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Behavior of Reinforced Concrete Frames on Flexible Foundations Subjected to Both Horizontal and Vertical Ground Motions

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Abstract
This paper investigates the behavior of combined horizontal and vertical accelerations on the seismic response of reinforced concrete resting on shallow foundations. Five and nine-storey buildings are considered to represent low- and medium- rise buildings. The building was assumed to be founded on shallow foundation with soil class corresponding to soft soil deposits, The soil of foundation was modeled by an equivalent massless spring coupled with a gap. The accelergrams used in this investigation in the nonlinear range are the horizontal and vertical components of the New Hall earthquake. Although most building codes postulate that SSI generally decreases the force demand of buildings, and increases the deformation demand, it was found that the inclusion of the vertical ground motion with SSI, the horizontal displacement are decreased compared to the case with only the horizontal component.
Keywords: horizontal and vertical accelerations, Shallow foundation, spring couple, Gap, nonlinear.

1. Introduction
Most of the seismic codes around the world tend to neglect the effect of vertical ground motion in the design of reinforced concrete buildings due to following reasons: it was believed that the intensity of vertical component is too weak compared to the horizontal component, the wave length of vertically propagation motions is shorter than that of the S-wave, and that structures are very rigid in the vertical direction compared to horizontal direction.
However, shear failure of concrete columns was one of the major causes of damage, as observed in recent destructive earthquakes such as the Loma Prieta (1989) and Northridge earthquakes (1994) in California and the Hyogo-ken Nanbu earthquake (1995) in Kobe, Japan. Significant damage to RC structures has been attributed to the reduction of shear strength caused by vertical ground motion effects. Many studies reported data showing that the effect of vertical ground motion can be important in some cases, Antoniou (1997) and Ghobarah and Elnashai (2003), Javed A et al (1997), Shakib H et al (2003), Collier and Elnashai (2001), Mwafy and Elnashai (2006).
Modern building codes for example NBC 105, IS 1893, UBC 97 and many other codes worldwide assume the vertical component of the ground motion to be ½ to 2\3of the horizontal component. However, the soil-structure interaction (SSI) effects were neglected. This is due to the difficulties involved in making simple but reliable dynamic models and also due to prohibitive cost associated with such analyses, and SSI involves structural and geotechnical engineering, traditional design practice has seen engineers in these two fields working independent of each other. Shallow isolated footings can be modeled using: the uncoupled approach (uncoupled translation and rotational springs); Winkler approach (distribution vertical and horizontal springs); plasticity-type macroelement formulation using foundation action bounding/yield surface; and continuum approach such as finite element or boundary element methods. The continuum approach is the most rigorous, but is computationally intensive and time consuming. The uncoupled approach can model the load-deformation behavior, but cannot predict settlement.
The macro-element approach is able to satisfactorily predict the complete foundation response because it accounts for nonlinear behavior and coupling between the responses in all directions. The Winkler model represents a trade-off between the uncoupled and macro-element model. It can simulate progressive mobilization of plastic capacity and the resulting settlement. Housne GW (1993) used the Winkler approach to evaluate the structural response accounting for foundation uplift; and Psycharis and Jennings (1984) and song and lee (1993) used it to examine the effects of rocking and uplift. More recently, Filiatrault *et al* (1992). Chaallal and Ghlamallah (1996), Anderson (2003) and Chen and Lai (2003) used the Winkler approach to examine the effect of both uplift and yield on structures.
These studies demonstrated the ability of the Winkler approach to provide reasonable prediction of the rocking response of various structures. This paper examines the effect of vertical ground motion on the nonlinear seismic behavior of reinforced concrete building with soil flexibly. A simple two-dimensional (2D) model was used to idealize the structures and the supporting soil. The nonlinear structural analysis program SAP2000 was chosen for this work.

2. Description of Building Models

The structures used in this study, are five and nine story buildings, the dimension in plan are 3m x 6m and the height is 3 m. The structures were designed according to the Algerian cod RPA (2003) and are located in high seismicity region with a peak ground acceleration of 0.32g. The dimensions of the beam and columns for the three structures are shown in Figure 1 and the reinforcement details are shown in Figure 2.

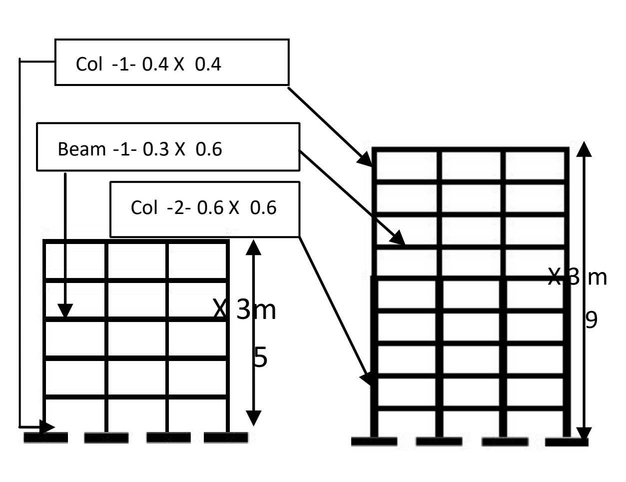


Figure 1. Structural models.

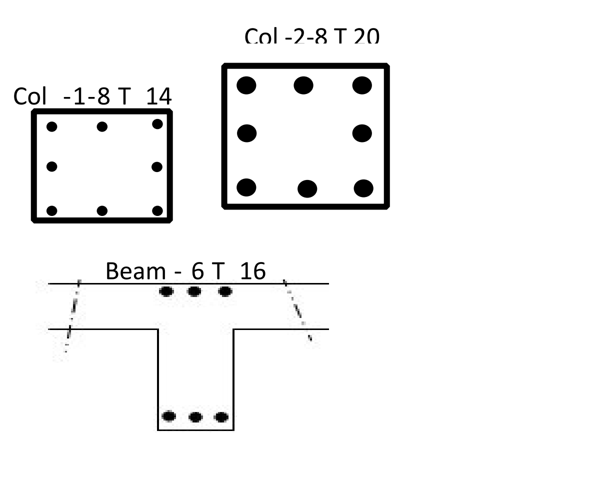


Figure 2. Reinforcement details.

3. Foundation System

The rigid footing and the soil beneath the footing in the zone of influence, considered as a "Gap element", were modeled by a NL Link element in order to model the pounding occurring between the soil and the footing. The nonlinear force–deformation relation for the Gap element is given by

$$f = \begin{cases} k(d+\delta) & \text{if } d+\delta < 0 \\ 0 & \text{otherwise} \end{cases} \quad (1)$$

Where k is the spring constant (or contact stiffness), d denotes deflection of the Gap element, and d is the initial gap separation ($\delta \geq 0$). Note that d < 0 for opening mode and do 0 for closing mode of the gap.

The vertical and horizontal elastic stiff nesses of the foundations are calculated using the frequency-independent formulas given by FEMA 356(1996) document. Allotey and El Naggar (2003) recommend using an effective shear modulus of 0.8 of the elastic shear modulus of the soil in calculating the vertical stiffness of the foundation. Therefore, the elastic shear modulus, G_0, of the soil was calculated from its shear wave velocity and mass density, then an effective shear modulus, $G=0.8\ G_0$, was used in calculating the vertical and horizontal stiff nesses of the footing.

4. Definition of Performance Levels

The design objective in current building codes address life safety, control damage in minor and moderate earthquakes, and prevent collapse in a major earthquake. FEMA 356 gives deformation capacities for the inelastic components corresponding to the three target performance levels for structural components as shown in Figure 3, namely: immediate occupancy (IO), life safety (LS) and collapse prevention (CP)

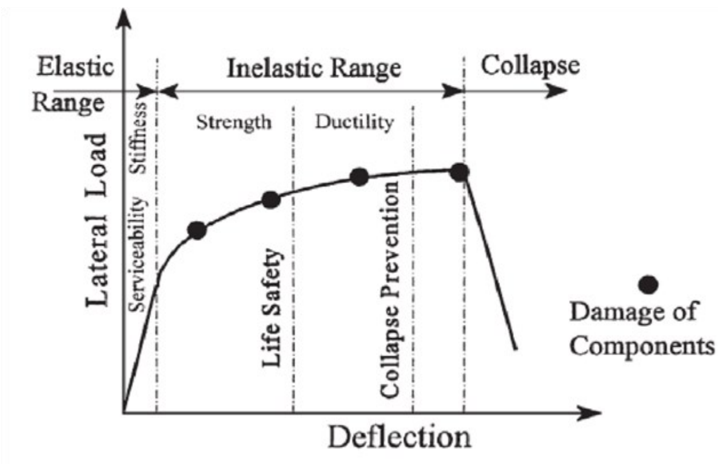


Figure 3. Typical performance curve for the structure.

The calculation of the flexural deflection of an RC member can be carried out by two different main methods. The first uses the finite element approach but needs a huge amount of computational effort, even for the fiber-model-based analysis. When the number of fibers in the cross section and the number of segments, along the member length, are not low, and a hysteretic analysis for a cyclic loading with a variable axial load is needed, the amount of required memory and computation are not comparable with other methods based on the assumption of a plastic hinge (Figure 4). It is evident that in the finite element method, the method of displacement approximation plays a significant role in the accuracy of the results.

In practical use, most often, the default properties provided in FEMA-356 (1996) and ATC-40 (1996) documents are preferred, due to convenience and simplicity. These default Properties can be implemented in well-known linear and nonlinear static and dynamic analysis programs such as DRAIN-2DX (1993), DRAIN-3DX (1994), PERFORM-2D, and SAP200. Some programs (i.e. SAP2000) have already implemented these default nonlinear properties. The use of this implementation is very common among the structural engineering profession and researchers. Mehmet Inel and Hayri Baytan Ozmen (2006) investigated the effect of plastic hinge properties in nonlinear analysis of reinforced concrete buildings, these analyses indicated that it is possible use this option (default-hinge) but cautiously, because the misuse of defaulthinge properties may result in relatively high displacement capacities.

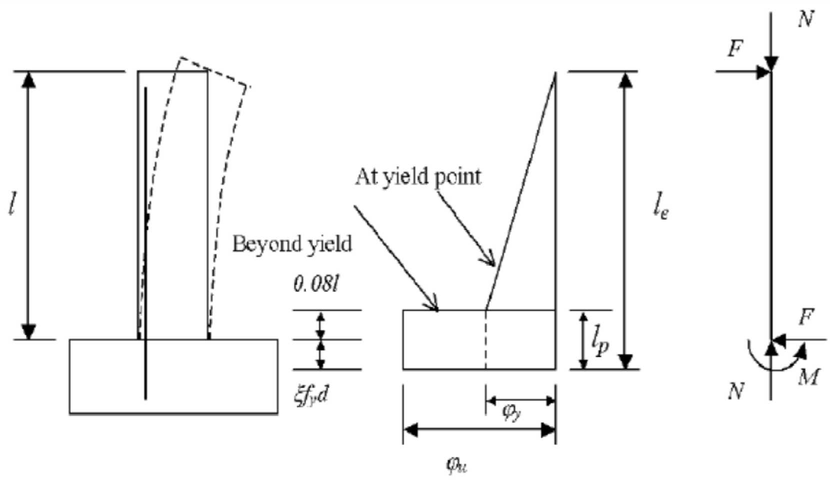


Figure 4. Typical plastic hinge method (Pristley and Park)

The column plastic hinge can be modeled in SAP2000 as a lumped plasticity model, using the Interaction PM_2M_3 Hinge. The applicability and limitations of the Interaction PM_2M_3 Hinge are similar to those of uncoupled hinge, except for its consideration of coupled behavior of the column in both orthogonal bending directions. The configuration of the column cross section must be taken into account in a separate moment-curvature analysis and interaction diagram, carried out to determine the nominal capacity M_{ne}, plastic capacity M_p, and ultimate capacity Mu of the column, as well as the rotations (θ) or curvatures (ϕ) related to those values, in discrete bending directions of the column. Whereas hinge based on bending moments were assigned to beam. Figure 6 shows the load-deformation curve of the hinges. The hinge properties and modeling parameters A, B, C, D, and E were specified according to FEAM 356 (2003).

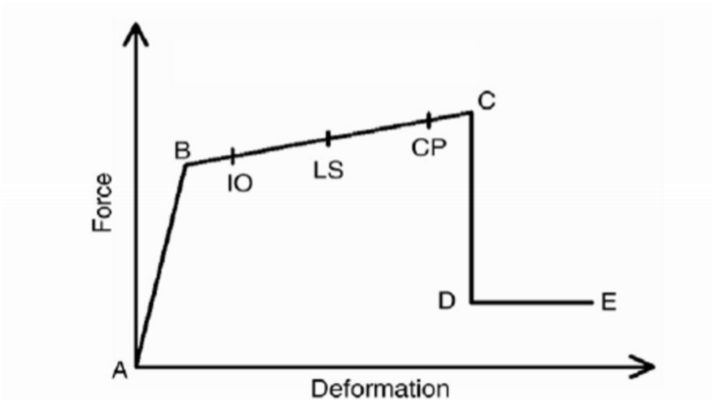


Figure 5. Force–deformation relationship of a typical plastic hinge.

The following points should be noted:
Point A is always the origin.

- Point B represents yielding. No deformation occurs in the hinge up to point B, regardless of the deformation value specified for point B. The displacement (rotation) at point B will be subtracted from the de formations at points C, D, and E. Only the plastic deformation be yond point B will be exhibited by the hinge.
- Point C represents the ultimate capacity for analysis.
- Point D represents a residual strength analysis.
- Point E represents total failure.

5. Earthquake Ground Motion

The accelerograms used in this investigation are the horizontal and vertical components of the New Hall earthquake record, figure 3. These records are believed to be representative of strong earthquake. Studies by Clough and Benuska (1966) indicate that structural response depends primarily on the peak acceleration impulse in the ground motion and that continuing motions of smaller amplitude have only a small effect on the maximum response. Therefore the duration of the earthquake used in this analysis was primarily limited to the first fifteen seconds of the earthquake. The peak ground acceleration of the horizontal component is 0.578 g and that of the vertical one is 0.537g.

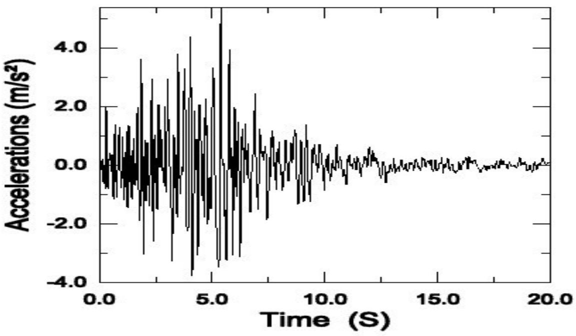


Figure 6. Earthquake records New hall accelerogram (north-south component)

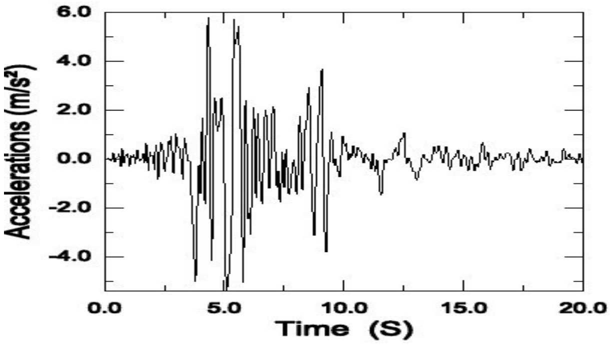


Figure 7. Earthquake records New Hall accelerogram (vertical component)

Acceleration response spectra of linearly elastic system with 0.05 damping subjected to the records are shown in figure 8. Comparison of the spectra shows clearly that the peak vertical acceleration falls in the small period range.

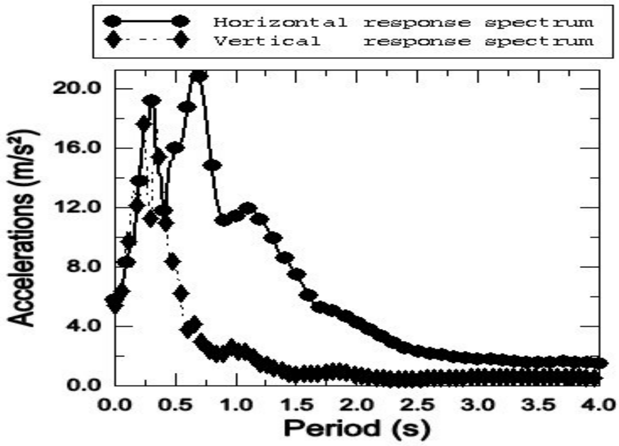


Figure 8. Acceleration response spectra of the records used.

6. Nonlinear Dynamic - Analysis Time History Analysis (THA)

In transient dynamic analysis, the following system of dynamic equilibrium equations is solved at each time t, Eq. 1.

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + (f_{\text{int}}(\{u\},\{\dot{u}\},\{\varepsilon\},\{\sigma\},t,\dots\dots)) = \{f_{\text{ext}}(t)\}$$

where $[M]$ and $[C]$ are the mass and damping matrices respectively, and $\{f_{\text{ext}}(t)\}$ is the vector of external forcing functions. The vectors $\ddot{u}(t)$, $\dot{u}(t)$ and $u(t)$ are the resulting accelerations velocities and displacements, respectively. The vector $f_{\text{int}}(\{u\}$is the internal set of forces opposing the displacements and is usually dependant on the displacements, velocities, and field of strains $\{\varepsilon\}$and stresses$\{\sigma\}$.

In this study, a Rayleigh type damping is adopted. It is based on a linear combination of the mass matrix $[M]$ and the linear elastic stiffness matrix or the tangent stiffness $[k]$ as follows, Eq. 2:

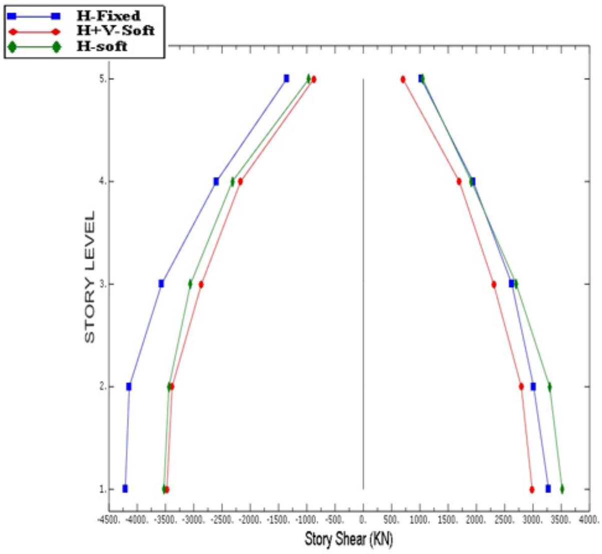
$$\alpha[M] + \beta[K]$$

where the coefficients α et β are determined to provide for two selected damping rations for two specific modes of vibration, ie the first two modes. The direct integration of the above equations is required. One of the most widely used methods is the HilbertHughes-Taylor method [15]. This method has superior numerical dissipation characteristics over the widely used Newmark method. For large number of degrees of freedom, the numerical damping feature is essential to suppress undesirable higher modes.

7. Results and Discussions:

7.1 Shear Forces

Figures 9 and 10 shows the envelope of storey shear demand of five and nine storey structures respectively. They indicate that the SSI with the horizontal excitation only results in a decrease of 16% in the base shear compared to the fixed based, but the analyses with SSI plus a combination of horizontal and vertical motions, the shear demand is decreased by 15% compared to the case of horizontal ground motion without and with SSI.



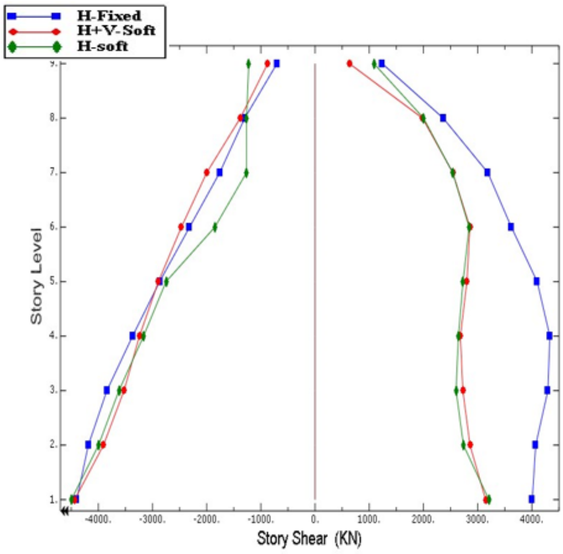
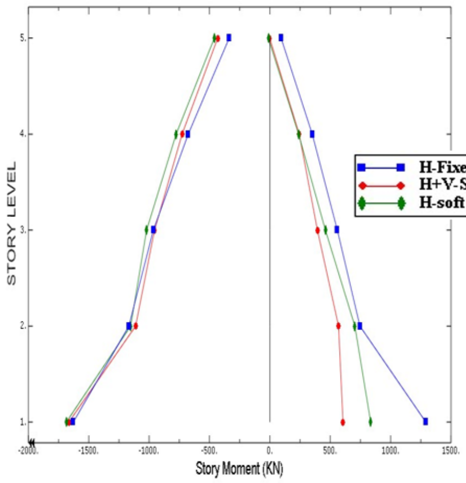


Figure 9. Five storey base shear.

Figure 10. Nine storey base shear.

7.2 Moment Demand

Figures 11 and 12 show the envelope of storey moment demand in interior and exterior columlns of the five storey building increased by 3% in analyses SSI plus the horizontal ground motion compared to the fixed case. However, for analyses involving SSI and both ground motions, the moment demand is decreased by 15% compared to analyses considering SSI and horizontal ground motion.



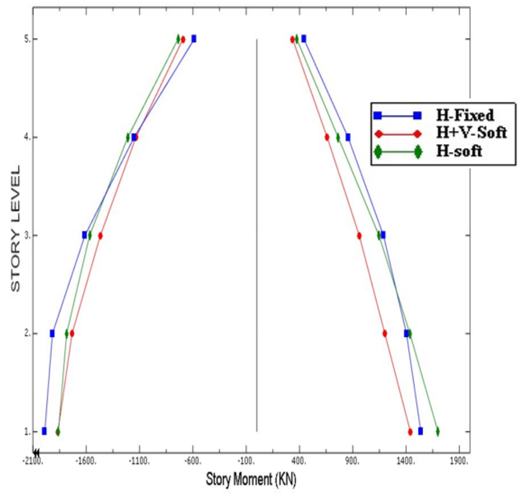


Figure 11. Five storey structure moment‟s exterior column

Figure 12. Five storey structure moments interior column.

Figs. 13 and Figure 14 compare the moment demand in the interior and exterior columns of the nine storey structure. The moment demand for analyses with SSI and horizontal ground motion component is increased by 28% compared to the fixed case. But for analyses with SSI and a combination of the horizontal and vertical ground motions, the moment demand is increased by 2% in compression compared to the case of the flexible footing under horizontal ground motion.

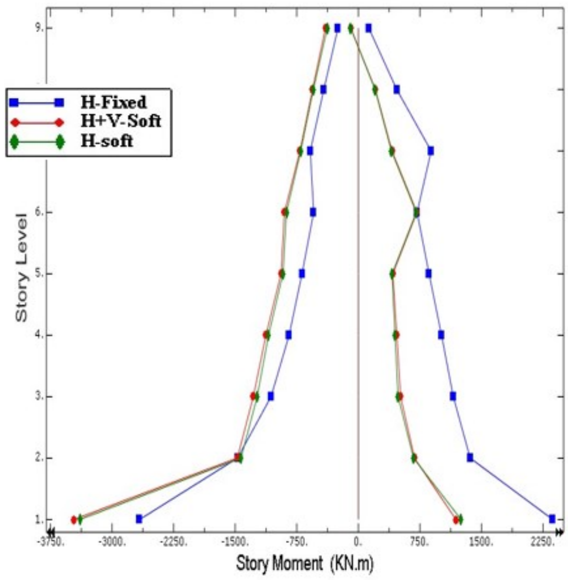


Figure 13. Nine storey structure moments exterior column

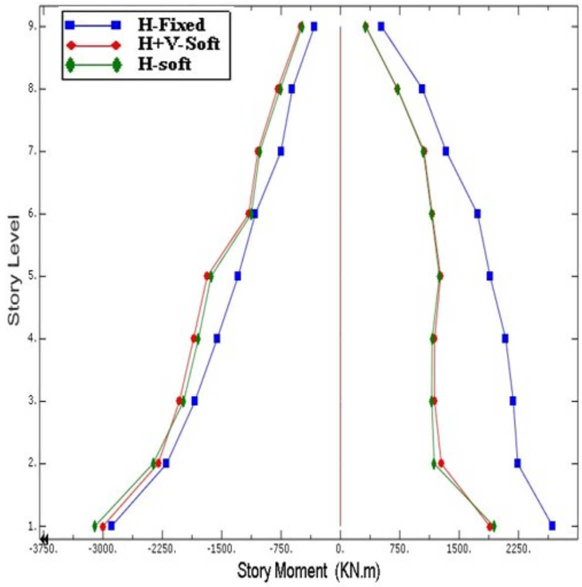


Figure 14. Nine storey structure moments interior column

7.3 Displacement

Generally, the displacements for analyses with SSI are increased compared to analyses fixed for the nine storey structure irrespective of the number of ground motions (refer Figures 15 to 16). For the five storey building, the displacements are increased compared to the fixed case when SSI and a combination of horizontal and vertical ground motions are considered.

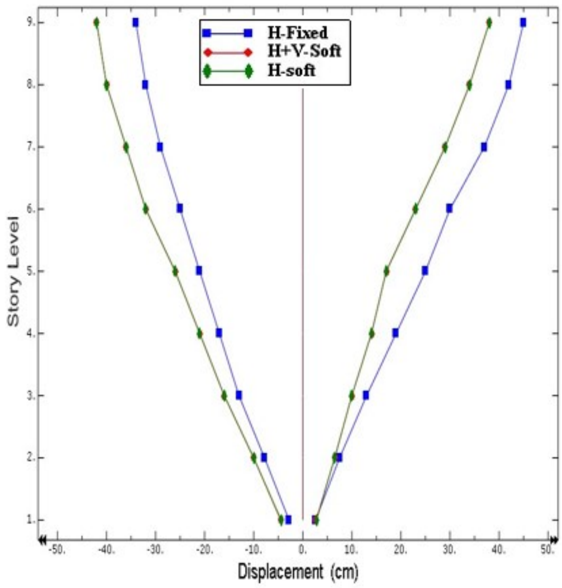


Figure 15. Horizontal Displacement of Nine Storey Structure

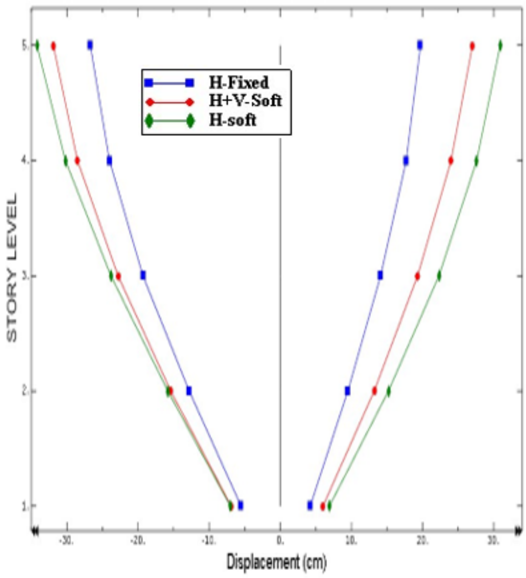


Figure 16. Horizontal Displacement of Five Storey Structure

8. Formation of Plastic Hinges and Accumulated Plastic Deformation:

The locations of plastic hinges formed in the 5 storey structures under New Hall earthquake are shown in Figure 17. First, it should be noted that all hinges were in the elastic domain before 4 s. Afterwards, there were some plastic deformations in the beams and the columns. SSI and a combination of horizontal and vertical ground motions can alter considerably the plastic hinges patterns compared to the fixed case. For instance, for both structures, the hinges at the base of the columns yielded at event E under SSI and both excitations.

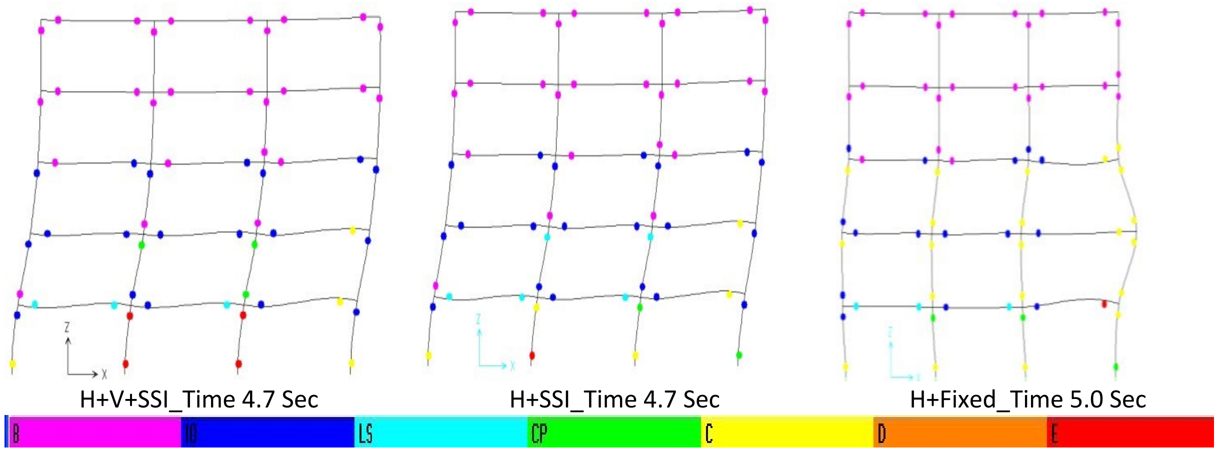


Figure17. Location of plastic hinges caused by New Hall earthquake

9. Conclusions

The results show that the SSI effects increases the base shear and moment demand on buildings founded on soft soil. This is contrary to the common assumption that SSI effect on seismic forces is always favorable, postulated by almost all the design codes. It is clearly demonstrated that the base shear and moment demand on short-period buildings could increase by up to 25% of the fixed base buildings values.

The deformations of the structural components of the buildings have also been affected by the SSI. The deformations of buildings with flexible bases have shown a considerable increase that ranged from 23% compared to the fixed base case for buildings founded on soil flexible. This would in turn increase the lateral deflection of the whole building. Thus, SSI can have a detrimental effect on the performance of buildings.

The effect of vertical acceleration is more pronounced in low and medium building, The SSI increased the displacements compared to the fixed case, this increase is absorbed by the energy dissipated in the structure which results in decreased in the forces, under combined vertical and horizontal ground plus SSI, the displacement is decreased compared to the case of only horizontal component plus SSI resulting in an increased force.

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Conflict of Interests

The authors declare no conflict of interest.

References

Allotey NK, El Naggar MH. Analytical moment–rotation curves for rigid foundations based on a Winkler model. Soil Dyn Earthquake Eng 2003;23(5): 367–81. https://doi.org/10.1016/S0267-7261(03)00034-4

Applied Technology Council, ATC-40. Seismic evaluation and retrofit of concrete buildings, vols. 1 and 2. California; 1996

Anderson DL. Effect of foundation rocking on the seismic response of shear walls. Can J Civil Eng 2003;30(2):360–5. https://doi.org/10.1139/l02-053

Antoniou, S., (1997). "Shear Assessment of R/C Structures under Combined Earthquake Loading", MSc Dissertation, ESEE, Imperial College.

Chaallal O, Ghlamallah N. Seismic response of flexibly supported coupled shear walls. J Struct Eng 1996;122(10):1187–97. https://doi.org/10.1061/(ASCE)0733-9445(1996)122:10(1187)

Chen X, Lai Y. Seismic response of bridge piers on elasto-plastic Winkler foundation allowed to uplift. J Sound Vib 2003;266(5):957–65. https://doi.org/10.1016/S0022-460X(02)01382-2

Collier, C.J. and Elnashai, A.S., (2001). "A Procedure for Combining Vertical and Horizontal Seismic Action Effects", Journal of Earthquake Engineering, Vol. 5 (4), 521-539. https://doi.org/10.1080/13632460109350404

Computers and Structures Inc; 2002.RPA 2003, Réglement Para-Sismique Algérien, DTR, 2003.

CSI. SAP2000 V-14. Integrated finite element analysis and design of structures basic analysis reference manual. Berkeley (CA, USA)

Clough, R.W, (1966), "*The Effect of Stiffness Degradation on Earthquake Ductility Requirements,*"" Report N66-16, Structural Engineering Laboratory, University of California, Berkeley, California, 1966. FEMA 256, (2003).

"NEHRP Guidelines for the Seismic Rehabilitation of Buildings", Federal Emergency Management Agency, 1996.

Filiatrault A, Anderson DL, DeVall RH. Effect of weak foundation on the seismic response of core–wall type buildings. Can J Civil Eng 1992;19(3): 530–9. https://doi.org/10.1139/l92-062.

Ghobarah, A. and Elnashai, A.S., (1998) . "Contribution of Vertical Ground Motion to the Damage of RC Buildings", 11 th European Conference on Earthquake Engineering, Paris.

H. Shakib , A. Fuladgar (2003). « Effect of vertical component of earthquake on the response of pure-friction base-isolated asymmetric buildings", Journal of Engineering Structures, Proceeding of ELSEVIER, Vol 25 (2003), pp. 1841–1850.

Housner GW. The behavior of inverted pendulum structures during earthquakes. Bull Seismol Soc Am 1963; 53:403–17. https://doi.org/10.1785/BSSA0530020403.

Javeed A. Munshi and Satyendra K. Ghosh (1997). " Analyses of seismic performance a code designed reinforced concrete building", Journal of Engineering Structures, Proceeding of ELSEVIER, Vol 20 (1997),pp.606-618. https://doi.org/10.1016/S0141-0296(97)00055-2.

Mwafy, A.M. and Elnashai, A.S., (2006). "Vulnerability of Code-Compliant RC Buildings under Multi-Axial Earthquake Loading", 4th International Conference on Earthquake Engineering, Taipei, Taiwan, Paper No. 115.

Mehmet Inel, Hayri Baytan Ozmen. Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. Engineering Structures 2006 ; 28 : 1494–1502.https://doi.org/10.1016/j.engstruct.2006.01.017

Newmark, N.M., Blume, J.A. and Kapur, K.K. (1973). "Seismic Design Spectra for Nuclear Power Plants", Journal of the Power Division, Proceedings of ASCE, Vol. 99, No. PO2, pp. 287–303.https://doi.org/10.1061/JPWEAM.0000753

Psycharis IN, Jennings PC. Rocking of slender rigid bodies allowed to uplift. Earthquake Eng Struct Dyn 1984; 11:57–76. https://doi.org/10.1002/eqe.4290110106

Prakash V, Powell GH, Campbell S. Drain–2DX base program description and user guide version 1.10. Structural engineering mechanics and materials report no. UCB/SEMM93-18. Berkeley: University of California; November 1993.

Prakash V, Powell GH, Campbell S. Drain–3DX base program description and user guide, and element description version 1.10. Structural engineering mechanics and materials report no. UCB/SEMM-94-07 and 94-08. Berkeley: University of California; August 1994.

RAM International. Perform-2D. West Carlsbad, CA 92008 at http://www.ramint.com.

Song Y, Lee D. Improved two-spring model for foundation uplift analysis. Comput Struct 1993;46(5):791–805. DOI:10.1016/0045-7949(93)90142-Z

Wilson, E.L. (1998). Three Dimensional Static and Dynamic Analysis: A physical Approach with emphasis on Earthquake Engineering. 2nd edition. Computer and Structures, Inc. Berkeley, California.