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THE EFFECT OF SOIL STRUCTURE INTERACTION ON COLLAPSE CAPACITY

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ABSTRACT

In this paper the effect of soil structure interaction (SSI) on structural collapse is investigated through parametric studies for representative key parameters. For this study, a simplified single degree of freedom system with stiffness and strength degradation hysteretic behavior has been used to model the structure. A discrete cone model has been used to represent the homogenous elastic half space of soil medium. The soil structure system was analyzed using a direct step-by-step integration method in time domain. The results of the analysis show that SSI tends to reduce the collapse capacity of the system and it also reduces the amount of cyclic deterioration and that collapse occurs due to non-cycle strength deterioration. Moreover, the study shows that significant insight on the collapse capacity of soil-structure systems can be gained using the equivalent system incorporated in this study.

Introduction

In the context of performance base earthquake engineering, collapse assessment has been attracting the attention of many researchers. Many collapse assessment studies have been conducted by including the P-δ effects in the seismic response of structures and incorporating deteriorating hysteretic model into the analysis (Ibarra 2003). Most of those studies have been concentrated on fixed base structures while it has been acknowledged that soil structure interaction influences the performance of the structure through the added foundation flexibility and added damping. As such, there is a clear need to better understand how SSI can be incorporated in collapse assessment.

For a given soil structure system, the collapse state may be defined as the inability of the structure to sustain the gravity load in the presence of the seismic load. Collapse occurs when the response of the system is on the post capping branch and the system can not withstand any further lateral displacement. The relative intensity measure (R) as introduced by Ibarra and Krawinker (2005) is used here as a means to calculate the collapse capacity of the soil structure system. This parameter is defined as the ratio of the ground motion intensity to a structure's strength parameter. In this study, the ground motion intensity is the spectral acceleration at the fundamental period of structure normalized by acceleration of gravity (S_a/g), whereas the structure strength parameter is the base shear strength of the system normalized by its weight (F_y/W). In order to obtain the collapse capacity, the intensity measure increases until the first dynamic instability observed in the program.

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Soil-Structure Model

The method employed to deal with soil structure interaction is the sub structure method. The soil beneath the structure is considered as a homogenous half-space and is modeled as a discrete element type based on the concept of Cone Models (Wolf 1994). Cone models, based on one dimensional wave propagation theory, can be used for modeling the soil with sufficient accuracy in engineering practices (Meek and Wolf 1993). Two degrees of freedom (DOF) for sway and rocking motions of foundation are considered in the model. Also, an additional DOF, θ_1, is introduced into the model in order to take into account the frequency dependency of dynamic stiffness of soil. The coefficient of springs and dampers for the sway and rocking DOFs are obtained through the following formulas:

$$k_h = \frac{8\rho V_s^2 r}{2-\upsilon} \qquad c_h = \rho V_s A_f \tag{1}$$

$$k_\varphi = \frac{8\rho V_s^2 r^3}{3(1-\upsilon)} \qquad c_\phi = \rho V_p I_f \qquad M_\phi = \frac{9\pi\rho r^5 (1-\upsilon)^2}{64(1-2\upsilon)} \tag{2}$$

Where $\rho, \upsilon, V_p \text{ and } V_s$ are the specific mass, Poisson's ratio and the dilatational and shear wave velocities of soil, respectively. Also, r is the radius of the equivalent circular foundation. For a compressible soil with Poisson's ratio between 1/3 and ½, two special features should be modified for the rocking motion: 1) the dilatational wave velocity should be selected as twice the shear wave velocity and 2) a tapped mass moment of inertia, ΔM_ϕ, as defined below is added to I_f. The Soil structure model is depicted in Fig. 1.

$$\Delta M_\phi = 0.3\pi(\nu - 1/3)\rho r^5 \tag{3}$$

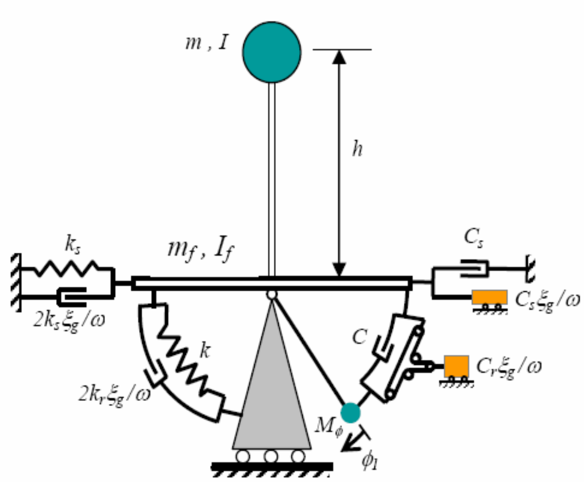


Figure 1. Soil Structure Model

The structure is considered as an SDOF with the same period and damping ratio as a fixed base structure in its first mode of vibration. The deteriorating model implemented in this study is a peak oriented hysteretic model with strength deterioration in the backbone curve (post capping stiffness branch) and cyclic deterioration of strength and stiffness. This model was introduced by Clough and Johnson (1966) and then modified by Mahin and Bertero (1976). More recently, Ibarra and Krawinkler (2003) adjusted model to include cyclic deterioration. Based on their suggestion, four cyclic deterioration modes may be activated once the yielding point is past in at least one direction. These are: basic strength, post-capping strength, unloading stiffness, and reloading stiffness deterioration. The cyclic deterioration rates are controlled by a rule which is based on the hysteretic energy dissipated when the system is subjected to cyclic loading. The cyclic deterioration in excursion "i" is defined by the parameter β_i defining the rate of deterioration in excursion i, which is given by the following expression:

$$\beta_i = \left(\frac{E_i}{E_t - \sum_{j=1}^{i} E_j} \right)^C \qquad (4)$$

Where $E_i, E_t = \gamma F_y \delta_y, \sum E_i$, and C are hysteretic energy dissipated in excursion i, hysteretic energy dissipation capacity, hysteretic Energy dissipated in all pervious excursions, and component defining the rate of deterioration parameters respectively. By using deterioration rule, the yield strength, strain hardening, post capping strength, and unloading stiffness deteriorate in accordance to the following formula as illustrated in Fig. 2.

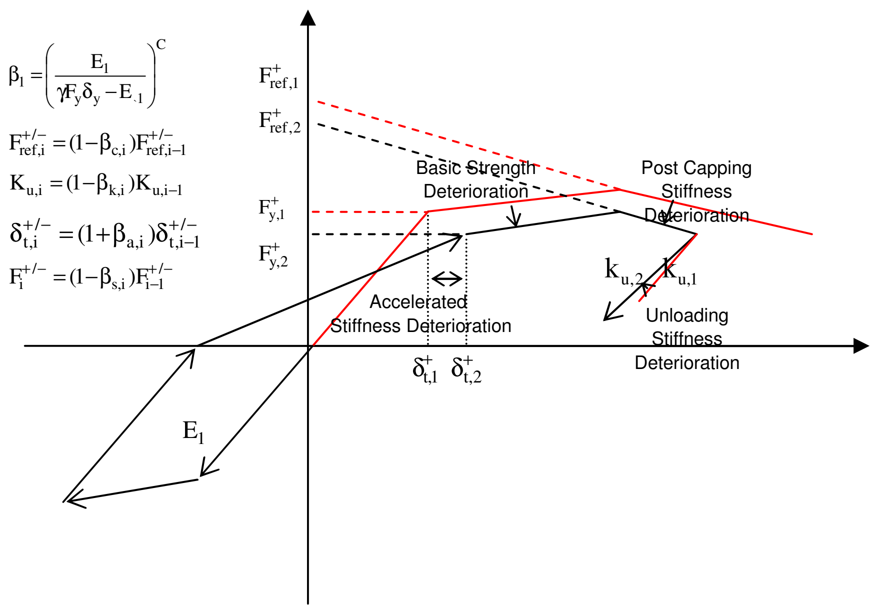


Figure 2. Peak Oriented Hysteretic Model with different mode of cyclic deterioration

Response of fixed base system to limiting forms of Ground Excitations

With a view of establishing certain guidelines for the interpretation of the results to be presented later, the collapse capacity of the fixed base structure and soil structure system to the limiting forms of ground excitation have been considered. Previously, simple expressions have been derived by Newmark (1973) and Miranda (1993) for fixed base systems with elasto-plastic behavior. In this study, the structure has a backbone with basic strength deterioration and the system with fixed base and flexible base support has been evaluated for these subjected pulses: an instantaneous displacement change, an instantaneous velocity change, and an instantaneous acceleration change.

Instantaneous displacement change

For a fixed base system subjected to an instantaneous displacement change, the extreme value of deformation is equal to this instantaneous displacement change, irrespective of whether the system behaves elastically or behaves in the plastic range, since there is no additional energy imparted to the system after the displacement has taken place. Furthermore, the strength and stiffness deterioration do not have any effect on the collapse capacity calculation. Accounting for this collapse capacity can be calculated using Eqs. 5 and 6.

$$R_c = \frac{U_e}{U_y} = \frac{U_m}{U_y} = \mu_u \tag{5}$$

Where μ_u is the ductility at collapse and is equal to

$$\mu_u = \frac{(1-\alpha_s) + \mu_c(\alpha_s + \alpha_c)}{\alpha_c} \tag{6}$$

Instantaneous velocity change

For a fixed base system subjected to an instantaneous velocity change, the energy imparted to system is equal to energy absorbed by the system up to the point of the maximum deformation, irrespective of whether the system remains elastic or behaves inelastically. By equating the imparted energy with the absorbed energy, the collapse capacity can be written as Eq. 7.

$$R_c^{\,2} = (\mu_c + \mu_u - 1) + \alpha_s(\mu_c - 1)(\mu_u - 1) \tag{7}$$

Instantaneous acceleration change

For a fixed base system subjected to an instantaneous acceleration change, the work done by the external force is equal to restoring energy up to the maximum deformation. The maximum deformation is a point that applied equivalent force crosses the post capping portion of backbone curve. This point meets both the energy and the dynamic equilibrium. The location of this point can be determined by Eq. 8 and the corresponding capacity spectrum can be determined by Eq. 9.

$$\mu_u = \sqrt{\mu_c^2\left(\frac{\alpha_s}{\alpha_c} + 1\right) + \frac{1-\alpha_s}{\alpha_c}} \tag{8}$$

$$R_c = \frac{(2\mu_u - 1) + \alpha_s(\mu_c - 1)(2\mu_u - \mu_c - 1) - \alpha_c(\mu_u - \mu_c)^2}{\mu_u} \tag{9}$$

A simplified model based on the idea of replacing the soil structure system with an equivalent fixed base SDOF model was proposed by Veletsos and Meek (1974) for an elastic structure and more recently by Ghannad and Ahmadnia (2006) for elasto-plastic system. In those methods, the effect of foundation mass is ignored and also constant coefficient for springs and dashpots are used in developing equivalent models. It has been show the equivalent system works well in the linear and nonlinear ranges and it provides a valuable insight into understanding of the effect of soil structure interaction. Fig. 3. shows schematically the behavior of the equivalent system and the corresponding fixed base structure.

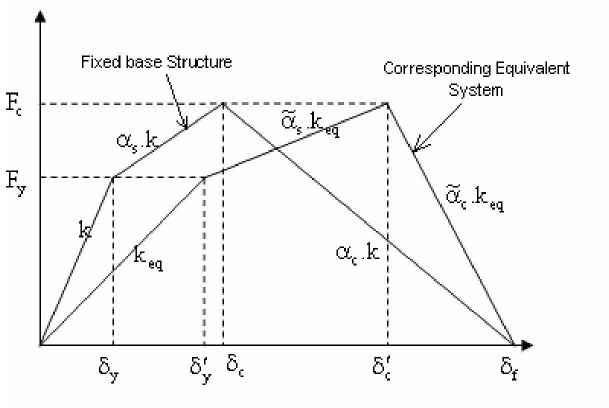


Figure 3. Backbone curve for hysteretic model and corresponding equivalent system

By referring to Fig. 3 and assuming conditions proposed for the equivalent system (Ghannad and Ahmadnia 2006) the equivalent period, ductility, post capping stiffness and stiffness hardening can be written as following expression.

$$\tilde{T} = T.\sqrt{1+\frac{k}{k_s}+\frac{k.H^2}{k_r}} \qquad \tilde{\mu}_c = (\mu_c - 1).(\frac{T}{\tilde{T}})^2 + \alpha_s.(\mu_c - 1).(1-(\frac{T}{\tilde{T}})^2) + 1 \tag{10}$$

$$\tilde{\alpha}_s = \alpha_s.\frac{\mu_c - 1}{\tilde{\mu}_c - 1} \qquad \tilde{\alpha}_c = \frac{(\frac{\tilde{T}}{T})^2}{1+(1-(\frac{\tilde{T}}{T})^2).\alpha_c}\alpha_c \tag{11}$$

As can be seen from the above expression, firstly, it can be observed that the effective period of soil structure system is larger than that of fixed base structure. Secondly, the equivalent ductility of the soil structure system is less than the ductility of the fixed base system and for systems without strain hardening it is less than that for the system with strain hardening. In other words, the SSI system usually requires less reduction factor to achieve certain ductility when compared to a corresponding fixed base system. This means that:

$$1 \le \tilde{\mu}_c(\alpha = 0) \le \tilde{\mu}_c(\alpha \ne 0) \le \mu_c \tag{12}$$

Thirdly, since the equivalent ductility is less than the structure ductility ($\tilde{\mu}_c \le \mu_c$), it can be said that the strain hardening of the equivalent system is greater than that of the structure. Finally, the negative post capping stiffness of the equivalent system is greater than that of the structure alone. That means the equivalent system is more susceptible to instability than the structure system after attaining post cap strength. Moreover, according to Eq. 11, for values of $\tilde{T}/T \ge \sqrt{1+1/\alpha_c}$, equivalent post capping stiffness ($\tilde{\alpha}_c$) can theoretically become infinity and negative. However, due to the existence of the damping and foundation mass this snap back behavior type doesn't create instability in the system.

For a soil structure system subjected to limiting form of ground excitation, the same expression as fixed base structure can be derived provided the modified value for strain hardening, post capping stiffness, and ductility are employed in the corresponding expressions (Eqs 5 to 9).

It would be expected that the first relationship involving conservation of the maximum deformation would also be applicable to systems subjected to ground displacements for which the rise time is small in comparison to the natural period of the system. The second relationship, involving conservation of energy, would be expected to apply also to quarter-cycle velocity pulses (half cycle acceleration pulse of short duration) for which the rise time is small compared to natural period of the system. Finally, the third relationship is also valid for acceleration pulses with a sharp rise and long duration in compared to natural period of the structure. In order to assess the validity of the above discussion to ground motion, five ground motions have been selected. Those five ground motions were recorded on soft soil and correspond to soil type E for Vancouver according to NBCC 2005. Using the deaggregation technique discussed by Halchuk and Adams (2004), the records were selected to meet the following criteria: a) the site to source distance ranges from 40 to 70 Km, and b) the magnitude ranges from 6.5 to 7.2. Accordingly, two records from the California 1989 Loma Prieta, one from the Japan 1995 Kobe, one from the California 1979 Imperia Valley Brawly and one from the California 1949 Elcentro earthquakes have been selected throughout this study. Fig. 4. illustrates the mean collapse capacity of the system subjected to the selected records, along with the result of the analytical solution discussed above. The results in Fig 4.a. are for systems with large Vs/r. As observed, the analytical expressions for instantaneous constant displacement pulse and instantaneous constant acceleration pulse work acceptably for both the constant displacement and constant acceleration region. However, the conservation of energy concept applies only to a small portion of the spectrum.

Fig .4.b shows the same comparison but for small values of Vs/r. Because of the elongation of the equivalent period, the instantaneous constant displacement pulse matches over broader period range to the mean capacity spectrum. These simple analytical expressions might be very useful in estimating the collapse capacity of a structure.

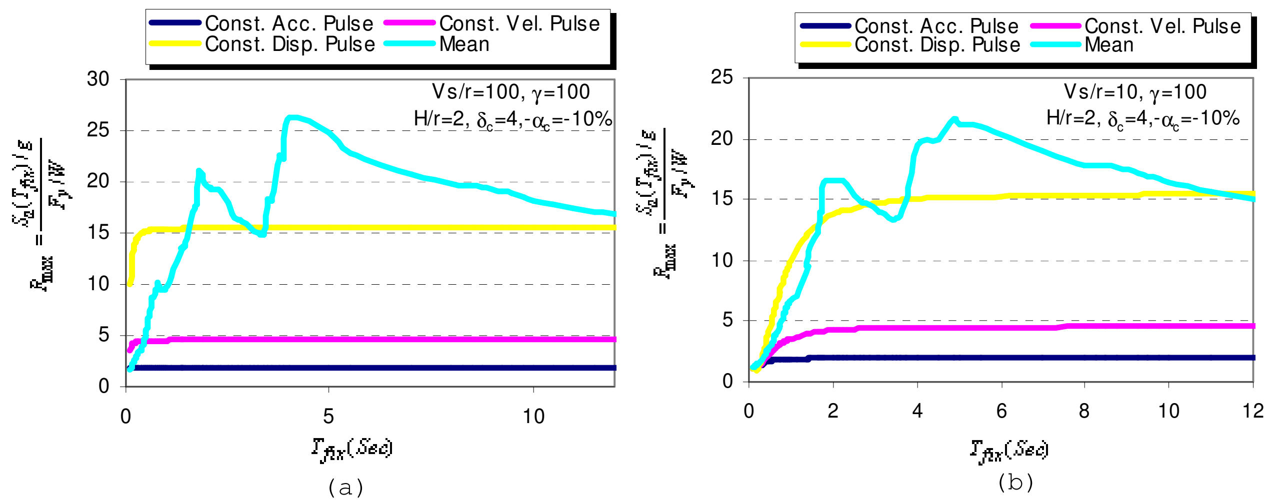


Figure 4. Comparison of mean collapse capacity response spectra with results from analytical expression

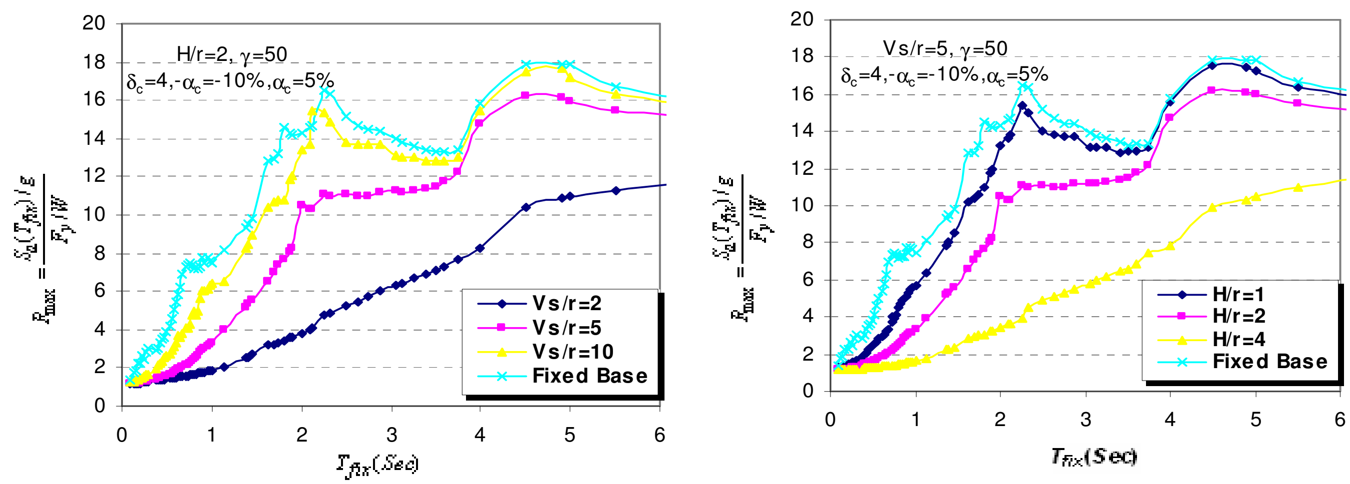
Parametric studies

Previous SSI studies by Ghannad and Ahmadnia (2003) demonstrated that the ratios V_s/r and H/r have pronounced effects on the response of the soil structure system. In this study, these two ratios have also been employed as the basis for our investigation. The third parameter is the rate of the cyclic deterioration which in conjunction with those mentioned parameters leads to study the SSI effect on the system with strength and stiffness deterioration. The other parameters have been fixed at their conventional values. These are:

$$\alpha_s = 5\% \ , \ \alpha_c = 10\% \ , c = 1 \ , \xi_s = 5\% \ , \ \delta_c = 4 \qquad (13)$$

To carry out a parametric study on the effects of SSI on collapse capacity, a large number of time histories analyses have been performed on the soil structure system with Vs/r=2,5,10, H/r=1,2,4, and γ=25,50,100. The effect of soil structure interaction on collapse capacity is shown in Fig. 5. for a system with cyclic deterioration γ=50. Fig. 5.a illustrates the mean collapse capacity spectrum for a soil structure system for

H/r=2, Vs/r=2, 5, and 10 with that for fixed base structure. Fig. 5.b displays the mean collapse capacity spectrum for a soil structure system with Vs/r=5, H/r=1, 2, and 4 with that for fixed based structure. As observed, for the system with low values of Vs/r and large values of H/r the mean collapse capacity reduces more in compared to the system with large value of Vs/r and low value of H/r. This trend can be explained by referring to equivalent system when Vs/r increases the equivalent period and post capping stiffness increase. Therefore increase in post capping stiffness leads to decrease in collapse capacity of the system. However, the amount of the reduction in collapse capacity is not the same for structures with different period. The stiffer structures (T<0.3) are influenced less than the flexible ones.



(a) Various Vs/r, H/r=2

(b) Various H/r,Vs/r=5

Figure 5. Effect of SSI on mean collapse capacity

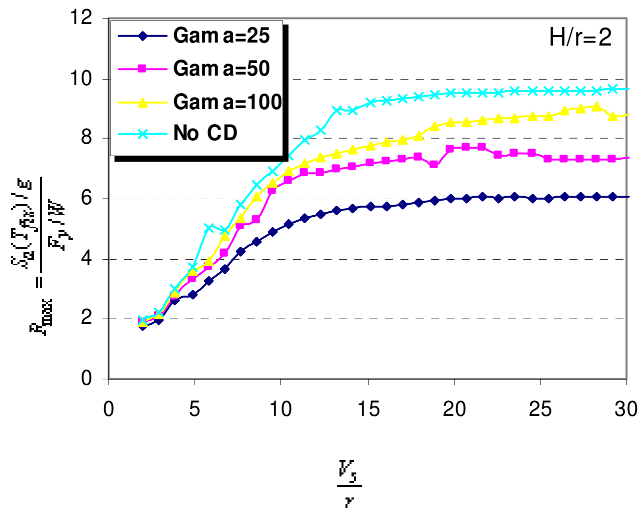
The effect of soil structure interaction on the cyclic deterioration is shown in Figure 6 for a system with a fixed period of T_{fix}=0.5 and one with T_{fix}=1. The mean collapse capacity was determined as a function of Vs/r or H/r for different levels of deterioration. Fig. 6.a. and Fig. 6.b. illustrate the collapse capacity versus Vs/r and Fig 2.6.c and Fig. 2.6.d display collapse capacity versus H/r for γ=25, 50, 100 along with collapse capacity for a system without cyclic deterioration. As observed, the SSI influences significantly the amount of deterioration. For a system with large values of H/r, namely H/r>3, and smaller value of Vs/r, namely Vs/r<6, the collapse capacity is almost the same for different levels of deterioration. The effect of Vs/r variation on the amount of cyclic deterioration is more pronounced than the effect of H/r variation. Based on those analyses, it can be concluded that the SSI reduces the amount of cyclic deterioration. Moreover, for the system when the SSI is important collapse mainly occurs due to non cyclic strength deterioration.

Conclusions

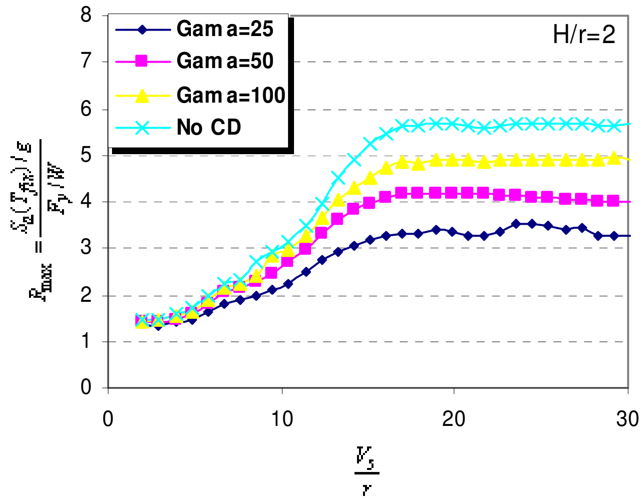
The aim of this study was to evaluate SSI effects on the collapse capacity of simple systems modeled with stiffness and strength deterioration hysteretic model. To accomplish this, an in-house nonlinear code has been developed to model both the structure and soil. Based on numerous parametric studies on the SSI model considered the following conclusions have been obtained:

a) SSI tends to decrease the collapse capacity of system. These decreases are more evident in system where the SSI is important.

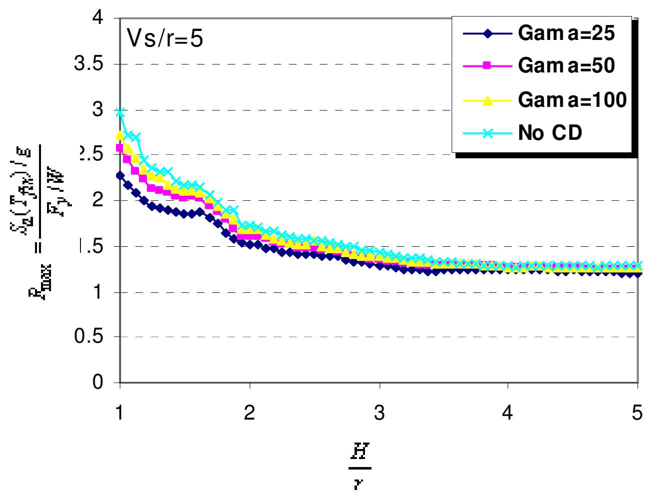
b) The most important parameters in assessment of SSI on collapse capacity are fixed period of structure in conjunction with V_S/r, and H/r ratio.



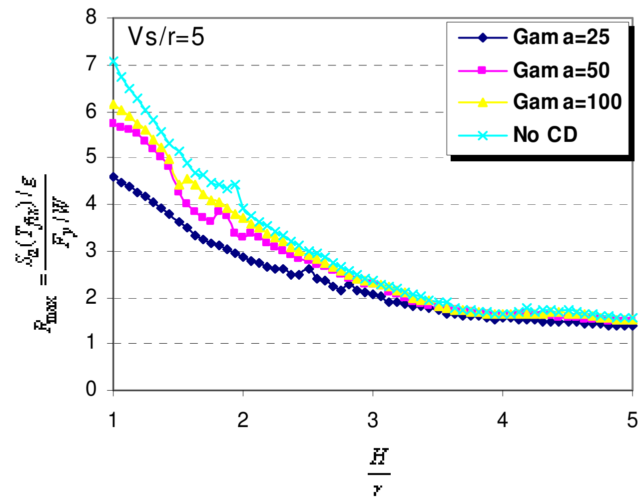
(a) Various CD and Vs/r, T_{fix}=1.0



(b) Various CD and Vs/r, T_{fix}=0.5



(c) Various CD and H/r, T_{fix}=1.0



(d) Various CD and H/r, T_{fix}=0.5

Figure 6. Effect of SSI on mean collapse capacity

c) For a condition which Vs/r is low and the structure is stiff the collapse capacity drops as much as three times. On the other hand, SSI reduces the effect of cyclic deterioration on collapse capacity and the collapse occurs due to basic strength deterioration or in other words due to negative post capping stiffness.

d) SSI tends to accelerate the negative post capping stiffness slope.

The proposed equivalent formula predicts reasonably the collapse capacity at both ends of the spectra. The concept of equivalent system shows that the equal displacement rule does not seem to work for SSI systems and it might be lead to unconservative results. Further studies need to be conducted to assess the validity of the equal displacement rule as a design basis when SSI is significant.

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