac{1}{4}(E_1 + E_2 - 2\nu\sqrt{E_1 E_2}) \end{bmatrix} \quad (5)$$

in which, E_1 and E_2 are the tangent moduli in the directions of the material orthotropy, and ν is the Poisson's ratio. The orthotropic material directions coincide with the principal stress directions for the uncracked masonry and these directions are parallel and normal to the cracks for the cracked masonry. The concept of the "equivalent uniaxial strain" developed by Darwin and Pecknold (1977) is utilized to relate the increments of stress and strain in the principal directions. Therefore, stress-strain curves similar to the uniaxial stress-strain curves can be used to formulate the required stress-strain curves in each principal direction.

The strength of masonry, σ_c, and the values of E_1, E_2 and ν are functions of the level of stress, and the stress combinations. The masonry strength when subjected to biaxial stresses is determined using the failure envelope developed by Kupfer et al. (1969). For the descending branches of both compression and tension stress-strain curves, E_i is set equal to a very small number, 0.0001, to avoid computational problems associated with a negative and zero values for Ei. The masonry is considered to be crushed, when the equivalent compressive strain in the principal directions exceeds the ultimate compressive strain of the masonry, ε_{cu}.

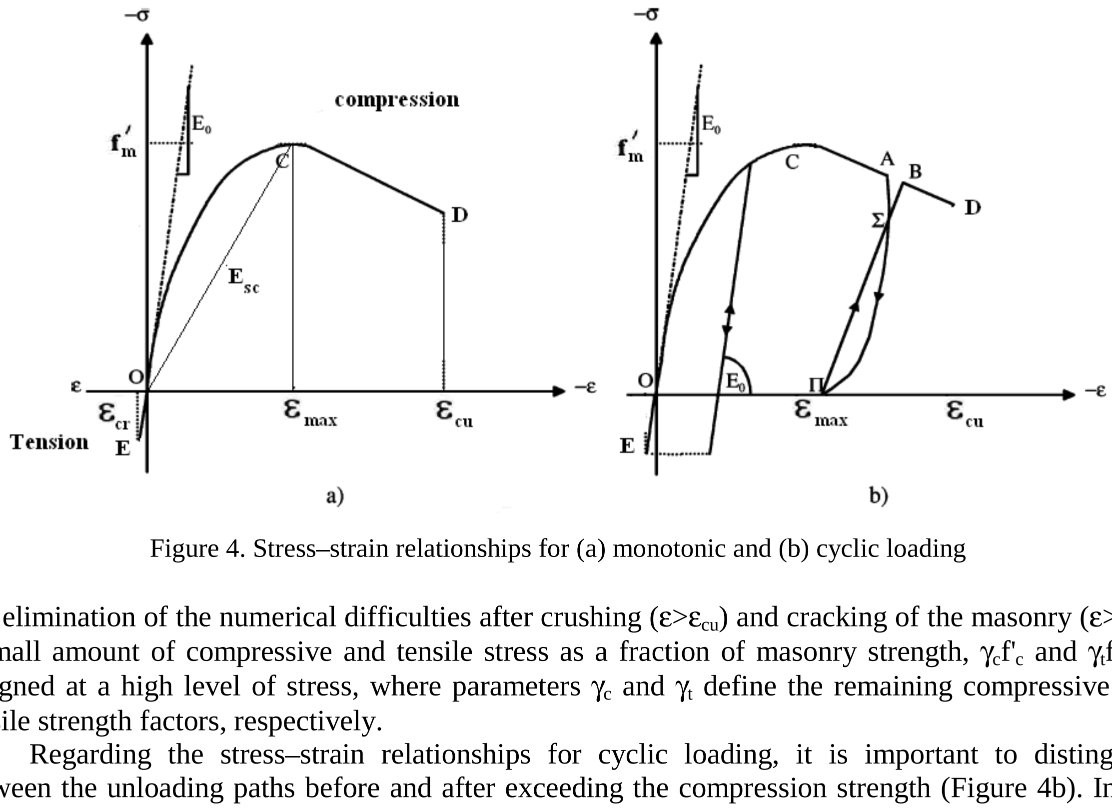

Figure 4. Stress–strain relationships for (a) monotonic and (b) cyclic loading

For elimination of the numerical difficulties after crushing ($\varepsilon > \varepsilon_{cu}$) and cracking of the masonry ($\varepsilon > \varepsilon_{tu}$), a small amount of compressive and tensile stress as a fraction of masonry strength, $\gamma_c f'_c$ and $\gamma_t f'_t$, is assigned at a high level of stress, where parameters γ_c and γ_t define the remaining compressive and tensile strength factors, respectively.

Regarding the stress–strain relationships for cyclic loading, it is important to distinguish between the unloading paths before and after exceeding the compression strength (Figure 4b). In the first case, the unloading path is a straight line defined by the elastic modulus E_0 and tensile stresses are still possible. In the second case the unloading path doesn't reach the tensile region. After exceeding the maximum tensile strength, cracks occur perpendicular to the principal stress direction. Based on a smeared crack model a smeared crack width is then calculated.

SOIL-STRUCTURE INTERACTION ANALYSIS

To assess the effect of SSI on the fundamental period of chimney, soil material is assumed to be linearly elastic. Figure 5 presents variation of fundamental period of the structure including soil elasticity (i.e. with flexible foundation), $T_{a,elastic}$, with soil stiffness for shear wave velocity, V_s, of 50 to 300m/s (shear modulus, G_s= 6.0 MPa to 150 MPa). To provide physical results, all values in Fig. 5 are normalized with respect to their corresponding values assuming structure to be fixed at base (i.e. assuming rigid foundation), $T_{a,fixed}$.

From Fig. 5, it is evident that ignoring SSI results in underestimation of the fundamental period by up to 125%. The effect of SSI on fundamental period becomes insignificant for soils with shear wave velocity of 250m/s or more. However, even for soils with shear wave velocity of 250m/s, ignoring SSI underestimates the fundamental period by about 30%. It worth noting that underestimation of the fundamental period (as a result of ignoring SSI) alters the earthquake spectral accelerations and, consequently, the seismic design forces as explained below.

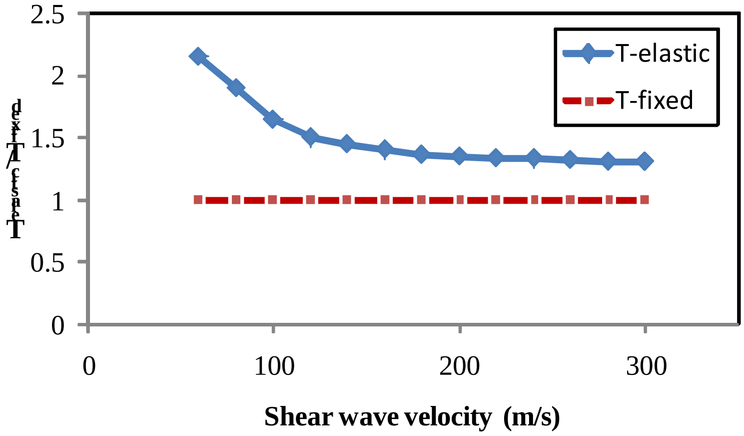

Figure 5. Effect of soil shear molulus on the fundamental period of chinmey

In order to investigate the effect of soil-structure interaction on altering seismic design, the spectral accelerations corresponding to the two conditions of including and excluding SSI effects are calculated and compared. For these purpose, the elastic design spectra recommended by Iranian earthquake code (2005) for four soil classes and depicted in Fig. 6 are used.

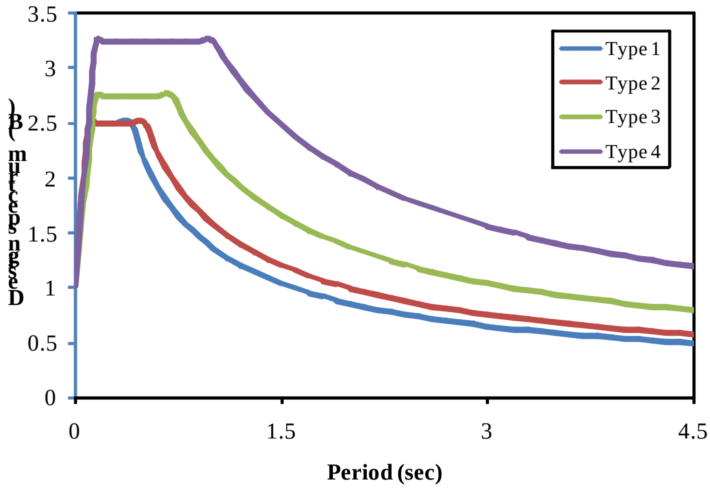

Figure 6. Design spectrum for soil type 1 to 4 according to the Iranian earthquake code (2005)

Now, for different elevations and soil stiffness, the spectral acceleration is calculated twice: first using Ta,elastic and then using Ta,fixed. As Ta,elastic is larger than Ta,fixed, ignoring SSI overestimates the building spectral accelerations with the percentages listed in Table 1 for various soil classes. Results listed in Table 1 shows that ignoring SSI could lead to large overestimation of spectral accelerations for the structures founded on loose to medium dense sandy soils. Underestimation of the fundamental period (as a result of ignoring SSI) alters the earthquake spectral accelerations and, consequently, the seismic design forces. In particular, for the response spectra recommended by Iranian earthquake code, the percentage overestimation of spectral acceleration due to overlooking SSI reached 275%. This would lead to uneconomic designs that can be avoided if SSI is accounted for.

Table 1. Percentage overestimation of spectral acceleration when SSI is ignored

Soil Class	Percentage
Class A	9 to 12
Class B	22 to 31
Class C	43 to 54
Class C	56 to 275

Realizing that the SSI effects are related to the structural elevation, four values of the soil shear wave velocity are considered: V_s= 50, 100, 200 and 600 m/sec. The earthquake record, Bam and Tabas, are normalized to peak ground accelerations (PGA) of 0.20g and 0.35g to model the maximum and minimum PGA values in Iran.

The results in Fig. 7 show that, compared to the correct "elastic foundation" response, the maximum base shear obtained when SSI is not accounted for (i.e. for fixed base) is increased by a factor of four. This is true for both earthquake records. The effect of frequency contents can be appreciated when comparing the base shear of different records. For fixed structure condition, the base shear under the Bam record is about 65% of its value under the Tabas one.

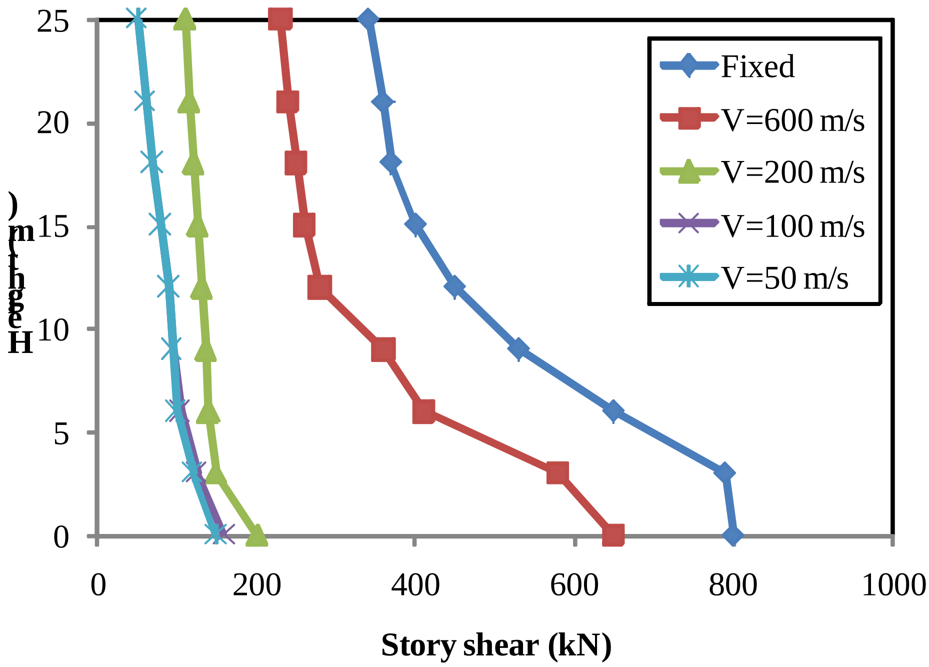

Figure 7. Maximum story shear for buildings due to Kobe earthquake (PGA = 0.3g)

Morover, time histories of base shear due to Bam and Tabas records (PGA=0.35g) for different values of the soil shear wave velocity show that SSI not only reduces the seismic shear forces but also modifies their time histories by altering the frequency content of the structural response. Results of the structural response to earthquakes with PGA of 0.2g showed similar trend as reported above with two observations. First, the effect of SSI was less-pronounced for the smaller PGA due to less soil nonlinearity. Second, the structural base shear for PGA=0.2g was about 31% to 39% of the corresponding value for PGA=0.35g. The above discussion confirms that neglecting SSI in seismic response analysis could lead to extremely overestimated actions that would considerably increase the construction costs.

CONCLUSIONS

Observations made during large earthquakes have specifically emphasized the importance of the dynamic soil-structure interaction. Based on these observations generally the effect of soil-structure interaction may increase or reduce the dynamic response, compared to the response of the fixed based structure, depending on the characteristics of the soil and the structure. In this paper, the effect of soil-structure interaction on the seismic response of a historical masonry chimney in Semnan, Iran was investigated. According to the investigation, the following conclusions are made:

1. Corresponding periods of vibration analysis that consider soil-structure interaction are greater than those without the consideration of the interaction (the soil is not a deformable support).
2. SSI has significant influences on the natural frequency and horizontal acceleration at top of chimney.
3. Soil structure interaction effect on inelastic displacement ratios should be considered for especially site classes C and D.
4. For soils shear wave velocity of 50 to 200m/s, ignoring SSI resulted in underestimation of the fundamental period by up to nearly 125%.
5. Underestimation of the fundamental period (as a result of ignoring SSI) alters the earthquake spectral accelerations and, consequently, the seismic design forces. In particular, for the response spectra recommended by Iranian earthquake code, the percentage overestimation of spectral acceleration due to overlooking SSI reached 275%.
6. Big differences in maximum story shear distribution are observed between strctures assumed to have fixed base and same buildings on elastic foundation. The maximum base shear obtained when SSI is not accounted for (i.e. for fixed base) can be as large as four multiples of its correct “elastic foundation” value.
7. Frequency contents of earthquakes have pronounced effects on building seismic response.

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