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Employing Foundation Nonlinearity to Mitigate Seismic Demand in Superstructure

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ABSTRACT

Because of difficulty in inspection and retrofit of foundation in comparison with other elements, the common design philosophy is to avoid any nonlinear deformation in the foundation. This paper shows that by employing controlled foundation nonlinearity, in predetermined sections with arrangements for inspection and retrofit, it is possible to reduce seismic demand on superstructure. Localizing nonlinear deformation to pre-specified zones in the foundation, it is possible to avoid wide spread nonlinear deformation across various members in the superstructure in the case of strong ground motions. To evaluate the efficiency of the proposed model, the response of steel braced frames is examined on rigid foundation, rocking elastic foundation and finally on rocking foundation with controlled nonlinear deformation. Results show that while rocking could be used to protect the superstructure elements from possible overloading during large earthquakes for low-rise structures; it has no remarkable effect on the response of high-rise structures. However, the proposed model with nonlinearity in the foundation could be used in both cases (low-rise and high-rise structures) to effectively control the response of structure.

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1. INTRODUCTION

Excluding nonlinearity in the structure, there are four sources of nonlinearity in the soil-foundation-structure interaction problems. These are nonlinearity in the soil mass, localized soil nonlinearity in the soil-foundation interface, foundation rocking, and nonlinear deformation in the foundation. There are two sources of nonlinearity in soil, nonlinearity in soil mass, or nonlinearity in soil-foundation interface. Nonlinearity in the soil mass is usually accounted for by reducing the soil shear modulus considering possible range of shear deformation that soil mass may undergo in the event of design ground motion [1]. To avoid localized nonlinear deformation in the soil near soil-foundation interface, it is usual to adopt a relatively large safety factor on soil (in comparison with superstructure), which results in larger foundation dimensions. However, as demonstrated by Gazetas and Apostolou [2], accounting for soil yielding in soil-foundation interface, it is possible to reduce the seismic demand on the superstructure. This form of nonlinearity in the case of deep foundation is usually considered by adoption of so called p-y models [3] and in the case of shallow foundation by Q-z models [4]. After pioneering work of Housner [5] demonstrating the importance of rocking in the good performance of some structures in Chile's 1960 earthquake, now foundation rocking is considered as practically feasible option either to reduce the seismic demand on the structure or to control the seismic demand on the superstructure [6]. Meek experimentally investigated response of a flexible structure with rocking foundation [7]. Chopra and Yim developed a simple method to evaluate the base shear of multi degree of freedom systems with rocking foundation [8]. They modified the design response spectrum accounting for larger energy dissipation due to foundation rocking. Makris and Konstantinidis [9] found that modification of the design spectrum for larger damping associated with rocking (as is also recommended in ASCE/SEI 41), could result in gross errors. Recognizing the importance of accounting for rocking, now there are ways to consider this effect in a systematic way in the evaluation of the seismic performance of structures [10]. Hutchinson et al. showed that rocking plays a dominant

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role in the force distribution between frame and bracing or shear wall in dual lateral force resisting systems [11]. On the other hand, it is renowned that rocking could have adverse impact on the seismic performance of structure as it is demonstrated by Gazetas [12]. They showed that rocking probably is the main cause of the failure of some structures in past earthquakes. Lu et al. [13] developed a simplified nonlinear sway-rocking model accounting for coupling between different motion components. Masaeli et al. [14] utilizing three dimensional nonlinear soil-structure interaction analyses investigated the reduction in seismic demand for structures of different geometrical dimensions subjected to a set of near fault records. Anastasopoulos and Kontoroupi [15] proposed a simplified approximate method capturing the effect of soil inelasticity and foundation rocking. It is shown that it could be effectively used for preliminary design. Apostolou et al. [16] investigated the effect of foundation rocking on the seismic response of slender rigid structures. Accounting for near field records, they idealized ground motion using Ricker wavelet. They found that estimating ground motion acceleration from overturned objects could be misleading. Genlagoti et al. [17] investigated the toppling probability for seismically isolated structures due to rocking. They developed a simplified models to estimate the seismic toppling displacement demand.

Due to difficulty in access and retrofit of foundations, one of the common design philosophies in the modern seismic codes is to avoid any nonlinear deformation in foundation. However, there are contradictory approaches to this problem in different codes. While some codes (e.g. Eurocode 8 [18] and NBCC [19]) uses capacity design concept to prevent any form of nonlinear deformation in the foundation, on the other hand, NEHRP [20] improvises a reduction in the overturning moment evaluation for foundation design. It is shown in this paper that by localizing nonlinear deformation in predetermined locations together with arrangements to have easy access to these locations, it is possible to reduce seismic demand on the superstructure. Localizing the nonlinear deformations in limited number of sections with provisions for easy access and retrofit has three advantages. The first one is a substantial reduction in the number of members which should be inspected and perhaps should be retrofitted after large ground motions. The second benefit is removing the need for capacity design of superstructure elements, which reduces the construction difficulty. The third advantage is limiting the retrofit area to limited locations in the foundation that minimizes the out of service time of the superstructure after any large earthquakes. In the following sections, after reviewing the finite element model used in the analysis, pushover analysis is used to evaluate the response of the system ignoring and accounting for foundation rocking in models with and without nonlinear deformations in the foundation.

2. STRUCTURAL MODELS

The model structures include 3- and 7-story steel braced frames. The design lateral force on the model structures is determined using NEHRP, while superstructure is designed using the seismic provisions of AISC [21] and foundation design satisfies ACI 318 regulations [22]. The mass of each node in all models is 6500 kg, and typical story height and span length are 3.2 and 5m, respectively. The model structures are designed for lateral force of about 0.1 times their weight. Figure 1 shows typical shape of the 7-story model, which is a 4 bays structure. Analyses are carried out by OpenSEES [23] finite element software. OpenSEES is an open source finite element analysis platform developed by Pacific Earthquake Engineering Research center (PEER). OpenSEES has a rich library of different materials and elements especially developed for seismic analysis of structures. To reach a balance between computational demand and accuracy for steel and concrete components Steel01 and Concrete01 material properties is used, respectivly. Material property 'Steel01' is used to model the steel columns, bracings and foundation rebars. This uni-axial material model benefits from a uni-axial bilinear stress-strain behavior with kinematic hardening (Figure 2). The material property used to model concrete is 'Concrete01'. This uni-axial material model ignores concrete tensile strength and employees a parabolic ascending and a linear softening region with a constant residual strength at large strains (Figure 2). For modeling of columns and bracings, 'nonlinear Beam Column' element is used. This is a force-based element which is able to model element buckling when accounting for large deflections. The bracings end condition is modeled by using EqualDof option in openSEES.

All of the beams are hinged at both ends, and are not part of lateral force resisting system. Therefore, they are modeled using elastic 'Truss' element. The beams end shear is applied as vertical load on the column.

The foundation in elastic region is modeled using 'elasticBeamColumn' element. In the predetermined nonlinear deformation region of the foundation the behavior is modeled using 'beamWithHinges' element. This element employing flexibility formulation models plasticity using concentrated plastic hinge with finite length in the element ends. The section property for elements modeled using 'beamWithHinges' is derived using Fiber section modeling of cross section, where aforementioned uniaxial material properties for concrete and steel rebars is used. In this study, foundation is modeled in three different ways: rigid foundation, rocking foundation with elastic behavior for foundation, and rocking foundation with nonlinearity in the predetermined locations in the foundation. Figure 1 illustrates the location of intended nonlinearity in the foundation (zones with length L_3) and Table 1 gives the foundation reinforcement and the location and length of predetermined regions with foundation nonlinearity. For each of 3 and 7 story structures, two models are considered. These models differ from each other in the location of intended nonlinear deformation. Comparing the response of these models gives an assessment of the effect of the location of nonlinear deformation zone on the performance of the structures. In both of the nonlinear models reinforcement used is lesser than that required by the code requirements.

Soil stiffness is modeled using ‘zeroLength’ elements. The Soil specifications were assigned based on ASCE/SEI 41. To model separation of foundation from supporting soil ‘QzSimple1’ material is used [24]. This material property has asymmetric backbone with nonlinear behavior in compression and insignificant or zero strength in tension. In the intended nonlinear deformation zones access to all side of foundation should be provided. Knowing that this could introduce constructional difficulty, however if applicable this maintains the possibility of easy access and retrofit to these zones. Therefore, in these regions no soil support is considered.

Nonlinear static analysis is performed using displacement-control until the roof displacement of equal to 0.03 times structures height. This displacement level is well above different codes prescribed target displacement. By following structures response at these drifts, development of any unintended failure modes is verified. Also, as could be inferred from Figures 3 and 4, this drift level is well above the point with peak response and the structures are pushed well in the softening branch.

3. SIMULATION RESULTS

Figure 3 demonstrates the deformed shape of the structure in the final loading stage for different models of 7-story structure. The same pattern of deformation is observed for 3-story models. As could be inferred from this figure, while in the case of rigid and rocking foundations with elastic foundation the final stage is accompanied by buckling of bracings, for the case of foundation with controlled nonlinearity, there is no buckling in the bracing for the final stage of loading.

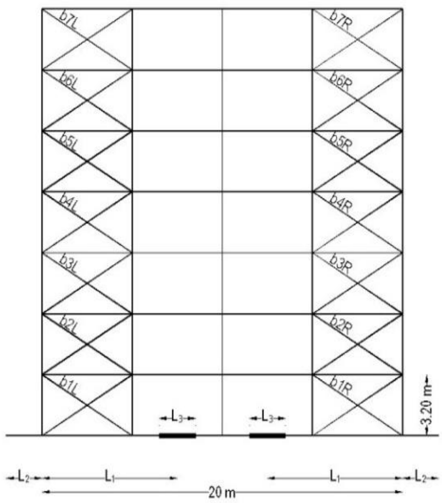
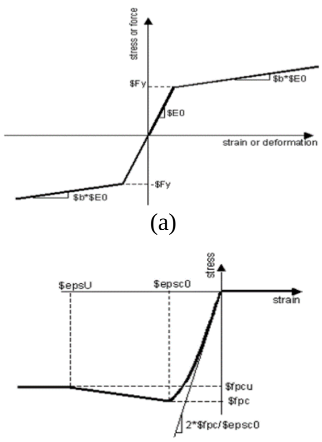


Figure 1. Model structure depicting intended nonlinear deformation zone in the foundation



(b)

Figure 2. Material properties used for a) Steel, b) Concrete

TABLE 1. Parameters used in the model with controlled nonlinearity in the foundation nonlinearity

Story No.	Foundation model	Foundation reinforcement used in the foundation with localized nonlinearity	Foundation reinforcement required by NEHRP lateral load	Foundation dimension width-depth (m)	Length L_1, L_2 (m)	Length of nonlinear deformation zone, L_3 (m)
3	A1	12T25	20T25	1.2x0.7	6.0, 1.0	1.2
	A2	12T25	15T25	1.2x0.7	8.5, 1.0	1.2
7	B1	20T25	42T25	2.4x1.2	6.5, 1.4	2.0
	B2	20T25	30T25	2.4x1.2	9.0, 1.4	2.0

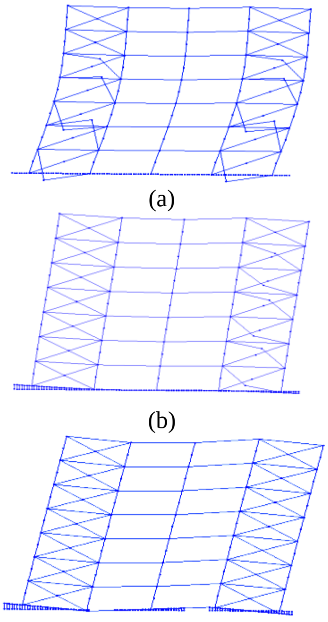


Figure 3. Deformed shape of 7-story model structure, a) rigid foundation (no rocking), b) rocking foundation with elastic foundation, c) rocking with foundation nonlinearity at predetermined locations.

Figure 4 shows the results of pushover analysis for model structures in the case of 3-story structure. Sequence of nonlinear deformation including bracing buckling and development of plastic hinge in the foundation are depicted in the figure. Figure 5 depicts the axial load-lateral drift diagram for bracings of different stories in different models. In this figure, the diagram for bracings in left and right are shown separately, where L and R denote the left and right bracings. For the case of rigid foundation nonlinearity in the behavior initiates with buckling of bracings in the first story. After buckling of bracings in the first story, there is only slight increase in the capacity of structure. Buckling in the bracings of second and third stories follow buckling in the first story and this triggers the softening in the response. As is evident from Figure 4a the post peak response in this case shows itself as a steep softening branch in the load-displacement diagram. Accounting for foundation rocking has great impact on the response of the structure. In the case of rigid foundation nonlinearity is due to buckling of bracings in different stories. Nonlinearity starts at small lateral displacements and again softening in the response is initiated by buckling of bracings in the same pattern as in the case of rigid foundation. Interesting point is that buckling occurs only for bracings in the right hand side of structure, while in the case of rigid foundation bracings at right and left hand side buckles simultaneously.

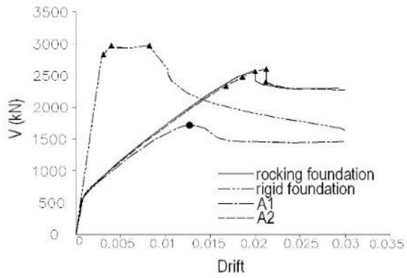


Figure 4. Results of pushover analysis for 3-story model structure in the case of different foundation models.

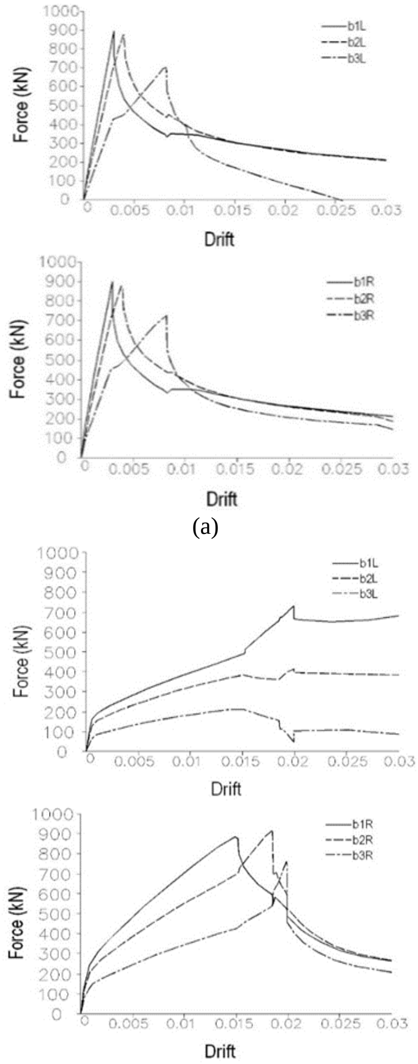
Figure 4 also depicts the response of the two models considering rocking nonlinear foundations. This figure shows that the performance of the nonlinear foundation model could be controlled by the location and length of the nonlinear deformation zone and also by the reinforcement considered in this region. While model A2 compared to rocking elastic foundation provides no superiority, model A1 gives a protection against the buckling of the bracings by reducing the maximum lateral load transferred to the structure. Moreover, in model A1 the post peak response has a good ductility and the structure is able to sustain the lateral load in large deflections without significant reduction in the lateral load resisting system. Figure 5 shows that by proper selection of design capacity for plastic hinge section in the case of rocking nonlinear foundation, it is possible to prevent buckling in the bracings. While for the cases of rigid, rocking elastic foundations and rocking nonlinear foundation A2, there are extensive buckling in the bracings, in the case of rocking nonlinear foundation A1 there are no buckling in the bracings and nonlinearity is mainly due to plastic hinging in the foundation. In the recent case, using reduced amount of reinforcement in the intended plastic hinge locations of the foundation, there is substantial reduction in the maximum lateral load exerted on the superstructure. This reduction guarantees elastic response in the superstructure, which means that there will be no need to extensive intervention in the superstructure even after large ground motion excitations. Localizing the locations of nonlinear deformation greatly reduces the time required for retrofit attempts that makes structure out of service. This makes this structural scheme an occupant friendly solution.

Due to large size of foundation and possibility of providing good ductility at these sections with small changes in the section reinforcement, limited nonlinear deformation zones are able to provide ductile response for global response of structure. Figure 6 depicts the moment-curvature diagram of the plastic hinge section for monotonic increase in the lateral displacement of structure for models A1 and A2.

Due to large size of foundation section providing effective confinement for core concrete requires large amount of transverse reinforcement that could lead to congestion of reinforcement and difficulty in construction. Considering this the moment curvature for plastic hinge section is derived assuming no confinement for core concrete. This explains the existence of a slight softening in the moment-curvature diagram in the post peak regime. However, the residual moment capacity of the section in large curvature is very good. This is due to use of compression reinforcement and near minimum longitudinal reinforcement used in the section.

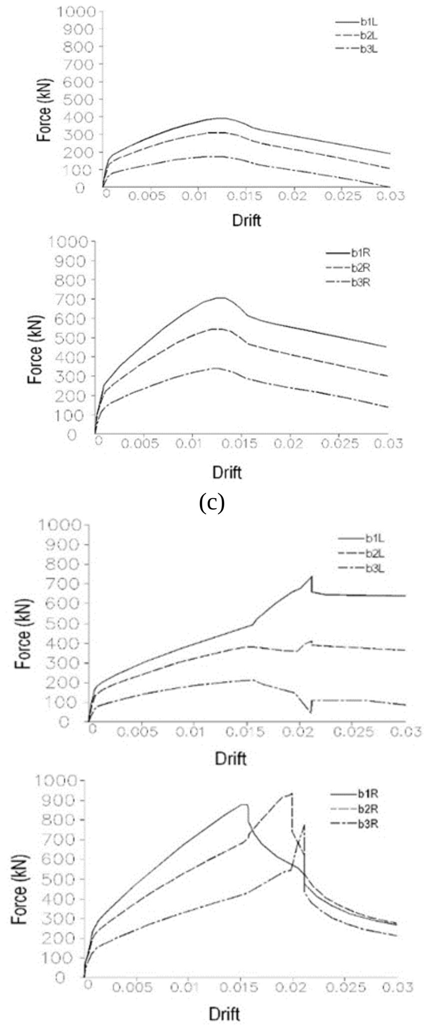
As could be inferred from Figure 5 in model A2 foundation remains elastic and main sources of nonlinearity are foundation rocking and bracing buckling. While in model A1, there are extensive nonlinear deformations in the foundation.

Figure 7 shows the lateral load-displacement diagram for 7-story model structure.



(a)

(b)



(c)

(d)

Figure 5. Bracings axial load-roof drift curves for 3-story model structures, a) model with rigid foundation, b) model with rocking elastic foundation, c) model with rocking nonlinear foundation A1, d) model with rocking nonlinear foundation A2.

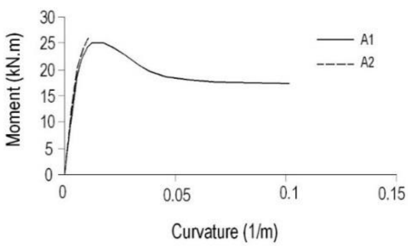


Figure 6. Moment-curvature diagram for plastic hinge sections of foundation in 3-story model structure.

In the case of rigid foundation, again sharp reduction is observed in the load carrying capacity of structure in post peak regime, similar to that observed in the case of 3-story model. Nearly, simultaneous buckling in bracings from story 1 to 7 initiates softening in the global response.

While in the case of 3-sory model, rocking in the foundation greatly changes the load-displacement diagram, for 7 story model the impact of rocking on the response is smaller and rocking only slightly delays initiation of buckling in the bracings. In other word, rocking has lesser impact on the global response of high rise structures. This means that for high rise structures rocking will not have appreciable impact on the global response as it has for low rise ones.

However, providing the foundation with controlled nonlinearity, it is still possible to control the global response in the same extent as it is done for 3-story model. Efficiency of the proposed nonlinear deformation scheme strongly depends on the distance between braced columns and plastic hinge sections.

Figure 8 gives the axial load-lateral drift diagram for bracings in different models. As could be seen for the case of rigid foundation bracings at different stories buckle nearly at the same lateral drift, which explains why there is sharp softening in the global response. Rocking in this case only introduces slight increase in the lateral drift corresponding to the buckling of bracings.

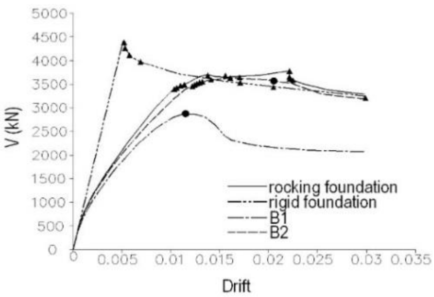
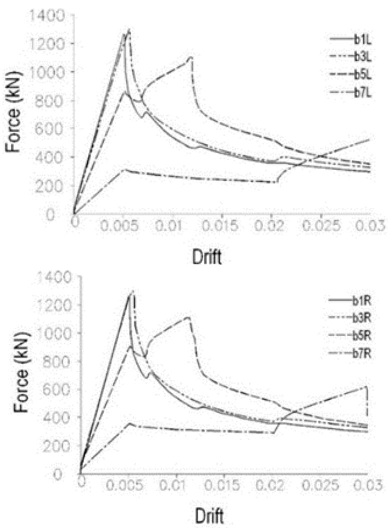
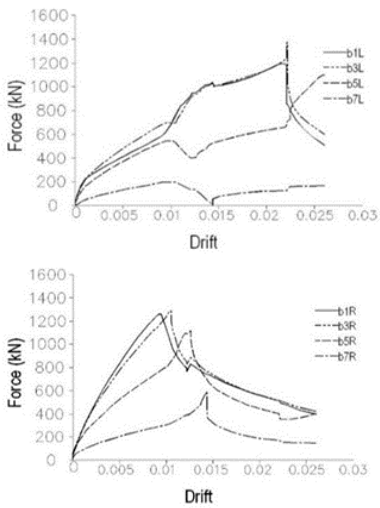


Figure 7. Results of pushover analysis for 7-story model structures.



(a)



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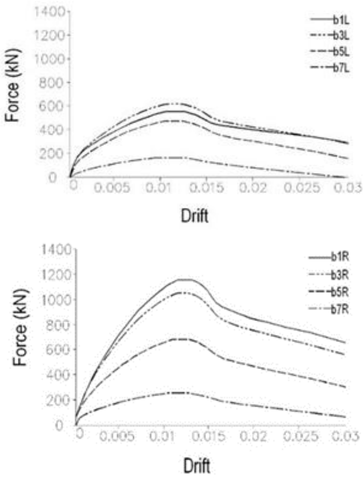

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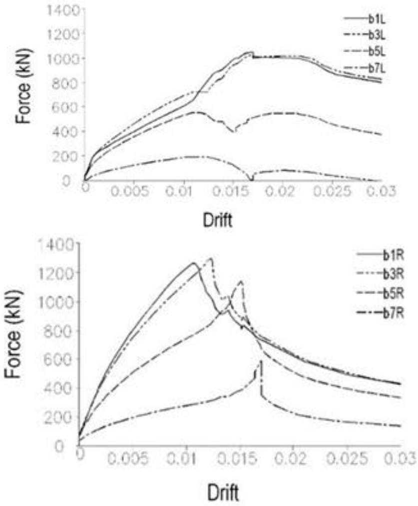

(d)

Figure 8. Bracings axial load-roof drift curves for 7-story model structures, a) model with rigid foundation, b) model with rocking elastic foundation, c) model with rocking nonlinear foundation B1, d) model with rocking nonlinear foundation B2.

Tables 2 and 3 gives the drifts corresponding to peak strength and 80 percent of peak strength on the softening braches. Interesting point is that due to rocking, the substantial increase in the drift at peak base shear without implementation of nonlinear deformation in superstructure. These tables also show that accounting for rocking, it is possible to reduce seismic demand on superstructure by as much as 16 percent, which by considering nonlinear deformation in the foundation this increases to about 36 percent.

Changing the dimension of plastic deformation zone in the foundation, it is possible to control the response.

While for model B2, abrupt decrease in the bracing load for large deflections are observable, in the case of model B1 there is no such behavior in the response of the bracings. This shows that by careful design of capacity and length of the plastic hinge deformation zones, even in the case of tall buildings that rocking has no remarkable effect on the global response, we could control the response of structure.

Figure 9 depicts the moment-curvature diagram of the plastic hinge section. As could be seen even assuming no confinement the section has good residual capacity at large deflections. In this case, nonlinear deformation in the foundation occurs for both models, however in greater extent for model B1.

Table 4 gives the value of nonlinear rotation at the foundation for different nonlinear models. The permissible rotation as per ASCE/SEI 41 for life safety and collapse prevention conditions are 0.025 and 0.05 rad, respectively. This shows that by proportioning of the nonlinear zone in the foundation, it is possible to control rotational demand in the foundation's nonlinear zone. To provide additional rotational demand confinement could be employed. By increasing the ultimate strain in concrete by as much as two times it is possible to reduce rotational demand to its half value. The ultimate strain of concrete enhanced by confinement (ε_{uc}) could be evaluated as $\varepsilon_{uc} = 0.004 + \frac{f_l}{4f_{cc}}$where f_l denotes provided lateral confinement, f_{cc} is confined concrete strength.

TABLE 2. Results of pushover analysis for 3-story model structure in the case of different foundation models.

Foundation model	V_{max}	Drift	Drift at 80% at V_{max}	Ratio of drift at V_{max} to drift at $0.8V_{max}$	$V_{max}/(V_{max})_{\text{Rigid Found.}}$
Rocking foundation	2500	0.020	0.032	1.60	0.87
Rigid foundation	2890	0.004	0.017	4.25	1.00
A1	1750	0.012	0.028	2.30	0.61
A2	2500	0.021	0.034	1.60	0.87

TABLE 3. Results of pushover analysis for 7-story model structure in the case of different foundation models.

Foundation model	V_{max}	Drift	Drift at 80% at V_{max}	Ratio of drift at V_{max} to drift at $0.8V_{max}$	$V_{max}/(V_{max})_{\text{Rigid Found.}}$
Rocking foundation	3750	0.022	0.032	1.45	0.84
Rigid foundation	4450	0.005	0.017	3.40	1.00
B1	2850	0.012	0.021	1.75	0.64
B2	3600	0.015	0.035	2.33	0.81

TABLE 4. Nonlinear rotation at drift corresponding to strength at 80 percent peak value on the softening branch.

Foundation model	Plastic curvature (1/m)	Plastic Rotation (rad.)
A1	0.050	0.02
A2	0.000	0.00
B1	0.060	0.04
B2	0.035	0.02

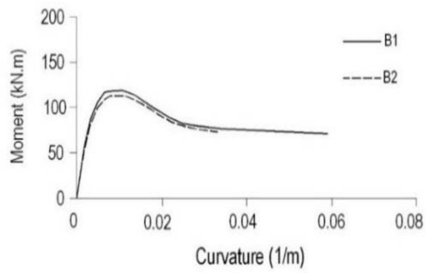

Figure 9. Moment-curvature diagram of the foundation plastic hinge section.

Minimum confinement as required by code requirement provides a confinement stress of about $0.06f_c$. This could result an increase of about 15 percent in concrete strength, which means f_{cc}=1.15f_c. This will lead to ultimate strain of about 0.017. Comparing this with ultimate strain of unconfined concrete (0.004) and assuming same neutral axis depth, this means an increase in rotational demand of as much as 4 times. It shows the possibility of significant enhancement in the rotational demand of the foundation's nonlinear zone by confinement.

4. CONCLUSION

Responses of a braced steel structure with rigid foundation, rocking elastic foundation and rocking foundation with controlled nonlinearity are evaluated. The results show that while accounting for rocking in low-rise structure could substantially reduce the seismic demand on the superstructure, in the case of high-rise structure, it could hardly affect the pattern of nonlinear deformation in the superstructure as compared to the model with rigid foundation. However, the model with controlled nonlinearity in the rocking foundation could effectively control the seismic demand on superstructure for both cases of low- and high-rise structures. The proposed model reduces the number of location with nonlinear deformation, and consequently limits the locations in need of inspection and perhaps retrofits. By this way, the proposed model reduces the out of service time of structure in case of the need for retrofit.

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6. REFERENCES

1. Engineers, A.S.o.C., "Seismic rehabilitation of existing buildings, ASCE Publications, Vol. 41, (2007), 154-163.
2. Gazetas, G. and Apostolou, M., "Nonlinear soil–structure interaction: Foundation uplifting and soil yielding", in Proceedings of the 3rd USA–Japan workshop on soil–structure interaction. Menlo Park., (2004), 53-61.
3. Brown, D.A., O'Neill, M., Hoit, M., McVay, M., El Naggar, M. and Chakraborty, S., "Static and dynamic lateral loading of pile groups, (2001), 77-87.
4. Chen, W.-F. and Duan, L., "Bridge engineering handbook: Construction and maintenance, CRC press, (2014), 96-105.
5. Housner, G.W., "The behavior of inverted pendulum structures during earthquakes", ***Bulletin of the Seismological Society of America***, Vol. 53, No. 2, (1963), 403-417.
6. Pollino, M. and Bruneau, M., "Seismic retrofit of bridge steel truss piers using a controlled rocking approach", ***Journal of Bridge Engineering***, Vol. 12, No. 5, (2007), 600-610.
7. Meek, J.W., "Effects of foundation tipping on dynamic response", ***Journal of the Structural Division***, Vol. 101, No. 7, (1975), 1297-1311.
8. Chopra, A.K. and Yim, S.C.-S., "Simplified earthquake analysis of structures with foundation uplift", ***Journal of Structural Engineering***, Vol. 111, No. 4, (1985), 906-930.
9. Makris, N. and Konstantinidis, D., "The rocking spectrum and the limitation of design guidelines", in Proceedings of the 15th ASCE engineering mechanics conference, Columbia University, New York, NY. (2002), 75-83.
10. Harden, C.W., "Numerical modeling of the nonlinear cyclic response of shallow foundations, (2003), 85-92.
11. Hutchinson, T.C., Raychowdhury, P. and Chang, B., "Nonlinear structure and foundation response during seismic loading: Dual lateral load resisting systems", in Proceedings of the Eighth US national conference on earthquake engineering. Paper. (2006), 98-105.
12. Gazetas, G., "Seismic design of foundations and soil-structure interaction", in First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland. (2006), 3-8.
13. Lu, Y., Marshall, A.M. and Hajirasouliha, I., "A simplified nonlinear sway-rocking model for evaluation of seismic response of structures on shallow foundations", ***Soil Dynamics and Earthquake Engineering***, Vol. 81, (2016), 14-26.
14. Masaeli, H., Khoshnoudian, F. and Tehrani, M.H., "Rocking isolation of nonductile moderately tall buildings subjected to bidirectional near-fault ground motions", ***Engineering Structures***, Vol. 80, (2014), 298-315.
15. Anastasopoulos, I. and Kontoroupi, T., "Simplified approximate method for analysis of rocking systems accounting for soil inelasticity and foundation uplifting", ***Soil Dynamics and Earthquake Engineering***, Vol. 56, No., (2014), 28-43.
16. Apostolou, M., Gazetas, G. and Garini, E., "Seismic response of slender rigid structures with foundation uplifting", ***Soil Dynamics and Earthquake Engineering***, Vol. 27, No. 7, (2007), 642-654.
17. Gelagoti, F., Kourkoulis, R., Anastasopoulos, I. and Gazetas, G., "Rocking-isolated frame structures: Margins of safety against toppling collapse and simplified design approach", ***Soil Dynamics and Earthquake Engineering***, Vol. 32, No. 1, (2012), 87-102.
18. Wei, D. and Wang, S., Design of structures for earthquake resistance. 2001, Wuhan Industrial University Press, Wuhan.
19. "Nbcc. National building code of canada ", in, Institue for Research in Construction, National Research Council of Canada, Ottawa, Ont. (1995).
20. Council, B.S.S., "Nehrp recommended provisions for seismic regulations for new buildings and other structures (fema 450): Provisions, Building Seismic Safety Council, National Institute of Building Sciences, (2004).
21. Construction, A.I.o.S., "Seismic provisions for structural steel buildings, American Institute of Steel Construction, (2002).
22. Kamara, M.E., Novak, L.C. and Rabbat, B.G., "Notes on aci 318-08, building code requirements for structural concrete: With design applications, Portland Cement Assn, (2008).
23. McKenna, F., Fenves, G. and Scott, M., "Open system for earthquake engineering simulation", ***University of California, Berkeley, CA***, (2000).
24. Boulanger, R., "The pysimple1, tzsimple1, and qzsimple1 material models, documentation for the opensees platform", ***URL: http:// opensees. berkeley. edu***, (2000).

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Steel Braced Frames

چکیده

به دلیل صعوبت‌های عملی مقاوم سازی فونداسیون در مقایسه با سایر المان‌های سازه ای، فلسفه کلی طراحی بر مبنای عدم ایجاد تغییرشکل‌های غیرخطی در فونداسیون استوار است. مقاله حاضر نشان می‌دهد که با لحاظ کردن تغییرشکل‌های غیرخطی کنترل شده در فونداسیون و با در نظر گرفتن مقاطع زمانی از پیش تعیین شده برای بازدید و مقاوم سازی، امکان کاهش تقاضای لرزه ای در رو سازه بوجود خواهد آمد. متمرکز سازی تغییرشکل‌های غیرخطی در نواحی از پیش تعین شده فونداسیون، مانع گسترش تغییرشکل‌های غیرخطی در المان‌های رو سازه در هنگام مواجه با زلزله‌های قوی خواهد شد. برای ارزیابی کارآیی مدل پیشنهاد شده در این تحقیق، پاسخ‌های لرزه‌ای قاب‌های مهاربندی شده فولادی قرار داده شده بر روی فونداسیون صلب، فونداسیون الاستیک دارای حرکت گهواره ای، و فونداسیون با امکان ایجاد تغییرشکل‌های غیرخطی کنترل شده دارای حرکت گهواره‌ای مورد بررسی قرار می‌گیرند. نتایج نشان دادند درحالیکه حرکت گهواره‌ای می‌تواند در سازه‌های کوتاه مرتبه مانع افزایش نیروهای لرزه‌ای المان‌های رو سازه شود، در سازه‌های بلند مرتبه اثر چندانی بر پاسخ‌های لرزه‌ای سازه ندارد. هرچند مدل پیشنهادی قابلیت کاربرد در هردو حالت فوق الذکر را دارا می‌باشد

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