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# Performance of soil-structure systems at collapse level: ASCE 7-10 to ASCE 7-22 criteria

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**ABSTRACT:** This paper examines the impact of soil-structure interaction on the collapse potential of structures and evaluates changes in the collapse margin ratio (CMR) when the design base shear is reduced according to ASCE 7 guidelines. The study considers a single degree of freedom (SDOF) structure with modified Ibarra Krawinkler deterioration model and characteristics of special moment resisting frames, considering both horizontal and rocking degrees of freedom in the soil. Incremental dynamic analyses are conducted to calculate CMR for three systems: fixed-base, flexible-base, and flexible-base with reduced design base shear based on ASCE 7-10, 7-16, and 7-22 criteria. The results indicate that soil-structure interaction enhances the performance of soil-structure systems at the collapse level, resulting in higher CMR values compared to fixed-base structures. The design base shear reduction recommended by ASCE 7-10 leads to non-conservative performance and lower CMR values, while ASCE 7-16 and 7-22 criteria, which consider the response modification factor, yield higher CMR values.

**Keywords:** Soil-structure interaction; Collapse margin ratio; Incremental dynamic analysis; ASCE 7.

## 1 INTRODUCTION

According to the properties of structures and seismic excitations, soil-structure interaction (SSI) can increase, decrease or make no change in the response of structures. The effect of SSI was first introduced into design codes by ATC 3-06 (1978) in which the soil-structure system was replaced by a fixed-base structure with an elongated period and a higher damping ratio. ASCE 7-10 (2010) employed the same strategy for the effect of SSI based on which, SSI causes reduction in the design base shear of buildings. These provisions are based on the linear behaviour of structures. However, under strong ground motions, especially at the collapse level, structures can experience strong nonlinear behaviour. The effect of SSI on nonlinear systems differs from linear systems, because yielding of structural elements reduces the structure-to-soil relative stiffness, increases the overall damping of the soil-structure system, elongating the vibration period, and producing significant resonance deformation modes of the whole system and changes the effectiveness of SSI. Hence, it is more likely that these regulations will change the performance of soil-structure systems such as CMR in comparison with fixed base structures.

Ghannad and Jahankhah (2007) showed that strength reduction factors of soil-structure systems are less than fixed-base structures, so flexible-base structures designed with the same strength reduction factor as fixed-base structures experience ductility demands higher than designed values, which is non-conservative. In addition, Ghannad and Ahmadnia (2006) concluded that although the SSI effect reduces the ductility demand on structures with long periods compared to fixed-base structures, ignoring the SSI effect on structures with short periods may cause non-consertive design. Khoshnoudian et al. (2015) concluded that soil-structure systems are potentially more susceptible to dynamic instability than fixed-based structures.

FEMA p695 (2009) provides a methodology to quantify the performance of buildings under seismic loading at collapse level, and provides the allowable collapse margin ratio (CMR) for load-resisting systems designed with a strength reduction factor. This paper evaluates the performance of soil-structure systems at collapse level by comparing the CMR values of soil-structure systems with those of fixed-based structures. Furthermore, it investigates whether and to what extent practicing ASCE 7-16 and 7-22 seismic design criteria for soil-structure systems changes the CMR values in comparison with those based on ASCE 7-10.

## 2 STRUCTURAL MODEL

In this study, the modified Ibarra Krawinkler (MIK) deterioration model (Ibarra and Krawinkler, 2005) is employed as a monotonic backbone curve to simulate stiffness, strength and inelastic deformation of structural elements under reversed cyclic loading. MIK model has been widely used in the literature for collapse assessment of structural components under seismic loading (Zareian and Krawinkler, 2007, Lignos and Krawinkler, 2011, Pourreza et al., 2021). As shown in

Figure 1, MIK model indicates that the initial elastic response is followed by hardening after yielding, and the subsequent post-peak strength and stiffness degradation leads to the residual strength. When structural deformation passes the post-capping deformation point, the structure loses all its strength and stiffness, resulting in the collapse of the system. The main parameters in this curve include effective yield strength and deformation ($F_y$ and $\delta_y$), effective elastic stiffness $K_e$, strength cap and associated deformation ($F_c$ and $\delta_c$), plastic deformation capacity $\delta_p$, post-capping deformation capacity $\delta_{pc}$, residual strength $F_r$, and ultimate deformation $\delta_u$. More detailed formulation of MIK backbone curve can be found in Ibarra and Krawinkler (2005).

Moment resisting frames are one of the well-known lateral load resisting systems introduced in ASCE 7 (2022) that has been categorized into ordinary, intermediate, and special moment resisting frames based on their expected ductility. Special moment resisting frames with strength reduction factor (R) equal to 8 has been chosen as a case study in this paper. The behavior of these frames with ductile beam to column connections have been extensively studied by experiments. FEMA p440a (2009) provided the parameters for the MIK model of special steel moment resisting frames, as shown in Table 1, to represent the cyclic behavior of these systems by simulating simplified single-degree-of-freedom oscillators in OpenSees and capturing the similar response as observed in the experiments as well as in the real earthquakes, including the Northridge earthquake. Although this idealized backbone curve does not represent the detailed mechanical behavior of the structural components, it will be used as the main characteristic to investigate the impact of SSI at the collapse level for a single-degree-of-freedom system.

As shown in Figure 2, the Bilinear Model (Ibarra and Krawinkler, 2005) with basic hysteretic rules and kinematic strain hardening is used to model the cyclic behavior of SDOF systems. During unloading in each loop, the slope of segment would be the same as elastic stiffness. Energy dissipation in each loading cycle reduces the stiffness of the system, so cyclic deterioration lowers force-deformation capacity in comparison with monotonic backbone which affects the collapse response of the system under cyclic loading. In this study, cyclic deterioration is neglected and the cyclic backbone curve is assumed to be the same as the monotonic backbone curve.

## 3 SOIL-STRUCTURE MODEL

The soil-structure system is modeled using the sub-structure method in which, the soil is modelled as a discontinuous, uniaxial deformable elements, and is connected to the structure at defined interface points (Wolf, 1985). As shown in Figure 3, a SDOF structure with mass $m$, mass moment of inertia $I$ and damping $c$ is located at height $h$ from a circular rigid foundation with mass $m_f$ and mass moment of inertia $I_f$. This SDOF structure is a simplified model of a multi-degree-of-freedom structure with the effective values of its first mode of vibration in order to use the current design regulations. The soil beneath the foundation is considered as a homogenous half-space. Sway and rocking degrees of freedom are assumed for horizontal and rotational movements of the foundation, each of them is modeled by a spring and a dashpot with stiffness and damping coefficients $k_h$, $c_h$ and $k_\phi$, $c_\phi$, respectively. Moreover, an internal DOF is considered by adding a mass polar moment of inertia $M_\phi$ to the rotational dashpot to take the frequency dependency of a soil-foundation system into account, which is the consequence of the soil mass participating in the foundation response. These 3DOF are defined based on the concept of Cone Models to model the soil-foundation system (Wolf, 2004). The mentioned sway and rocking springs and dashpots just consider the radiation damping of the soil. In order to include the material damping as well, a dashpot and a mass are added parallel to each spring and dashpot, respectively. The parameters shown in Figure 3 are defined as follows:

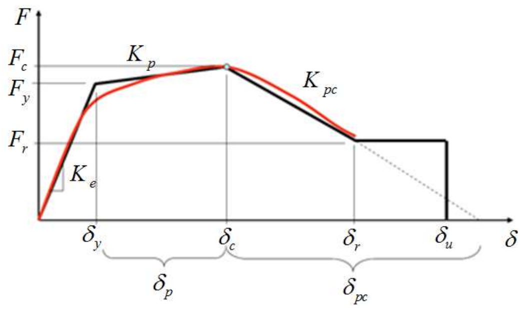


*Figure 1. Backbone curve of MIK model (Ibarra and Krawinkler, 2005)*

*Table 1: Parameters of MIK model for special steel moment resisting systems (FEMA p440a, 2009)*

| $\delta_p$ | $F_c$ | $\delta_{pc}$ | $\lambda = F_r/F_y$ | $\delta_u$ |
|---|---|---|---|---|
| $3\delta_y$ | $1.05F_y$ | $3.5\delta_y$ | 0.45 | $8\delta_y$ |

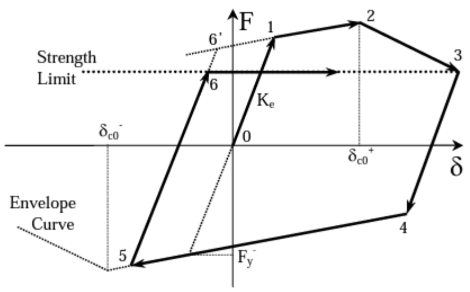


*Figure 2. Bilinear hysteresis behavior(Ibarra and Krawinkler, 2005)*

$$k_h = \frac{8\rho V_s^2 r}{2-\nu} \quad (1)$$
$$c_h = \rho V_s A_f \quad (2)$$
$$k_\varphi = \frac{8\rho V_s^2 r^3}{3(1-\nu)} \quad (3)$$
$$c_\varphi = \rho V I_f \quad (4)$$
$$M_\varphi = \frac{9\rho\pi r I_f(1-\nu)}{32}\left(\frac{V}{V_s}\right)^2 \quad (5)$$
$$\xi_0 = \xi_g \frac{T_{SSI}}{T_n}\frac{h}{r}\frac{1}{a_0} \quad (6)$$
$$\omega_0 = \frac{V_s}{r} \quad (7)$$

where $\rho$, $\nu$, $V_s$ and $V$ are specific mass of soil, Poisson's ratio, shear wave velocity and longitudinal wave velocity of the soil, respectively. In addition, $r$ is the radius of equivalent circular foundation, $A_f$ is the area of foundation, and $T_{SSI}$ is the natural period of soil-structure system. $\xi_0$ and $\omega_0$ are soil frequency-dependant damping ratio and the natural frequency when dimensionless frequency is one.

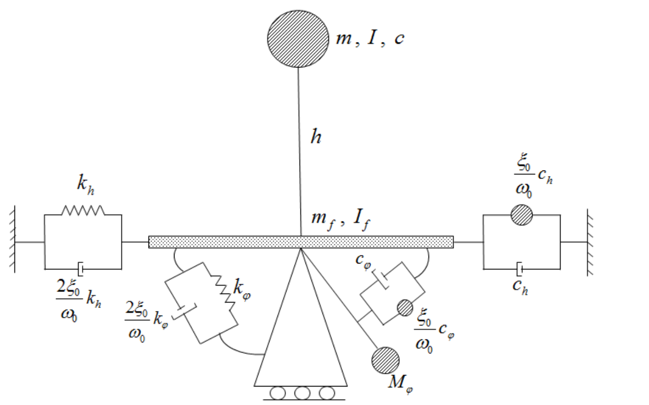


*Figure 3. Soil-structure model*

All soil parameters discribed above can be rewritten in terms of some dimensionless parameters which describe main aspects of the soil-structure systems, as summerized in Table 2. These non-dimensional parameters are widely being used in the literature (Ghannad and Jahankhah, 2007, Ghannad and Ahmadnia, 2006) for considering the SSI. SSI effects can mainly be considered by dimensionless frequency, which indicates the severity of the SSI effects ($a_0 = 0$ for fixed-base structures and $a_0 = 2$ for soil-structure systems with a significant SSI effect) and aspect ratio, which indicates the slenderness ratio of the structure.

The parameters in Table 2 are adequate for constructing the necessary mass, stiffness, and damping matrices to solve the nonlinear equation of motion. These matrices only require the mass ratios between the structure and soil, as well as the foundation and structure, rather than the individual masses of the structure, soil, and foundation. The simplified 4DOF model is analyzed in time domain using Newmark β step-by-step method (Chopra, 2007) and subjected to the uni-directional loading.

*Table 2: Non-dimensional parameters and their values*

| **Description** | **Parameter** | **Values** |
|---|---|---|
| Dimensionless frequency | $a_0 = \omega_n{}^* h/V_s$ | 0, 0.5, 1, 2 |
| Aspect ratio | $h/r$ | 0.5, 1, 2 |
| Natural period of fixed-base structure | $T_n$ | 0.5 to 2s (0.1s inter.) |
| Structure to soil mass ratio | $\bar{m} = m/\rho r^2 h$ | 0.62 |
| Foundation to structure mass ratio | $m_f/m$ | 0.25 |
| Soil Poisson's ratio | $\nu$ | 0.3 |
| Damping ratio of the structure | $\xi_s$ | 0.05 |
| Soil damping ratio | $\xi_g$ | 0.08 |

$^*\omega_n$ is the natural circular frequency of the fixed-base structure

## 4 FAR-FIELD RECORD SET

FEMA p695 (2009) provides twenty-two far-field record sets (44 individual components) for incremental dynamic analysis. These ground motions are all strong motions with PGA>0.2g and large magnitude (M>6.5) events selected from the Pacific Earthquake Engineering Research Center (PEER) database. Strong motions are more important in the collapse assessment of nonlinear deteriorating models because they typically have a longer duration of shaking and can dominate the collapse risk. These records are from soft rock and stiff soil sites (mostly site class C and D).

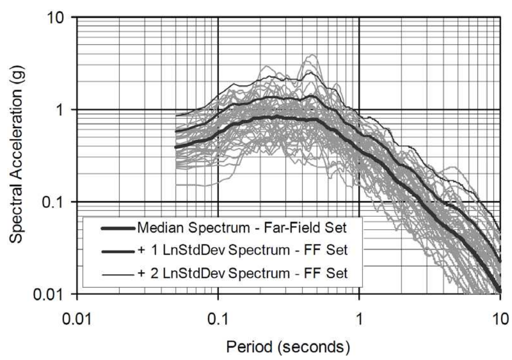


*Figure 4. Spectral acceleration responses of far-field set (FEMA p695 (2009))*

## 5 DESIGN METHODOLOGY

### *5.1 SSI regulations in ASCE7-10 to ASCE7-22*

The most common and traditional method of designing structures is neglecting soil flexibility and assuming structures on fixed bases. Design guidelines in ASCE 7 propose seismic design base shear "V" for fixed-base structures which is achieved based on an acceptable level of safety dedicated to this code. Due to the effect of SSI, ASCE 7 standard allows engineers to design soil-structure systems with reduced design base shear using a linear static procedure. The reduction in the design base shear depends on the damping ratio of the soil-structure system, the ratio of elongated period of SS system to the period of fixed-base structure, and expected ductility demand of the structure. The challenge is to determine whether and to what extent reducing the design base shear will affect the performance of structures, and whether it will provide the necessary margin of safety against collapse of fixed base structures.

Since the formulations for design base shear reduction of SS systems has been changed from ASCE 7-10 to ASCE 7-22, these regulations are evaluated and the results are compared with those of fixed-base structures. The significant improvement from the formulation of ASCE 7-10 to 7-16 is consideration of the effect of the response modification factor (R) on the maximum allowable reduction in design base shear. Based on ASCE 7-10, the maximum allowable reduction in design base shear is 0.3V independent of the seismic force resisting system. However, in both ASCE 7-16 and 7-22, the maximum allowable reduction in design base shear is less when R is larger. There is only a slight difference between the formulation in ASCE 7-16 and 7-22, which does not affect the conclusion. The special moment resisting frame (R=8) used for the structure model in this study has the same maximum allowable reduction in design base shear (0.1V) based on both ASCE 7-16 and 7-22, which is much less than the value provided in ASCE 7-10. Therefore, only the results based on ASCE 7-10 and 7-22 are provided and discussed in this paper.

The studied cases are classified into four systems:

1) System A is a SDOF structure with seismic design base shear, V, staying on a rigid base with no reduction of base shear.

2) System B is the same as system A with the same seismic design base shear, V, staying on a flexible base. This system is an example of a structure that is designed according to fixed-base design regulations and is constructed on soil in reality.

3) System C is a SDOF structure with reduced seismic design base shear based on ASCE 7-10, due to the effects of SSI and staying on a flexible base. This system is an example of a structure that is designed according to SSI provisions and is constructed on soil in reality.

4) System D is the same as system C, but the reduced seismic design base shear, is calculated based on ASCE 7-22 regulations.

For all four systems, the MIK model of Figure 1, with parameters of special steel moment resisting systems in Table 1, as well as the dimensionless parameters of Table 2 are considered.

## 5.2 Incremental dynamic analysis (IDA)

Collapse capacity of structures can be assessed through the concept of Incremental Dynamic Analysis (IDA). To achieve IDA curves, each record should be scaled to increasing intensities until the collapse of the structure happens. In this paper, IDA curves for collapse assessment are plotted in terms of spectral acceleration at fundamental period of structure ($Sa(T_n)$) as intensity measure (IM) versus maximum relative displacement of structure as damage measure (DM). At lower intensities, the behaviour of the structure is linear, and by increasing the ground motion intensity, the slope of the IDA curve decreases. By increasing the scale factor of each ground motion, there would be a point at which a small increase in the intensity causes a large difference in the displacement of the structure, i.e. the slope of the IDA curve would yield to zero. Flattening of the IDA curve indicates dynamic instability. The collapse intensity has been defined as the last point at which the curve flattens (Zareian and Krawinkler, 2007); however, FEMA 350 (2000) defines collapse point as the last point with a tangent slope equal to 20% of the elastic slope. Figure 5 shows the IDA curves for four systems (A to D).

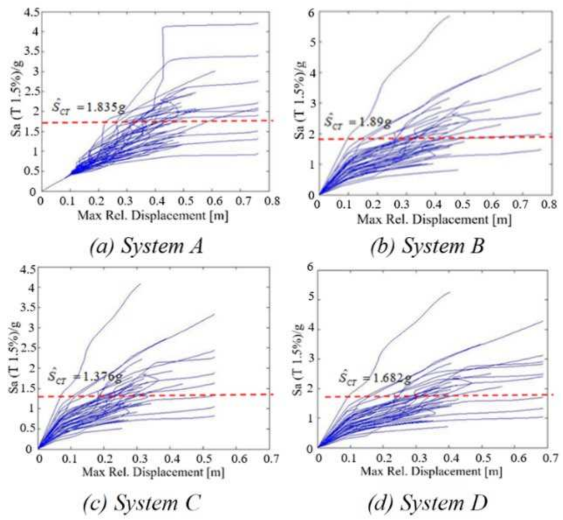


*Figure 5. IDA curves for four systems (in the SSI systems, $a_0$=2, h/r=2)*

## 5.3 Collapse Margin Ratio, CMR

The collapse margin ratio quantifies the safety of the structures against collapse, and is defined as the ratio of $\hat{S}_{CT}$ to $S_{MT}$ according to Equation 8 (FEMA p695, 2009):

$\hat{S}_{CT}$: the median 5%-damped spectral acceleration of the collapse level ground motions, which is derived from IDA curves, as shown in Figure 5.

$S_{MT}$: the 5%-damped spectral acceleration of the maximum considered earthquake (MCE) ground motions at the fundamental period of the seismic force resisting system. MCE ground motion represents the maximum level of ground shaking that is expected to occur at a specific location during the design life of a structure. A higher CMR value means there is a greater margin of safety against collapse.

$$\mathrm{CMR} = \frac{\hat{S}_{CT}}{S_{MT}} \tag{8}$$

$S_{MT}$ is calculated from the response spectrum of MCE ground motion at the fundamental period, $T_n$ of the structure and is defined based on Equation (9):

$$\begin{cases} S_{MT} = S_{MS} \text{ for } T_n \leq T_s \\ S_{MT} = \frac{S_{M1}}{T_n} \text{ for } T_n > T_s \end{cases} \quad (9)$$

According to ASCE 7 (2022), $S_{MS}$ and $S_{M1}$ are MCE spectral response acceleration parameters for short periods and 1 second respectively, adjusted for site class effects. Transition period $T_s$ defines the boundary between the region of constant acceleration and constant velocity of the MCE response spectrum and is defined as $T_s = S_{M1}/S_{MS}$.

# 6 RESULTS

## 6.1 *Effect of SSI on CMR*

Figure 6 compares the CMR values of the fixed-base structure (System A) with those of the flexible-base structure designed with the fixed-base structure base shear (System B), considering different $a_0$ and *h/r* values. The results show that at low $a_0$ values which means low soil-structure interaction, the CMRs of systems A and B are very close to each other. This is because the soil stiffness is still much higher than the structure stiffness and soil beneath the structure is acting as a fixed-base. Increasing $a_0$ values decreases the relative stiffness of soil to structure, which reduces the seismic demands on the structure, so the structure is expected to collapse at higher ground motion intensities, which results in larger CMR values. By increasing *h/r* in soil-structure systems, CMR values decrease which means that squat structures collapse at higher ground motion intensities and have larger CMR values because of higher soil radiation damping.

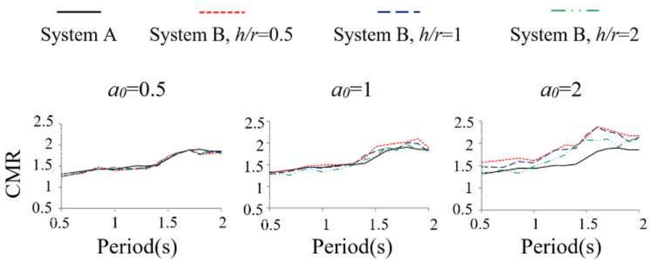


*Figure 6. Comparison of CMR versus period for systems A and B with different $a_0$ and h/r values*

## 6.2 *Effect of design base shear reduction based on ASCE 7-10 on CMR*

The ASCE 7-10 standard offers engineers two design methods, one without considering SSI effects, and one that utilizes its positive effects to reduce design base shear. The results of employing these two methods are investigated in this part by comparing the CMR curves of Systems A and C, as shown in Figure 7 for various $a_0$ and *h/r* values. Figure 7 shows that all systems with reduced design base shear (systems C) have lower CMR values than fixed-base structures (systems A). This indicates that the regulations of ASCE 7-10 for design base shear reduction are non-conservative at collapse levels. Moreover, increasing the *h/r* ratio increases the CMR, because the more radiation damping of soil in squat structures increases the effectiveness of SSI, which leads to more reduction in design base shear.

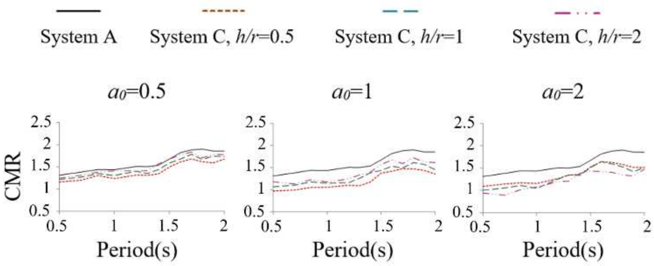


*Figure 7. Comparison of CMR versus period for systems A and C with different $a_0$ and h/r values*

Figure 8 illustrates the ratio of change in design base shear proposed by SSI provisions in ASCE 7-10 ΔV to the original base shear computed for a fixed-base structure V for different soil-structure parameters. It shows that systems with $a_0$=2 have the maximum allowable reduction in design base shear, i.e. 0.3V. It means that all systems with $a_0$=2 independent of the aspect ratio have the same design base shear. Moreover, larger reduction in the elastic strength demands of squat buildings causes smaller CMR values for these systems as shown in Figure 7.

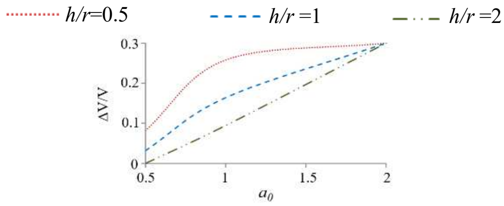


*Figure 8. Ratio of reduction in design base shear to the design base shear of fixed-based structure based on ASCE 7-10*

## 6.3 *Effect of design base reduction based on ASCE 7-22 on CMR*

ASCE 7-22 proposes provisions for soil-structure systems that vary significantly from those suggested by ASCE 7-10. The main differences include design base shear reduction, maximum allowable reduction in design base shear and soil related damping. To explore their effects, variation of CMR values versus different structural periods are illustrated in Figure 9. ASCE 7-22 restricts the maximum reduction of design base shear to 0.1V for systems with the response modification factor of 6 and larger. Figure 10 illustrates the ratio of ΔV/V based on ASCE 7-22 for different soil-structure parameters. It shows that all systems with $a_0$=1 and 2 get the maximum allowable reduction in design base shear

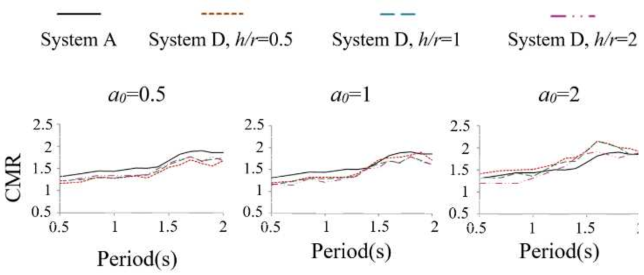


*Figure 9. Comparison of CMR versus period for systems A and D with different $a_0$ and h/r values*

equal to 0.1V, and other systems get lower reductions. Figure 9 illustrates that long-period structures with $a_0$=2 have greater CMR values than fixed-base structures. This behavior does not happen in systems designed according to ASCE 7-10 SSI regulations, which shows the conservative changes that happened in ASCE 7-22 and that the effectiveness of SSI outweighs the reduction in design base shear.

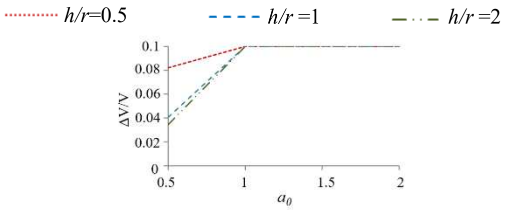


*Figure 10. Ratio of reduction in design base shear to the design base shear of fixed-based structure based on ASCE 7-22*

## 7 CONCLUSION

This paper evaluated the performance of soil-structure (SS) systems at collapse level under seismic loading using collapse margin ratio (CMR). In addition, the effects of employing ASCE 7-10 and ASCE 7-22 seismic design criteria for reducing design base shear for soil-structure systems were studied. A single degree of freedom structure representing special moment resisting systems was modeled based on the modified Ibarra Krawinkler backbone curve. The soil beneath the structure was modeled using horizontal and rotational springs, dashpots, and masses. Different values for dimensionless frequency ($a_0$) and aspect ratio (*h/r*) were considered as two indicators of the main aspects of SS systems. Incremental dynamic analysis (IDA) was conducted using twenty-two pairs of far-field records to calculate the median 5%-damped spectral acceleration of collapse level ground motions, $\hat{S}_{CT}$. Calculation of CMR for SS systems with different $a_0$ and *h/r* values resulted in the following conclusions:

• Increasing the effect of SSI (increasing $a_0$ and decreasing *h/r*) mostly improves the performance of SS systems at collapse level compared to fixed-base structures, resulting in greater CMR values.

• Structures designed based on the recommended reduced design base shear of ASCE 7-10 reached the collapse level at lower ground motion intensities compared to fixed-base structures. However, this approach leads to non-conservative performance and smaller CMR values compared to fixed-base structures.

• ASCE 7-16 and 7-22 criteria, which suggest a maximum reduction in design base shear depending on the response modification factor, yield more conservative results than ASCE 7-10. For systems with high SSI effects (higher $a_0$ and lower *h/r*), the effectiveness of SSI outweighs the negative effects resulting from reduction in design base shear, so CMR values of SS systems with reduced design base shear are greater than CMR values of fixed-base structures.

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