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Enhancing Displacement Coefficient Method for Multi Degree of Freedom Buildings (MDOF) Considering Nonlinear Soil Structure Interaction

--Manuscript Draft--

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Abstract:	One of the main challenges in seismic assessment of existing structures is estimating the displacement demand under earthquake motions. Displacement coefficient method introduced in current code and instructors correlates displacement of an equivalent single degree of Freedom (ESDOF) system to roof or any story of corresponding MDOF. An important coefficient in this method, , defines the ratio of inelastic to elastic displacement of ESDOF. The effects of soil structure interaction (SSI) on parameter for SDOF has been investigated by many researchers, however, this parameter on MDOF system, itself, has not been properly investigated. This is a challenging issue since many influential behaviors cannot properly be addressed in ESDOF systems such as: P-delta effects, higher mode effects, forming of plastic hinges and their sequences considering strength and stiffness deterioration, and nonlinear SSI. In this study, to investigate this approach, seven buildings representing a reasonable range of effective period as MDOF systems were selected (three moment resisting frames and four shear wall buildings) and designed with different strength reduction factors (R=3, 4, 5 & 7). To investigate the effect of SSI on responses, the foundations were designed with (1.5, 3, 4 & 5) factor of safety vertical (FSV) to cover all probable rocking and uplifting behaviors. All designed buildings were analyzed for far-field Design Basis Earthquake (DBE), Maximum Considered Earthquake (MCE), and near field pulse-like earthquake records. The responses were investigated for the effects of SSI on global response of buildings considering both R-factor and FSV as well as MDOF displacement inelastic ratio ( ) and modified amplification factor coefficient ( ). The results showed that the method suggested by ASCE-41-17 for prediction of inelastic displacement ratio underestimates the responses in shorter period buildings and overestimate for longer period buildings for all values of R-factors and FSV. The results showed that the coefficient of introduced in ASCE-41- 17 should include the effects of structural and SSI nonlinearity instead of elastic mode participation factors. Based on results, two practical equations and methods were proposed to enhancing displacement coefficient method considering SSI effects on MDOF systems.

Suggested Reviewers:	Faramarz Khoshnoudian khoshnud@aut.ac.ir He has published some relevant paper in this field
	Mohammad Ali Ghannad he has some valuable papers in the field of SSI
Response to Reviewers:	Reviewer #2: The revised paper can now be accepted. Please in the Proofs correct "5 Conclusion" to "5 Conclusions" Thanks for the comment. The correct form is noted in the text. Reviewer #3: The authors have made a good effort to revise their manuscript, but there still lingers many problem, including: 1. Answers to comments 1, 2, 3, 5, 7, and 9 should be somehow reflected in the revised manuscript. Please note that comments are raised to clarify the content, not only to receive answers. Thanks for the comments, further clarification about the previously mentioned comments has been noted in the corresponding sections in the text of the revision and also a manuscript file is attached with “track changes” to clearly indicate the added parts. 2. Answer to comment 6 is surprising. There are two serious mistakes on the part of the authors. First, "It is noted that the scaling method is not really important in this type of researches, because the ratio of responses are included": It is clearly wrong because this is a nonlinear analysis. Second, "nearly in all current codes such as ASCE7-16 or ASCE7-2022 the scaling methods for all parts of use, suggest using different scale factor for each single record not a single scale factor for all records": It is proved to be wrong by directly referring to the mentioned codes. Clearly, they promote use of a unique scale factor. The approach taken to compute the scale factor has an important influence on such nonlinear analysis. Thanks for the comments, the authors definitely agree with the reviewer’s opinion that in nonlinear analysis the responses do not properly change linearly with simple scaling of records. The authors’ meaning by mentioning the phrase of “not really important” for the answering this comment was initially to rise this fact that, scaling method as well as record selection wasn’t the aim of this study. In this research, earthquake records should be selected for two hazard levels(DBE and MCE). Instead of selecting and scaling the record, the authors preferred to use the records introduced by Baker et al., which have been used in many researches of this type. The selected records and their scale factors values are exactly taken from the Baker’s research (Baker et al. 2011) . The scaling method in Baker et al.'s research was the use of geometric mean method. In this method, different scale factors are considered for different records in order to adjust the geometric mean responses on the target spectrum. It is a method that is accepted in technical literature. It should be noted that with this number of records considered (32 records), the obtained ratios provide a suitable dispersion of the results and logically, there will be no concern about the scaling method on the dispersion of the obtained data. For the second part of this comment, respecting the reviewer's opinion, the authors still emphasize on using different scale factors for different records but the average of all records spectra should be matched with design spectra on defined period range. Of course, this issue is out of the scope of this research, but to clarify the issue a bit more, in the following, the clause of 16.2.3.2 of ASCE-7-22 chapter 16 is presented as a sample of design code. “16.2.3.2 Amplitude Scaling For each horizontal ground motion pair, a maximum-direction spectrum shall be constructed from the two horizontal ground motion components. Each ground motion shall be scaled, with an identical scale factor applied to both horizontal components, such that the average of the maximum-direction spectra from all ground motions generally matches or exceeds the target response spectrum over the period range defined in Section 16.2.3.1. The average of the maximum direction spectra from all the ground motions shall not fall below 90% of the target response spectrum for any period within the same period range.”

	This approach can be seen in other codes and instructions. The geometric mean scaling method which has been used in our research is in the line of current approaches. 3. I am surprised why the authors did not correct the wrong referencing to the important reference ASCE41-17 (wrongly mentioned in several places in the paper as ASCE-41-17) as “Engineers, 2017”!! Thanks for the comment all the citation for ASCE-41-17 now is modified and change to the ASCE-41-17 as the reviewer noted.

Enhancing Displacement Coefficient Method for Multi Degree of Freedom Buildings (MDOF) Considering Nonlinear Soil Structure Interaction

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Abstract

One of the main challenges in seismic assessment of existing structures is estimating the displacement demand under earthquake motions. Displacement coefficient method introduced in current code and instructors correlates displacement of an equivalent single degree of Freedom (ESDOF) system to roof or any story of corresponding MDOF. An important coefficient in this method, C_1, defines the ratio of inelastic to elastic displacement of ESDOF. The effects of soil structure interaction (SSI) on parameter C_1 for SDOF has been investigated by many researchers, however, this parameter on MDOF system, itself, has not been properly investigated. This is a challenging issue since many influential behaviors cannot properly be addressed in ESDOF systems such as: P-delta effects, higher mode effects, forming of plastic hinges and their sequences considering strength and stiffness deterioration, and nonlinear SSI. In this study, to investigate this approach, seven buildings representing a reasonable range of effective period as MDOF systems were selected (three moment resisting frames and four shear wall buildings) and designed with different strength reduction factors (R=3, 4, 5 & 7). To investigate the effect of SSI on responses, the foundations were designed with (1.5, 3, 4 & 5) factor of safety vertical (FSV) to cover all probable rocking and uplifting behaviors. All designed buildings were analyzed for far-field Design Basis Earthquake (DBE), Maximum Considered Earthquake (MCE), and near field pulse-like earthquake records. The responses were investigated for the effects of SSI on global response of buildings considering both R-factor and FSV as well as MDOF displacement inelastic ratio ($C_{1_{MDOF}}$) and modified amplification factor coefficient (C_m). The results showed that the method suggested by ASCE-41-17 for prediction of inelastic displacement ratio underestimates the responses in shorter period buildings and overestimate for longer period buildings for all values of R-factors and FSV. The results showed that the coefficient of C_0 introduced in ASCE-41- 17 should include the effects of structural and SSI nonlinearity instead of elastic mode participation factors. Based on results, two practical equations and methods were proposed to enhancing displacement coefficient method considering SSI effects on MDOF systems.

Keywords: Soil-structure-interaction, displacement demand, rocking motion, nonlinear soil-structure system, parallel processing

1 Introduction

Nowadays, structures evaluation and design require having the precise details of the demands they are exposed to and their capacities. Regarding demand, knowing the displacement demand (or target displacement) of structures under earthquake ground motion is one of the tools for assessing structural performance under earthquake. The methods for deriving this target displacement is one of the challenges that researchers and engineers are trying to address for years. During the last decades, enormous researches have been conducted to determine an acceptable estimation of structural target displacement. This approach, if it is accessible and reliable, can save engineers from implementing complex and time-consuming nonlinear time history analyses to assess seismic performance of structures. Following this approach, current codes and instructions proposed an equation relating linear displacement of single degree of freedom system (SDOF) to nonlinear multi degree of freedom system (MDOF) with fixed base condition. FEMA440 (2005) and ASCE41-17 suggested an equation to calculate target displacement (δ_t) which is called displacement coefficient method. This target displacement approximates the displacement demand of an equivalent SDOF system to roof or any desired story of the same building (i.e., MDOF).

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{1}$$

Where, C_0 is a coefficient to relate displacement of SDOF system to each story of MDOF system calculated by modal analysis; C_1 is a modification factor relating elastic displacement demand to nonlinear displacement demand of an elastic perfect plastic SDOF system; and C_2 imposes the strength deterioration, stiffness degradation, and pinching effects on target displacement demand. T_e is the effective fundamental period of the system; S_a is the response spectral acceleration at corresponding effective fundamental period, and g is gravity acceleration. In this equation, C_1 and T_e which can significantly increase or decrease the target displacement are the more influential parameters. Both parameters are mainly suggested based on the studies carried out on SDOF systems in which defining effective period and hysteretic rules are well established. However, in current codes and standards, to investigate the displacement demands of buildings such as MDOF systems, it is suggested that displacement demand of MDOF system should be determined by an SDOF system response and then convert the SDOF response to the MDOF one. In this procedure, some important behaviors may not be addressed properly. One of these behaviors expected to different from SDOF to MDOF system, is the foundation behavior and soil structure interaction. In previous research conducted on soil-structure interaction, as is explained in the following part, the effect of foundations has been investigated using simple concentrated models (cone models) or beam on nonlinear Winkler springs in which just one-dimensional horizontal beam attached to a vertical single column which represents structures as SDOF system (e.g. single column bent of bridge's pier). While in real structures, the foundation is designed as strip, mat and, in less practical case, as spread. The effects of strip or mat foundation on structural system have not been thoroughly studied. The whole system behavior under earthquake in terms of rocking and uplifting motion which definitely has significant influences on the natural periods and system damping are not clearly addressed so far based on the author’s survey. These behaviors as well as higher mode effects and degrading of behavior in plastic hinges of frames cannot be properly addressed in SDOF systems. In addition, the challenges issued here on C_1, T_e, can also be extended on the terms of C_0 and C_2 which are out of the scope of the present research.. However, before discussing the methodology of the research, a brief literature review on the related studies is presented in the following.

First, by examining a wide range of SDOF systems, Veletsos and Newmark (1960) proposed equal displacement rules. Then, further research was conducted on displacement amplification factor for fixed-base structures (e.g. (Chopra and Goel, 2000, Fajfar and Fischinger, 1988, Shimazaki and Sozen, 1984)). Miranda and Ruiz-García (2002) presented an equation for estimating C_1 by considering site effect. Then, Ruiz-García and Miranda (2007) studied the displacement coefficient method for fixed base structures built on soft soil.

Foundation flexibility has significant effects on dynamic characteristic and structural displacement response of structures, especially for stiff structures (Stewart et al. (1999)). The influences of rocking and uplifting motion first have been studied on rigid block by Housner (1963). Meek (1975) demonstrated that uplifting motion reduced the base shear demand. Moreover Yim and Chopra (1983) assigned complex behavior for soil and pointed that the ratio of height to the width of the structure was one of the influential parameter and uplifting motion was more probable for structure with shorter period. Kawashima and Hosoiri (2003) considered the response of Pier Bridge with uplifting potential. They showed that moment rotation response reduced significantly compared to the fixed base case. Furthermore, the displacement increased 27% to the fixed base with no residual displacement. These observations resulted that some researchers claimed that soil structure interaction might be beneficial in some cases ((Gajan et al., 2005, Gazetas et al., 2003, Pecker, 2003)).

The parametric studies of SSI in the literature mainly investigated SDOF systems. In these research an elastic half space is considered for representing the soil beneath the foundation. Ghannad and Jahankhah (2007) and Ganjavi and Hao (2012) studied the effect of SSI on strength reduction factor. Khoshnoudian and Behmanesh (2010) and Khoshnoudian et al. (2013) evaluated the accuracy of C_1 equation proposed by FEMA440 (2005) by modeling SSI with cone model concept. Harden (2005) proposed a new C_1 for uplifting systems with rigid block as their superstructure. Later (Ghannad and Jafarieh, (2014, Jafarieh and Ghannad, 2014) studied the effects of foundation uplift on displacement demand for elastic and inelastic structures, respectively. In this research new equation was proposed for determining C_1 coefficient by considering some effective parameters. Their results indicated that uplifting motion should be prevented for structures with shorter period due to instability potential. Simultaneous effects of nonlinear soil-structure-interaction rarely considered. Much research assumed linear half space for the soil beneath the foundations or investigated the soil nonlinearity behavior while structures assumed to be elastic. Khanmohammadi and Mohsenzadeh (2018) in a complete and thorough research, presented new equation for displacement amplification factor by considering soil structure interaction nonlinearity and foundation uplift and showed that existing equation could be led to underestimate values. SSI increased the rotation demand in beams to the rotation compatibility in rocking motion (Carbonari et al., 2012). (Karapetrou et al., 2015) investigated the effect of SSI and site effect on a non-ductile RC frame with direct methods. They showed that SSI could reduce the structure inter story drift ratio (IDR). Recently similar studies assessing the SSI effect on multi story buildings under earthquake excitation has been done(G. et al., 2020; Karakostas et al., 2021), these studies showed that, SSI can significantly affect the structural responses especially the inter story drift ratio (IDR). In a parametric study (Asadi-Ghoozhdi & Attarnejad, 2020)investigated the effect of SSI on MDOF systems with vertically stiffness irregularities, their parametric study indicated that the SSI effect can reduces the ductility demands for structures and its much significant for systems with shorter periods.

Ruiz-García and González (2014) calculated the inelastic displacement ratio for the buildings located on soft soil. Their results showed that inelastic displacement ratio for MDOF systems differed from SDOF systems. The foundation flexibility did not take into account in their research. They proposed a C_M modification factor for the effects of frame such as higher mode effects and frame mechanism. In that study only the site effects were considered. More recently, Gholamrezatabar et al. (2017) estimated inelastic displacement factor of shallow foundation MDOF systems by considering higher mode effects. They assumed the soil as a linear elastic half space and utilized cone model for SSI numerical model. They indicated that using SDOF inelastic displacement ratio for MDOF system, overestimated target displacement. In this study one new factor which can convert the nonlinearity effects of SDOF system to MDOF on target displacement has been introduced. However, their numerical model was not able to capture uplifting and soil material nonlinearity beneath the foundation.

As explained before, there are a limited number of research works on estimating target displacement with rocking and uplifting motion, especially in MDOF system, which is important in viewpoint of energy dissipation capacity, dynamics characteristics, settlement, radiation damping, higher mode effects, and frame mechanism. Current study considers Simultaneous effects of rocking, uplifting, structural nonlinearity, and soil behavior to discover their effects on C_1 and a coefficient was proposed based on the conducted nonlinear time history analysis. To consider a wide range of responses, a set of 7 types of building with a wide range of foundation types and periods were introduced to cover rocking, uplifting, and structural nonlinearity conditions. To this end, more than 23,520 nonlinear dynamics time history analyses were run for multiple ground motions with different level of hazard and near and far field motions. A modification coefficient was proposed for C_1 ; and the effect of involved parameters in behavior was investigated. For practical engineers, two different procedures were proposed to capture the effects of MDOF system, considering SSI, on SDOF target displacement based on the results.

2 Methodology

The inelastic displacement ratio C_1 is defined as the maximum lateral inelastic displacement of elastic perfectly plastic response of equivalent SDOF structures to its linear elastic corresponding system. In current research, quite the same definition is employed for MDOF system as being schematically illustrated in Fig. 1. It is worth noting that instead of the center of mass, the roof of the building is addressed. Therefore, to estimating the inelastic to elastic displacement ratio in MDOF systems by considering soil-structure-interaction, ${C_1}_{MDOF}$ coefficient is defined as Eq (2):

$$ {C_1}_{MDOF} = \frac{\Delta_{inelastic}^{\mathrm{MDOF}}}{\Delta_{elastic}^{\mathrm{MDOF}}} \qquad (2) $$

where $\Delta_{inelastic}^{\mathrm{MDOF}}$ is the maximum displacement of MDOF system in which all structural nonlinearity as well as foundation response are included (Fig. 1a and 1b), $\Delta_{elastic}^{\mathrm{MDOF}}$ is the maximum elastic displacement of the same linear elastic MDOF system with foundation flexibility in which the structure has elastic linear behavior (including p-delta) and foundation is modeled with elastic linear Winkler springs to present flexibility (Fig. 1 1c and 1d). It is important to emphasize that, where the MDOF system is solved for fundamental period, the effect of SSI should include. Hence in this research where ${C_1}_{MDOF}$ is calculated, the effect of SSI on fundamental period and consequently spectral acceleration, was considered using elastic spring beneath the foundations as illustrated in Figure 1c. following this procedure can guarantee that all possible source of nonlinearity is included in the defined coefficient without counting SSI effect twice.

the whole described process is followed for SDOF systems and for all corresponding equivalent single degree of freedom systems used in this study the $C_{1_{SDOF}}$ was defined which contains all nonlinearity for SDOF system This procedure is followed for SDOF system; therefore, C_{SDOF} coefficient defined as Eq. (3)

$$C_{1_{SDOF}} = \frac{\Delta_{inelastic}^{\mathrm{SDOF}}}{\Delta_{elastic}^{\mathrm{SDOF}}} \tag{3}$$

In Eq. (3) $\Delta_{inelastic}^{\mathrm{SDOF}}$ and $\Delta_{elastic}^{\mathrm{SDOF}}$ is maximum inelastic and elastic displacement of equivalent SDOF system, respectively, derived from its equivalent corresponding MDOF system. The details of modeling are explained in the following.

$C_{1_{MDOF}}$ contains all structure-foundation nonlinearity effects on displacement amplification factor for MDOF system and $C_{1_{SDOF}}$ has these features for SDOF system. In calculation of $C_{1_{MDOF}}$ all structural nonlinearity, near field soil nonlinearity, foundation rocking and uplifting, P-delta effects, and higher mode effects are included, while in SDOF system the effects of P-delta, higher modes, and in cases, foundation's rocking and uplifting cannot properly be modeled. Therefore, to extract such behavior in which the equivalent SDOF and MDOF responses are distinguished, the coefficient of C_m is defined in this procedure as in Eq. (4):

$$C_m = \frac{\mathrm{C}_{1_{\mathrm{MDOF}}}}{\mathrm{C}_{1_{\mathrm{SDOF}}}} \tag{4}$$

The coefficient of C_m is one of the important parameters which the previous research on SDOF system were not able to be accurately addressed while inherently is embed in response of MDOF system.

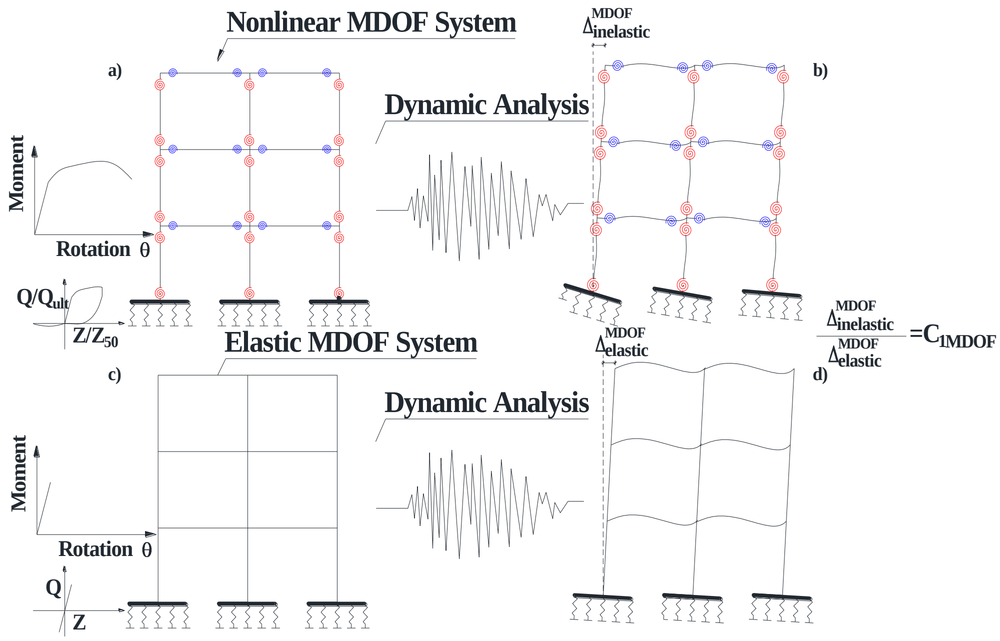


Fig. 1 Schematic methodology in present study

2.1 Equivalent SDOF

To generate equivalent SDOF systems (ESDOF) from MDOF frames, the following steps are implemented: 1) performing nonlinear pushover analysis under first mode load pattern. To perform pushover analysis all sources of nonlinearity were considered including structural plastic hinges, soil behavior, rocking and uplifting of foundations, and P-delta effects. Fig. 2a shows a sample of modeling of 5-bay 3-story building. 2) Idealizing the base shear versus roof displacement to provide a tri-linear curve. Fig. 2b shows a sample of analyses for the same building introduced in part a, and 3) using the first mode characteristics, convert the idealized curve to capacity curve of ESDOF system. To do this, the procedure suggested in ATC-40 (1996) is used as illustrated in Fig. 2c. With this regard, the effective height, mass, and modal participating ratio determined from modal analysis were used. The equivalent effective mass and effective height were used to replace the response of lateral force versus lateral displacement to base moment versus rotation simply by multiplying the lateral shear by effective height and dividing the lateral displacement by effective height. To accurately capture the lateral stiffness of equivalent SDOF (ESDOF) (e.g., Fig. 2d), the stiffness of vertical member was adjusted accordingly. Fig. 2d shows a sample of generated SDOF system extracted from MDOF (3-story building). Another challenging issue on generating ESDOF from MDOF is hysteretic responses. Post yield hardening, strength and stiffness degradation, and pinching behavior should calibrate using cyclic analysis. To this end, in current research cyclic push-over analysis was carried out on all frame buildings using first mode load pattern (i.e., MDOF system) and the derived response curve was examined for the best hysteretic curve matched by ESDOF system. Clough Material is used as hysteresis models of ESDOF systems. The resulted parameters on matching are used for all other nonlinear time history analyses on SDOF systems. Fig. 3 shows two samples of such response fitting for 3 and 27 story buildings considered in current research. The main idea from such response fitting are including the hysteretic behaviors. This approach implying that both ESDOF and MDOF systems have the same strength degradation. Therefore, this procedure can eliminate the coefficient of C_2 from target displacement equation by dividing $C_{1_{MDOF}}$ to $C_{1_{SDOF}}$. Fig. 2 illustrates the procedure of generating equal SDOF systems from MDOF. This procedure is implemented for all selected buildings described in the following part.

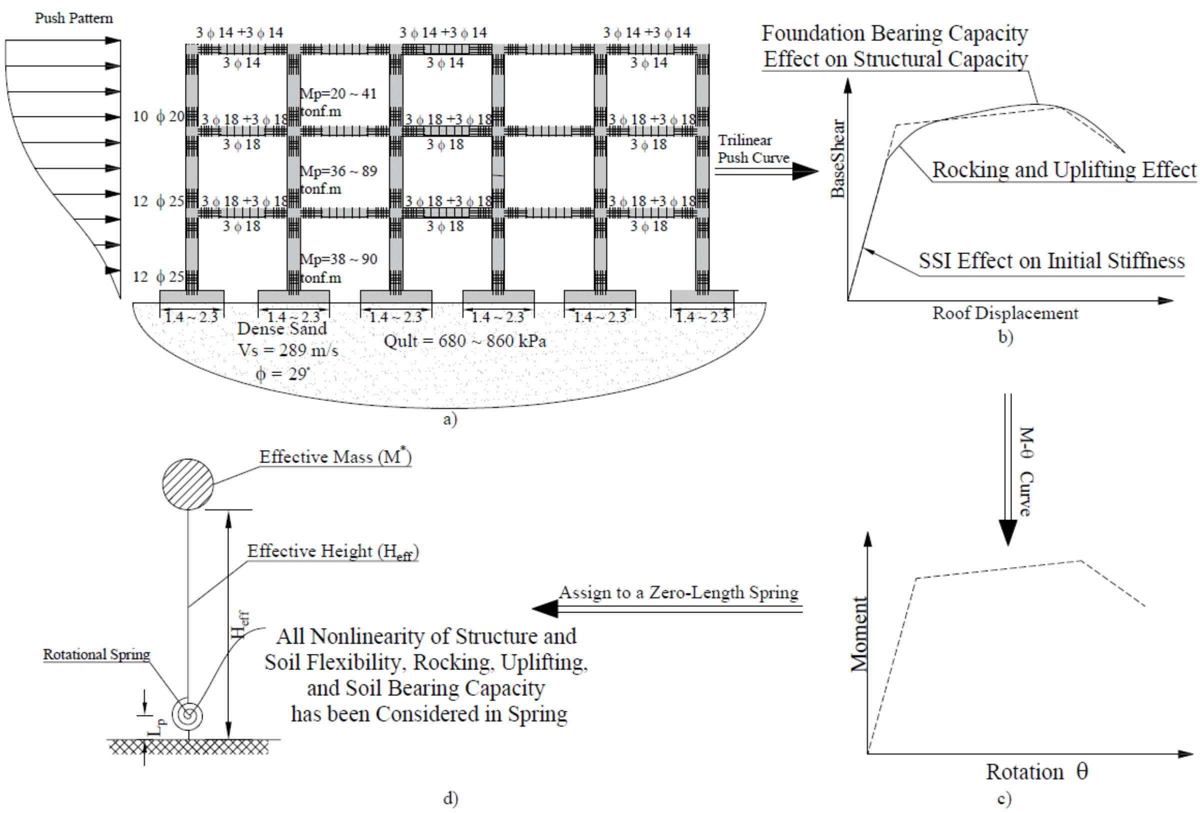


Fig. 2 Calculating equivalent SDOF system procedure, a) Pushover analysis on SSI model, b) Bilinear capacity curve of MDOF system, c) ESDOF moment rotation, d) Conceptual model of ESDOF system

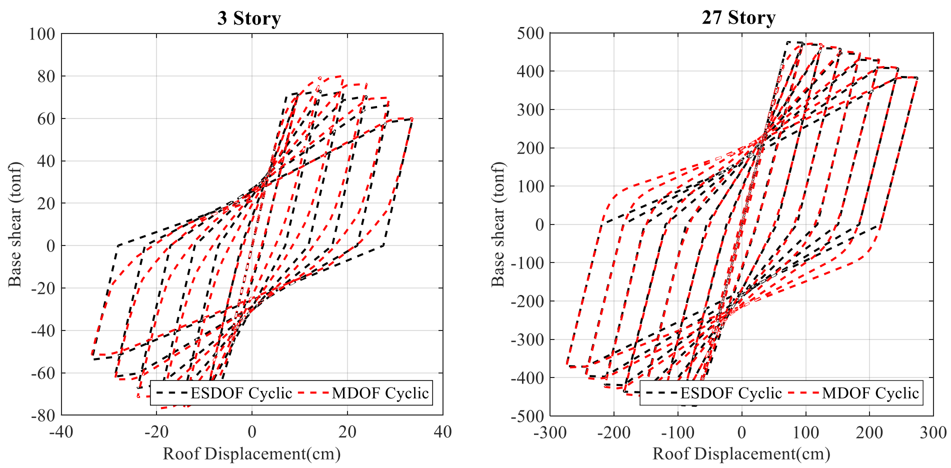


Fig. 3 Cyclic Response of 3 and 27 story with R=1.7 and FSV=3

3 Numerical Modeling

3.1 Structures

In order to consider all effective parameters on displacement amplification factor such as structural period, lateral capacity, soil shear wave velocity, foundation capacity and frame mechanism and multi degree influences, seven RC buildings with different stories (ranges from three to twenty seven stories) are considered. In these structures three of them are moment resistance frame and the others have shear walls as their lateral resisting system. Four types of lateral strength capacity are set, and the foundation of these structures have different dimension to cover different vertical factor of safety. In any type of considered building, a fixed base model was also designed to take a better comparison between fixed base and flexible based models.

These structures covered a range of 0.25 to 4.5 seconds fundamental periods which are thought cover many practical cases of designs from stiff to flexible frames. Since the foundation's dimensions affects the response of SSI, shear wall buildings also considered in design. Loading and designing of structures were carried out using ASCE-7 (2010) and ACI318-14 (2014), respectively. The soil type was considered to be type D and for seismic loading the Iranian standard seismic load (2013) was implemented. All moment resisting frame buildings are designed for intermediate level of ductility and shear walls are designed for special ductility. To investigate the effect of yielding strength on responses and displacement coefficients like C_{1MDOF} and C_m, all structures redesigned for four different level of yielding strength. With changing the strength reduction factor introduced in the Iranian standard seismic load (2013) such as R_u=3, 4, 5 & 7. all the buildings were redesigned and detailed for seismic requirements. Fig. 4 demonstrates typical plane and elevation of structures. The "H" and "B" notations are related to the structural total height and width which is reported for all considered frames in Table 1, and the “L” and “b” notations are foundations length and width dimensions as illustrated in Figure 4f, and 4g and listed in Table 2. To condense annotating, in the remaining parts of paper, all moment resisting frames were shown by number of story and symbol of M, and shear wall buildings were noted by number of story and the notation SW. For instance, 3 M and 6 SW represented the 3-story moment resisting frame and 6-story shear wall building, respectively. In present study, a dense sand soil with 284 m/s shear wave velocity, 0.25 Poassion ratio, and 18.84 kN/m^3 density has been selected. Four types of foundations with different vertical factors of safety were considered to investigate both soil plasticity and rocking and uplifting of foundations effects. The soil ultimate bearing capacity (Q_{ult}) was calculated through Vesic (Vesic, 1973) equation. The maximum allowed foundation settlement criterion has been considered in portioning the foundation geometry for both single and strip footing. All geometrical foundations were designed as a spread footing except the 9-story structure which is a strip foundation due to not enough spacing of spans. To investigate the effect of foundation behavior on responses (rocking, uplifting, and yielding the soil), and different level of SSI, four vertical factors of safety (FSV=1.5, 3, 4, and 5) to represent small to large foundations were considered in current research. The FSV controls the dimensions of foundation and consequently the level of nonlinearity of soil. Since, one of the main goals of the research was the studying the effects of soil nonlinearity of foundation and structure behaviors and their effects on $C_{1_{MDOF}}$. Therefore, in this study through choosing different FSV for the foundation, different level of SSI nonlinearity is assumed, and the results were compared. The selected FSVs were in range of practical cases and for comparison a fixed based also designed. The fundamental period of buildings and achieved strength reduction factors are listed in Table 1. The design parameters of foundations including dimensions and bearing capacity for each FSV were listed in Table 2 and Table 3 for moment frame structures and shear wall structures, respectively.

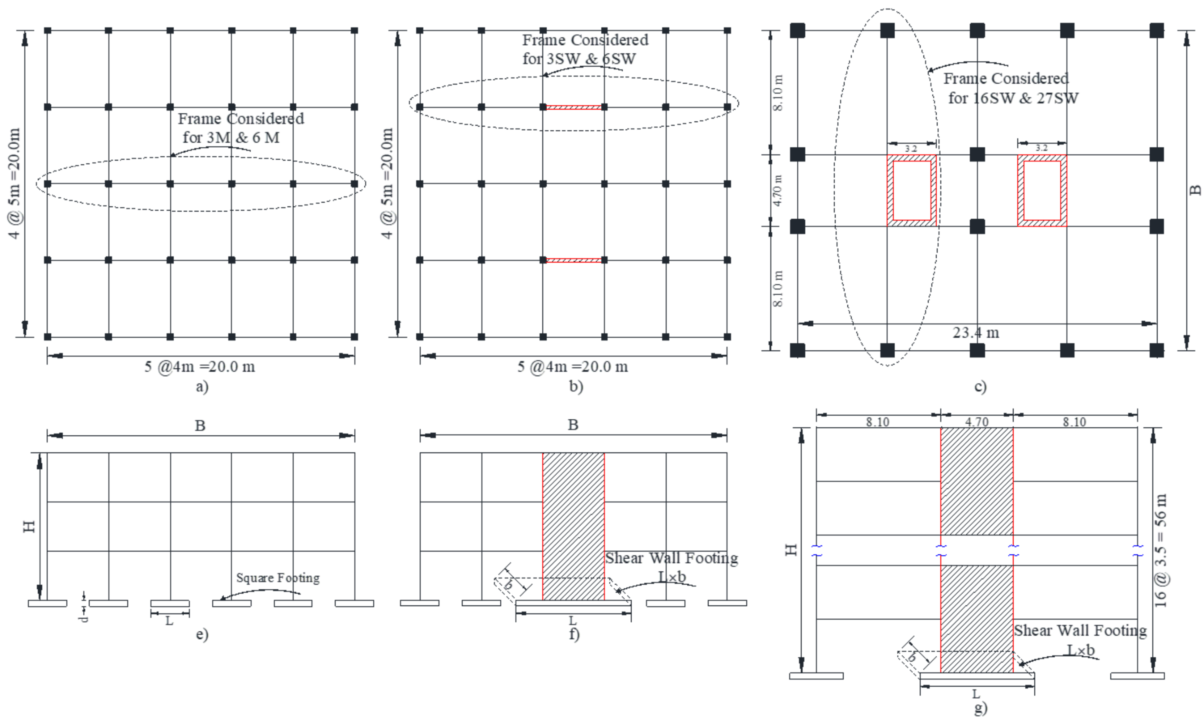


Fig. 4 Building Plane and Elevation, a) planes of 3M,6M, and 9M buildings, b) plan of 3SW and 6SW buildings, c) plan of 16SW and 27SW buildings, e) elevation of 3M, f) elevation 3 SW, g) elevation 16SW

As it is shown in Table 1, structural period increases by decreasing foundation vertical factor of safety. The width of the structures is constant, and the structural height is varied from 9.2 to 94.5 meter. The expected strength reduction factor(R) was estimated for each level of hazard according to Eq. (5) ASCE41-17:

$$R = \frac{mS_a}{F_y} \qquad (5)$$

Where m is the effective mass, S_a is average response spectra acceleration of records at its effective periods, and F_y is the structural lateral strength determined from static pushover analysis which explained in Fig. 2b. it is important to note that this expected reduction factors are different from those used in design of building's design codes (R_u). The R_u is code-based while the R values depend to hazard level and actual level of strength of building. As listed in Table 1, the value of expected strength reduction factors varies from each hazard level and each type of foundation (different FSV) since the values of S_a and F_y are different for these conditions. The values of R listed in Table 1 vary from 1.3-3.4 which are in the range of values expected for buildings designed based on current seismic codes.

Table 1. Main structures characteristics

	Periods (sec)					Dimension (m)		Expected Strength Reduction factor(R)		
Type	Fixed	FSV=5	FSV=4	FSV=3	FSV=1.5	B	H	DBE	MCE	Near-Field
3 M	0.775	0.785	0.79	0.81	0.88	20	9.6	1.3,1.4,1.7,2.2	1.6,1.9,2.3,2.9	1.7,2.05,2.6,3
6 M	1.277	1.29	1.3	1.33	1.43	20	19.2	1.3,1.4,1.6,2.1	1.6,1.9,2.1,2.8	1.7,2.1,2.65,3
9 M	1.66	1.82	1.83	1.86	2	20	28.8	1.5,1.7,2.1,2.4	1.8,2.3,2.8,3.2	1.9,2.3,2.8,3.4
3 SW	0.23	0.27	0.29	0.32	0.38	20	9.2	1.35,1.4,1.7,2.3	1.7,2.1,2.5,3.1	1.8,2.2,2.7,3.3
6 SW	0.51	0.6	0.62	0.65	0.71	20	19.2	1.3,1.45,1.7,2.2	1.75,2.1,2.6,3.1	1.8,2.2,2.8,3
16 SW	2.4	2.575	2.605	2.65	2.9	20.9	56	1.7,1.9,2,2.2	2,.2,4,2.8,3.1	2.1,2.4,3,3.4
27 SW	4.4	4.54	4.59	4.78	4.9	20.9	94.5	1.7,1.9,2.1,2.2	2,2.4,2.7,3.3	2.1,2.4,3,3.4

Table 2. Foundation characteristics for moment frame structures

Square Foundation				Middle foundation		Corner foundation			Strip Foundation	
3 M	L×L (m)	Qult (kPa)	6 M	L×L (m)	Qult (kPa)	L×L (m)	Qult (kPa)	9 M	b×L (m)	Qult (kPa)
FSV=1.5	1.4	614	FSV=1.5	1.5	640	1.8	680	FSV=1.5	3×22	1000
FSV=3	1.8	656	FSV=3	2	690	2.4	750	FSV=3	2.5×22	900
FSV=4	2.1	710	FSV=4	2.4	725	3	810	FSV=4	2×22	825
FSV=5	2.3	720	FSV=5	2.7	780	3.6	880	FSV=5	1.3×22	725

Table 3. Foundation Characteristics for shear wall structures

	Column Foundation		Shear Wall Foundation			Column Foundation		Shear Wall Foundation	
3 SW	L×L (m)	Qult (kPa)	b×L (m)	Qult (kPa)	6 SW	L×L (m)	Qult (kPa)	b×L (m)	Qult (kPa)
FSV=1.5	1.4	630	0.8×5	600	FSV=1.5	1.8	650	1×6	650
FSV=3	1.6	680	1×6.5	610	FSV=3	2.1	690	2×6.5	680
FSV=4	1.8	700	1.25×7	635	FSV=4	2.4	725	2×7	771
FSV=5	2	720	1.25×8	664	FSV=5	2.7	780	2×8	805

	Shear Wall Foundation		Column Foundation			Shear Wall Foundation		Column Foundation	
16 SW	b×L (m)	Qult (kPa)	L×L (m)	Qult (kPa)	27 SW	b×L (m)	Qult (kPa)	L×L (m)	Qult (kPa)
FSV=1.5	6.6×8	1400	3	810	FSV=1.5	7.5×9	1550	4.2	900
FSV=3	6.6×9.2	1520	4	920	FSV=3	7.5×10.5	1720	5	1000
FSV=4	6.6×10.2	1600	4.5	1000	FSV=4	7.5×12	1900	5.6	1100
FSV=5	6.6×11	1700	5	1100	FSV=5	7.5×13	2100	6	1200

3.2 Nonlinear Modeling

As explained before, to have a precise estimation of the target displacement, all structural nonlinearities should be modeled properly in the numerical simulation. The Open System for Earthquake Engineering Simulation (McKenna et al., 2010) is employed for the modeling and analyses of structural and soil elements.

3.2.1 Structural Nonlinear Modeling

Modeling of the beam and column elements is performed using concentrated plasticity approach. The response curve of the beam and column elements was based on empirical models proposed by (Haselton et al., 2008). The methods proposed in (Ibarra et al., 2005) was used for modeling the deterioration of strength and stiffness (Fig. 5). In all building models, P-Δ effects and 5% Rayleigh damping were considered at the first and third frequencies of each structure. Since structures in this study are regular in height and plane, therefore, through the modal analysis it is revealed that by considering 3 modes, more than 90 percent modal participation would be achieved which could lead to a quite fair estimation for calculating the structural damping with Rayleigh method.

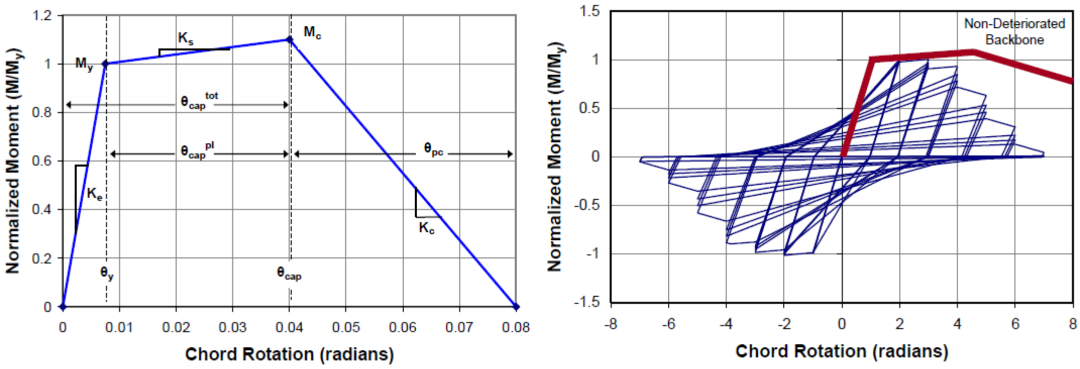


Fig. 5. Clough material implemented in OpenSees (Haselton et al., 2008)

The concentrated plasticity (moment-rotation) model based on moment-curvature and plastic hinge length is implemented for Shear wall elements as well. The section of shear walls were divided to confined and unconfined concrete using Mander (Mander et al., 1988) model(concrete 02) and rebars are modeled using menegtopito (Menegotto, 1973)(reinforcing bar) and a fiber-based approach implemented for section response. To consider the effect of strength degradation and stiffness deterioration on monotonic pushover curve, the recommended reduction coefficient in the Applied Technology Council report ATC72 (2010) is used.

3.3 SSI modeling and verification

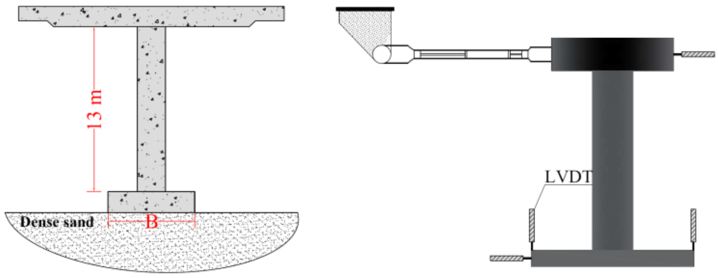
Soil structure interaction is modeled by substructure method (Wolf, 1985) in which actual structures and springs representing the foundation effects are implemented in dynamic motion and no extra nodes are defined for the soil interior. The foundation sway and rocking and uplifting motion are simulated by beam on nonlinear Winkler foundation theory. Qzsimple2, Pysimple2, and Tysimple2 were used as given materials for soil spring ((Boulanger, 2000, Harden, 2005, Raychowdhury, 2008, Raychowdhury and Hutchinson, 2009)). The initial stiffness of these springs are calculated via equations suggested by (Gazetas,

1991) and foundation element were modeled by elastic beam column elements. Soil structure interaction radiation damping was simulated by Laysmer's spring (Lysmer and Kuhlemeyer, 1969). To ensure that, this simulation could capture all rocking and uplifting motion properly, this code was verified and calibrated with experimental research reported by (Drosos et al., 2012).

3.3.1 Verification

(Drosos et al., 2012) reported experimental research on rocking response of a SDOF bridge pier constructed with 1/20 scale on dense sand under cyclic and harmonic excitation. Details of prototype, model, experimental set-up, and test results can be found at (Anastasopoulos et al., 2010, Anastasopoulos et al., 2012, Anastasopoulos et al., 2013, Drosos et al., 2012). Experimental set-up is shown in Fig. 6 . Through extensive parametric analysis, minimum number of required springs and appropriate equation for calculating the end length ratio which are more compatible with the experimental results are determined. This proposed model is revalidated with an experimental investigation conducted by (Faccioli et al., 1999) to ensure that the proposed model would be working properly and can satisfy different types of test conditions.

Natural periods of experimental fixed base and flexible base structure was 0.04 and 0.1175 seconds, respectively. The current simulation is resulted in 0.0377 and 0.1167, respectively for fixed and flexible based, which indicates acceptable degrees of accuracy with only 5.0 and 0.65 percent relative error respectively.



a) b)

Fig. 6 Experimental Setup (Drosos et al., 2012), a) Prototype scale, b) schematic location of measuring equipment

Based on cyclic response, the proposed model is calibrated to simulate nonlinear soil structure interaction. The number of springs has a direct influence on initial stiffness and foundation moment capacity. In current research minimum distance ratio between middle and end length regions springs are set to be 0.1 and 0.25 respectively. For the length of the end region Harden et al (2005) used 0.25L, Ghannad and Jafari 2014 used no end region length, and NEHRP(2012) suggest a value between 0.15L-0.3L. ASCE-41 -17 and FEMA-440 suggest L/6(adopted from ATC40) , for calculating the end length ratio of shallow foundations. In this research, the same value of 0.15L which is more consistent with L/6 was selected for modelling. is.. Among the proposed equations to calculate end region stiffness (e.g. (ATC, 1996, Harden, 2005, Venture, 2012)), the most appropriate equation which is more compatible to experimental results is the equation proposed by NEHRP (2012). The suggested model and numerical modeling of experimental set-up are been shown in Fig. 7.

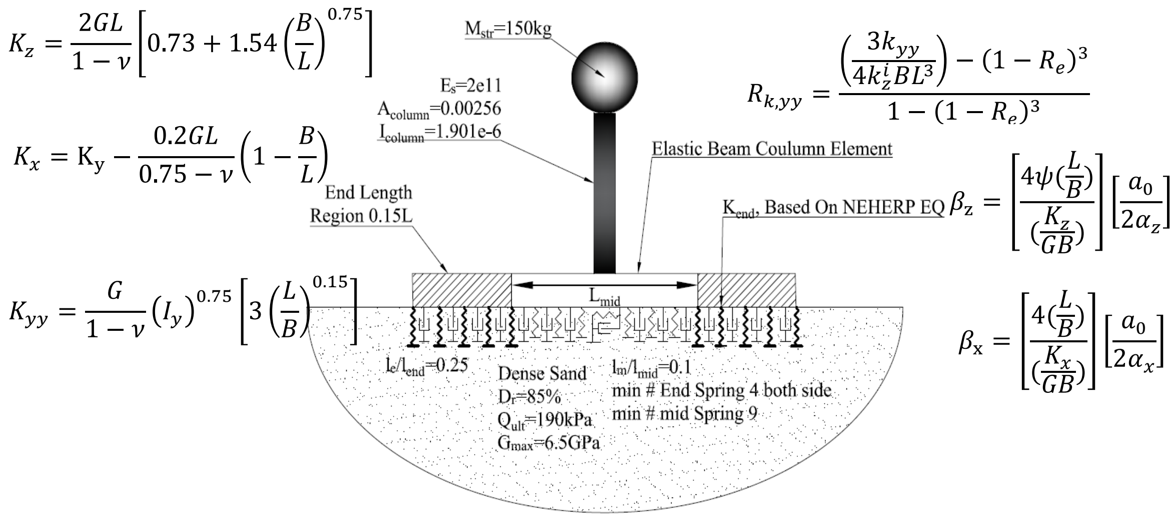


Fig. 7 Proposed SSI modeling in OpenSees software

In addition, in modeling SSI with beam on nonlinear Winkler foundation, horizontal stiffness spring in the middle of foundation is defined to capture sway motion properly. Comparison between numerical and experimental results are shown in Fig. 8.

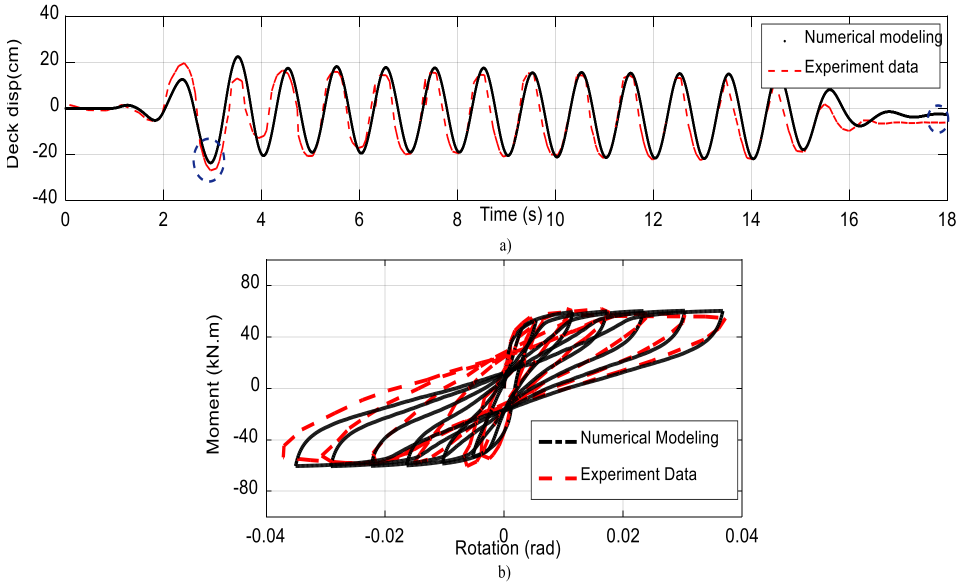


Fig. 8 Experimental and Numerical response of bridge pier. a) Dynamic Response, b) cyclic response

Fig. 8 a shows the experimental and numerical response of the deck. Fig. 8b illustrates cyclic response of base moment versus rotation. The rotation calculated from difference of vertical displacement measured at end points of foundation both for experimental and numerical models as displayed in Fig. 6b. Both responses presented in Fig. 8 showed an acceptable modeling approach on experimental test. This modeling approach and parameters were tested again for another test results adopted from(Faccioli et al., 1999). For the sake of brevity, the detail of test is not shown here. Fig. 9 also shows the results of proposed model with experimental test. The results on moment rotation of foundation and base shear versus foundation horizontal displacement reveal that the modeling approach can accurately present the behavior of SSI. Using the concept of presented modeling approach, all foundations designed for selected buildings introduced in Table 2 and Table 3 are generated beneath structural frame.

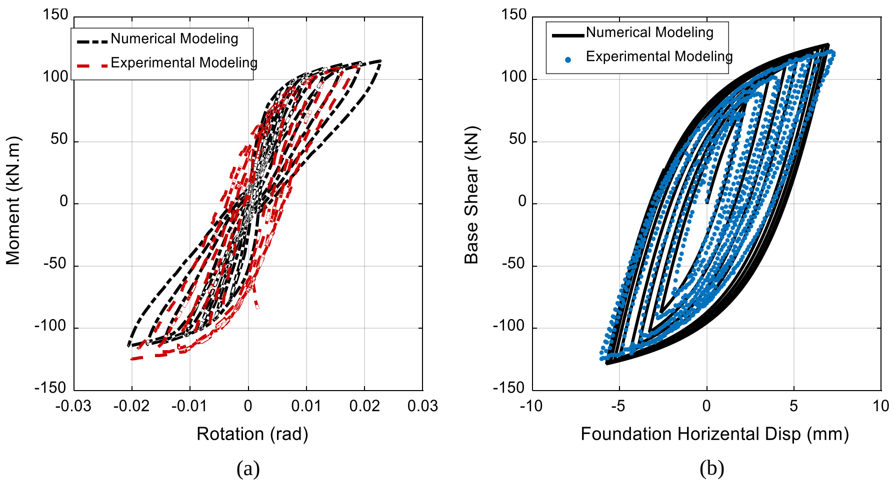


Fig. 9 Cyclic response of experimental and numerical model. a) Moment-Rotation, b) Shear Force-Foundation Displacement

3.4 Earthquake ground motions

A set of 32 ensemble earthquake motioned from PEER-NGA database suggested by Baker *et al* (Baker et al., 2011) was consulted for selection earthquake records in MCE Level with 0.6 g PGA on mean values (Peak ground motion Acceleration). These records were scaled with mean geometry method to match with DBE level with 0.4 g PGA. Table 4 lists the characteristic of selected earthquakes including scaling factors for DBE and MCE records. The records numbers and scaling values provided in Table 4 are the same as records identities used in (Baker et al., 2011); hence, the number was not continuous. For investigating the effects of rupture directivity at near-fault sites on the displacement amplification factor, 20 unscaled pulse-like ground motions were used(Baker et al., 2011). This set consists of unscaled ground motions containing strong velocity pulses of varying periods in their strike normal (SN) component. To include the effect of velocity pulse period of forward directivity component on responses, the records were selected in such a way that they have a variety of pulse periods to figure out the effect of pulse period on the displacement demand of systems (Table 5). The selected records cover a wide range of frequency content, duration, and spectral accelerations to provide a wide range of interested responses. Fig. 10 shows the elastic response spectra of far-field and near field records and mean spectra.

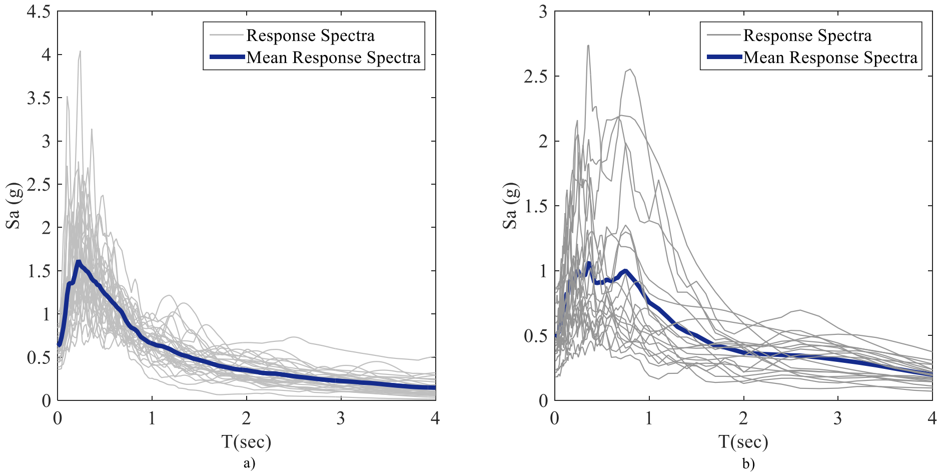


Fig. 10 Elastic Response Spectra. a), Far-field records MCE level, b) Near-field records

Table 4. Far field ground motion

#Rec	Name	Year	Station	Mag	Closet Distance (Km)	Vs (m/s)	Scale Factor (MCE)	Scale Factor (DBE)
1	Imperial Valley-02	1940	El Centro Array #9	7	6.1	213	2.13	1.73
4	Imperial Valley-06	1979	Chihuahua	6.5	7.3	275	2.08	1.77
7	Imperial Valley-06	1979	El Centro Array #12	6.5	17.9	197	4.22	3.38
15	Imperial Valley-06	1979	Parachute Test Site	6.5	12.7	349	5.44	3.37
16	Imperial Valley-06	1979	Westmorland Fire Sta	6.53	15.25	194	5.83	5.21
17	Victoria, Mexico	1980	Chihuahua	6.3	19	275	3.42	3.25
19	Chalfant Valley-02	1986	Bishop - LADWP South St	6.2	17.2	271	3.64	1.93
20	Superstition Hills-01	1987	Wildlife Liquef. Array	6.2	17.6	207	5.58	2.90
21	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.5	18.2	192	1.84	1.64
22	Superstition Hills-02	1987	Westmorland Fire Sta	6.5	13	194	2.28	1.89
23	Loma Prieta	1989	Gilroy Array #3	6.9	12.8	350	1.94	0.89
24	Loma Prieta	1989	Gilroy Array #4	6.9	14.3	222	2.26	1.40
26	Northridge-01	1994	Arleta - Nordhoff Fire Sta	6.7	8.7	298	2.11	1.64
27	Northridge-01	1994	Canoga Park - Topanga Can	6.7	14.7	267	1.44	1.17
33	Northridge-01	1994	Sun Valley - Roscoe Blvd	6.7	10.1	309	1.86	1.45
34	Northridge-01	1994	Sylmar - Converter Sta East	6.7	5.2	371	0.85	0.79
35	Duzce, Turkey	1999	Bolu	7.1	12	326	1.13	0.74
36	Duzce, Turkey	1999	Duzce	7.1	6.6	276	1.08	0.98
29	Northridge-01	1994	LA - Sepulveda VA Hospital	6.69	8.44	380	0.94	0.60
30	Northridge-01	1994	N Hollywood - Coldwater Can	6.69	12.51	446	2.28	0.48
31	Northridge-01	1994	Newhall - Fire Sta	6.69	5.92	269	0.61	0.44
8	Northern Calif-01	1941	Ferndale City Hall	6.4	44.68	219	6.05	3.96
95	Managua_ Nicaragua-01	1972	Managua_ ESSO	6.24	4.06	289	1.89	1.26
158	Imperial Valley-06	1979	Aeropuerto Mexicali	6.53	0.34	260	1.77	1.21
160	Imperial Valley-06	1979	Bonds Corner	6.53	2.66	223	0.97	0.71
162	Imperial Valley-06	1979	Calexico Fire Station	6.53	10.45	231	2.29	1.62
163	Imperial Valley-06	1979	Calipatria Fire Station	6.53	24.6	206	6.49	4.11
166	Imperial Valley-06	1979	Coachella Canal #4	6.53	50.1	336	4.89	3.59
167	Imperial Valley-06	1979	Compuertas	6.53	15.3	260	4.61	2.79
172	Imperial Valley-06	1979	El Centro Array #1	6.53	21.68	237	5.92	3.29
176	Imperial Valley-06	1979	El Centro Array #13	6.53	21.98	250	5.24	3.36
186	Imperial Valley-06	1979	Niland Fire Station	6.53	36.92	212	5.20	3.77

Table 5. Near field ground motions

# Record	Name	Year	Station	Magnitue	Min Distance	Pulse Period (s)	V_s (m/s)
1	Imperial Valley-06	1979	EC County Center FF	6.53	7.31	4.515	192.1
2	Imperial Valley-06	1979	EC Meloland Overpass FF	6.53	0.07	3.346	186.2
3	Imperial Valley-06	1979	El Centro Array #4	6.53	7.05	4.613	208.9
4	Imperial Valley-06	1979	El Centro Array #5	6.53	3.95	4.046	205.6
5	Imperial Valley-06	1979	El Centro Array #6	6.53	1.35	3.836	203.2
6	Imperial Valley-06	1979	El Centro Array #7	6.53	0.56	4.228	210.5
7	Imperial Valley-06	1979	El Centro Array #8	6.53	3.86	5.39	206.1
8	Imperial Valley-06	1979	El Centro Differential Array	6.53	5.09	5.859	202.3
13	Landers	1992	Yermo Fire Station	7.28	23.62	7.504	353.6
14	Northridge-01	1994	Jensen Filter Plant	6.69	5.43	3.528	373.1
16	Northridge-01	1994	Newhall - Fire Sta	6.69	5.92	1.036	269.1
17	Northridge-01	1994	Newhall - W Pico Canyon Rd.	6.69	5.48	2.408	285.9
18	Northridge-01	1994	Rinaldi Receiving Sta	6.69	6.5	1.232	282.3
19	Northridge-01	1994	Sylmar - Converter Sta	6.69	5.35	3.479	251.2
20	Northridge-01	1994	Sylmar - Converter Sta East	6.69	5.19	3.528	370.5
22	Kobe, Japan	1995	KJMA	6.9	0.96	0.952	312
23	Kobe, Japan	1995	Takarazuka	6.9	0.27	1.428	312
26	Chi-Chi, Taiwan	1999	CHY101	7.62	9.96	4.599	258.9
36	Chi-Chi, Taiwan	1999	TCU101	7.62	2.13	10.038	272.6
40	Chi-Chi, Taiwan	1999	WGK	7.62	9.96	4.396	258.9

4 Analytical Results

As described previously, to determine the effects of nonlinear soil-structure interaction on responses, linear and nonlinear time history analyses on both MDOF and ESDOF system are carried out. More than 23,520 response history analyses (280 models under 84 records) were performed to investigate the displacement demand of MDOF and its corresponding ESDOF systems. Based on linear and nonlinear time history results, the defined coefficient ($C_{1_{MDOF}}$ and $C_{1_{SDOF}}$) were estimated through averaging on each level of records for all selected building. The Coefficient of Variation (COV) is calculated to interpret the data dispersion appropriately. Global response of structures and the influence of SSI on displacement demand of structures are presented and discussed in the following chapter. For the sake of brevity, just MCE and some near field results are presented here. To reduce the runtime of analysis, for large structures, parallel processing in OpenSees software were utilized (McKenna and Fenves, 2008, Mckenna, 1997) which was significantly reduced the time of runs.

4.1 Global dynamic results

The results of nonlinear dynamic time history analysis of selected buildings considering vertical factor of safety (FSV) and strength reduction factor R=1.7(as sample of results) as well as fixed base are presented in Fig. 11. The results of maximum interstory drift ratio (IDR) versus number of stories indicated that, SSI reduces maximum roof drift in general and changed the location of maximum drift. Under severe SSI condition (e.g. lower shear wave velocity and FSV), the rotation of foundation is increased and structural hinge rotation demands decreased (see Fig. 13 and Fig. 14). This conclusion has been confirmed by another research (Karapetrou et al., 2015, Liu et al., 2015, Sáez et al., 2013).

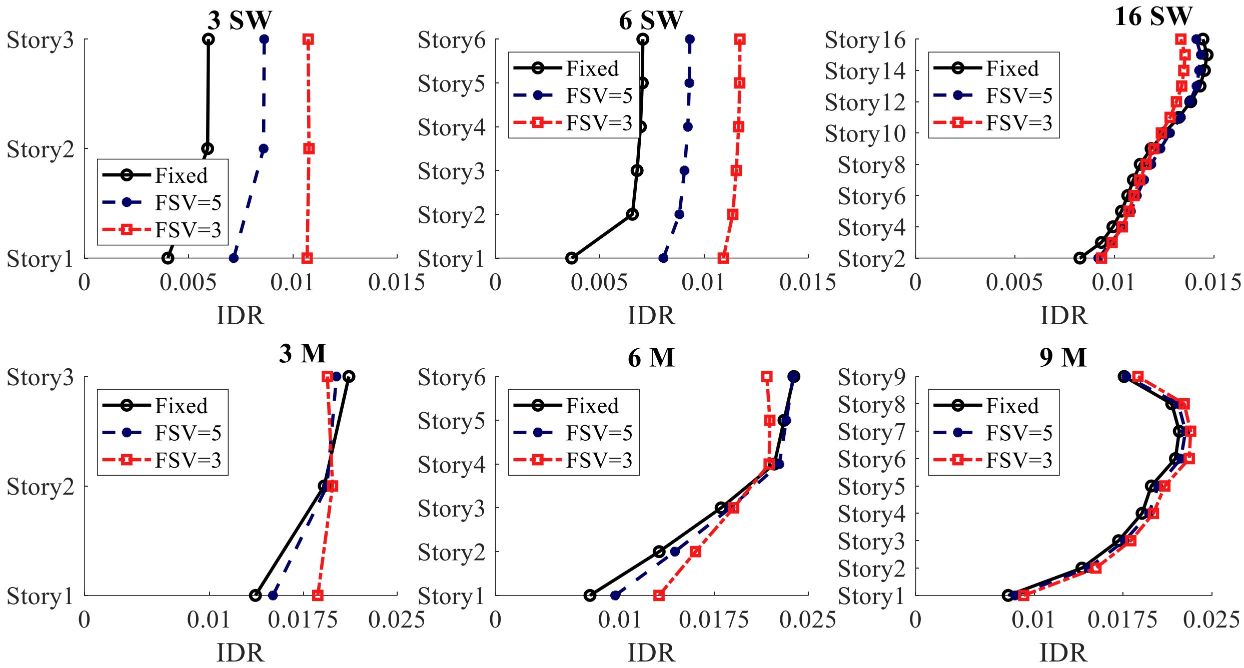


Fig. 11 Average of maximum Interstory drift ratio IDR Profile under MCE excitation for R=1.7 and different vertical factor of safety

The results of Figure 11 reveal that, in comparison with fixed-base building, the effects of SSI in shorter and stiffer structures are much more than taller and flexible structures, so that in 9-story moment frame building, the effects of SSI can be ignored, but in 16-story shear wall structures, especially lower FSV are suggested to considered (27 SW results have not shown here). In short structures with shear walls (up to 6 floors), the SSI effects cause a rigid rotation of wall at base due to rocking or uplifting of the foundation. This effect is such that the drift does not change in height, i.e., the wall rotates rigidly. In buildings with moment frames up to 6 floors, due to the use of spread foundations, there is a significant change in the distribution of the drift profile over the height of buildings, particularly with decreasing the FSV, although the amount of maximum drift does not change much. Fig. 12 presents the average of IDR of the buildings (regardless to location over the height) with different lateral resistance systems with FSV=3. It reveals that, as the strength reduction factor increases from 1.7 to 3.0, the difference between fixed-base and flexible-base drift ratios decreases which indicate the more contribution of structural response than soil-foundation behavior. However, for shorter and stiffer buildings (e.g. shear wall buildings), this differences are also meaningful. With increasing the height of structures (longer natural period), the effect of soil-foundation responses on maximum IDR decreases or insignificant. It is important to note that this concluding is regardless to the location of maximum IDR over the height of building and also effect of SSI on C_{1MDOF} which is important in displacement coefficient method.

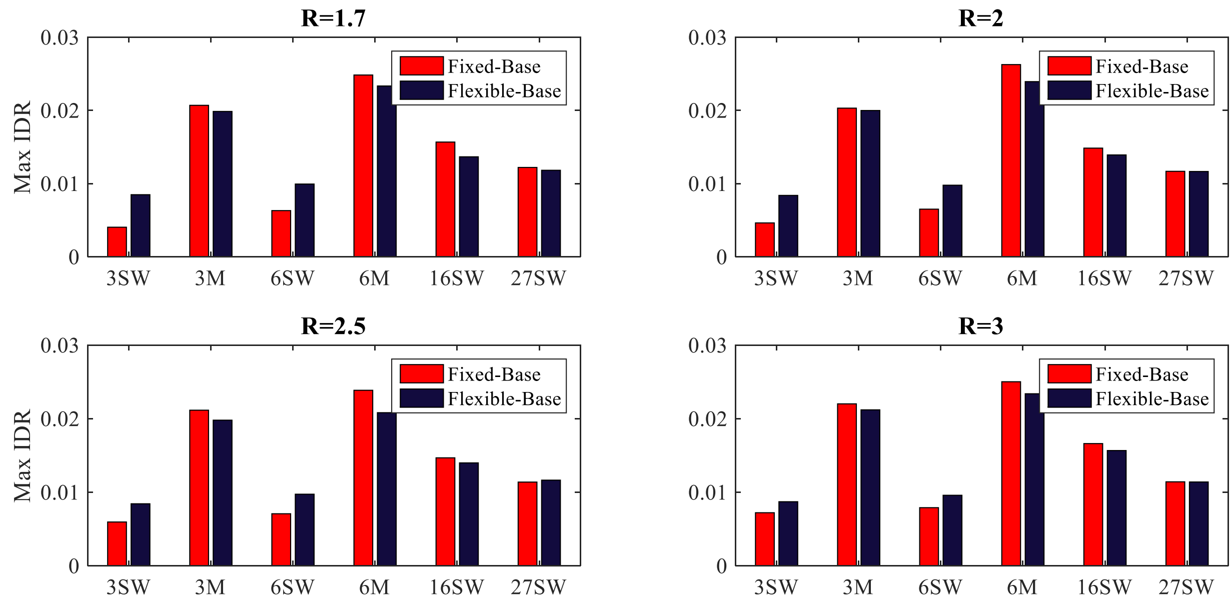


Fig. 12 the average of Maximum IDR comparison between fixed and flexible system and different R factors FSV=3

The contribution of foundation rotations due to rocking and uplifting was a main source of drift and displacement demands over the height of buildings particularly in short stiffer systems. To investigate it, Fig. 13 illustrates the values of mean of maximum foundation rotation for all shear wall buildings, different strength reduction factors(R), and FSV. The results show that the 3- and 6-story shear wall buildings (low rise) experience the maximum rotation at foundation level for all FSV. For 16- and 27-story shear wall buildings, no significant difference between rotation demands is seen (for all reduction factor and FSV). Generally, the rotation demands of buildings designed with lower FSV (smaller foundations) are more than others and as a consequence, smaller foundations experience both uplifting and sinking. With increasing the strength reduction factor from 1.7 to 3, the rotation demand of foundations is being to decrease; consequently, the displacement demands of buildings is compensated by rotation of plastic hinges of members (shear walls or frames) over the building height. To investigate such results, Fig. 14 presents the normalized rotation demands of shear wall plastic hinges for all buildings.

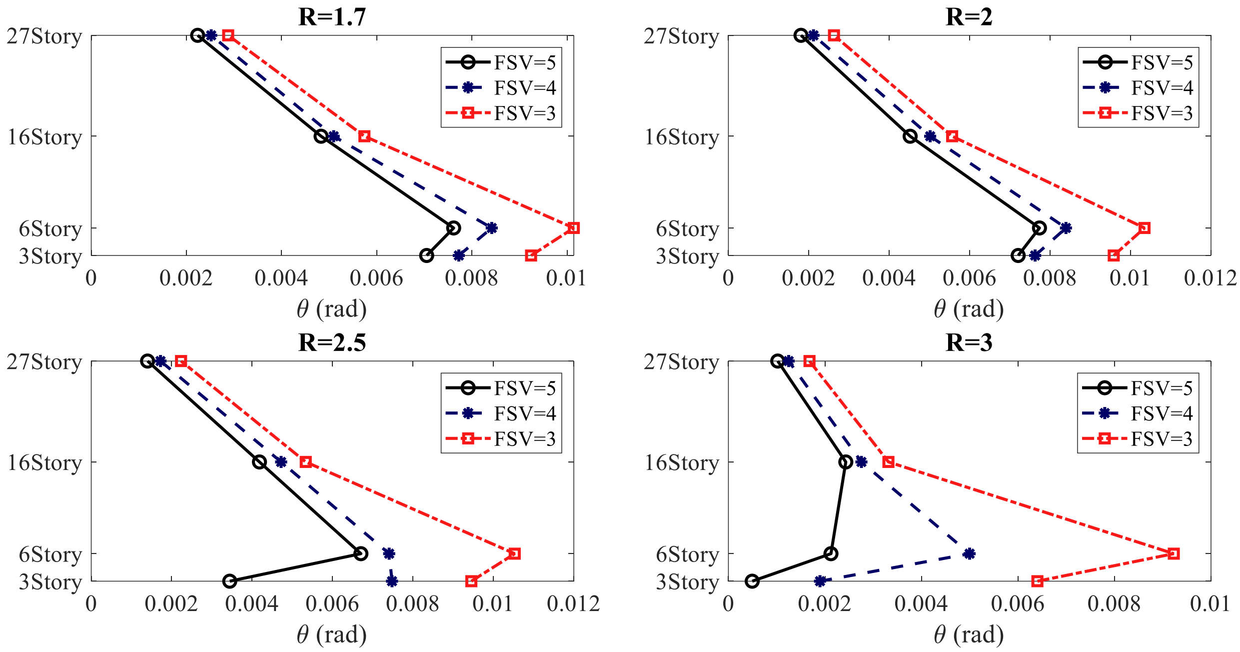


Fig. 13 the mean of maximum Foundation rotation for shear wall structures

The ratio of wall rotation at plastic hinge to its yielding rotation θ/θ_y (rotation ductility demand) for 3,6,16 and 27-story buildings is presented in Fig. 14 for different R factor and factor of safety (FSV). For better comparison, the results of fixed base are also presented here. The results show that for all buildings with lower reduction factors of 1.7-2.5, (larger lateral strength) no significant yielding of shear wall at base is found; hence, all structural displacement demands are provided by foundation's rotation, while in such cases, the fixed base buildings experience considerable plasticity. This finding is acceptable for all factors of vertical safeties. The low-rise shear wall building which are designed with higher strength reduction factor, experienced significant rotation demand regardless of their vertical factor of safety. In contrast, while higher shear wall buildings experience no, or minor rotation ductility demands at base (between 1-2). In this case, the behavior of building with the safety factor of 5 is near to fixed base responses. The comparison of the results of Fig. 13 and Fig. 14 shows that for buildings with R=1.7, 2, and 2.5 (except FSV=5), all sources of the displacement demands are provided by foundation rotations, while for R=3 both foundation and structural yielding are influential on amount of the displacement demand.

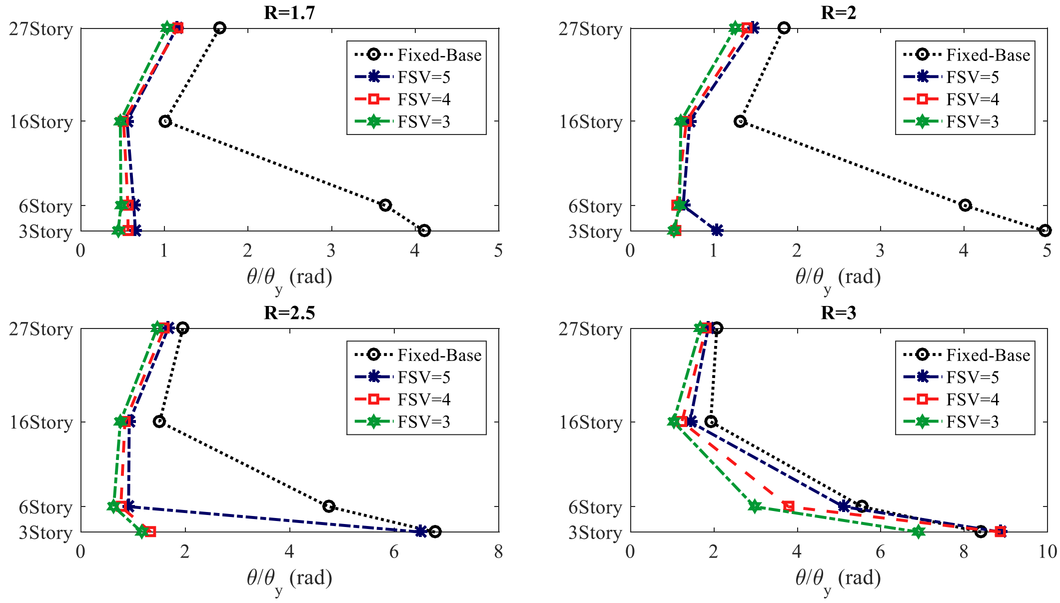


Fig. 14 the average of rotational ductility demands of shear walls at base for different FSV and strength Reductions factors

Ruiz-García and González (2014) assessed the distribution of displacement over the height of buildings for elastic and inelastic buildings built on soft soil site considering T/Tg, however, effects of SSI on the inelastic displacement versus elastic responses needs more investigation. In current research, based on ratio of T/Tg elastic and inelastic displacement distribution along height of buildings considering SSI is evaluated. T_g is the ground period which is estimated based on shear wave velocity of each record and depth of elastic medium beneath foundation. Fig. 15 as a sample of results presents the displacement profile of demands for three buildings in both elastic and inelastic cases with R=2.5 and FSV=3. These results derived from time history analysis of different records to provide four ranges of T/Tg. The results indicate that, if the T/T_g be greater than 1, then, the inelastic displacement will be smaller than the elastic system and for T/T_g smaller than 0.5, inelastic displacement is greater than elastic displacement. For T/Tg between 0.5 and 1, it seems the equal displacement principal is applicable and the inelastic and elastic displacements are nearly match, while for T/Tg less than 0.5, the equal energy principle seems to be eligible. Fig. 15 shows a linear trend for building with shear wall as their lateral resistance system. The linear trend is happened for this specific building (6SW) because of its lateral resistance system. The results showed that the rocking motion is dominant in this type of building and the responses are mostly influenced by rocking motion. However, the trend for other cases is not always linear and depends on height, strength reduction factor and foundation capacity could be different.

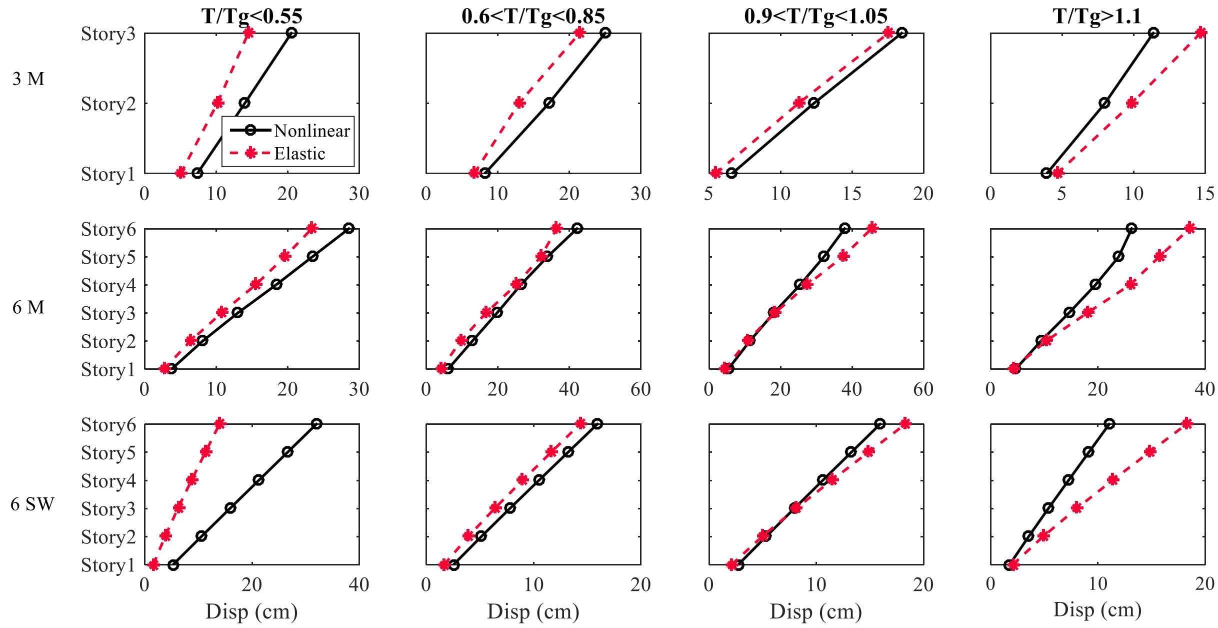


Fig. 15 Displacement Profile of 3 buildings subjected to 4 records with different predominant periods with R=2.5 and FSV=3

4.2 SSI effects on inelastic to elastic displacement ratio

The pure contribution of soil-structure-interaction on displacement amplification factor is introduced by dividing ${C_1}_{MDOF}$ (equation 2) to the same coefficient for fixed base structures. The displacement amplification factor for fixed MDOF system is defined in Eq. (6).

$$ {C_1}_{fixed} = \frac{\Delta_{fixed}^{inelastic}}{\Delta_{fixed}^{elastic}} \tag{6} $$

Where, $\Delta_{fixed}^{inelastic}$ and $\Delta_{fixed}^{elastic}$ are the buildings nodal displacement without SSI effect for inelastic and elastic condition respectively (see Fig. 1)The SSI effects on displacement amplification factor are defined as C_{soil} in current research. The mathematical form of this coefficient is presented in Eq. (7). In this coefficient all the effects of SSI on MDOF system are considered including to change in displacement demands due to shifting in effective period due to SSI, changing the values and location of maximum IDR, and rocking and uplifting of foundations. Fig. 16 illustrates the C_{soil} values versus story numbers for all buildings, different FSV, and two R samples of factors (1.7 and 3). The values of C_{soil} are the mean of the maximum responses resulted from the case by case time history analysis.

$$ C_{soil} = \frac{{C_1}_{MDOF}}{{C_1}_{fixed}} \tag{7} $$

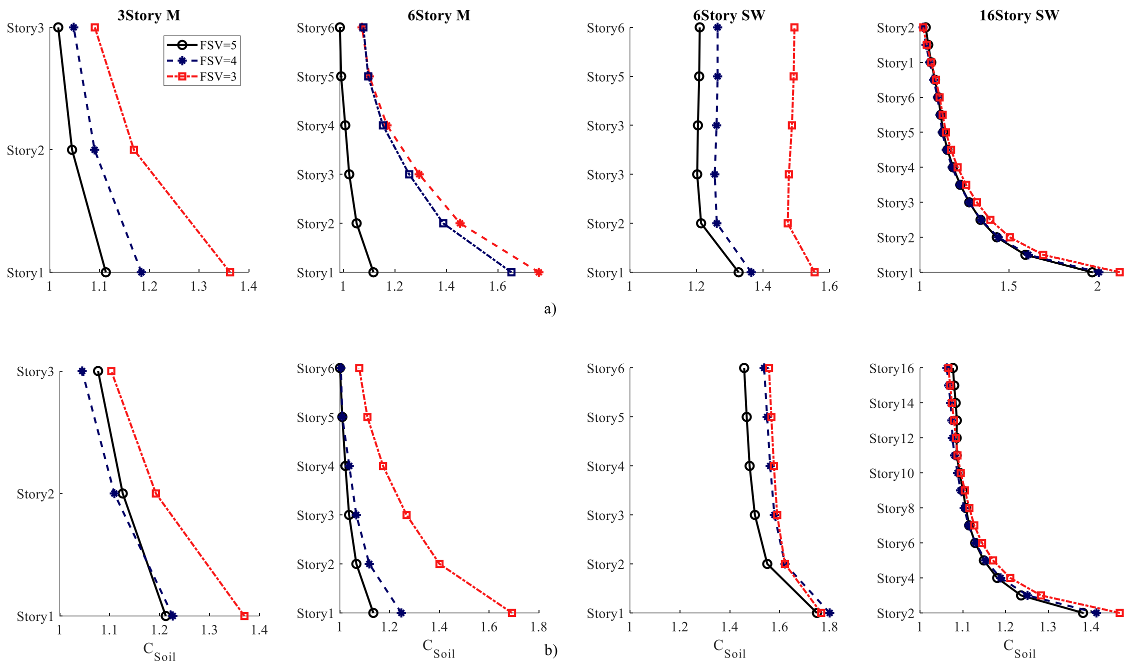


Fig. 16 SSI effects on displacement amplification factor profile C_{soil}, over the height of buildings a) R=1.7, b) R=3.0

Parameter of C_{Soil} and its variation along the height of buildings is influenced by effective periods, strength reduction factor(R), and vertical factor of safety (FSV). In general speaking, for taller or flexible buildings (16 SW or moment frames more than 6-story) all the values of C_{soil} are near to 1 at roof and increase up to 2.0 at first story. The results reveal that in taller buildings (higher natural periods), the variation in vertical factor of safety, FSV, does not lead to significant changes in C_{soil}, however, in stiffer and shorter buildings (3SW, 3M and 6SW structures) SSI effects have a remarkable contribution in changing the predicted displacement demands based on displacement coefficient method of ASCE-41(i.e. C1fixed). The distribution of C_{soil} along height of buildings (different periods) shows that the effect of SSI on inelastic displacement are more obvious in lower half parts of buildings, particularly for smaller FSV and higher buildings (e.g. 6M, 16SW buildings for FSV=3,4). For moment frame buildings the effects of SSI are being to decrease with increasing the height of buildings specially where the FSV is larger (the 6 story building with FSV=5 behave near to fixe-based). Fig. 16 also indicates that in buildings higher than 6 story the C_{soil} coefficient at roof is near to the one. Hence, it could be concluded that including the SSI effects on roof displacement (the method suggested by current code such as ASCE-41-17) can not appropriately estimates the effects of SSI over the height of building. This finding for shorter buildings (e.g. 3M and 3SW) is also applicable, especially for lower FSV (e.g. FSV=3 and less). For these type of buildings the effects of SSI on roof displacement are varies between 1-1.2 for 3M and 6M buildings and 1.5-1.8 for 3SW (therefore, These findings implied that, whether the common push-over analysis including SSI being set(just similar to model illustrated in Fig1a) for roof as target of displacement point, it can not present reliable displacement profile over the stories along the height of buildings both for taller buildings and particularly for short and stiffer buildings. It is important to note that the coefficient of C_1 suggested in ASCE41-17 is claimed that can predicts target displacement at any location over the height of structure and is not limited to roof. Although this problem is out of the scope of this research, however, it is a challenging concern. A short discussion at the last part of this research explores this issue.

4.3 Effect of FSV on $C_{1_{MDOF}}$

In current codes such as ASCE-41-17, the values of C_1C_2 was defined the anticipated displacement demand on elastic perfectly plastic SDOF systems based on Eq (8) and Eq (9), respectively. The comparison between $C_{1_{MDOF}}$ defined in present study (Eq (2)) and the C_1C_2 coefficient in ASCE-41, can present the differences between the MDOF including SSI responses and equivalent SDOF equated for MDOF responses. It is important to note that $C_{1_{MDOF}}$ and C_1C_2 have the same meaning while the location of comparison on building heights are different. The location of the former is the roof and for the latter is equivalent height of SDOF representing MDOF. Since the results indicated that the earthquake intensity had a minor influence on defined coefficient, just MCE and near-field results are presented in this study. Fig. 17 shows the effects of variation of FSV on responses of $C_{1_{MDOF}}$ for MCE records where the R factor is kept constant. In this figure, the horizontal axes are effective periods derived from analysis, and vertical axes are $C_{1_{MDOF}}$. For better comparison, the values of C_1C_2 calculated based on ASCE-41-17 are illustrated as well. The results show that the effects of FSV on responses are more significant where the reduction factors increase. In such cases both structural nonlinearity and foundation behavior (rocking and uplifting with soil yielding) are influential on values of $C_{1_{MDOF}}$ particularly in range of shorter periods (less than 1 sec). The results also reveal that there is no difference between the results with FSV more than 4. This could be inferred from the values of R-factors from 1.7-2.5. For effective periods longer than 1.5 seconds the vertical factor of safety is not dominant parameter for displacement amplification coefficient. The Fig. 17 illustrates that from effective period of 0.8 second, the values of $C_{1_{MDOF}}$ on average are less than 1.0. It means that the elastic displacements were more than inelastic displacements. This results were also reported by (Erduran and Kunnath, 2010) for fixed-based MDOF systems. The values of C_1C_2 plotted in Fig. 17 in comparison with results presented in this research shows an underestimation on shorter period range and overestimation on longer periods for all values of R-factors and FSV

$$C_1 = 1 + \frac{R-1}{aT^2} \tag{8}$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \tag{9}$$

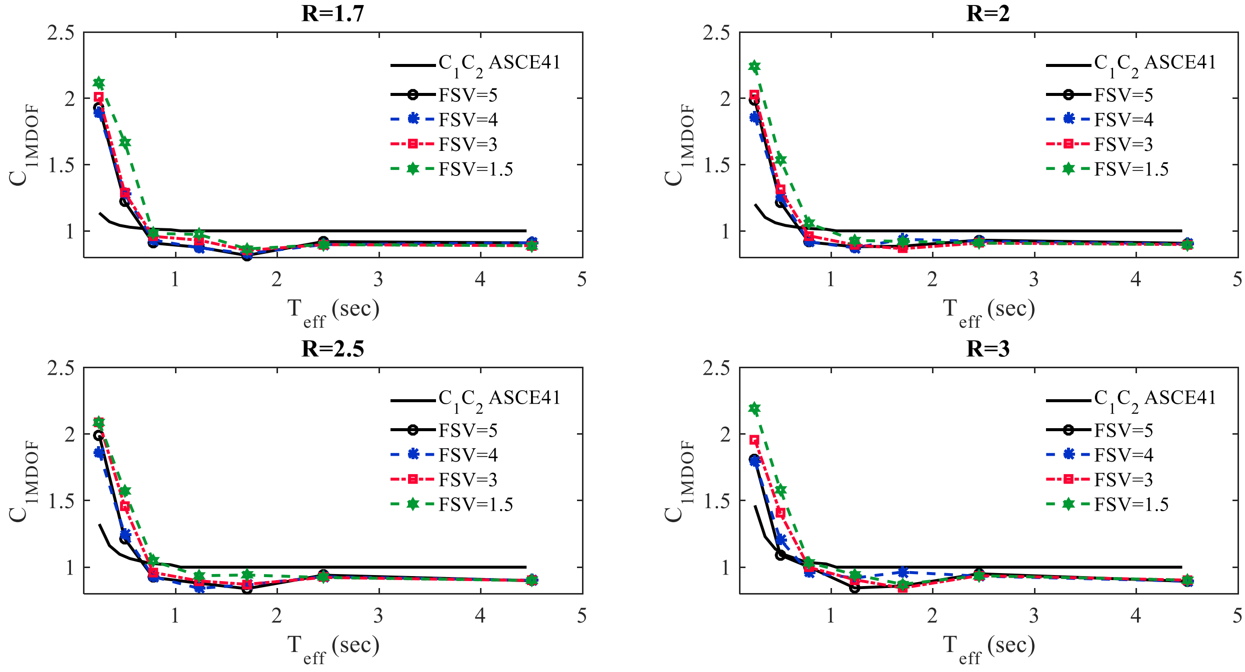


Fig. 17 the effects of ***FSV on C1$_{MDOF}$ under MCE records***

4.4 Proposing new relationship for prediction of $C_{1_{MDOF}}$

For MDOF flexible based system, $C_{1_{MDOF}}$, suggested in current research, can be implemented instead of C_1C_2 proposed in current code to evaluate target displacements. To predict an equation for resulted responses on $C_{1_{MDOF}}$, a nonlinear regression was carried out on all results from earthquakes and structures considering FSV and R-factors. This equation adopted from general form of C_1 suggested by ASCE-41-17 for SDOF systems, takes into account foundation flexibility, rocking and uplifting motion, strength and stiffness degradation, and strength reduction factors. Eq (10) expresses $C_{1_{MDOF}}$ coefficient general functions and represents its corresponding coefficient as function of R and FSV. a, b, and c equations are presented in Table 6. for MCE records.

$$C_{1_{MDOF}} = a + b/T^c \tag{10}$$

Fig. 18 illustrates Eq (10) and the C_1C_2 values in companion with analyzed data. Fig. 18 shows that with an acceptable degree of precision, the proposed equation can predict the $C_{1_{MDOF}}$ both in short and long periods. However, the values of C_1C_2 calculated based on ASCE-41-17 underestimated the inelastic displacement ratios in the range of periods less than 0.8 and 1 seconds, respectively for FSV more than 4 and less than 3. The best prediction between ASCE-41-17 and C_{1MDOF} for short period range was detected for buildings with higher R-factor and lager foundations (e.g. R=2.5,3 and FSV=4,5). The worst prediction of C_1C_2 in comparison C_{1MDOF} were for lower R-factors and smaller foundations (i.e. R=1.7, 2 and FSV=1.5, 3). The results also showed that for shorter period (less than 0.5 sec) the systems tend to be unstable and dynamic instability is anticipated. The coefficient of variations (COV) of the C_{1MDOF} was also illustrated in Fig. 19. Based on Fig. 19, the values of COV in most cases were less than 0.4 for shorter period ranges (less than 1 seconds) and less than 0.2 for the period more than 2 seconds (long period buildings) which were reasonable in assessments of structures under earthquake. However, in general, the trend of COV distribution followed the same trend of C_{1MDOF} on mean values which were more important as one can expect in average.

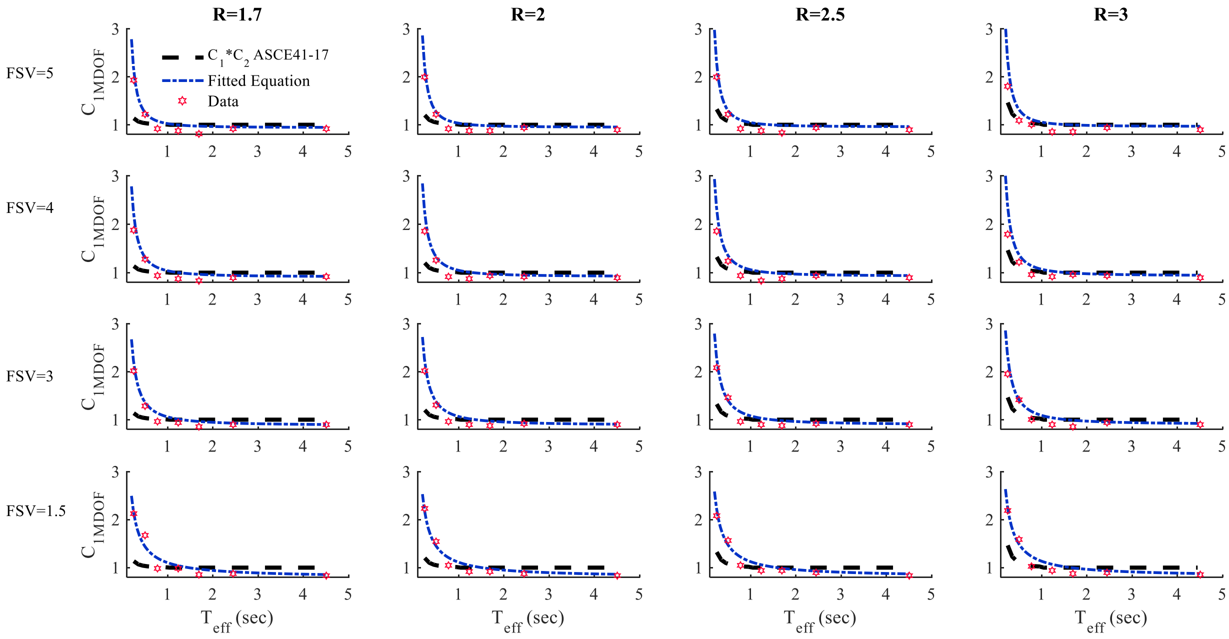


Fig. 18 Comparison between C_{1MDOF} and C_1C_2 equation presented in ASCE41-17

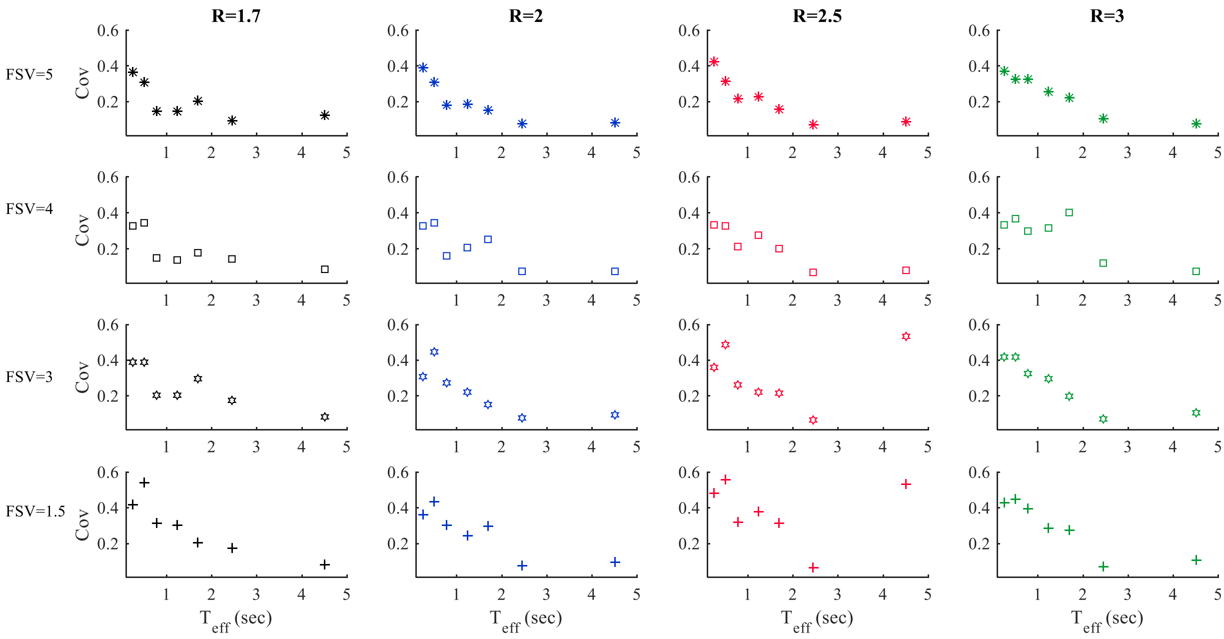


Fig. 19 C_{1MDOF} Coefficient of Variation

4.5 Estimation of modified amplification factor coefficient (C_m)

As discussed earlier, in calculation of $C_{1_{MDOF}}$, all structural nonlinearities including stiffness and strength degradations, yielding the soil beneath the foundation, foundation rocking and uplifting, P-delta effects, and higher mode effects are contributing to whole structural responses, while in ESDOF system the effects of P-delta, higher modes, and in cases, foundation rocking and uplifting cannot properly be modeled. Therefore, to extract such behavior distinguishing the equivalent SDOF and MDOF responses, the coefficient of C_m was defined in this research as Eq (4). The coefficient of C_m is one of the important parameters which the previous research on SDOF system have not been able to accurately address while it is inherently embedded in response of MDOF system. The importance of introducing this parameter is clear in calculating target displacement using the equations proposed using SDOF systems (e.g. ASCE-41, etc) and needs to be considered the effects of MDOF on target displacement. In this case, it is enough to calculate C_1C_2 from code and multiply it by C_m and C_0 if needed.

Fig. 20 shows all results of the C_m coefficient versus effective periods-which is the natural frequencies of structures by including SSI effect-for all vertical factors of safety and strength reduction for MCE records. The results indicated that, foundation's contribution in determining whole structural behavior and multi degree effects is more dominant in structures with higher strength (R=1.7). However, as foundation dimensions become smaller (decreasing the FSV), in shorter periods, the $C_m$increases and foundation effects on equal SDOF systems are more significant. In structures with higher strength reduction factor(R=3.0), the structural elements behavior controls the nonlinearity of building, and the effects of foundation and SSI are less important. However, in such cases, just the foundations with lower vertical factor of safety (FSV=1.5, 3) have relatively low impact on responses of MDOF including SSI. This trend is more sophisticated as the R factor is in the middle of range(R=2,2.5), because both soil and structure could experience nonlinear behavior simultaneously. As Fig. 20 demonstrated the trend of C_m is different in this case.

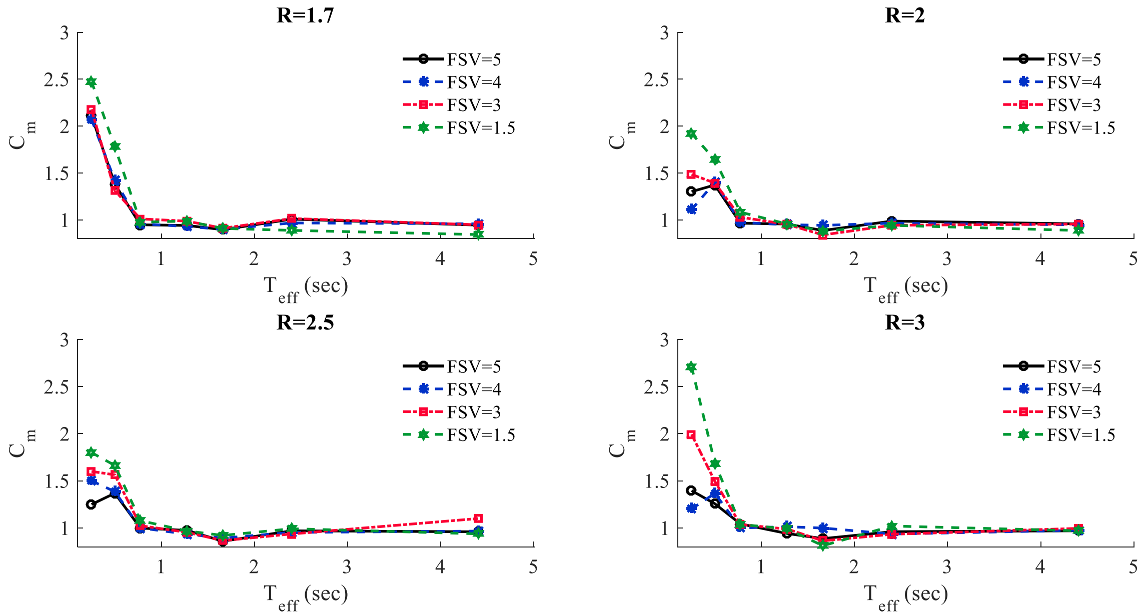


Fig. 20 mean of maximum values of Cm versus effective periods considering different FSV and R-factors

Nonlinear regression analyses using Levenberg-Marquardt method (Bates and Watts, 1988) is performed to derive an equation for C_m coefficient. The results indicate that the value of C_m approximately is independent of the earthquake intensity .Eq (10) estimates the C_m coefficient for structures. As described before, structural nonlinearity and especially the soil effect on roof displacement decreased significantly in systems with higher periods. Moreover, the trend of Fig. 20 and Fig. 21 suggests that the homographic form for C_m equation could be a rational and simple assumption which is more convenient for engineers to use. Table 6 shows the equations used for a, b, and c coefficient in Eq (10 and 11)

$$C_m = a + b/T^c \qquad (11)$$

Table 6 coefficient of a, b and c

Coeff	a	b	c
Eq (10)	$0.72 + 0.125\ln(FSV) + 0.04\ln(R)$	$0.38 - 0.19\ln(FSV) + 0.015\ln(R)$	$0.65 + 0.25(FSV) + 0.019(R)$
Eq (11)	$0.81 - 0.032\ln(FSV) + 0.034\ln(R)$	$0.31 - 0.04\ln(FSV) + 0.0035\ln(R)$	1.1

The results of predicted equation and data resulted from analysis on mean values are illustrated in Figure 23. The coefficient of variation (COV) of results is also presented in Fig. 22. The dispersion of results in the range of shorter periods (e.g. less than 1 second) is more than longer periods; however, generally the values of COV were acceptable in the field of seismic analysis. The trend of predicted equation in Fig. 21 seems acceptable particularly in the range of periods more than 0.4 second and for smaller reduction factor or larger foundations (higher FSV and lower R factor).

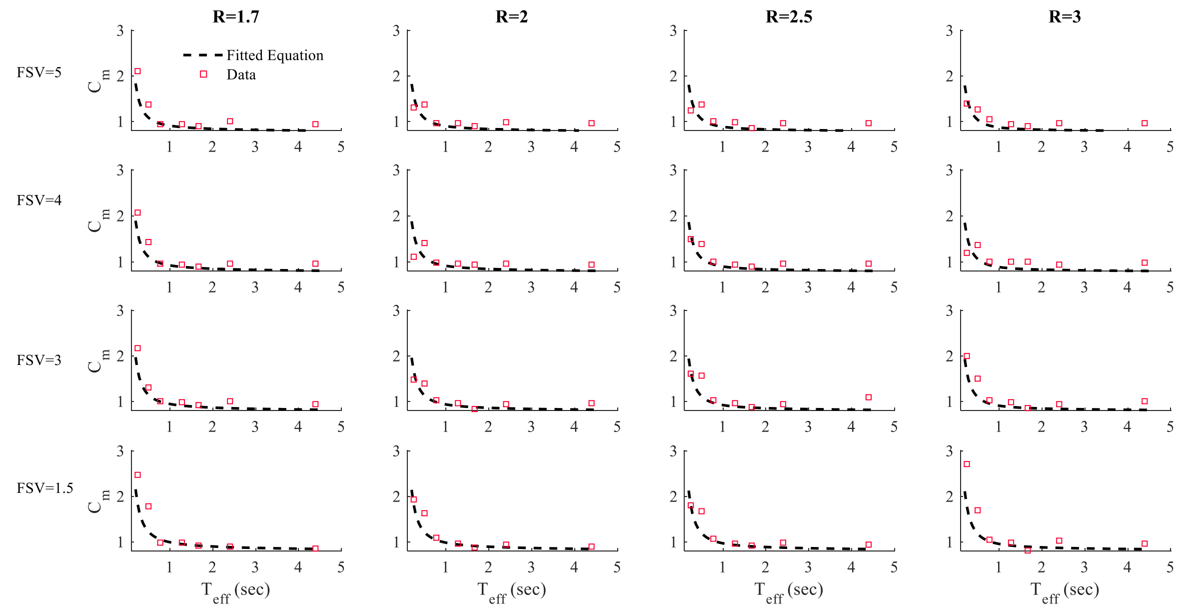


Fig. 21 C_m coefficient to modify displacement amplification factor for different types of FSV and R

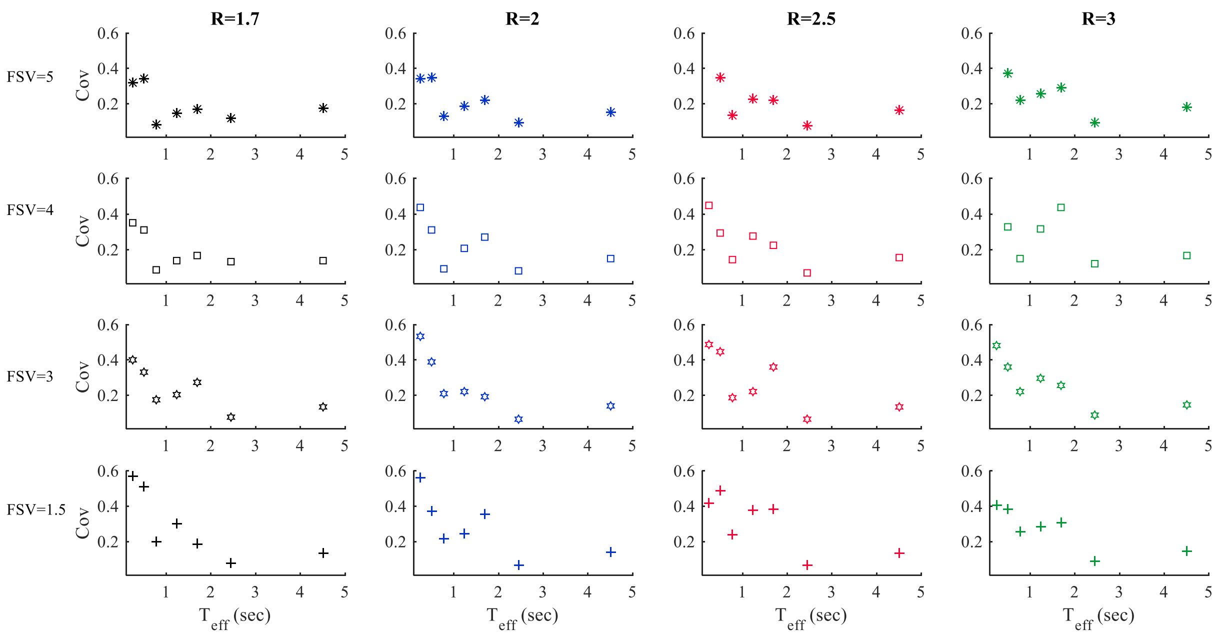


Fig. 22COV of C_m coefficient for different types of FSV and strength reduction factor

4.6 Pulse like effect

The derived responses on displacement amplification factor ($C_{1_{MDOF}}$) under near fields earthquake records are illustrated in Figure 25. The results showed that the same equation proposed for prediction of $C_{1_{MDOF}}$ (Eq (9)) can predict the effects of near field responses in the range of FSV and R-factor considered in this study.

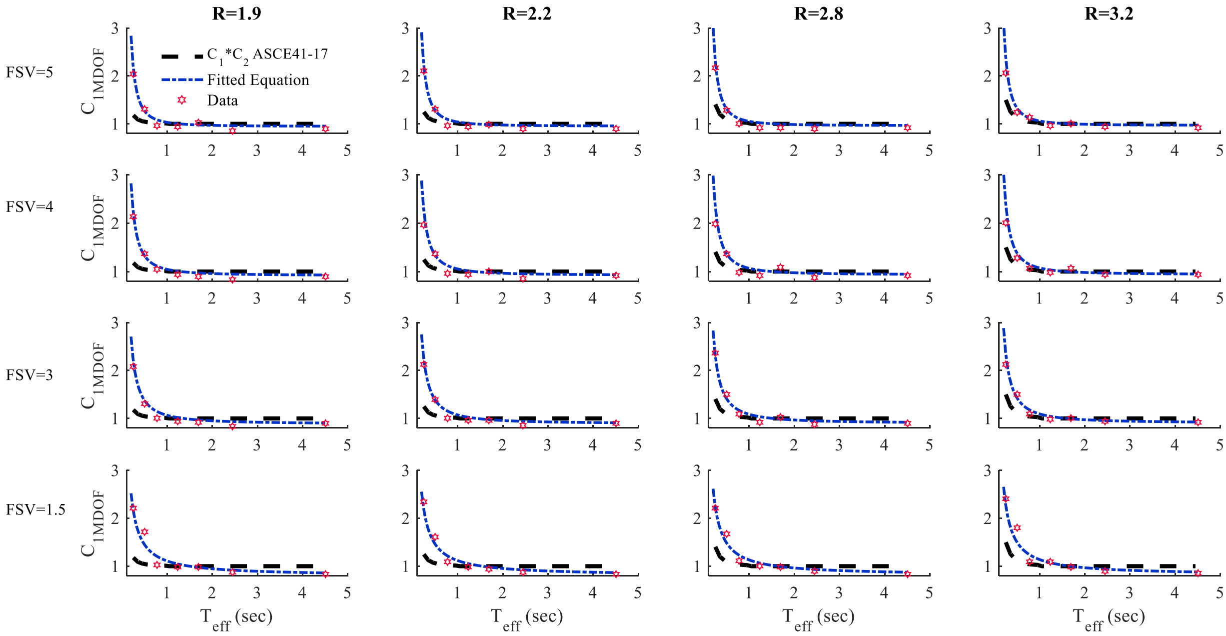


Fig. 23 C_{1MDOF} Coefficient for near field records for different types of FSV and strength reduction factor

4.7 Discussion

It is worth noting that, as mentioned before, all defined coefficients like $C_{1_{MDOF}}$ and C_m have been presented for the roof of all structures which are common in current static push-over analysis. However, the results of IDR and C_{Soil} as well as the results Fig. 16 presented here reveal that, the depends on location on height the values of drift ratios are affected by rocking and uplifting. Higher modes effects as well as strength degradations can be changed significantly in comparison with fixed- base responses. Although performing static nonlinear analysis, in which roof is selected as target point, can address parts of actual nonlinearity throughout the height of building, however that parts of displacements including to: structural degradations, higher mode effects, and nonlinear rotation due to foundation rocking and uplifting cannot explicitly be considered in analysis. As a result, using the roof as a reference point and pushing the building up to target displacement at roof cannot properly indicate the distribution of drift along the height of building accordingly. The ASCE-41-17 allows to use multiplying C_1C_2 by the term of C_0 to any points on building height. To investigate the difference between selecting the roof or any other story as target point, in this study first story results which is thought to be more influential in this field, are illustrated in Fig. 24 and Fig. 25 for C_{1MDOF} and C_m, respectively. Moreover, the values of C_1C_2 of ASCE41-17 corresponding to first story are also plotted. The results of Fig. 24 is different with adjusted C_1C_2 for first story and also with Fig. 17 and Fig. 20 both in terms of values and trend of prediction. The difference between the results determined for the first and top stories speculated that C_0 coefficient should be revised in such a way that to include the effects on structural and SSI nonlinearity instead of elastic mode shape participation factors.

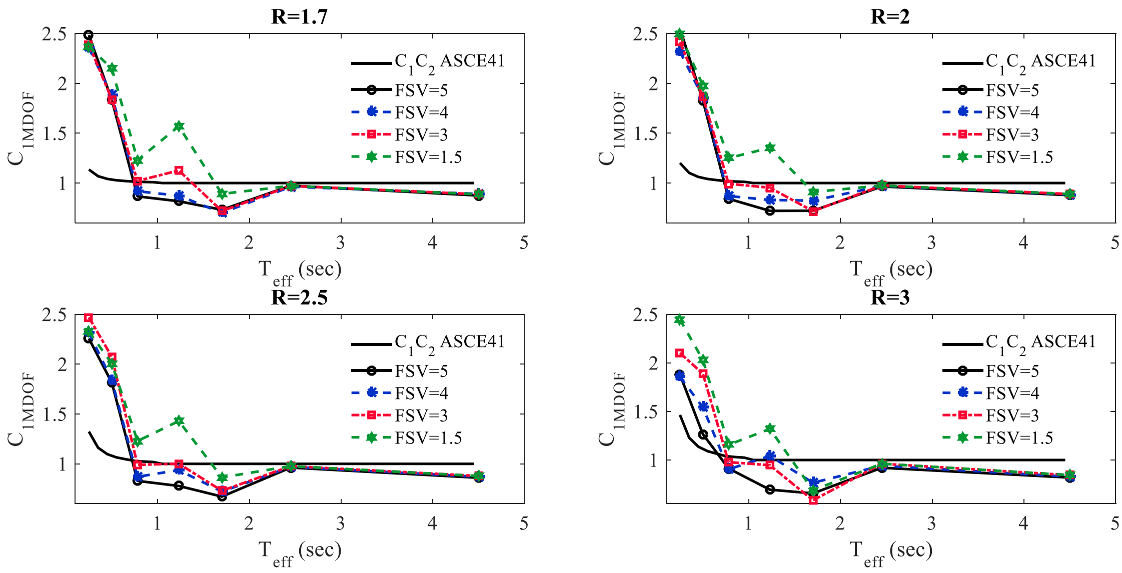


Fig. 24 C_{1MDOF} Coefficient for first story

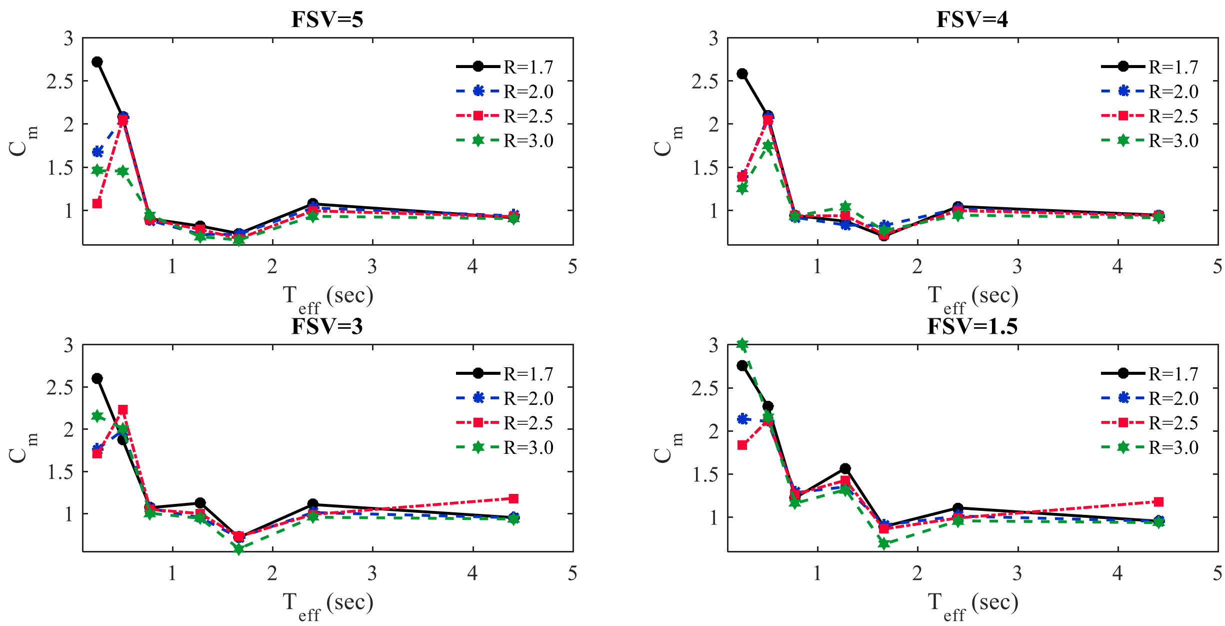


Fig. 25 C_m Coefficient to modify displacement amplification factor for first story

4.8 Implementing SSI effects on displacement Amplification factors for practical cases

Two methods are proposed to determine structural target displacement by considering all soil-structure nonlinearity. First method applies to the condition that dynamic characteristics of fixed base ESDOF system was available (Fig. 26 a). In this case, procedure illustrated in Fig. 26 can be followed. 1) Using the given dynamic characteristics of ESDOF system(effective period including SSI), the elastic displacement of ESDOF system is determined (Fig. 26 b and c). 2) The proposed C_1 coefficient in previous research like the one conducted by (Khanmohammadi and Mohsenzadeh, 2018) to account the SSI effects for SDOF system, is multiplied to the elastic displacement calculated in pervious step for the purpose of the estimation the inelastic displacement for SDOF system. 3) As shown in Fig. 26d and e, the other effects including higher mode effects, frame mechanism, and SSI effects on MDOF on roof target displacement, is implemented by multiplying C_m coefficient introduced in this study (Eq 11). The engineer can prepare a nonlinear model including to structural nonlinearity (plastic hinges) and soil nonlinearity beneath the footing and the push the structure to target displacement determined at roof.

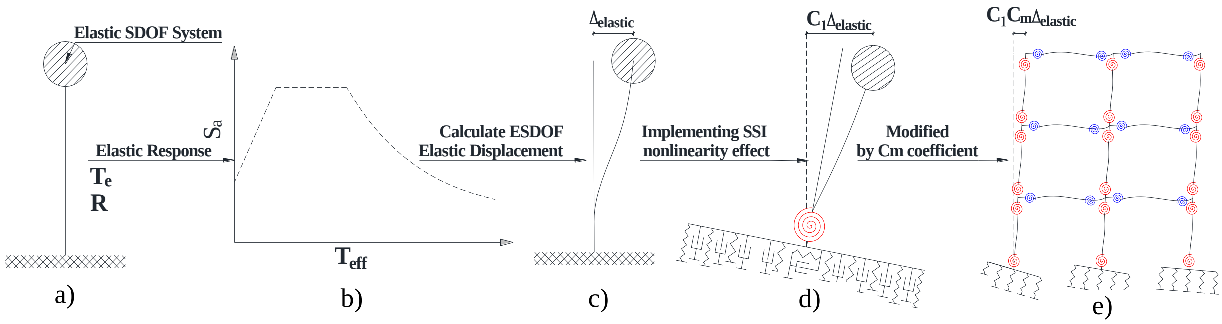


Fig. 26 Implementing SSI nonlinearity on displacement amplification factor using SDOF response

The second method is applied to the condition that the elastic MDOF model of the building is available (Fig. 27 a). In this situation 1) using the response spectra analysis (RSA), the elastic displacement of MDOF system would be determined (Fig. 27 b and c). 2) Estimating roof target displacement directly by multiplying, the elastic displacement calculated in step (1) by $C_{1_{MDOF}}$ coefficient introduced in (Eq 10). It is worth noting that the foundation flexibility needed to be considered in the linear model by modelling the relevant elastic springs beneath the foundation. since the $C_{1_{MDOF}}$ coefficient only contains the effect of structural and soil nonlinearity. As an alternative method, the engineer can determine the fundamental period of building from a fixed-base model and modify for SSI effects based on procedure suggested in ASCE41-17 or

FEMA440 and follow the same procedure which is proposed in Figure 27c and 27d. Regarding previous discussions, the strength degradation and stiffness deterioration coefficient have been inherently considered in the $C_{1_{MDOF}}$ coefficient thus, C_2 coefficient can be ignored.

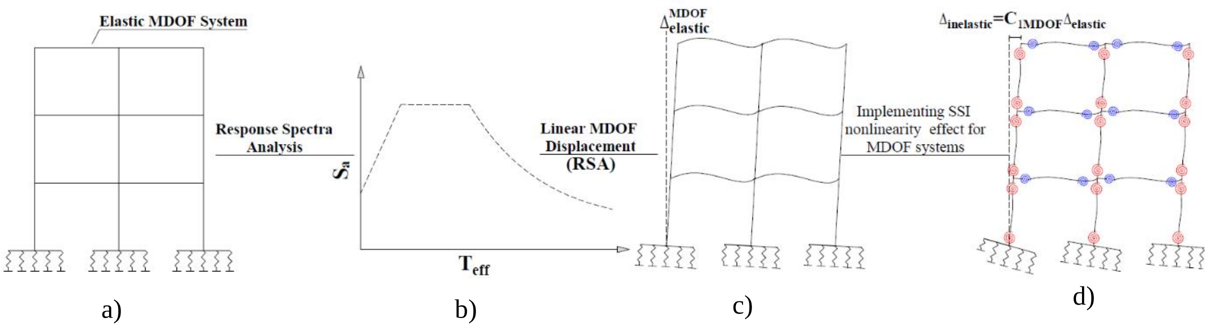


Fig. 27 Implementing SSI nonlinearity on displacement amplification factor using $C_{1_{MDOF}}$

5 Conclusions

The purpose of this study is to investigate the effects of SSI on displacement amplification factor of MDOF system introduced in current codes (e.g. ASCE41-17). A new index, adopted from SDOF system, introduced on MDOF systems named C_{1MDOF} which normally covered the source of nonlinearity from elastic and inelastic SSI, higher mode effects, strength and stiffness degradations and distribution of plastic hinges over the buildings. The same procedure is implemented for equivalent SDOF of the selected buildings (i.e. C_{1SDOF}) and ratio of C_m is defined by dividing $C_{1MDOF}\ toC_{1SDOF}$. which covers all behavior of MDOF systems that cannot be addressed in ESDOF. The effects of SSI behavior on responses of MDOF buildings were also investigated using the introduced index C_{soil} ($C_{soil} = \frac{C_{1MDOF}}{C_{1_{fixed}}}$). The results are compared with C_1C_2 introduced in ASCE 41-17 and based on the derived results using nonlinear regression, two relationships were proposed for C_{1MDOF} and C_m for practical cases. Finally, two methods are proposed for implementing the SSI effects on displacement amplification factor. The main results of this study are as following:

- SSI effects on responses can significantly change the values and distribution of drift ratio over the height of low and mid- rise (3 and 6 story buildings MRF or shear wall) of buildings particularly in lower strength reduction factor and FSV. In these cases, the rotation of foundation due to uplifting is the main source of drift demands.

- The effects of SSI on responses of shear wall buildings increases with increasing the height of buildings and strength reduction factors (e.g. more than 2.5) particularly in FSV more than 4.
- The effects of SSI on inelastic displacements versus elastic displacement over the height of buildings showed that: for the range T/T_g greater than 1, the inelastic displacements are smaller than the elastic, while for T/T_g smaller than 0.5, inelastic displacements are more than elastic displacements. For T/Tg between 0.5 and 1, the equal displacement principal is applicable.
- The results showed that, the effects of SSI on inelastic to elastic displacement ratio ($C_{1_{MDOF}}$) respect to the same ration in fixed-based buildings (C_{Soil}) can increase the responses up to twice as much. This concluding is more sensitive to values of FSV, particularly in low-rise buildings (e.g. 3 and 6 story) in comparison with higher buildings (e.g.16 and 27 story).
- The comparison of results on $C_{1_{MDOF}}$ considering SSI with values of C_1C_2 suggested by ASCE-41-17 shows that the values of C_1C_2 underestimate the responses in shorter period range and overestimate for longer periods for all values of R-factors and FSV.
- The comparisons between proposed equations for C1MDOF and Cm with results of analysis considering SSI on studied buildings showed an acceptable degree of prediction on mean values and COV.
- The results of comparison between responses of first and roof of buildings considering SSI showed that the coefficient of C_0 introduced in ASCE-41 17 should include the effects of structural and SSI nonlinearity instead of elastic mode participation factors. The C_0 coefficient could not cover all features of MDOF system.
- Two practical methods were proposed to considering SSI effects on displacement Amplification factors.

References

(BHRC), B. A. H. R. C. 2013. Standard No. 2800. Iranian Code of Practice for Seismic Resistant Design of Buildings, 4th

Edition, 2013. Iran.

ACI, A. 2014. 318–14. Building Code Requirements for Structural Concrete. *American Concrete Institute, Farmington Hills, Michigan*.

ANASTASOPOULOS, I., GEORGARAKOS, T., GEORGIANNOU, V., DROSOS, V. & KOURKOULIS, R. 2010. Seismic performance of bar-mat reinforced-soil retaining wall: Shaking table testing versus numerical analysis with modified kinematic hardening constitutive model. *Soil Dynamics and Earthquake Engineering,* 30**,** 1089-1105.

ANASTASOPOULOS, I., KOURKOULIS, R., GELAGOTI, F. & PAPADOPOULOS, E. 2012. Rocking response of SDOF systems on shallow improved sand: An experimental study. *Soil Dynamics and Earthquake Engineering,* 40**,** 15-33.

ANASTASOPOULOS, I., LOLI, M., GEORGARAKOS, T. & DROSOS, V. 2013. Shaking table testing of rocking—isolated bridge pier on sand. *Journal of Earthquake Engineering,* 17**,** 1-32.

Asadi-Ghoozhdi, H., & Attarnejad, R. (2020). The Effect of Nonlinear Soil–Structure Interaction on the Ductility and Strength Demands of Vertically Irregular Structures. *International Journal of Civil Engineering*, *18*(11), 1209–1228. https://doi.org/10.1007/s40999-020-00529-0

ATC, A. 1996. 40, Seismic evaluation and retrofit of concrete buildings. *Applied Technology Council, report ATC-40. Redwood City*.

BAKER, J. W., LIN, T., SHAHI, S. K. & JAYARAM, N. 2011. New ground motion selection procedures and selected motions for the PEER transportation research program. *Peer Report 2011,* 3.

BATES, D. M. & WATTS, D. G. 1988. *Nonlinear regression analysis and its applications*, Wiley New York.

BOULANGER, R. 2000. The PySimple1, TzSimple1, and QzSimple1 material models, documentation for the OpenSees platform. *URL: http://opensees. berkeley. edu*.

CARBONARI, S., DEZI, F. & LEONI, G. 2012. Nonlinear seismic behaviour of wall-frame dual systems accounting for soil–structure interaction. *Earthquake Engineering & Structural Dynamics,* 41**,** 1651-1672.

CHOPRA, A. K. & GOEL, R. K. 2000. Evaluation of NSP to estimate seismic deformation: SDF systems. *Journal of Structural Engineering,* 126**,** 482-490.

COUNCIL, A. T. 2010. Modeling and acceptance criteria for seismic design and analysis of tall buildings.

DROSOS, V., GEORGARAKOS, T., LOLI, M., ANASTASOPOULOS, I., ZARZOURAS, O. & GAZETAS, G. 2012. Soil-foundation-structure interaction with mobilization of bearing capacity: Experimental study on sand. *Journal of Geotechnical and Geoenvironmental Engineering,* 138**,** 1369-1386.

ENGINEERS, A. S. O. C. 2010. ASCE7-10: Minimum Design Loads for Buildings and Other Structures. ASCE, Structural Engineering Institute.

ENGINEERS, A. S. O. C. Seismic Evaluation and Retrofit of Existing Buildings: ASCE/SEI, 41-17. 2017. American Society of Civil Engineers.

ERDURAN, E. & KUNNATH, S. K. 2010. Enhanced displacement coefficient method for degrading multi-degree-of-freedom systems. *Earthquake Spectra,* 26**,** 311-326.

FACCIOLI, E., VANINI, M. & PAOLUCCI, R. 1999. *TRISEE: 3D site effects and soil-foundation interaction in earthquake and vibration risk evaluation*, European commission. Directorate-general 12. Science, research and development.

FAJFAR, P. & FISCHINGER, M. N2-A method for non-linear seismic analysis of regular buildings. Proceedings of the ninth world conference in earthquake engineering, 1988. 111-116.

FEMA, A. 2005. 440, Improvement of nonlinear static seismic analysis procedures. *FEMA-440, Redwood City*.

G., Arboleda-Monsalve Luis, Mercado Jaime A., Terzic Vesna, and Mackie Kevin R. 2020. "Soil–Structure Interaction Effects on Seismic Performance and Earthquake-Induced Losses in Tall Buildings." *Journal of Geotechnical and Geoenvironmental Engineering* 146(5): 4020028. https://doi.org/10.1061/(ASCE)GT.1943-5606.0002248.

GAJAN, S., KUTTER, B. L., PHALEN, J. D., HUTCHINSON, T. C. & MARTIN, G. R. 2005. Centrifuge modeling of load-deformation behavior of rocking shallow foundations. *Soil Dynamics and Earthquake Engineering,* 25**,** 773-783.

GANJAVI, B. & HAO, H. 2012. Effect of structural characteristics distribution on strength demand and ductility reduction factor of MDOF systems considering soil-structure interaction. *Earthquake Engineering and Engineering Vibration,* 11**,** 205-220.

GAZETAS, G. 1991. Formulas and charts for impedances of surface and embedded foundations. *Journal of geotechnical engineering,* 117**,** 1363-1381.

GAZETAS, G., APOSTOLOU, M. & ANASTASOPOULOS, J. Seismic uplifting of foundations on soft soil, with examples from Adapazari (Izmit 1999 earthquake). BGA International Conference on Foundations: Innovations, observations, design and practice: Proceedings of the international conference organised by British Geotechnical Association and held in Dundee, Scotland on 2–5th September 2003, 2003. Thomas Telford Publishing, 37-49.

GHANNAD, M. & JAHANKHAH, H. 2007. Site-dependent strength reduction factors for soil-structure systems. *Soil Dynamics and Earthquake Engineering,* 27**,** 99-110.

GHANNAD, M. A. & JAFARIEH, A. H. 2014. Inelastic displacement ratios for soil–structure systems allowed to uplift. *Earthquake Engineering & Structural Dynamics,* 43**,** 1401-1421.

GHOLAMREZATABAR, A., GHODRATI AMIRI, G., SHAYANFAR, M. A. & GANJAVI, B. 2017. Estimation of inelastic displacement factor of soil-shallow-foundation MDOF systems incorporating higher modes effect. *The Structural Design of Tall and Special Buildings,* 26**,** e1402.

HARDEN, C. W. 2005. *Numerical modeling of the nonlinear cyclic response of shallow foundations*, Pacific Earthquake Engineering Research Center.

HASELTON, C. B., GOULET, C. A., MITRANI-REISER, J., BECK, J. L., DEIERLEIN, G. G., PORTER, K. A., STEWART, J. P. & TACIROGLU, E. 2008. An assessment to benchmark the seismic performance of a code-conforming reinforced-concrete moment-frame building. *Pacific Earthquake Engineering Research Center*.

HOUSNER, G. W. 1963. The behavior of inverted pendulum structures during earthquakes. *Bulletin of the seismological society of America,* 53**,** 403-417.

IBARRA, L. F., MEDINA, R. A. & KRAWINKLER, H. 2005. Hysteretic models that incorporate strength and stiffness deterioration. *Earthquake engineering & structural dynamics,* 34**,** 1489-1511.

JAFARIEH, A. H. & GHANNAD, M. A. 2014. The effect of foundation uplift on elastic response of soil-structure systems.

Karakostas, C., Morfidis, K., Rovithis, E., & Theodoulidis, N. (2021). Soil-structure interaction effects on the seismic response of a public building in Lefkas, Greece. *Bulletin of Earthquake Engineering*. https://doi.org/10.1007/s10518-021-01278-8

KARAPETROU, S., FOTOPOULOU, S. & PITILAKIS, K. 2015. Seismic vulnerability assessment of high-rise non-ductile RC buildings considering soil–structure interaction effects. *Soil Dynamics and Earthquake Engineering,* 73**,** 42-57.

KAWASHIMA, K. & HOSOIRI, K. Rocking response of bridge columns on direct foundations. Proceedings, 2003.

KHANMOHAMMADI, M. & MOHSENZADEH, V. 2018. Effects of foundation rocking and uplifting on displacement amplification factor. *Earthquake Engineering and Engineering Vibration,* 17**,** 511-525.

KHOSHNOUDIAN, F., AHMADI, E. & NIK, F. A. 2013. Inelastic displacement ratios for soil-structure systems. *Engineering Structures,* 57**,** 453-464.

KHOSHNOUDIAN, F. & BEHMANESH, I. 2010. Evaluation of FEMA-440 for including soil-structure interaction. *Earthquake engineering and engineering vibration,* 9**,** 397-408.

LIU, W., HUTCHINSON, T. C., GAVRAS, A. G., KUTTER, B. L. & HAKHAMANESHI, M. 2015. Seismic Behavior of Frame-Wall-Rocking Foundation Systems. I: Test Program and Slow Cyclic Results. *Journal of Structural Engineering,* 141**,** 04015059.

LYSMER, J. & KUHLEMEYER, R. L. 1969. Finite dynamic model for infinite media. *Journal of the Engineering Mechanics Division,* 95**,** 859-878.

MANDER, J. B., PRIESTLEY, M. J. & PARK, R. 1988. Theoretical stress-strain model for confined concrete. *Journal of structural engineering,* 114**,** 1804-1826.

MCKENNA, F., FENVES, G. & FILIPPOU, F. 2010. OpenSees.

MCKENNA, F. & FENVES, G. L. 2008. Using the OpenSees interpreter on parallel computers. *Network for earthquake engineering simulations*.

MCKENNA, F. T. 1997. *Object-oriented finite element programming: frameworks for analysis, algorithms and parallel computing*, University of California, Berkeley.

MEEK, J. W. 1975. Effects of foundation tipping on dynamic response. *Journal of the Structural Division,* 101.

MENEGOTTO, M. Method of analysis for cyclically loaded RC plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending. Proc. of IABSE symposium on resistance and ultimate deformability of structures acted on by well defined repeated loads, 1973. 15-22.

MIRANDA, E. & RUIZ-GARCÍA, J. 2002. Evaluation of approximate methods to estimate maximum inelastic displacement demands. *Earthquake engineering & structural dynamics,* 31**,** 539-560.

PECKER, A. Aseismic foundation design process, lessons learned from two major projects: the Vasco de Gama and the Rion Antirion bridges. ACI International Conference on Seismic Bridge Design and Retrofit. La Jolla, California, 2003.

RAYCHOWDHURY, P. 2008. *Nonlinear winkler-based shallow foundation model for performance assessment of seismically loaded structures*, University of California, San Diego.

RAYCHOWDHURY, P. & HUTCHINSON, T. C. 2009. Performance evaluation of a nonlinear Winkler-based shallow foundation model using centrifuge test results. *Earthquake Engineering & Structural Dynamics,* 38**,** 679-698.

RUIZ-GARCÍA, J. & GONZÁLEZ, E. J. 2014. Implementation of Displacement Coefficient method for seismic assessment of buildings built on soft soil sites. *Engineering Structures,* 59**,** 1-12.

RUIZ-GARCÍA, J. & MIRANDA, E. 2007. Probabilistic estimation of maximum inelastic displacement demands for performance-based design. *Earthquake engineering & structural dynamics,* 36**,** 1235-1254.

SÁEZ, E., LOPEZ-CABALLERO, F. & MODARESSI-FARAHMAND-RAZAVI, A. 2013. Inelastic dynamic soil–structure interaction effects on moment-resisting frame buildings. *Engineering structures,* 51**,** 166-177.

SHIMAZAKI, K. & SOZEN, M. A. 1984. *Seismic drift of reinforced concrete structures*, Hazama-gumi.

STEWART, J. P., FENVES, G. L. & SEED, R. B. 1999. Seismic soil-structure interaction in buildings. I: Analytical methods. *Journal of Geotechnical and Geoenvironmental Engineering,* 125**,** 26-37.

VELETSOS, A. & NEWMARK, N. M. Effect of inelastic behavior on the response of simple systems to earthquake motions. 1960. Department of Civil Engineering, University of Illinois.

VENTURE, N. C. J. 2012. Soil-structure interaction for building structures. *Nist Gcr***,** 12-917.

VESIC, A. S. 1973. Analysis of ultimate loads of shallow foundations. *Journal of Soil Mechanics & Foundations Div,* 99.

WOLF, J. 1985. *Dynamic soil-structure interaction*, Prentice Hall, Inc.

YIM, C.-S. & CHOPRA, A. K. 1983. *Effects of transient foundation uplift on earthquake response of structures*, University of California, Earthquake Engineering Research Center.