Effects of SSI on the seismic response of older structures before & after retrofit

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ABSTRACT: Comprehensive experimental and analytical studies have been conducted to understand the behaviour of frame buildings constructed before the introduction of modern design codes. This usually has been done assuming a fixed-base structure while ignoring the flexibility of soil.

The interaction between the super-structure and sub-structure (SSI) is investigated by modelling the soil as simple as possible to capture the overall response of the system.

As new analytical hysteresis rules and more advanced tools of analysis have been developed in recent years, and as part of a more comprehensive investigation on the response of older structures before and after retrofit, focus will be herein given on the response of SDOF systems representing a broad range of existing, newly designed and, retrofitted structures, while allowing for flexibility of the soil-foundation system.

The results of this study suggest that the compliance of simply modelled soil for typical building structures have in average beneficial effects in terms of structural demand especially in the case where as a consequence of implementing a retrofit strategy, the stiffness of the structure might increase. On the other hand, the governing component of these effects, i.e. rocking of foundation, can result on average in higher absolute displacement of floors.

1 INTRODUCTION

Destructive earthquakes in urban areas emphasized the need for assessment of existing buildings constructed before the introduction of capacity-designed-based seismic codes. As seismic assessment is the first step within a retrofit strategy, comprehensive experimental-analytical studies have been conducted in the past years to investigate the seismic response of pre-1970's structures. The results generally confirmed the inherent weaknesses and deficiencies of these buildings (Magenes and Pampanin, 2004; Pampanin et al, 2002; Calvi et al, 2002; Klingner and Bertero, 1978).

Typically the studies on assessment and retrofit of existing buildings have assumed the foundation of the structure to be rigid and compliance of foundation soil under seismic loading has been ignored, because the SSI considerations bring more complexity to the model and analytical procedure in general. It is known that flexibility of foundation usually is accompanied with lengthening of the fundamental period of the soil-structure system and an increase in the damping. Using typical code spectra, this generally leads to a reduction in the spectral acceleration and, consequently, to lower seismic demand (in terms of forces) for the superstructure but for actual structures this may not be always the case (Gazetas and Mylonakis, 2001). Whilst new analytical hysteresis rules and more refined modelling tools, based on extensive validation with experimental and/or observed data, have been developed in recent years, there is still a critical need for further studies to better understand the overall response of existing RC buildings prior and after retrofit intervention under earthquake loading in presence of a flexible soil-foundation system.

As part of a more comprehensive investigation on the response of older structures before and after retrofit, focus of this paper is to investigate the overall response of single-degree-of-freedom systems modelled on flexible foundation, and therefore to evaluate the SSI effects on the structural response albeit through the use of relatively simple soil models. The important parameters which may increase the effect of SSI have been considered such as the level of flexibility of the soil, the structural strength, intensity of ground motion etc. Different hysteresis rules assigned to SDOF systems were assumed as representative of different multi-degree-of-freedom (MDOF) structural types (including frames, walls or dual systems) prior and after retrofit period (Fig. 2, top).

2 PARAMETRIC STUDIES ON SDOF STRUCTURE

To understand the behaviour of structures considering the soil-structure interaction (SSI), it is desirable to investigate the behaviour of SDOF systems which in turn can be used to approximately represent multi-degree-of-freedom (MDOF) systems. This can be used later to compare how far SDOF results match with their corresponding results for MDOF systems. The statistical results of SDOF systems' response are organized to evaluate the effect of SSI by varying critical parameters such as: the period of the structure, the level of flexibility of the soil, the relative lateral strength of the structure, and the intensity of the ground motion. In addition, the influence of different hysteresis rules for the superstructure is also investigated.

The main structural response herein used to evaluate the performance is the peak deformation demand (e.g. peak roof displacement etc.). Relative displacements of the SDOF system are presented which can be representative of the level of damage in the super-structural components of the system.

2.1 *Parameters*

A set of ten historical strong ground motion records has been selected (from Christopoulos et al 2002) for use in time history analyses using Ruaumoko 2D (Carr, 2008). These records monitored in California on soil class C or D (according to NEHRP classification, corresponding to soil class B and C according to the NZS1170.5 guidelines), are representatives of ordinary earthquakes with no directivity and near fault effects. The target response spectrum (BSE-1) corresponds to the design spectrum defined by the International Building Code as two thirds of the Maximum Considered Event (MCE, BSE-2) spectrum which has a PGA = 0.6g. In addition to these two levels, half of the BSE-2 level spectrum is used in the analyses to apply three different intensity levels.

Four hysteretic rules are employed to model the cyclic force-deflection characteristics of the SDOF representation of the buildings considered: a linear model, an elastoplastic model, a stiffness degrading model (modified-Takeda model), and a pinched and stiffness degrading model (Pampanin model) (see Fig. 1).

The elastic model is a benchmark for parametric studies on SDOF systems. The elastoplastic model is regarded as representative of steel framed structures and also a point of reference for analyses of non-linear SDOF systems. The modified Takeda hysteretic model can be representative of reinforced concrete structures, whilst the Pampanin pinching hysteretic model is representative of existing pre-1970 (frame or wall) structures.

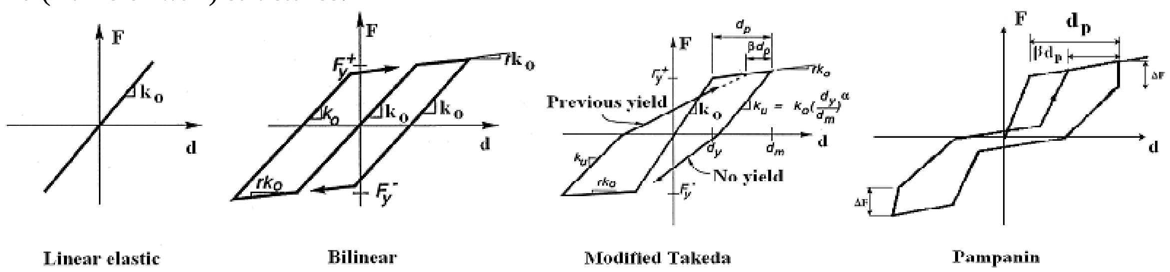


Figure 1: Hysteretic Models Used in the Analyses (Ruaumoko library, Carr, 2008)

To model the semi-infinite half-space, an appropriate simple cone model developed by Wolf (1994) can be implemented in the program Ruaumoko to take into account the flexibility of the foundation and soil. Two key parameters, i.e. stiffness and damping, are evaluated to be added in each degree of freedom of the foundation as a function of soil and foundation properties in the form of springs and viscous dampers (Fig. 2). Note that the viscous damper represents only the frequency independent radiation damping of the soil. For horizontal degree of freedom, in addition to equivalent linear approach, a nonlinear Ramberg-Osgood (R-O) (Ramberg & Osgood, 1943) curve is adopted to take into account the material damping (Fig. 2).

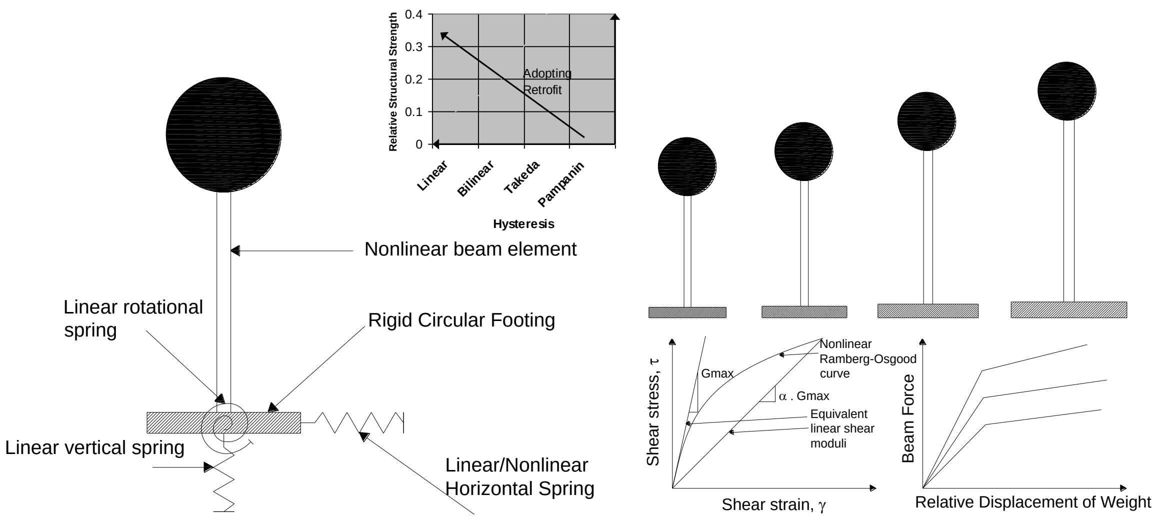
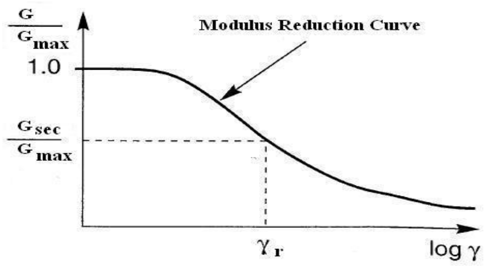


Figure 2: SDOF representation of MDOF structures (left), varying parameters of super- and sub-structure system: Period, soil stiffness and structural strength (right), schematic view of retrofitting of SDOF (top).

As the soil enters into its non-linear status, the shear modulus of the soil reduces substantially which means a decrease in shear wave velocity correspondingly (Fig. 3). In the equivalent linear approach, it is assumed that the soil is first at its initial state and then in a degraded state with reduced stiffness up to 90%. This degraded stiffness is used in the linear model described above (Fig. 2).



Characteristics	Value
Soil Shear Wave Velocity, C_s (m/s)	150
Poisson Ratio ν	0.33
Mass Density, ρ (kg/m^3)	1700

Figure 3: Variation of shear modulus with shear strain γ, (left), adopted soil parameters (right).

Ramberg-Osgood Model: This model allows accounting for the nonlinear soil behaviour more accurately. The model parameters are classified into those that have physical meanings such as shear modulus and correlation parameters that do not have physical meanings. The mathematical expression of Ramberg-Osgood model is:

$$\gamma = \frac{\tau}{G_{\max}}\left(1+\alpha\left|\frac{\tau}{\tau_f}\right|^{\beta-1}\right) \tag{1}$$

where G_{max} denotes the shear modulus at small strains, τ_f denotes shear strength of the soil, α and β are parameters. Note that τ_f need not be the actual shear strength and it can be also treated as a correlation parameter.

To determine those parameters, a silty sand (SM) type of soil under an effective confining pressure of $\sigma`_m$ = 50 kPa is assumed. It is possible to derive the reference shear strain from G/G_{max}-γ relationships (Fig. 4). If the reference strain, γ_r is defined as the strain where G/G_{max}=0.5 (Japanese Geotechnical Society, 1998), therefore, it can be assumed that γ_r = 0.04% for small number of loading cycles of N<1000 (Stokoe, 1999) (Fig. 4). Then τ_f can be computed from the equation below (Japanese Geotechnical Society, 1998):

$$\tau_f = G_{max}\gamma_r = C_s{}^2 \times \rho \times \gamma_r = 150^2 \times 1700 \times 0.0004$$
$$\tau_f = 15.3kN/m^2 \tag{2}$$

where γ_r is the reference strain and C_s and ρ are shear velocity and soil density, respectively. Bilinear factor β can be computed by the following equation (Japanese Geotechnical Society, 1998):

$$\beta = \frac{2+\pi\xi_{max}}{2-\pi\xi_{max}} = \frac{2+\pi \times 0.15}{2-\pi \times 0.15} = 1.616 \tag{3}$$

where ξ_{max} is the maximum damping ratio from ξ-γ relationships (Fig. 4).

To evaluate α, Ruaumoko (Carr, 2008) employs the following equation:

$$\alpha = 2^{\beta-1} = 2^{1.616-1} = 1.533 \tag{4}$$

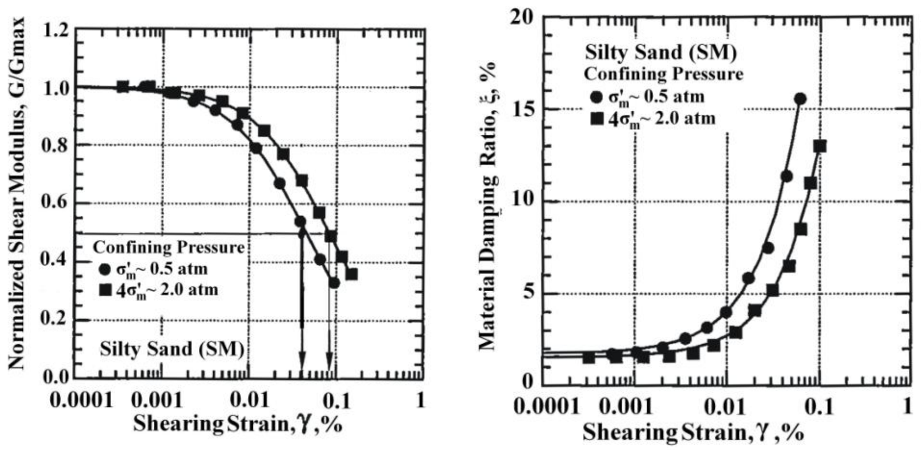


Figure 4: Effect of effective pressure on the G/G_{max} – logγ and on the ξ – log γ relationships for silty sands (Stokoe, 1999)

Equation 1 expresses the shear stress – shear strain relationship. This needs to be converted to a force-displacement relationship in order to obtain the required inelastic parameters of nonlinear spring. If the horizontal shear strain of the cone model is integrated over the semi-infinite medium:

$$D = z_0 \, . \, \gamma_0 \tag{5}$$

where D denotes the horizontal displacement of base foundation, z_0 is the apex height of the translational cone and γ_0 is the shear strain of the soil at ground surface (Arefi, 2008). Hence, the reference horizontal displacement, D_r and reference horizontal force can be evaluated by substituting the reference shear strain at surface, $\gamma_{0(r)}$.

$$D_r = 73mm$$
$$F_r = K_H . D_r = 375.8kN \tag{6}$$

where K_H is the elastic stiffness of horizontal spring and F_r the reference horizontal force.

2.2 *Results and Discussion*

A benchmark SDOF system has been assumed in this section and the important parameters varied to observe the sensitivity of SSI. The benchmark SDOF system is subjected to BSE-1 level earthquake and in the case of nonlinearity in the structure it yields at $F_y = 0.2W$, in which W is the weight of the structure. In addition, it is assumed that soil is in the initial stage of stiffness with maximum shear modulus.

2.2.1 *Influence of foundation flexibility*

Two levels of stiffness in the foundation soil are considered. First, with the initial soil stiffness $G = G_{max}$, and second when the shear strain in the soil is well beyond the linear threshold and has resulted in a reduced modulus of $G = 0.1G_{max}$. Moreover, an R-O spring models incrementally the nonlinear behaviour in the soil (Fig. 4). The relative displacement of the SDOF mass to the moving base is compared. Figure 5 illustrates such relative displacement response spectra for different hysteresis loops of super-structure. Note that the presented results are the average of 10 analyses by 10 different earthquake ground motions.

It is clear that for the structures with fundamental periods of less than one second, ignoring the SSI effects leads to a conservative response in terms of relative displacement. On the other hand, for less stiff structures with period more than one second, flexibility of the soil and foundation has increased the ratio between the flexible- and fixed-base structures, though this increase is less than 15%.

It should be added that the displacements presented in Figure 5 are relative structural displacements (SDOF superstructure relative to its base), which typical indicators of structural damage. The total-displacement spectra of the SDOF mass is not presented here. This can be important when pounding of adjacent structures is considered.

In a more general view, Figure 5 shows that considering SSI might have either advantageous or detrimental influences on the structural response; it is worth noting however that in most of the cases such variation is contained within 20%. It is clear that stiff soil and degraded soil results in similar trends while the R-O soil model illustrates very conservative answers. This may be due to inherently high material damping effects in R-O soil model which is ignored in equivalent linear approach.

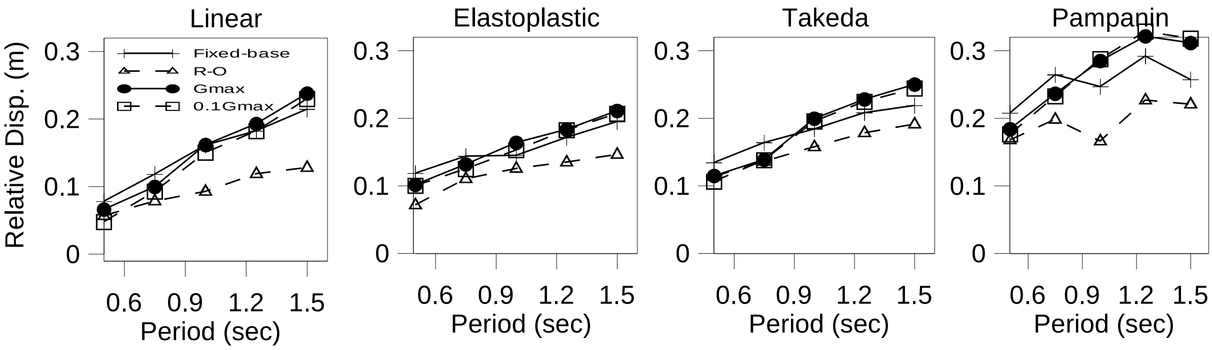


Figure 5: Relative displacement response spectra for different levels of soil flexibility (10 EQ's)

2.2.2 *Influence of strong ground motion intensity*

Three levels of strong motion were employed. The soil is considered to be within its initial stiffness i.e. $G = G_{max}$. BSE-1 is used as a reference throughout this section. The strength of the super-structure is kept so as the yielding force is 20% of the weight of the structure, $F_y= 0.2W$.

To better compare the fixed- and flexible-base foundation systems under different levels of ground shaking, it is essential to normalize all the spectral ordinates. To do this all the spectral displacement ordinates of flexible foundation is divided by their corresponding one with fixed-base foundation. Figure 6 illustrates the ratio of spectral displacement of flexible foundations to fixed-base one for each level of ground motion. The three spectra corresponding to elastoplastic, Takeda and Pampanin hysteresis have similar trends which may be due to the fact that they include induced damping in the structure. The pinched hysteresis loop, however, seems to be more significantly affected by the increase in earthquake intensity.

It is clear that for stiff structures, the BSE-1 level earthquake is comparatively more effective on the structure when fixed-base and flexible-base SDOF are analyzed. The other two levels of strong ground motion, namely, BSE-2 (=1.5 BSE-1) and 0.5 BSE-2 (=0.75 BSE-1) have minimal influences. This may mean, dealing with these two levels of motion considering or ignoring the SSI effect might not change the results significantly for the structure with $F_y = 0.2W$. This can show that when capacity and demand are disproportional, the effect of SSI could be ignored.

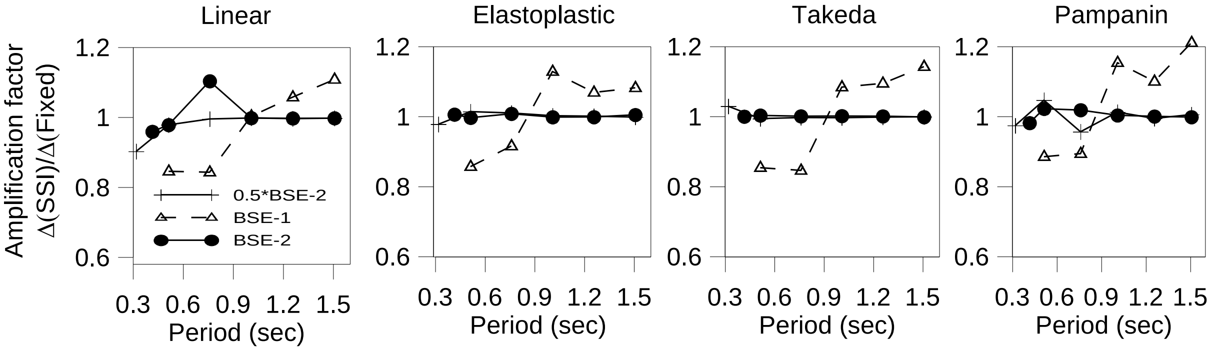


Figure 6: Amplification factor of each intensity level relative to fixed-base motion, F_y = 0.2W, G= G_{max}.

2.2.3 *Influence of super-structural strength level*

The super-structure strength is varied from as low as F_y = 0.1W to F_y = 0.4W, where W is the weight of the structure. The nonlinear displacement spectra are shown in Figure 7. The level of earthquake shaking in this section is assumed to be fixed at BSE-1. To find out the effects due to soil-structure interaction, the displacement of each level of structural strength with flexible soil is divided by the displacement of the structure with fixed-base foundation (Fig. 8). Note that for structures with F_y = 0.1W and F_y = 0.4W the SSI shows negligible effect and this factor is close to one, whereas, for the structures with F_y = 0.2W subjected to BSE-1 level earthquake the effect is bigger. It is expected that very high and low demands do not affect the response in terms of SSI analyses.

It is suggested to carry out more analyses to investigate the effect of SSI for a F_y= 0.1W under 0.5BSE-2 and also for F_y= 0.4W under BSE-2 level earthquake. From the trends obtained, it is expected that for those two case studies, the effect of SSI may not be negligible. This can be regarded also as proportional capacity and demand which needs to be verified.

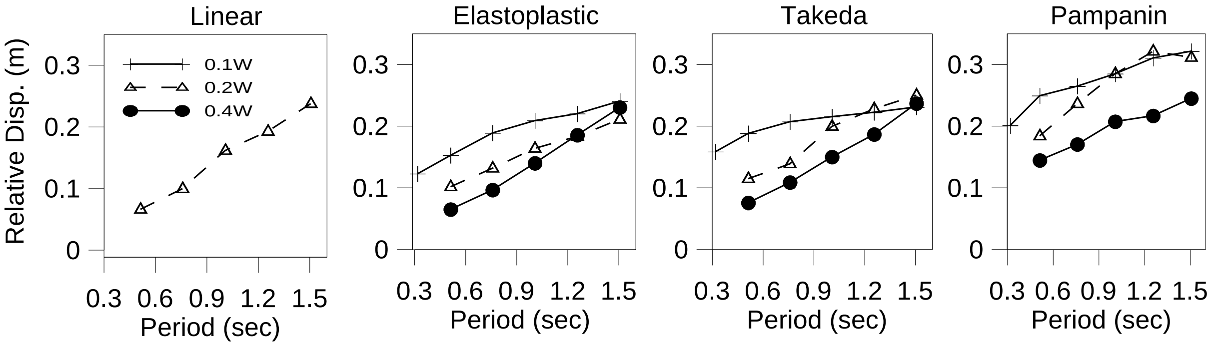


Figure 7: Relative displacement spectra of flexible foundation, BSE-1 level earthquake, G = G_{max}.

2.2.4 *Influence of selection of strong ground motion*

In the previous sections, the results were the average values of different analyses by 10 earthquake inputs. Although each ground motion record is scaled similarly, it may have different properties. Therefore, discrepancies amongst the results for each of the records are expected.

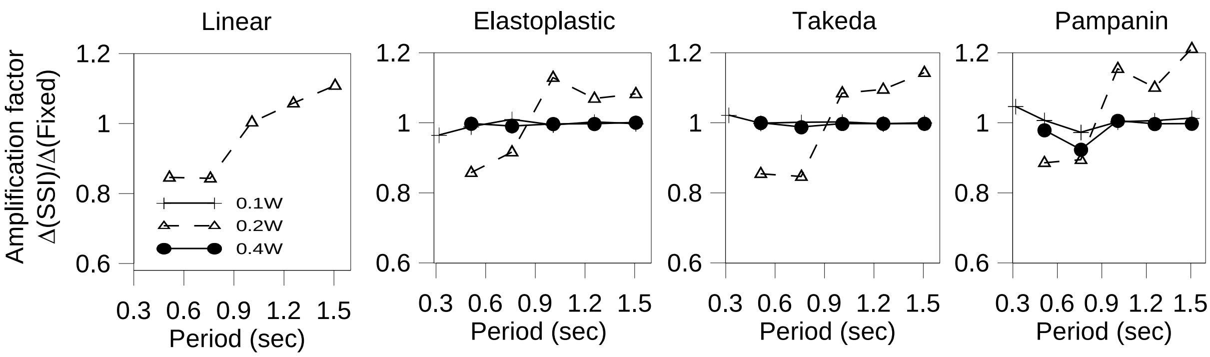


Figure 8: Amplification factor of flexible-base structure response to fixed-base, BSE-1, $G = G_{max}$.

Two sets of results obtained by two particular accelerograms are presented in following, namely, Loma Prieta and Northridge. The following graphs (Figs. 9 and 10) show the ratio of the relative flexible-base displacement to the displacement of a fixed-base foundation. It is apparent that the spectral displacements for flexible base are slightly different from the one for fixed base if the initial stiffness is used. This shows that for most of the cases, the SSI effect is negligible when soil is in its initial stage $G = G_{max}$. Note that $V_s = 150$ m/s is adopted for the soil.

The discrepancy amongst the displacement spectra is more evident for SDOF system with a flexible and degraded soil. For this particular level of shear modulus of the soil, considering SSI may have detrimental or advantageous influences as depicted in Figures 9-10. This was concluded by Gazetas and Mylonakis (2001) that some specific structures subjected to particular accelerograms may show detrimental effects due to SSI.

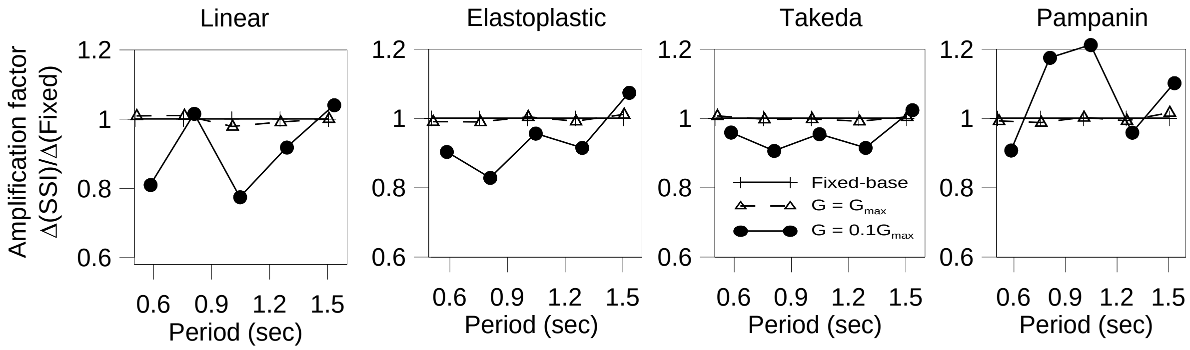


Figure 9: Amplification factor of flexible-base relative to fixed base foundation, Loma Prieta (1998) – Hollister St., BSE-1, $G = G_{max}$.

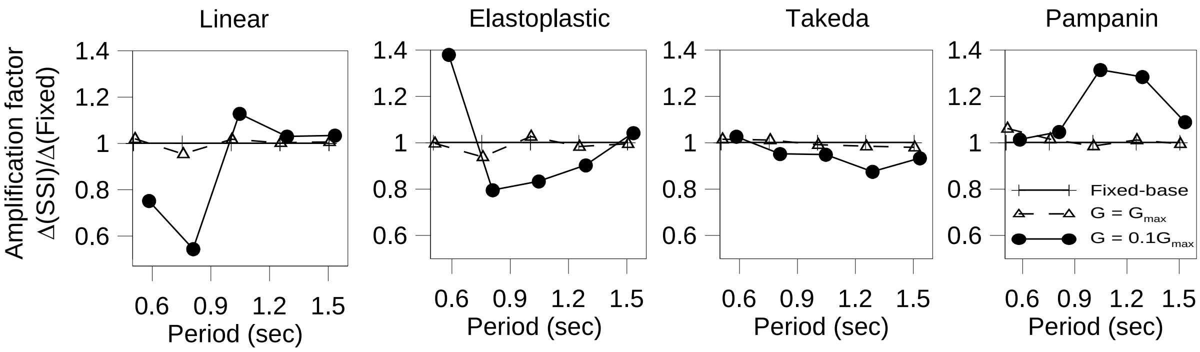


Figure 10: Amplification factor of flexible-base relative to fixed base foundation, Northridge (1998) – Hollywood store station, BSE-1, $G = G_{max}$.

3 CONCLUSIONS

Parametric analyses to study soil-structure interaction effects on the response of SDOF systems in the presence of other influential factors were carried out. Different nonlinear hysteretic rules were considered for SDOF structure.

Although some useful trends were observed, it is worth noting that the characteristics of the earthquake input motion play a major role on the output results, in particular when dealing with stiffness-degrading and pinched hysteresis loop, more representative of older and substandard designed structures. The wide scatter results among the different strong motions clearly indicates this.

In most of the cases, just introducing any level of flexibility of foundation soil is sufficient to trigger SSI effects; variation of the level of flexibility, in particular towards a reduction of the soil-foundation stiffness due to higher shear strain, may not change the results significantly.

The intensity of ground motion revealed an important parameter in terms of SSI effects. A system with a particular capacity subjected to a high or low demand may not be affected by SSI consequences.

Likewise, for a particular ground motion intensity level, low capacity and high capacity systems do not show the effects of SSI on the response. It can be concluded to take advantage of beneficial effects of SSI, stiffness-only upgrading strategies are more favourable.

It is important to notice the diverse SDOF response with different hysteresis. Pinching behaviour introduces very peculiar response of SDOF system. Hence assuming SSI in the course of adopting an upgrading selective scheme either by stiffness only or strength only results in more robust solutions.

4 ACKNOWLEDGEMENT

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