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Effect of Soil Structure Interaction on Seismic Pounding in Adjacent RC Buildings

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Abstract

This study presents the effects of soil structure interaction on the seismic pounding response of adjacent RC buildings. Regular 2D frame of 3, 5 and 7 storey buildings considering two buildings at a time are taken for analysis with both fixed and flexible base cases with no gap between the buildings. Non-linear modeling and analysis of building and soil is accomplished in OpenSees, a finite element based software framework. 7 different ground motions are used for time history analysis. The results are obtained in the form of pounding force and storey displacement values. The results indicate an increase in pounding force value due to consideration of SSI. The force has increased by 34% for pounding between 3 and 5 storey buildings whereas it has increased by 62% for pounding between 3 and 7 storey buildings. Due to combined effect of pounding and SSI, there is increase in displacement of 7 storey building by upto 104% on the pounding side when collided with 3 storey building whereas there is 12% reduction for 3 storey building on the pounding side for the same case. Compared to shorter building, the taller building is more affected due to SSI which becomes more vulnerable with the combined effect of both pounding and SSI. Finally, a relation to approximate the pounding force that may be considered in the design of buildings is proposed.

Keywords

Pounding, Soil-structure interaction, Non-linear analysis, OpenSees

1. Introduction

Nepal lies in seismically active zone which is prone to moderate to strong ground shaking causing damage to infrastructures. Rapid population growth, unplanned urbanization and increased land costs in the cities has changed building constructions to be built very close to adjacent buildings with little to no gap between them. Due to this, pounding between adjacent buildings may occur during seismic events.

1.1 Pounding

Pounding is the impact of adjacent buildings on each other during seismic events due to the out of phase vibration when the separation gap between them is insufficient than that required for the free vibration. Pounding is one of the major causes of damages in buildings during earthquake events. Pounding induced building damage happened during the 1985 Mexico city and 1989 Loma Prieta earthquakes [1]. Pounding significantly amplifies the seismic response of adjacent buildings which could lead to building damage [2]. Several researchers [1, 3, 4, 5] had the primary notable contribution in the study of pounding phenomenon.

1.2 Soil Structure Interaction (SSI)

In conventional building design, buildings are generally considered to be fixed at the base. But this assumption is valid only for structures founded on rocky strata or soil with high stiffness. Soil structure interaction refers to the response of structure due to influence of soil and the response of soil due to motion of structure [2]. SSI analysis of buildings takes the combined response of the superstructure, its foundation system, and the soil beneath under seismic excitation through settlement, sliding and rocking behavior of the foundation. SSI has been traditionally thought to be of little benefit for typical seismic analyses as it reduces the demand on the buildings. But it might not always be the case as the flexibility of whole system is increased due to consideration of leading to increased time period, increased displacements and reduced base shear [6].

1.3 Pounding Considering SSI

SSI and influence on the pounding of the adjacent buildings have been studied by several researchers [2, 5, 7, 8]. Rahman et. al. [7] found shift in period due to underlying soil altered the time at which first impact occurred which had consequences on the subsequent poundings. Naserkhaki et. al. [2] investigated two MDOF models with lumped mass, viscous dampers and linear springs. SSI was taken into account. A linear viscoelastic impact element was used the study. Results indicated that the underlying soil negatively impacts the response of buildings. Madani et. al. [8] studied the effects of pounding and SSI on different combinations of adjacent steel buildings resting on soil. Time history analysis using 7 different earthquake records was done on the model. Soil was modeled using BNWF model. They concluded that soil flexibility had increasing effect on the pounding forces and made it happen even at farther clear distances.

As soil structure interaction is not generally considered in design of buildings, the provided gap may not be sufficient to prevent pounding. Thus, effets of SSI should be studied properly and incorporated in design to be safe from the damage during earthquake events.

1.4 Objectives

1. To determine the effects of SSI on pounding between adjacent RC buildings.
2. To evaluate the pounding force required to be considered in design of buildings.

1.5 Limitations

1. The study only considers regular 2D buildings with equal storey heights.
2. Stiffness of infill wall is not considered.

2. Structural Model

2.1 Building Frame

For the purpose of our study, regular 2D framed buildings of varying heights of 3, 5 and 7 storey with 3 bays are taken. M25 concrete with Fe500 steel rebar has been considered for the RCC. The beam column cross-section properties for the buildings are shown in table 1. Same storey height of 3m has been taken for all buildings. The bay length is also constant for all cases equal to 4m.

Table 1: Building Description

Storey	Column	Beam
3	350×350	250×350
5	450×450	300×450
7	550×550	350×550

Same storey height of 3m has been taken for all buildings. The bay length is also constant for all cases equal to 4m.

The three building models are shown in figure 1. The schematic model of superstructre is shown in figure 2.

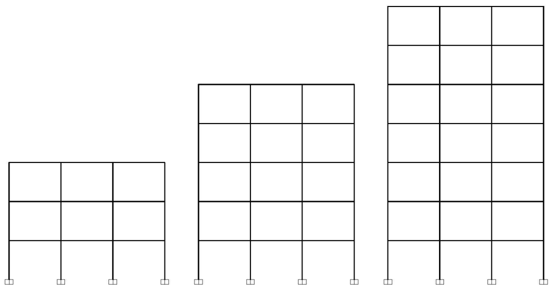

Figure 1: 3, 5 and 7 Storey Buidings (Fixed Base)

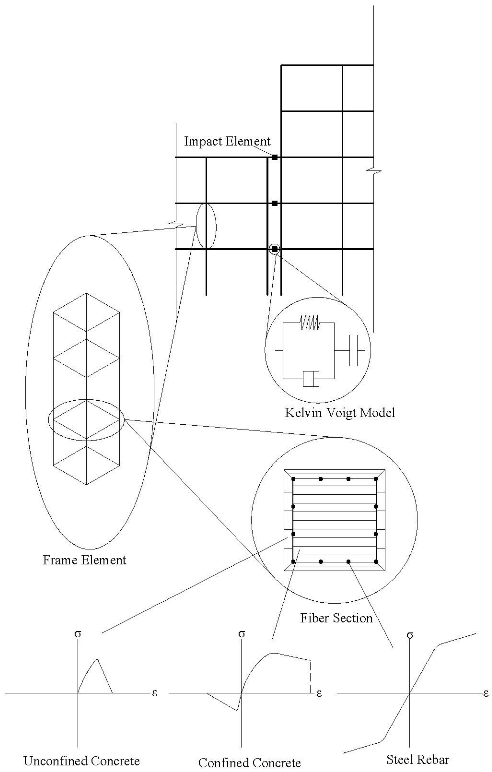

Figure 2: Schematic Diagram of Superstructure, Gap Modeling and Material Behavior

2.2 Non-linear Modeling of Members

The non-linear behavior of beams and columns of the building frames are modeled with force based distributed plasticity approach. The cross section consists of a number of meshed fibers running from one end to other end of the member. Each fiber is assigned with uniaxial stress strain behavior of a particular material.

The RC section is made up of three distinct materials; steel rebars, confined concrete inside the stirrups and unconfined concrete. The steel rebars are defined using steel02 material available in OpenSees [9] whereas concrete02 material is used to define both confined and unconfined concrete. The properties of rebars is obtained from Giuffre Menegotto Pinto model whereas, confined concrete properties are determined from Mander's confinement model [10].

2.2.1 Validation of RCC Modeling

For validation of the used models for RCC, an experiment conducted by Tanaka and Park was modeled in OpenSees and its cyclic displacement curve was compared with the experimental result. Tanaka and Park specimen #5 is shown in figure 3.

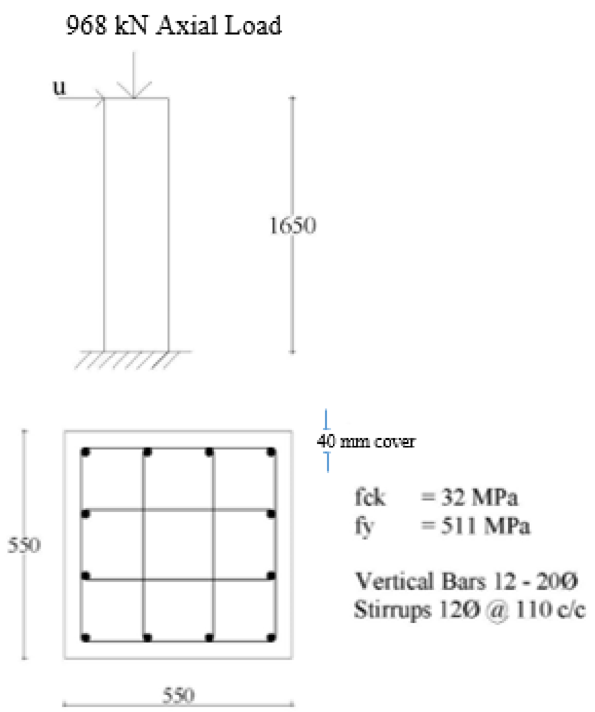

Figure 3: Experimental Setup and Cross-Section Details

From figure 4 it is seen that the displacement obtained from numerical model in good agreement with experimental results.

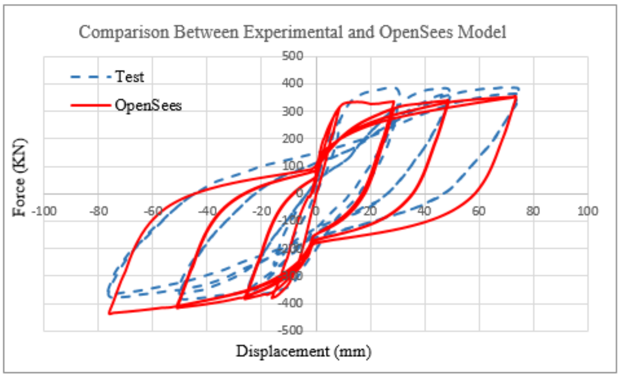

Figure 4: Cyclic Displacement Comparison Between Experimental and OpenSees Model

2.3 Pounding Model

Pounding phenomenon between adjacent buildings is simulated by using force based contact elements. A viscoelastic gap element also known as Kelvin-Voigt model is used in our study. This model was first proposed by Anagnostopoulos [3] and is widely used in modeling pounding phenomenon due to its simplicity and ability to model energy dissipation.

This model consists of a spring and damper material in parallel combination and gets activated when the predefined gap closes. equation 1 gives the equation of impact force for the model during pounding.

$$F = k_k(u_1 - u_2 - gap) + c_k(\dot{u_1} - \dot{u_2});\ u_1 - u_2 > gap \quad (1)$$

Here, u_1 and u_2 are displacements of adjacent buildings and gap is the separation distance between the two. c_k is the damping coefficient given by Equation 2. k_k is the stiffness of impact material and m_1, m_2 are the masses of the connecting nodes. The damping ratio is derived from the restitution coefficient e, given by equation 3. A rational value of 0.65 was taken for coefficient of restitution [5].

$$c_k = 2\xi\sqrt{k\frac{m_1\, m_2}{m_1 + m_2}} \quad (2)$$

$$\xi = -\frac{ln(e)}{\sqrt{\pi^2 + (ln(e))^2}} \quad (3)$$

To calculate the stiffness value for the model, several researchers have proposed different relations. The relation used in this study was is shown in equation 4.[11]

$$k_k = \left[\frac{1-\nu_1^2}{E_{Dyn,1}} + \frac{1-\nu_2^2}{E_{Dyn,2}}\right]^{-1} \quad (4)$$

Where, ν is the Poisson's ratio and E_{Dyn} is the dynamic modulus of elasticity given by equation 5.

$$E_{Dyn} = 5.82\left(E_{Static}\right)^{0.63} \quad (5)$$

Poisson's ratio and static modulus of elasticity for concrete were taken as 0.2 and 25 GPa respectively.

The accuracy of this model was validated by Jankowski and Mahmoud (2015) by comparing the experimentally obtained displacement time history with that obtained from numerical modeling.

2.4 SSI Model

The interaction between the structure and underlying soil is modeled using Beam on nonlinear Winkler foundation (BNWF) model [12]. This model was first proposed for pile foundations by Boulanger et. al. [13] and Harden [14] suggested to use this model for shallow foundations. Later, the model was calibrated for shallow foundations by Raychowdhury. This model accurately predicted experimentally measured footing response in terms of moment, shear, settlement and rotation demands.

The BNWF model consists of series of vertical zero length elements (Q-z springs) below the footing and two horizontal zero length springs (T-x and P-x springs). Q-z springs simulate the settlement and rocking behavior of the footing whereas P-x spring simulate lateral passive behavior of soil and T-x spring simulate sliding behavior of soil. The model is shown in figure 5.

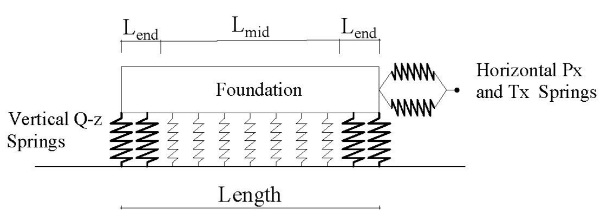

Figure 5: BNWF Model

The force-displacement behavior of Q-z springs is defined by QzSimple2 material in OpenSees library. This material has asymmetric hysteretic behavior with large capacity in compression and a small capacity in tension. PySimple2 material in OpenSees is used to model P-x springs which has pinching hysteretic behavior to model the potential gaping of embedded shallow foundation under seismic loading. TzSimple2 material in OpenSees is used to model T-x springs to account for frictional behavior of foundation due to sliding.

A non uniform distribution of vertical springs is present with stiffer springs at the edges. This is to account for the stiffened soil due to foundation rotation. The softer inner springs account for remaining vertical stiffness. The distribution length and stiffness of springs are made such that provided vertical and rotational stiffnesses should be equal to the vertical and rotational stiffnesses of foundation. The equations 6, 7, 8 satisfy the above condition.

$$K_{mid} = \frac{K_z}{BL} \quad (6)$$

$$K_{end} = K_{mid} + \left(\frac{K_{\theta y}}{I_y}\right) C_{R-V}^{K} \quad (7)$$

$$C_{R-V}^{K} = \frac{K_{\theta y} - (K_z/A) I_y}{K_{\theta y}} \quad (8)$$

Here, K_{mid} and K_{end} are the stiffness of vertical springs in middle and end zones. K_z and $K_{\theta y}$ are the vertical and rotational stiffness of the foundation. I_y is moment of inertia of foundation and C_{R-V}^{K} is the residual rotational stiffness ratio. The width of exterior zone of foundation containing stiffer springs is given by equation 9.

$$L_e = 0.5L - L\left[\frac{1}{8}\left(1 - C_{R-V}^{K}\right)\right]^{1/3} \quad (9)$$

K_z and $K_{\theta y}$ are calculated from Gazetas equations [15] which are shown in equations 10 and 11.

$$K_z = \frac{GL}{1-\nu}\left[0.73 + 1.54\left(\frac{B}{L}\right)^{0.75}\right] \quad (10)$$

$$K_{\theta y} = \frac{G}{1-\nu} I_y^{0.75}\left[3\left(\frac{L}{B}\right)\right] \quad (11)$$

Horizontal stiffness according to Gazetas is given in equation 12.

$$K_x = \frac{GL}{2-\nu}\left[2+2.25\left(\frac{B}{L}\right)^{0.85}\right] + \frac{0.1\ GL}{0.75-\nu}\left[1-\frac{B}{L}\right] \tag{12}$$

The ultimate bearing capacity, ultimate passive resistance and ultimate sliding resistance are required for nonlinear modeling of springs. The ultimate bearing capacity is determined from Meyerhof's equations.

The other necessary parameters in defining the spring model in OpenSees can be found in research paper by [8, 12, 14].

2.5 Loading

Gravity loading and earthquake loading has been considered in this study. For gravity loading, dead load and live load are considered. Details about earthquake loading are given in section 3.

For the dead loads, self weight of slab, beam, column and 230mm brick wall are taken. 1 kN/m^2 floor finish is taken on the slabs. Live load of 2 kN/m^2 is taken for all cases out of which 30% from inner storey is taken for gravity load.

Unit weight of RCC is taken as 25 kN/m^3 and that of brick masonary is taken as 20 kN/m^3.

3. Earthquake Records

Selection of earthquake time history plays significant role in the non-linear analysis of structures. NBC recommends to use minimum 3 ground motions for 2D analysis and a minimum of 7 ground motions allows to take average of the structural responses. For this study, 7 ground motions are selected. Selection for ground motion is done in Pacific Earthquake Engineering Research Center (PEER) database.

The selected ground motion should correspond to the seismic hazard level of the site. It, in our case, is defined by the design response spectrum provided in NBC 105:2020 modified for Kathmandu valley with soil type D, zone factor 0.35, importance factor 1.0 and overstrength factor 1.5. The magnitude range (Mw 6.5-8), source to site distance (0 km to 150 km) and shear wave velocity (50 m/s to 200 m/s) are taken for searching [16]. From the search results, the ground motions which have similarity in shape of response spectrum with that of target response spectrum and scale factor ranging between 0.33 to 3 were selected and are shown in table 2.

Table 2: List of Earthquake Data

Year	Location	Predominant Period (s)
1970	Imperial Valley	0.50
1989	Loma Prieta	1.06
1995	Kobe	1.12
1999	Kocaeli	0.42
1999	Chi Chi	0.62
2010	Darfield	0.44
2015	Gorkha	1.38

The response spectrum of the ground motions in table 2 are shown in figure 6.

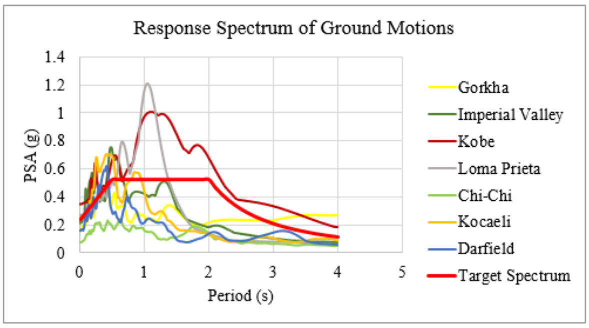

Figure 6: Response spectrum of selected earthquakes

These ground motions are matched to the target spectrum to be able to use in time history analysis. Spectral matching is performed in Seismomatch 2022 software. The response spectrum of ground motions after matching are shown in figure 7.

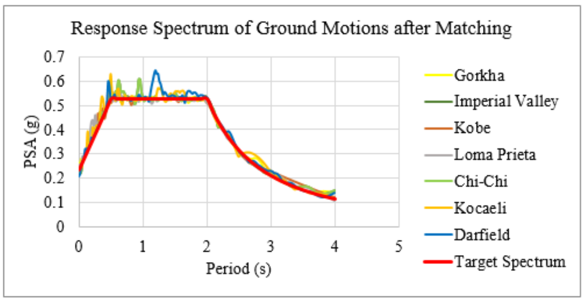

Figure 7: Response spectrum of earthquakes after matching

4. Non-linear Time History Analysis

Non linear time history analysis of the system was done in OpenSees. Three pair of buildings; 3v5, 3v7 and 5v7 were taken for adjacency case. As the direction of seismic excitation also affect the seismic response, the mirrored configurations 5v3, 7v3 and 7v5 were also analyzed as shown in figure 8. 2 different base conditions were taken for all the building pairs; one as a fixed base case and another as soil structure interaction case. All these combinations of building pairs were analyzed for 7 different ground motions as mentioned in table 2. The seismic gap in all these cases were taken to be 1 mm simulating the case of buildings constructed without any gap between them. Hence, total of 84 different cases were taken for time history analysis.

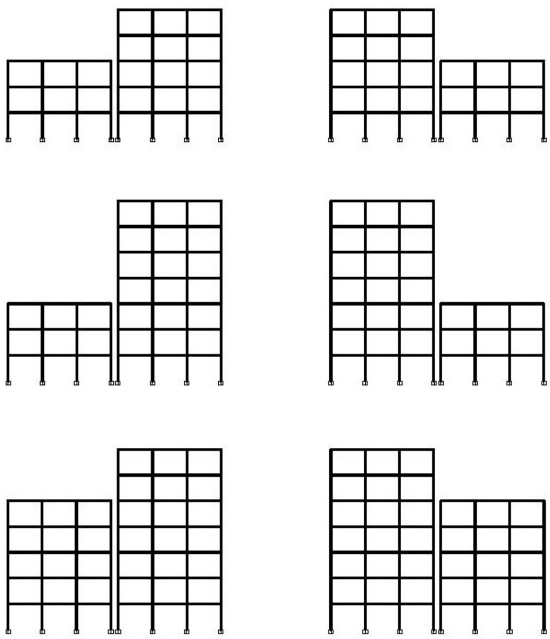

Figure 8: Building Configurations Taken for Study

The time history analysis results obtained are presented taking the maximum pounding force, storey displacement and interstorey drift averaged between the 7 ground motions. Due to the space limitations, results are presented only for the effect of pounding on 3 storey building.

5. Results and Discussion

5.1 Effect of SSI on Fundamental Time Period

Fundamental time period for all 3 buildings were obtained from eigen value analysis for both fixed base and SSI cases. The results are presented in table 3.

Table 3: Fundamental Time Period

Case	Fixed	SSI	Period Ratio
3 Storey	0.59s	0.66s	1.12
5 Storey	0.68s	0.79s	1.16
7 Storey	0.74s	0.92s	1.24

The fundamental time period increased for SSI case in all the buildings. There is 12% increase in 3 storey building whereas there is 24% increase in 7 storey building. The foundation soil and the structure can be considered as springs which are connected in series due to which the overall stiffness of the system reduces.

5.2 Effect of SSI on Pounding Force

Pounding force was calculated based on equation 1. The force in the top floor of lower storey buildings averaged from 7 ground motions are presented in table 4.

Table 4: Pounding Force in kN

Case	Fixed	SSI
3v5	2766	3708
3v7	3484	5647

Consideration of SSI increases the displacement demand of the buildings due to the introduced flexibility so, even more space is required for free vibration of buildings. Due to this, the pounding force will increase on SSI case compared to fixed base case. As seen from table 4, the pounding force in 3v5 case is increased by 34% and that on 3v7 case is increased by 62% when SSI is considered.

Based on the findings of the study, the equation 1 is proposed to approximate the pounding force that may be considered in the design of buildings.

$$Force(kN) = 550\, N\, \beta^2 \qquad (13)$$

Where,
N = Number of storey of taller building
β = Period elongation ratio due to SSI of taller building

Table 5: Comparison of Pounding Force (Fixed Case)

Case	β	Predicted	Obtained
3v5	1	2750	2766
3v7	1	3850	3484

Table 6: Comparison of Pounding Force (SSI Case)

Case	β	Predicted	Obtained
3v5	1.16	3700	3708
3v7	1.24	5920	5647

5.3 Effect of SSI on Displacement

When SSI is considered in the analysis of single building, the displacement of buildings in all cases are increased. In case of 3 storey building, there is only 13% increase in top storey displacement and 43% increase in inter storey drift value. Whereas in 5 storey building, there is 48% increase in top storey displacement and drift is increased by 62%. In the case of 7 storey building, there is increase in displacement by 100% and drift by 140%. It is observed that the highest drift always occured in the first storey for SSI case. This occurs because the fixed base boundary condition is released [8].

5.4 Effect of SSI on Displacement due to Pounding

The results of combined effects of pounding and SSI on the displacements of adjacent buildings are presented in this section. Graphs of displacement for only 3 storey building are shown here for brevity.

Figures 9 and 10 show the storey displacement for 3 storey building when pounded with 5 storey building in both fixed base case and SSI case. There is reduction in displacement on both sides for fixed base case whereas there is increase in displacement on non-pounding side for SSI case. The combined effect due to pounding and SSI is 5.6% increase on PS and 23% increase on NPS.

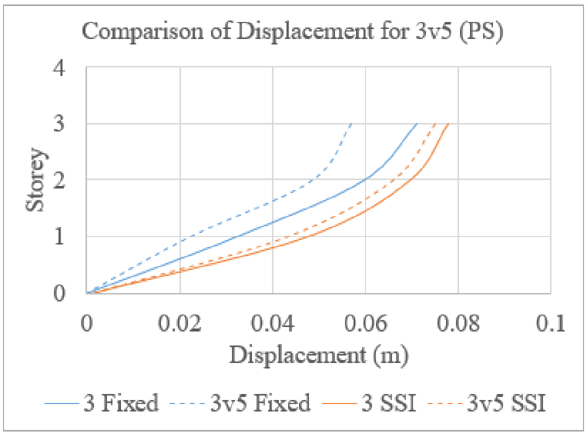

Figure 9: Displacement of 3 Storey Building on Pounding Side (3v5 case)

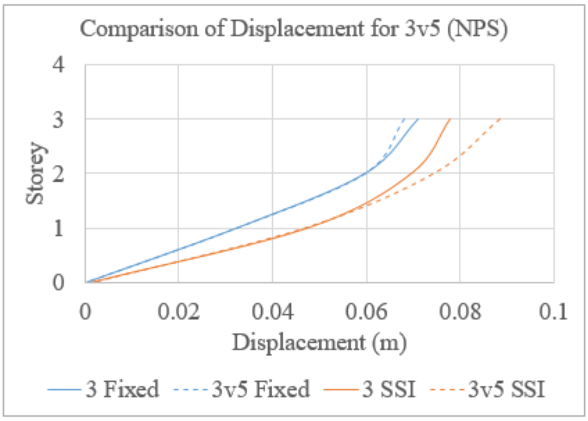

Figure 10: Displacement of 3 Storey Building on Non-Pounding Side (3v5 case)

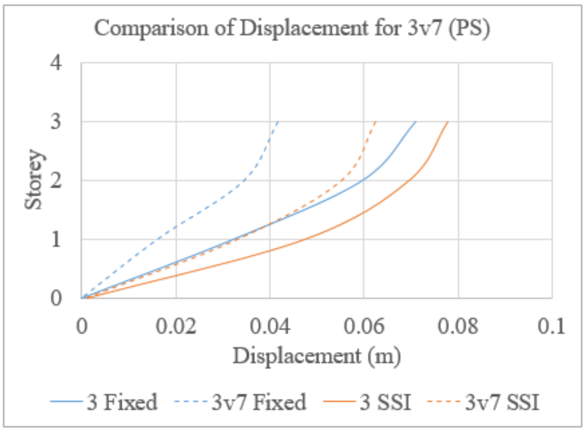

Figure 11: Displacement of 3 Storey Building on Pounding Side (3v7 case)

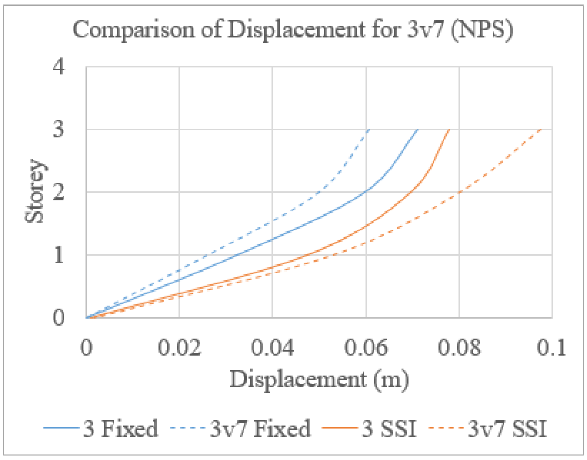

Figure 12: Displacement of 3 Storey Building on Non-Pounding Side (3v7 case)

Similarly, figures 11 and 12 show the storey displacement for 3 storey building when pounded with 7 storey building in both fixed base case and SSI case. The results are similar to 3v5 case but the reduction or increment is greater for this case. The combined effect is 12% reduction in displacement on PS and 37% increment on NPS. The displacements are generally reduced for shorter buildings and increased for taller buildings due to pounding but the displacements in all cases are increased due to SSI.

As for 5 storey building pounding with 3 storey building, the combined effect is 58% increase in displacement on PS and 71% increase on NPS. building. For pounding with 7 storey building, combined effect is 36% increase in displacement on PS and 60% increase in NPS.

For pounding between 7 and 3 storey buildings, the combined effect is 102% increase in displacement on PS and 104% increase on NPS. Combined effect of 85% increase in displacement on PS and 108% on NPS is observed for pounding with 5 storey building.

6. Conclusion

In this research, a comprehensive study was done on the effects of soil structure interaction on the pounding response of RC buildings. Different cases of adjacency with 3v5, 3v7, 5v7 with no gap between them and resting on rigid base as well as flexible base were investigated. 7 ground motions were applied for each building pair for each base condition and maximum averaged responses were taken.

Consideration of soil structure interaction increased the fundamental time period, storey displacement and interstorey drifts. SSI also increased the pounding force in all the floor levels. Considering only pounding caused reduction in seismic responses for shorter building whereas increased them for taller building.

Considering the increased displacement, the taller buildings seem to be more vulnerable to damage due to the combined effect of pounding and soil-structure interaction when compared to shorter buildings.

Finally, based on the findings, an empirical relation to approximate the pounding force that may be considered in the design of buildings that are constructed with no gap in between them is proposed.

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