Numerical parametric study on inelastic foundation-building system

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SUMMARY:
Numerous experimental and numerical efforts have demonstrated that a rocking foundation can possess a re-centering tendency, and inherently absorb a large amount of seismic energy at the foundation-soil interface when the capacity of the foundation is mobilized. However, when integrating a rocking foundation into a building system, there is still a need to explore how this behavior dynamically interacts with superstructure elements, which are traditionally designed with inelastic behavior to absorb seismic energy. Investigating this sharing of seismic demand is the subject of a recent centrifuge test program conducted by the University of California, San Diego and Davis at the UC Davis-NEES centrifuge facility. This paper presents pre-test numerical parametric analysis, which aims to identify a structural model with an improved seismic performance. Several key parameters have been proposed to characterize the elastic and inelastic behavior of the foundation-building system. The sensitivity of each parameter is then evaluated through using numerical simulations conducted in OpenSees. Results show that equivalence between structural fuse yield coefficient and foundation rocking yield coefficient may lead to a better controlled seismic-resistant performance.

Keywords: rocking foundation, structural fuse, seismic analysis, numerical modeling

1. INTRODUCTION AND BACKGROUND

1.1. Background

Performance-based seismic design encourages the development of nonlinear inelastic behavior of designated structural components during a moderate or intense earthquake shaking. Nonlinear soil-foundation-structure interaction (SFSI) effect, particularly under the circumstance when the foundation is allowed to uplift or its capacity is mobilized, has been gradually acknowledged as an effective and natural technique for incorporation into performance based seismic design strategies. Adoption of nonlinear SFSI supports dissipation of seismic vibrational energy and thereby can reduce seismic demand to the superstructure. Although several foundation component tests uniformly verify these benefits, current seismic design guidelines have not yet considered and directed this footing-rocking mechanism into system-level design. In addition, inadequacy of system-level experimental evidence impedes its further development and application. Hence, a comprehensive investigation is desired in this regard, to experimentally and numerically assess the seismic performance of foundation-rocking at a system-level.

1.2. Previous work on foundation rocking

The earliest findings from Housner (1963) indicate that the rocking behavior of blocks benefit seismic stability. Furthermore, investigators in New Zealand in the 1970s (e.g. Bartlett 1976, Wiessing 1979) conducted a series of 1-g experiments on shallow foundation-rigid shear-wall systems with various sizes and factors of safety of the footing. Their testing results further substantiate the merit of a rocking footing. To date researchers across the United States and Europe have conducted a variety of test programs, centrifuge and 1-g experiments to investigate the mobilization of soil-foundation capacity (see e.g. Rosebrook, 2001; Gajan and Kutter 2008; Anastasopoulos et al., 2010; Deng et al., 2012; Hakhamaneshi et al., 2012; Gelagoti et al., 2012; Drosos et al., 2012). Since the centrifuge test is able to create equal confining stress level compared with prototype model, the stress-dependent soil nonlinear behavior can thereby be reasonably captured. A common strategy in centrifuge testing has involved placing a number of simple structural assemblies, such as a shearwall-footing system or a single degree-of-freedom (1-DOF) lollipop structural model supported on a shallow foundation element. Foundations have been designed with various types of geometries, vertical factors of safety (FS_v), and different types of soil condition. In general, centrifuge tests concur that irrespective of the soil condition, design of the footing element with a relatively high FSv (around 10) will induce a better rocking performance without significant settlement (Deng et al., 2012; Hakhamaneshi et al., 2012). A joint research program in Europe and Japan undertook large-scale 1-g experimental tests on shallow foundations (e.g. Taylor and Crewe 1996, Negro et al. 2000, Paolucci et al. 2008 and Shirato et al. 2008), which simultaneously verified the advantages of yielding rocking foundations. Given the demonstrated benefits of the rocking foundation component, the merits of integrating it into the overall soil-structure system needs to be studied on level playing field with other inelastic mechanisms. To the authors' knowledge, only a handful of studies have undertaken system-level studies in this regard (Chang et al., 2007; Trombetta et al., 2012; Deng et al. 2012). Chang et al. (2007) examined a 2-dimensional physical shearwall-frame building system model considering the nonlinear SFSI effect at the foundation coupled with inelastic structural fuses within the superstructure. Trombetta et al. (2012) constructed a pair of 3-dimensional frame models, each with inelastic structural fuses and one of these with shallow footings. Strong motion amplitudes in these tests facilitated rocking of the shallow footings. Deng et al. (2012) incorporated the foundation rocking mechanism into a bridge structural system while also accounting for a column hinging mechanism. Complementary to these studies, a systematic investigation of the dynamic interaction and balance in load carrying demand between foundation and structural fuse elements particularly for frame-braced structural systems has yet to be undertaken.

1.3. Motivation and scope

In general, a typical earthquake-resistant structural configuration in a building, e.g. moment-resisting frame structural system, usually incorporates structural fuse mechanism within the superstructure components to dissipate seismic energy. Weak-beam-strong-column strategy, for example, is a common design practice in concrete frame building system which aims at localizing yielding behavior at the ends of beam elements. When the proven foundation rocking mechanism is simultaneously implemented into the structural system, how these two inelastic components dynamically interact with each other and share the seismic demand is still unreported and warrants a future investigation. In this research program supported by National Science Foundation (NSF), the researchers at UC San Diego and UC Davis are dedicated to investigating the seismic performance of foundation-building system. However, prior to designing and testing structural models in details, we comprehensively conducted a numerical parametric study on this system within OpenSees analysis program. The primary goal of this study is to understand the effect of variation of the governing parameters on the system's dynamic behavior, and further to seek an optimal configuration to guide the real model design in the testing program. This paper hereby presents the study results and provides preliminary insights on the seismic performance of this foundation-building system.

2. PROPOSED FOUNDATION-BUILDING CONFIGURATION

In present study, a one-bay two-story structural frame model is selected as a target superstructure configuration which attempts to formulate a normal seismic moment-resisting frame system. An idealized structural configuration is depicted in Figure 1 (a). The elliptical circles in this graph, located at the bottoms of columns at each level, represent the potential plastic hinge behavior triggered by fuse and foundation respectively. To simplify load transmission mechanism, it is assumed that the horizontal beam at each floor level, with the columns at first story together, performs with infinitely-rigid flexural behavior comparing with second story columns. As a result, this configuration can be reasonably treated as 2 degree-of-freedom (2-DOF) system and its dynamic characteristics can thereby be analytically determined. The lateral stiffness at each story level will be provided by the foundation rotational flexibility and second story column flexural stiffness respectively. In the meantime, pin connections are applied at the top of the columns at each level for two reasons: (i) to interpret the strength of each inelastic component into two ensuing dimensionless strength parameters, and (ii) to promote foundation rocking mechanism. Figure 1 (b) describes an ideal nonlinear load-deformation diagram for both fuse and footing, in which k_e, k_p and k_u denote elastic stiffness, plastic stiffness and unloading stiffness respectively.

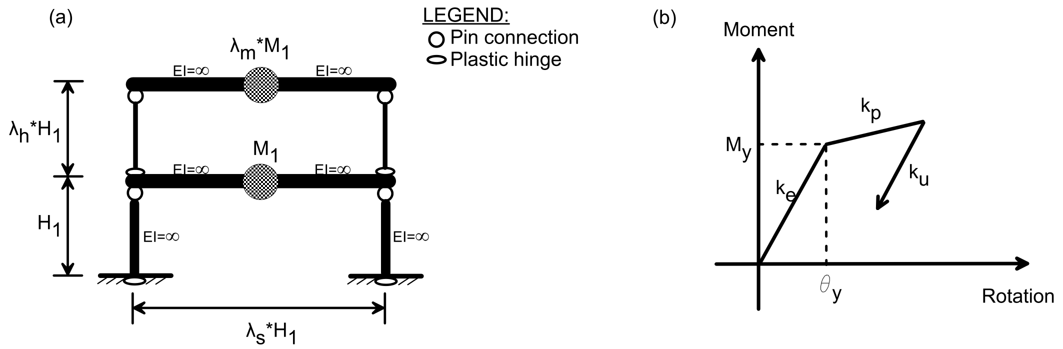

Figure 1. (a) Structure configuration schematics; (b) idealized load-deformation for fuse and footing

In accordance with the proposed configuration, several key parameters are identified in Table 2.1. to govern the structural elastic and inelastic behavior. These parameters include the natural periods of two vibrational modes of the system (T_1, λ_t), the elastic stiffnesses and inelastic strength parameters for the fuse and footing component (k_p/k_e, k_u/k_e, C_r, C_y), the masses at each floor level (M_1, λ_m) , and the system geometry information (H_1, λ_h, λ_s).

Table 2.1. System parameter and its baseline values

Parameter	Parameter Description	Baseline value
M_1	First story mass	1.0
λ_m	Ratio of mass at each level (M_2/M_1)	1.0
T_1	Flexible base 1st natural period	0.3 s
λ_t	Ratio of first two natural periods ratio (T_2/T_1)	0.33
H_1	Column height at first story	3
λ_h	Story height ratio (H_2/H_1)	1.0
λ_s	Ratio of first story column height to spacing	0.25
k_p/k_e	Stiffness hardening ratio for fuse and footing	0.01
k_u/k_e	Unloading stiffness ratio for fuse and footing	1
C_y	Structural fuse yield coefficient	0.3
C_r	Foundation rocking yield coefficient	0.3

It is desirable herein to introduce the proposed two dimensionless parameters, C_y and C_r to characterize the inelastic capacity of fuse and footing respectively. To normalize the yield strength of structural fuse by the mass it will support, structural fuse yield coefficient C_y is proposed. As the second floor column cantilevers (Figure 1), the yield moment capacity of the structural fuse M_{y_fuse} can be correlated with knowledge of the second story mass M_2 and height of the second floor ($H_2 = \lambda_h * H_1$), i.e.,

$$M_{y\_fuse} = C_y * M_2 * g * H_2 \tag{2.1}$$

where g = acceleration of gravity. Correspondingly, another key dimensionless parameter, foundation rocking yield coefficient C_r, is introduced to identify the relationship between the moment needed to mobilize the soil-footing capacity and the superstructure seismic weight. Considering the effect of the pinned connection at the top of the columns, the yield moment capacity of the footing $M_{y\_footing}$ may be related to C_r, assuming a uniform seismic lateral force distribution along the building height, as:

$$M_{y\_footing} = C_r * (M_1 + M_2) * g * H_1 \quad (2.2)$$

where M_1 and H_1 = the mass and height of the first story.

With a reference to general layout and dynamic characteristics of a regular moment-resisting building, the baseline value of each parameter is proposed in Table 2.1 as well. Note that the mass values are assigned as unity for simplicity in this study, which does not indicate the physics of a real building.

3. NUMERICAL PARAMETRIC STUDY

3.1. Numerical model construction

A simplistic numerical model is developed in OpenSees analysis program (OpenSees, 2012), employing several types of beam-column elements and inelastic material sections to attain the proposed elastic and inelastic behavior. Figure 2 (a) describes the basic modeling strategy. In order to capture the hinging behavior of structural fuse, a "beam with hinge" (BWH) element is utilized for the second story columns in which 10% of the column height is set as the hinge length (OpenSees, 2012). As for the foundation element, a sophisticated bed-of-spring model, Beam-on-Nonlinear-Winkler-Foundation (BNWF) model for example, would not be a feasible option at pre-design stage due to the lack of underlying soil information and foundation geometry. In this regard, one single inelastic spring element, modeled by zero-length-element (ZLE), is implemented at the bottom of first story columns to simulate the footing nonlinear rocking behavior. With respect to the inelastic behavior at section state and element state, an idealized elasto-plastic material "Steel01" in OpenSees with kinematic hardening, as illustrated in Figure 2 (b), is applied within BWH and ZLE elements, where the main governing parameters and its values are provided in Table 2.1. The remaining elements of the model are simulated by elastic beam-column element, including horizontal beam elements and columns at first story level. In addition, the two lumped masses are assigned at the central nodes at each floor level in the lateral loading direction. Corotational geometric formulation is implemented in the deformed state of the structure throughout the analysis to facilitate consideration of P-Δ effects.

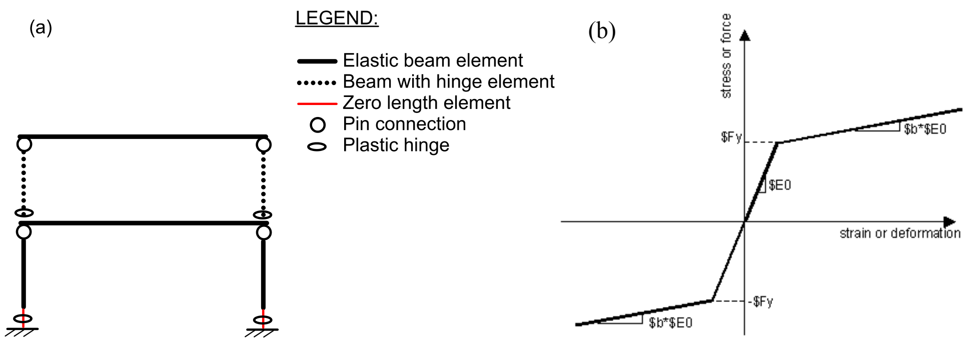

Figure 2. Numerical modeling in OpenSees: (a) model construction; (b) Steel-01 elasto-plastic behavior (adapted from OpenSees website, 2012)

3.2. Selected ground motions

Three recorded ground motions from different seismic events are selected for dynamic motion analysis, denoted as GZ motion, SF motion and MG motion respectively. The spectral acceleration of each motion, overlaid with the targeted flexible-based natural periods of the system, is plotted in

Figure 3 (a). Part (b) provides the acceleration time history plots. Those graphs indicate that the selected motions fundamentally possess different frequency content and motion amplitude. For instance, MG motion is dominated with high frequency content, and has the highest peak acceleration compared with the other two. GZ motion, however, has low frequency content and SF has the smallest PGA. The diversity in frequency and amplitude will help us to thoroughly examine the system's seismic behavior. The motion characteristics and some key features are documented in Table 3.1. To be noted, the spectral acceleration of the first natural period is generally larger than that of second natural period, except for MG motion case.

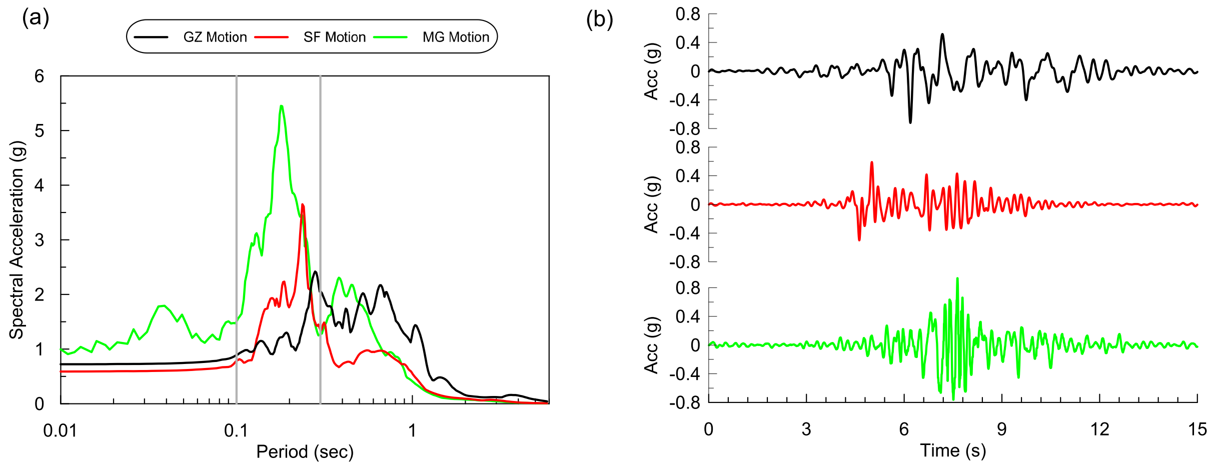

Figure 3. Motion characteristics: (a) Elastic 3% damped spectral accelerations; (b) Acceleration time histories

Table 3.1. Motion source characteristics (PGA= peak ground acceleration, PGV = peak ground velocity, Tp = predominant period as determined from the peak in spectral acceleration, Sa = spectral acceleration)

Motion name	Earthquake event	Tp (sec)	PGA (g)	PGV (cm/s)	$Sa(T_1,3\%)$	$Sa(T_2,3\%)$
GZ motion	1976 Gazli (USSR)	0.28	0.72	57.0	2.05	0.88
SF motion	1971 San Fernando (USA)	0.24	0.59	66.9	1.34	0.77
MG motion	1984 Morgan Hill (USA)	0.18	0.93	80.2	1.28	1.47

3.3. Parameter candidates

Table 3.2 identifies five different predominant parameters, which serve to control the elastic and inelastic behavior of the foundation-building system, and its respective range of values considered in this study.

Table 3.2. Predominant parameter and its range of values considered in this study

Pred. parameter	Description	Range of values	Total cases considered
C_y	Structural fuse yield coefficient	0.1,0.2,0.3,0.4,0.5	5 X (3motions) = 15
C_r	Foundation rocking yield coefficient	0.1,0.2,0.3,0.4,0.5	5 X (3motions) = 15
λ_t	Ratio of first two natural periods	0.1,0.333,0.4	3 X (3motions) = 9
λ_h	Story height ratio	0.5,0.75,1.0,1.25,1.5	5 X (3motions) = 15
λ_m	Ratio of mass at each level	0.5,0.75,1.0,1.25,1.5	5 X (3motions) = 15

A systematic parametric study is then conducted within the numerical model by varying one parameter at a time and keeping the rest constant at their baseline values listed in Table 2.1. The objective is to investigate the effect of each parameter on system's seismic performance. It can be envisioned that an adoption of different value for each parameter will lead to variation of the model in terms of physical geometry or material properties of elements, e.g. EI, My etc. According to structural dynamics theory, they can be analytically determined given the values of predominant parameters associated with the knowledge of equations 2.1 and 2.2 (Liu et al. 2012). In this regard, one needs to update the numerical models' information correspondingly for each case. The ultimate goal of this sensitivity study is to pursue an optimized value for each parameter such that the balanced design can be ideally realized and its physical model construction can be guided prior to actual testing.

3.4. Numerical results

After running intensive dynamic motion analyses for different model cases and motion cases, the seismic performance is systematically compared with each other. A number of engineering demand parameters (EDPs) are selected herein to evaluate its performance, which include maximum structural fuse rotation normalized by fuse yielding rotation, maximum roof drift normalized by the total building height, residual roof drift normalized by the building height, maximum footing rotation normalized by its yielding rotation and maximum base shear normalized by the dead weight. Note that yielding rotation of fuse and footing can be determined with the knowledge of yield capacity (M_y) and rotational stiffness (k_e), as indicated in Figure 1 (b). It is also worth mentioning that the obtained values of each interested EDP cannot directly indicate its behavior considering the unity mass value, thus they should not be judged or compared by the seismic design guidelines or provisions. Instead, the variation trend is more important to be observed to obtain a deep insight on this variation. Figure 4 through 8 compare the seismic behavior of the proposed foundation-building configuration with respect to the sensitivity of each interested parameter.

Effect of Structural Fuse Yield Capacity - Figure 4 shows the effect of variation of structural fuse yield capacity, indicated by C_y, on the system's seismic performance. The increasing of C_y from 0.1 to 0.5 indicates a fivefold increase in strength of the structural fuse. The plots clearly show that the superstructure seismic demand is gradually reduced with an adoption of higher C_y regardless of motion characteristics. For instance, the normalized maximum fuse rotation is dramatically decreased from 12 or above to 4 or below when C_y approaches to 0.3 or higher. Although the response of roof drift ratio and its residual performance exhibit with a slight amplification for the case of 0.2, the plots overall show the decreasing trend. To be pointed, superstructure performance becomes stationary as

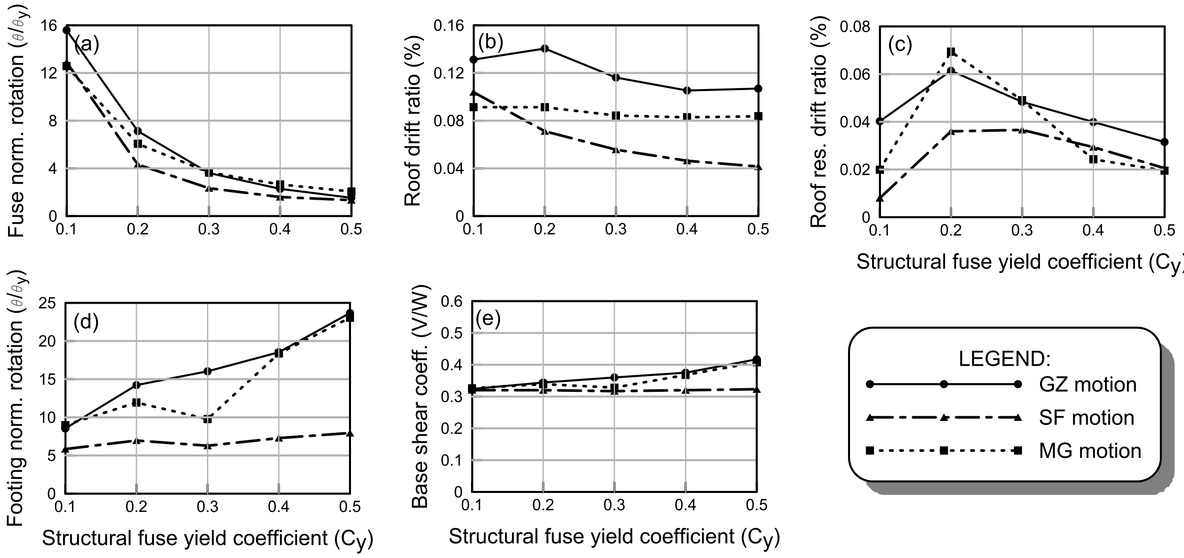

Figure 4. Effect of varying C_y on system seismic performance (baseline parameters refer to Table 2.1)

C_y exceeds 0.3. With respect to substructure's performance, it is expected that the footing is more vulnerable to seismic loading as it becomes relatively weaker. However, it is interesting to observe that footing rotation demand is not monotonically amplified, as indicated in part (d). Particularly when C_y reaches 0.3, the footing rotation ductility demand experiences a restrained increase or even a reduction (SF and MG motion case). As for the maximum base shear demand acting on the foundation, it is not particularly sensitive to variation of C_y, which is mostly located at between 0.3 and 0.4 of the dead weight. Hence, one may conclude that targeting C_y at range of 0.3-0.4 may be able to achieve a better performance for both superstructure and substructure.

Effect of Foundation Rocking Capacity - Similarly, Figure 5 examines the corresponding system performance by varying the foundation rocking strength (C_r). Structural fuse rotation ductility demand in this case increases as rocking foundation capacity becomes stronger. However, the maximum roof drift actually decreases. This can be understood by the fact that roof displacement is contributed from rotation of fuse and footing both. As C_r increases, the footing rotation ductility demand is remarkably reduced, as indicated in part (d), which in turn restrains further development of roof beam horizontal movement. The roof drift and its residual performance also show that they reach a steady state when C_r is being selected 0.3 and higher. On the foundation level, its rotation ductility demand is extremely sensitive with C_r value. For instance, the maximum normalized rotation dramatically decreases from around 100 for the case of C_r=0.1 to approximate 2.0 with a selection of 0.5. Nonetheless, a higher C_r will not necessarily benefit the foundation response. The maximum base shear demand plot, as shown in part (e) indicates that the foundation is subjected to significant amount of base shear when C_r reaches up to 0.4 or 0.5 irrespective of motion characteristics. Therefore, a compromised value of C_r, e.g. 0.3-0.4, should be able to result in reduced demand on the whole system.

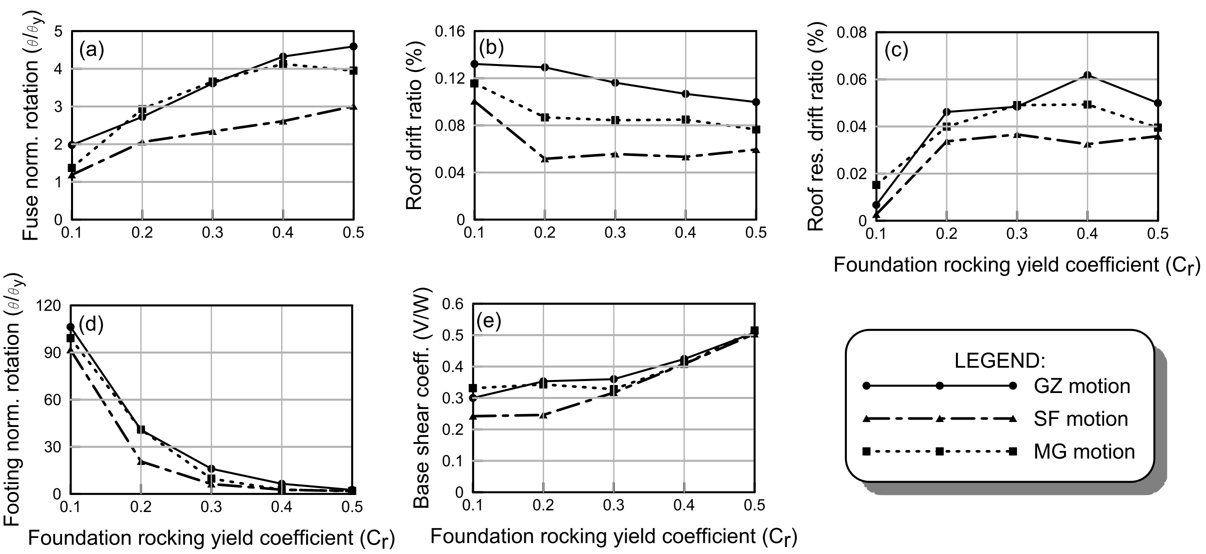

Figure 5. Effect of variation of C_r on system seismic performance (baseline parameters refer to Table 2.1)

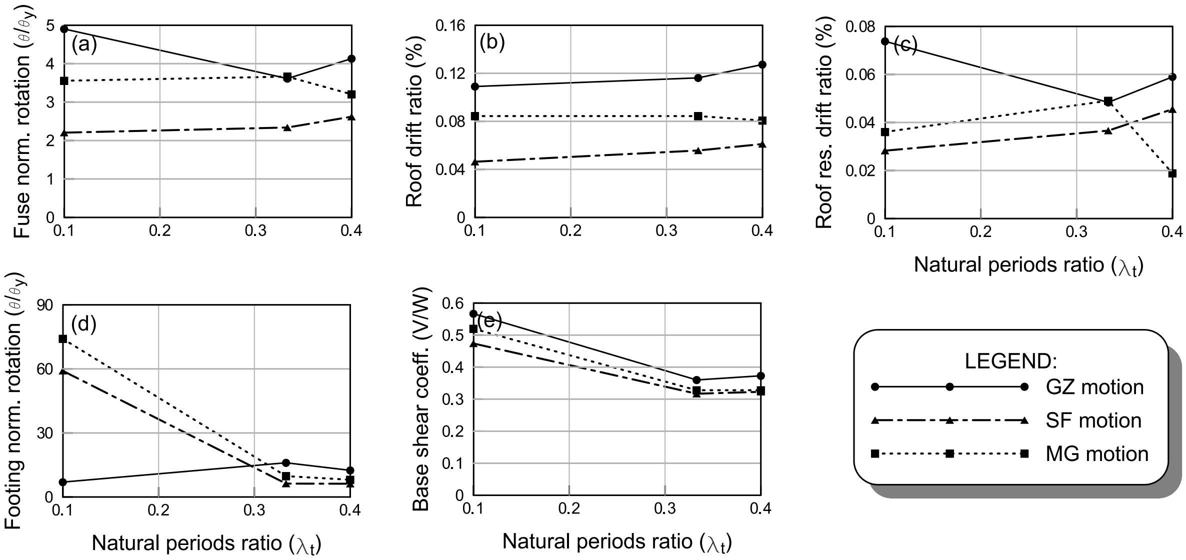

Figure 6. Effect of varying λ_t (T_2/T_1) on system seismic performance (baseline parameters refer to Table 2.1)

Effect of Elastic-State Parameters - Figures 6-8 evaluate the performance's sensitivity in terms of the elastic-state parameters, i.e. natural periods' ratio (λ_t), column height ratio (λ_h) and mass ratio (λ_m). Figure 6 focuses on the effect of second mode natural period variation. Selection of the period ratio (λ_t) requires some care to ensure a real physically meaningful value; some arbitrarily selected period ratios were found to produce imaginary lateral stiffnesses of the system. In this study, the period ratios 0.1, 0.333 and 0.4 were validated and selected. The plots generally show that seismic response of superstructure component is barely influenced by the second mode period. Particularly for the roof drift, the almost identical response for each λ_t scenario within one motion implies that it is merely dependent on the first mode period and motion characteristics. In contrast, the footing element is sensitive to changes in the second mode period. Part (d) and (e) straightforwardly show that the demand (normalized rotation and base shear) is greatly reduced as one elongates the second natural period. Within the motions considered in present study, the natural periods' ratio should be optimally controlled in the range of 0.333 to 0.4 to reduce seismic demand on foundation.

The column height ratio (λ_h) is the one important parameter affecting system's geometry. As the second story stretches, its bending stiffness (EI) and fuse yield strength should be correspondingly stiffer and larger provided that the natural periods and yield coefficient are fixed. As displayed in Figure 7, the structural fuse's rotation ductility demand increases with a higher λ_h, while the maximum roof drift ratio is found to be effectively restrained. Residual roof drift ratio appears to be motion-dependent, without showing any definite trends with λ_h. On the other hand, seismic demand on the foundation level, maximum normalized rotation and maximum base shear, is less sensitive to the variation of column height ratio particularly when it exceeds 1. For the motions considered in this study, an intermediate value (0.8-1.0) works as well as others.

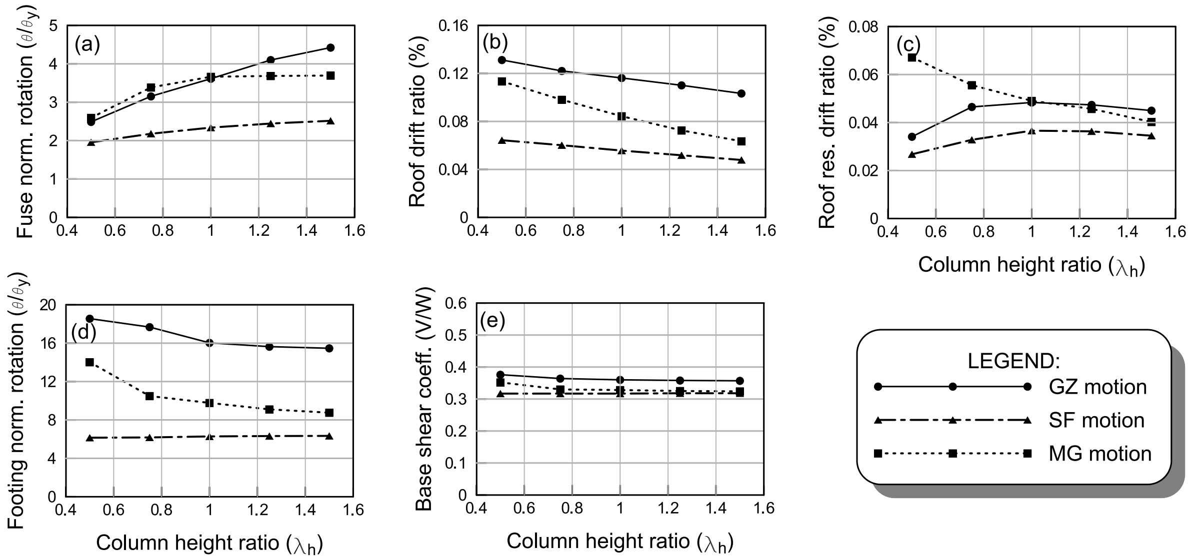

Figure 7. Effect of varying λ_h (H_2/H_1) on system seismic performance (baseline parameters refer to Table 2.1)

In a similar fashion, plots in Figure 8 examine the effect of varying mass ratio (λ_m) on the system's performance. It can be easily envisioned that an implementation of higher mass on second floor will not only require stronger column stiffness and fuse yield capacity, but will also need a stronger rocking foundation at the same time. The plots show that structural fuse ductility demand is enlarged as one increases λ_m , while the normalized roof lateral displacement has a steady response regardless of the roof mass. Meanwhile, the roof residual drift demand appears to be more motion-dependent instead of mass-dependent characteristics. As for the footing response, a heavier mass on second story results in a reduced ductility demand on the footing since its capacity is accordingly strengthened, which implies that the amplification of second story mass will reduce footing rotation ductility demand instead given a fixed foundation rocking yielding coefficient. This is probably attributed to the fact that foundation demand is more sensitive to the second mode, in which the participation of second story mass will yield a balancing inertial force and further result in reduced moment demand. Meanwhile, normalized base shear acting on the foundation is insensitive to variations in mass ratio with each case, regardless of λ_m and motion case.

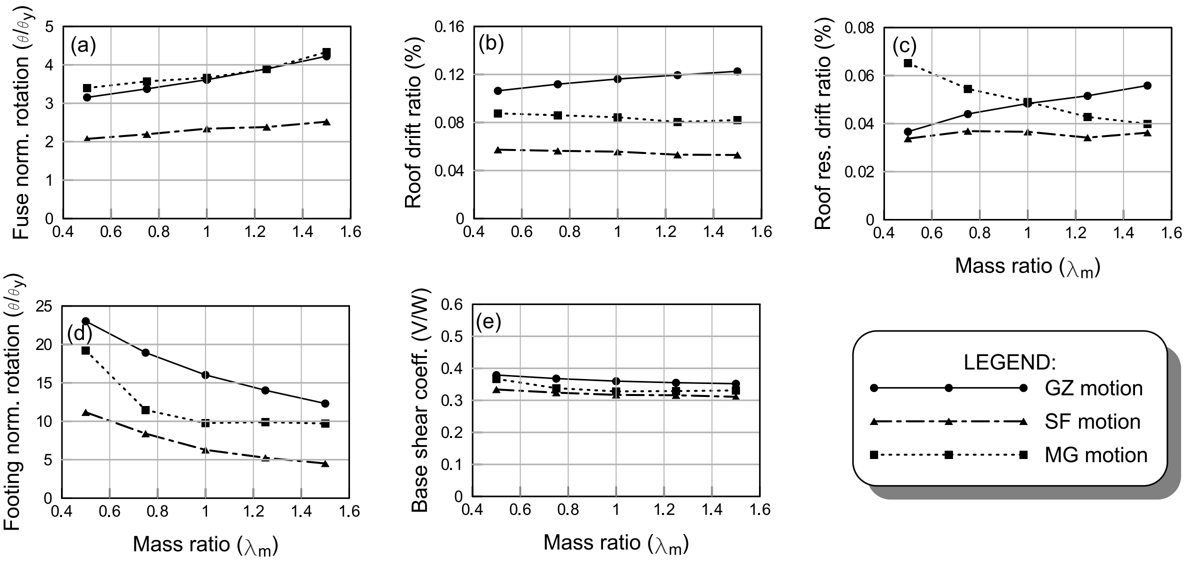

Figure 8. Effect of varying λ_m (M_2/M_1) on system seismic performance (baseline parameters refer to Table 2.1)

4. CONCLUSIONS

This paper presents a parametric study on assessing the seismic performance variation of an inelastic foundation-building system. It is performed in OpenSees finite element analysis program by varying some key building configuration parameters, including structure fuse yield coefficient, foundation rocking yield coefficient, natural periods' ratio, story column height ratio and story mass ratio. The major outcomes from this study include the following:

(1) As the structural fuse becomes stronger, the superstructure seismic ductility demand can be greatly reduced, at the cost of higher ductility demand on the foundation level. However, foundation ductility demand is not monotonically amplified; a local reduction appeared when C_y approaches to 0.3 for one of the ground motions. Meanwhile, maximum base shear acting on the footing is less sensitive to the variation of the fuse strength.

(2) When foundation rocking capacity is enlarged, footing rotation ductility demand and roof drift ratio are both reduced, and reach a steady state when C_r exceeds 0.3. The structural fuse, however, steadily receives higher ductility demand as C_r increases. Also, normalized base shear is prominently amplified when C_r surpasses the C_y value.

(3) The seismic performance of superstructure component is not very sensitive to second mode natural period if the first mode period is held constant while varying the period ratio (λ_t). However, the footing generally has a reduced ductility demand as the second natural period increases. Base shear will also be decreased as one increases the period ratio.

(4) As the height of second story increases, the structural fuse will have a growing rotation ductility demand, but the normalized roof drift ratio will be reduced instead. On the other hand, foundation's ductility demand, such as maximum normalized rotation and maximum normalized base shear, are less sensitive to the variation of second story column height.

(5) When the second story mass is increased, the fuse normalized rotation is linearly increased with mass ratio (λ_m). The roof drift ratio is not significantly amplified. This may be a consequence of the system of parameter variation; as the mass ratio was increased, the strength and stiffness of the columns was increased to maintain constant C_r, C_y, T_1, and T_2. Meanwhile, foundation rotation ductility demand is dramatically reduced since it is highly dependent on second mode shape. The normalized maximum base shear however performs with the same regardless of the assigned masses.

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