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Seismic analysis of shear wall buildings incorporating site specific ground response

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Abstract. During earthquake, the motion of ground is affected significantly by source characteristics, source-to-site path properties and local site conditions. Due to the influence of local soil conditions different places experience distinctive amplitude of surface ground motion. Ground response analysis of a specific site utilizing the borehole information at different locations is done in present study. The ground motion with the highest peak ground acceleration for this site obtained from the ground response analysis is used in finite element soil-structure interaction analysis of multi-storey shear wall buildings with various positions of shear walls. The variation in seismic response of buildings and advantageous position of shear wall are determined. The study reveals that providing shear wall at the core of buildings at the specific site is advantageous among all shear wall configurations considered.

Keywords: ground response analysis; soil-structure interaction; shear wall

1. Introduction

Ground responses are essentially influenced by the local soil conditions during earthquakes. The principal components that influence local modifications to the underlying motion are the topography of site and nature of depositional soil (Raju, Ramana *et al*. 2004). Approximation of site-specific dynamic response of a layered soil deposit is pertained to as a site-specific response analysis.

Effect of local soil conditions on intensity of ground shaking is known. The impact of local soil on the 1906 San Francisco earthquake was studied by (Wood 1908, Reid 1910). Later, from recordings of earthquake at sites with different subsurface conditions Gutenberg formulated site-dependent amplification factors. The influence of local site conditions on ground motions have been an area of intense research because the local site effects play a major role in earthquake resistant design of structures.

Site response study of selected sites of New Madrid seismic zone was carried out by Wang, Zeng *et al*. (1996) to study the site effects and strong motion characteristics of the area. New Madrid seismic zone area is susceptible to extreme damage by local site effect due to the thick

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unconsolidated alluvial deposit overlying Paleozoic bedrock. Site response studies in Victoria, B.C., Canada was carried out by Molnar, Cassidy *et al*. (2004) to study the amplification of seismic waves due to the local geology. Due to the impact of local soil, variation in PGA and MMI were observed from the study across greater Victoria. Ground response analysis for seismic design in Fraser river delta, British Columbia to find the significant amplification in the earthquake motions was carried out Uthayakumar and Naesgaard (2004). After the occurrence of few devastating earthquakes in the recent past, the importance of ground response analysis has been attained great momentum in India. A case study on ground response analysis for a site in close proximity to Sabarmati river area, Ahmedabad was carried out by Raju, Ramana *et al*. (2004) to determine the varying degrees of damage in multi-storey buildings due to amplification of ground motion during Bhuj earthquake. This study expressed the engineering importance of site-specific ground response analysis, difficulties faced in carrying out a complete ground response analysis and steps to be adopted in conducting a meaningful site amplification study. The site response study of Vijayawada city which falls in Zone III (IS 1893: 2002) was carried out by Manne, Chowdary *et al*. (2011) to estimate the local site effects of Vijayawada city. Similar site amplification studies and seismic hazard analysis of Coimbatore region carried out by Chandrasekaran, Bharadwaja *et al*. (2012) and for Kolkata Metropolitan District area by Roy and Sahu (2012), Shiuly and Narayan (2011), Shiuly, Sahu *et al*. (2015).

Conventionally, buildings are analysed presuming the base of building to be fixed. Whereas in reality, response of structure is greatly influenced by the supporting soil over which the structure is constructed. Movement of substructure elements due to natural ability of soil to deform affects the response of structure. The significance of considering soil-structure interaction in seismic analysis of structures is learnt from the severity in damage occurred in structural elements during past earthquakes. During earthquakes, displacements of structure and soil are interdependent. This interdependency in response between the soil and structure is termed as soil-structure interaction (SSI).

The potential severities of neglecting the effects of SSI in the seismic design of buildings are shown in the studies carried out by Mylonakis, Nikolaou *et al*. (1997), Roy and Dutta (2001a, b). Bielak (1975), Stewart, Fenves *et al.* (1999a), Stewart, Seed *et al*. (1999b) reported the decrease in lateral stiffness of structural system due to flexibility of supporting soil causing the lengthening in lateral natural. These studies have shown a considerable variation in seismic structural responses due to lengthening of lateral natural period. Related studies on the implications of lengthening of natural period in low-rise buildings having fundamental lateral period in short period region of the design response spectrum was performed by Bhattacharya and Dutta (2004). Soil-structure interaction studies on massive concrete structures using finite element software ANSYS and LS DYNA were carried out by Rajasankar, Iyer *et al*. (2007) to determine the stress resultants in substructure and normal and shear stresses developed at interface between the foundation rock and raft. Analyze of the effect of the liquefaction of sand on the seismic response of the SSI system using ANSYS software was carried out by Li, Lu *et al*. (2004). Dynamic soil-structure interaction assessment of an ethylene tank, in the Philippines was carried out by Lubkowski, Pappin *et al.* (2000) to determine the effects of kinematic interaction on the piled foundations.

An attempt is made in the present study to bring forth the effects of site specific ground motion employing ground response analysis. ProSHAKE software (Lasley, Green *et al.* 2014) was used in ground response analysis to determine the ground level time history of acceleration for an input motion. Geotechnical data from twenty bore holes near to the Arabian Sea coast were considered in the analysis. This specific site has a lot of variation in the geotechnical profile and the depth of bed rock varies from 7m to 15m below the ground level. Due to unavailability of any recorded strong ground motion at the considered study area, Elcentro earthquake motion (1940) possessing a wide band of dominant frequencies (0.39-6.39 Hz) was selected. This ground motion record was scaled down to a maximum acceleration of 0.1g to represent an artificial ground motion expected at the specific site as per the zonal classification for zone III (IS 1893: 2002). The surface ground motion having the highest PGA generated using ProSHAKE was further utilized in the three dimensional finite element soil-structure interaction analyses of multi storey shear wall buildings to evaluate the seismic response and hence to determine the advantageous location of shear wall which attracts the least earthquake force in buildings with aspect ratio (*h/d*) of 1to 4 having natural frequency in the range of 0.31 to 5.55Hz.

2. Soıl-structure interactıon

The interaction among the structure, its foundation and the soil medium below the foundation vary the structural response as anticipated with a fixed base condition under any type of loading. The movement of supporting soil varies the response of structure and the response of structure varies with the movement of soil. SSI analyses are classified into two main categories, namely direct method and substructure method (Wolf 1985). In direct method, the response of integral structure-foundation-soil system is determined by modeling and analysing the integrated system in a single step. However, in substructure method, analysis in parts is performed for the components of the system and the final response is obtained based on the principle of superposition.

Soil medium in SSI problems are generally modelled using Winkler spring model and elastic continuum model. Soil medium on which the foundation slab lies is assumed to be made of a series of closely spaced springs in Winkler spring model. Soil medium is assumed to be a finite continuum and divided into elements interconnected at finite number of nodes in elastic continuum model. In the analysis of integrated structure-foundation-soil system, present study adopts the direct method of SSI and the soil is represented by elastic continuum finite element model. SSI analysis was carried out on multi storey shear wall buildings of height-to-base ratio (aspect ratio) ranging from 1 to 4, considering the site specific ground motion incorporating the local soil effects.

3. Fınıte element formulatıon of perfectly matched layer (PML) in tıme domaın

The direct method of soil-structure interaction is followed in this study. The governing equations of motion for the structure incorporating soil interaction are relatively complex. The dynamic equilibrium equation depicting the motion of structure subjected to a transient external load can be written as

$$[M]\{\ddot{u}\}+[M]\{\dot{u}\}+[M]\{u\}=\{F_{dyn}\}+\{F_{st}\} \tag{1}$$

where,

$[M]$, $[C]$, $[K]$ are characteristic matrices for consistent mass, damping and stiffness of a system.

$\{F_{st}\}$ is the pre-dynamic load vector including self-weight of the structure and $\{F_{dyn}\}$ is the dynamic load vector.

$\{u\}$ is the vector of nodal displacements and a super imposed dot indicates the time derivative.

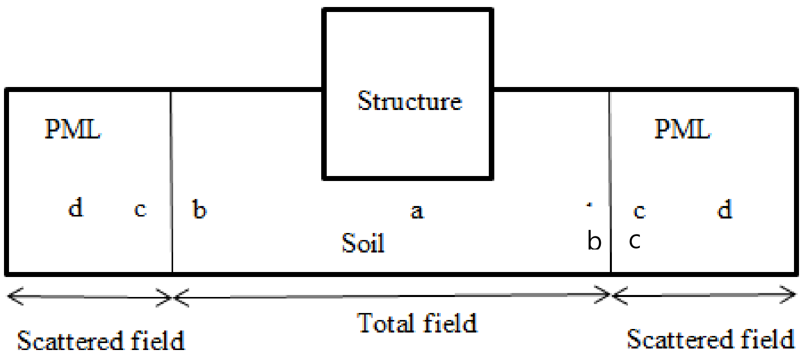


Fig. 1 Partition of DOF's of SSI system (Lee 2006)

To simulate the infinite soil regions in this study, fınıte element based PML formulation following the displacement based approach introduced by Basu and Chopra (2003) was employed. The governing equations for PML domain were found by means of a coordinate transformation involving stretching function determined with complex numbers. Governing equations for finite element formulation of PML in time domain is given by

$$[M_{PML}]\{\ddot{u}\}+[C_{PML}]\{\dot{u}\}+[K_{PML}]\{u\}+\{f_{PML}{}^{\text{int}}\}=\{f_{PML}{}^{ext}\} \tag{2}$$

where,
$[M_{PML}]$, $[C_{PML}]$ and $[K_{PML}]$ represent the mass, damping and stiffness matrices for a PML medium modulated by stretching functions for evanescent waves. $f_{PML}{}^{\text{int}}$ is internal force vector and represents the true external forces to the PML domain.

A computational model of structure-soil system considered is as shown in Fig. 1 (Lee 2006), where the soil region is of infinite magnitude in the horizontal directions and thus PMLs are rendered outside the region of interest. For considering seismic excitation, the domain is divided into two regions, one where the field variables are expressed in total motion and the other in scattered motion. Scattered field motion u^s is defined as the difference between the total motion and free field motion.

$$u^s = u - u^f \tag{3}$$

Where, free-field motion u^f is the motion of the soil deposit (due to an earthquake under consideration) without any structure on it and u is the total motion.

For the region expressed by the total field, the equations of motion are written as (Lee 2006)

$$\begin{bmatrix} M_{aa} & 0 \\ 0 & M_{bb} \end{bmatrix}\begin{Bmatrix} \ddot{u}_a \\ \ddot{u}_b \end{Bmatrix}+\begin{bmatrix} C_{aa} & C_{ab} \\ C_{ba} & C_{bb} \end{bmatrix}\begin{Bmatrix} \dot{u}_a \\ \dot{u}_b \end{Bmatrix}+\begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & K_{bb} \end{bmatrix}\begin{Bmatrix} u_a \\ u_b \end{Bmatrix}=\begin{Bmatrix} 0 \\ F_b \end{Bmatrix} \tag{4}$$

Where, the subscript b refers to the DOFs on the interface within soil between the total and scattered field and subscript 'a' denotes DOFs within total field.
Similarly, the scattered-field region is governed by

$$\begin{bmatrix} M_{cc} & 0 \\ 0 & M_{dd} \end{bmatrix}\begin{Bmatrix} \ddot{u}_c \\ \ddot{u}_d \end{Bmatrix}+\begin{bmatrix} C_{cc} & C_{cd} \\ C_{dc} & C_{dd} \end{bmatrix}\begin{Bmatrix} \dot{u}_c \\ \dot{u}_d \end{Bmatrix}+\begin{bmatrix} K_{cc} & K_{cd} \\ K_{dc} & K_{dd} \end{bmatrix}\begin{Bmatrix} u_c \\ u_d \end{Bmatrix}=\begin{Bmatrix} F_b \\ 0 \end{Bmatrix} \tag{5}$$

Where, the subscript c refers to the DOFs on the interface within PML and subscript '*d*' denotes DOFs within scattered field.

To combine Eqs. (4) and (5) and to invoke the interface relation between the total displacement and scattered field displacement

$$u_b = u_c + u^f \tag{6}$$

and the balance of interaction forces as

$$F_b = -(F_c + F^f) \tag{7}$$

Where, F^f is the equivalent nodal force due to free-field motion.

$$\begin{bmatrix} M_{aa} & 0 & 0 \\ 0 & M_{bb}+M_{cc} & 0 \\ 0 & 0 & M_{dd} \end{bmatrix} \begin{Bmatrix} \ddot{u}_a \\ \ddot{u}_b \\ \ddot{u}_d \end{Bmatrix} + \begin{bmatrix} C_{aa} & C_{ab} & 0 \\ C_{ba} & C_{bb}+C_{cc} & C_{cd} \\ 0 & C_{dc} & C_{dd} \end{bmatrix} \begin{Bmatrix} \dot{u}_a \\ \dot{u}_b \\ \dot{u}_d \end{Bmatrix}$$

$$+ \begin{bmatrix} K_{aa} & K_{ab} & 0 \\ K_{ba} & K_{bb}+K_{cc} & K_{cd} \\ 0 & K_{dc} & K_{dd} \end{bmatrix} \begin{Bmatrix} u_a \\ u_b \\ u_d \end{Bmatrix} = \begin{bmatrix} 0 \\ M_{cc}\ddot{u}^f + C_{cc}\dot{u}^f + K_{cc}u^f \\ C_{dc}\dot{u}^f + K_{dc}u^f \end{bmatrix} \begin{Bmatrix} 0 \\ -F^f \\ 0 \end{Bmatrix} \tag{8}$$

The effect of seismic excitation included as external forces to the discrete structure-soil system is as shown in Eq. (8).

4. Methodology

One dimensional equivalent linear ground response analysis of the considered site was carried out using ProSHAKE software (Roy and Sahu 2012) to determine the amplification of ground motion due to local site conditions. The study zone falls under Zone III as per Indian Seismic Zone classification wherein a ground motion with PGA of 0.1 g can be expected Because of unavailability of recorded solid ground motion information of the study zone, time history of Elcentro earthquake motion was downsized to a maximum ground acceleration of 0.1 g and was utilized as input motion for ground response analysis. Further, the surface ground motion generated using ProSHAKE software showing the highest PGA was used in soil-structure interaction analysis.

In the finite element SSI analysis, three-dimensional finite element model of the whole structure-foundation-soil system was generated using finite element software LS DYNA. The ground motion showing the highest PGA for the specific site was applied in the global *X* direction for the integrated structure-foundation soil model. The damping ratio equivalent to 5% of critical damping was assumed for structures and soil. The lateral loads due to other causes were neglected. Natural frequencies of soil strata at different borehole locations were determined using the method proposed by Zeeveart (1972) taking the total thickness of soil deposit (D) and shear wave velocity (Vs) as the parameters for calculation. Variations in structural responses due to the varying shear wall positions were analysed to identify the suitable position of shear walls. The seismic responses in building founded on flexible base were compared with conventional rigid base to determine the effect of local soil conditions.

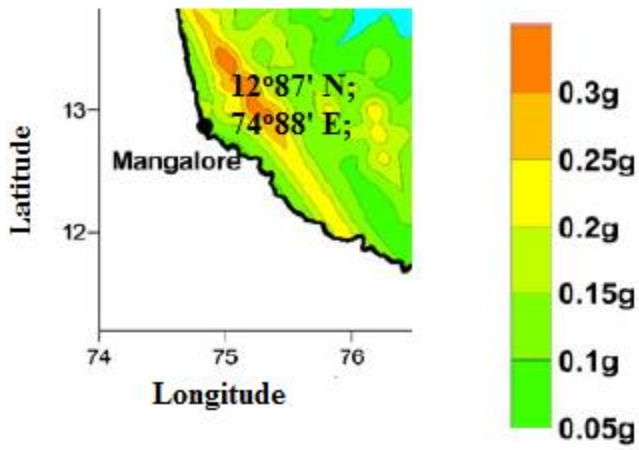


Fig. 2 Spatial variation of mean PHA (g) values at bedrock in study area (Sitharam *et al*. 2011)

4.1 Ground response analysis

Site response studies carried out by Finn (1991), Boore, Joyner *et al*. (1993), Anderson, Lee *et al*. (1996) showed that the earthquake ground motion is based on the geotechnical properties in shallow depths. Borcherdt (1970) reported that the top 30m soil column was responsible for site amplification. The geological study area covering the coastal belt at latitude 12°87'N and longitude 74°88'E, nearly 300 m from the Arabian sea shore was considered for ground response analysis. The effect of subsurface conditions on seismic response of buildings for an educational institution of national importance is the focus of this work. The location of the site and the spatial variation of PHA (g) values at bedrock are as shown in Fig. 2 which falls under Zone III, zonal classification as per Indian seismic code IS1893 (2002).

Lateritic soil is found in abundance in this region. Geotechnical investigation of the site shows that the surface soil generally consists of sandy silt which is locally known as 'Shedi soil'. Borehole data at 20 locations of this study area, located within 300m from Arabian Sea shore, were collected for ground response analysis. There is considerable variation in soil properties within a small area. Upper layers of soil mainly consist of sandy silt for 4-10 m depth and lower layers consist of weathered rock. Here, the depth of bed rock varies from 7-15 m. Typical borehole log of the site drawing comparably higher ground acceleration is shown in Fig. 3.

ProSHAKE program was used for the one dimensional equivalent linear ground response analysis. The necessary inputs for the analysis were shear wave velocity, bedrock acceleration time history, damping curves and shear modulus reduction curve of soil layers at the site.

Based on the Seed and Idriss (1981) empirical relation as shown in Eq. (1), shear wave velocity (Vs) of each soil layer of the site was estimated using SPT (N) values.

$$V_s = 61N^{0.5} \tag{9}$$

Damping curve and shear modulus reduction curve of the soil layer were selected from the upper bound curves as in Seed, Wong *et al*. (1986). Elcentro earthquake motion was modified and chosen as an artificial time history for the input bed rock motion due to unavailability of strong motion data in the study area and since this ground motion contains strong frequency contents

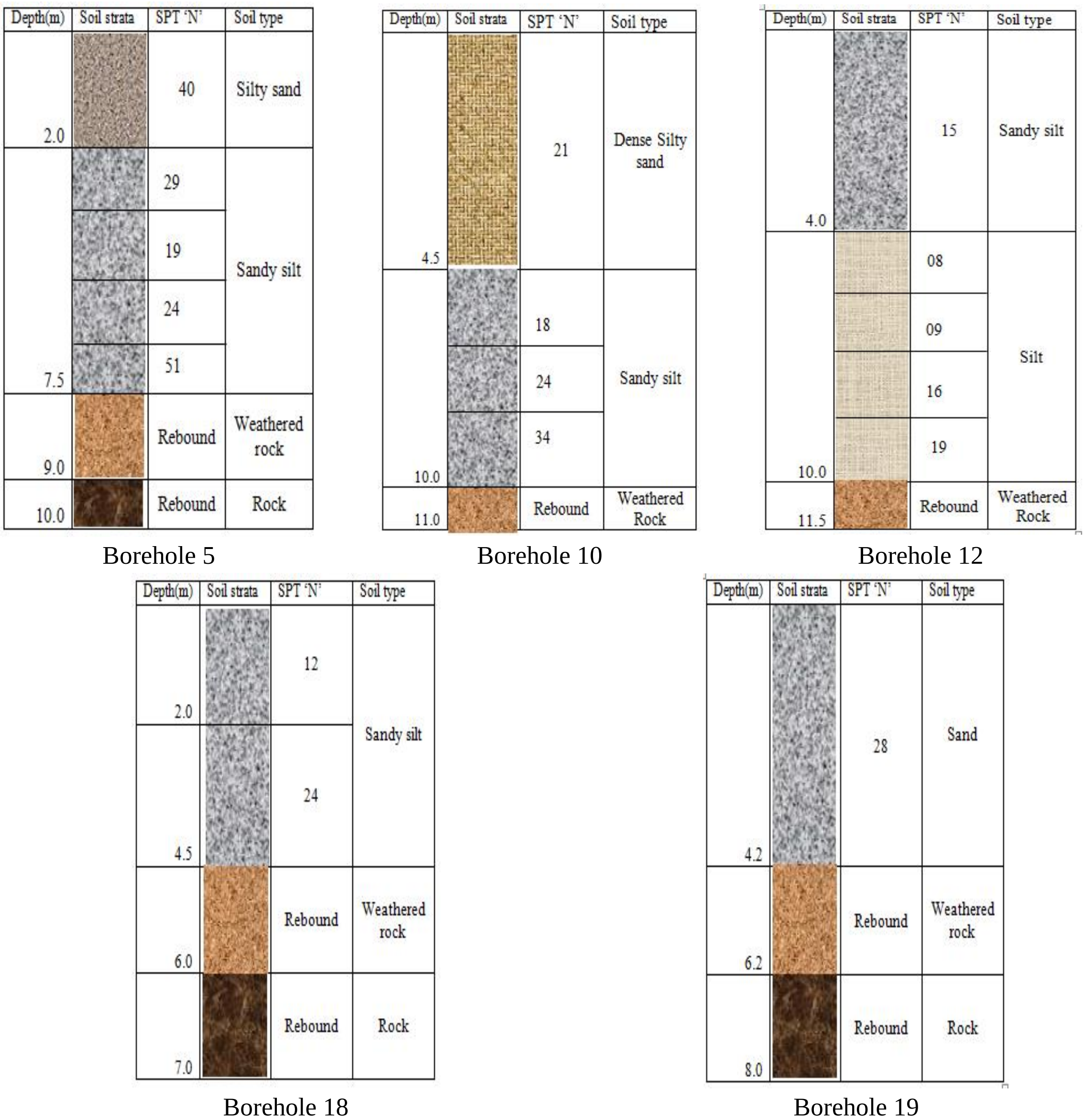
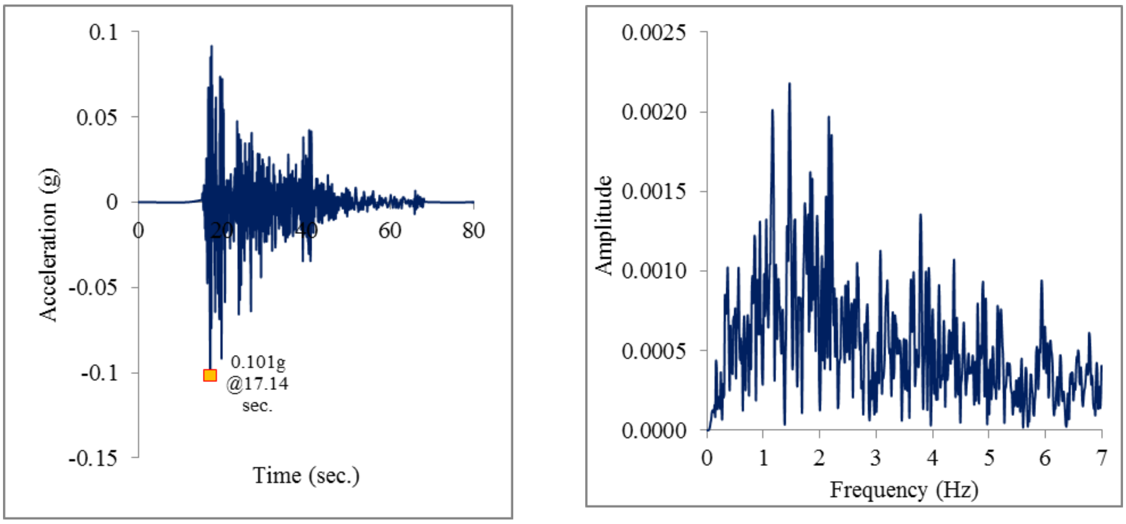


Fig. 3 Typical soil profile at the site

corresponding to the frequency range of midrise buildings considered. Based on seismicity and seismic hazard study by Sitharam, James *et al*. (2011), input ground motion for the study area was generated by scaling down the Elcentro ground motion to represent a maximum ground acceleration of 0.1 g. The acceleration time history of this input motion is shown in Fig. 4(a) and the corresponding Fourier spectrum is as shown in Fig. 4(b). Higher amplitude frequency contents exist above 0.36 Hz for this input ground motion.

Ground response analysis of the site considering the geotechnical data from all 20 boreholes was carried out. The peak acceleration time history of free field motion obtained from this analysis



(a) Time history (b) FFT

Fig. 4 Input motion

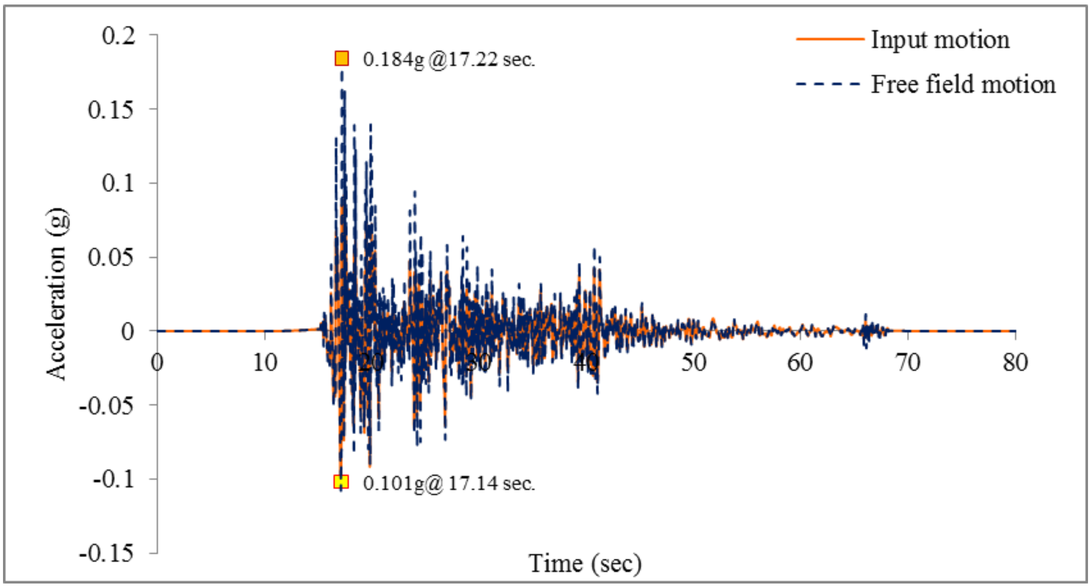


Fig. 5 Acceleration time history of free field motion at borehole 18

is as shown in Fig. 5. This corresponds to borehole location 18 which contains the soil strata drawing the highest acceleration of all the locations considered. From Fig. 5 it is observed that the maximum acceleration of the surface ground motion is greater than the maximum input acceleration demonstrating the amplification of motion due to local site characteristics. The maximum acceleration of 0.184 g is observed at borehole location 18. Among all boreholes, the data at boreholes 5, 10, 12, 18 and 19 were selected for the site response analysis as the free field ground motions at these locations show higher amplifications of bed rock motion. These bore hole

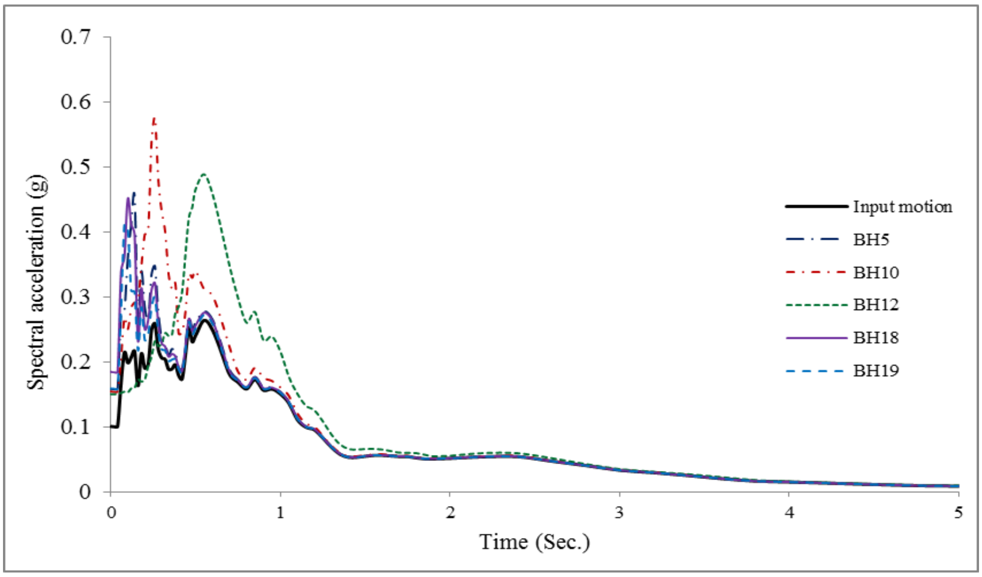


Fig. 6 Response spectra of the surface motion at various bore hole locations

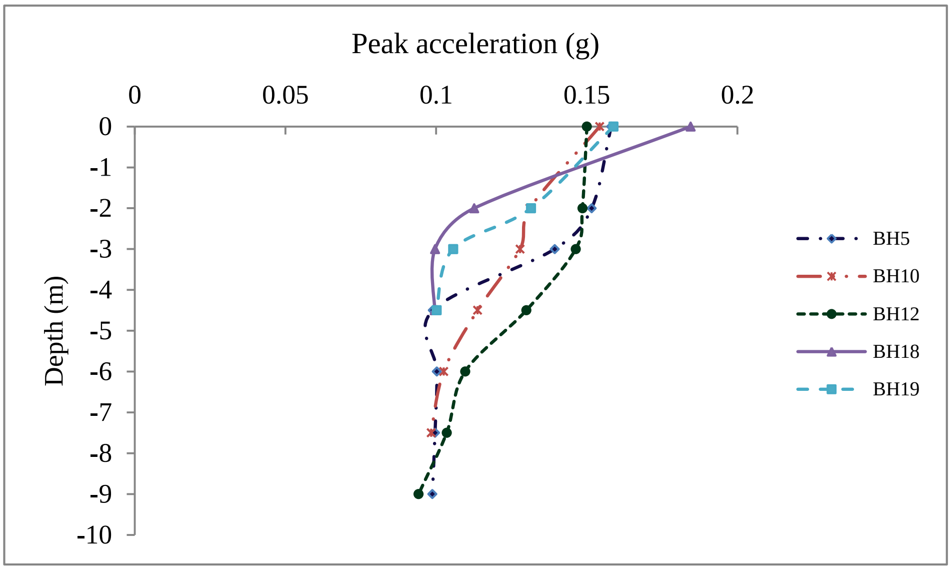


Fig. 7 PGA variation with depth

locations are designated as BH5, BH10, BH12, BH18 and BH19 respectively.

Response spectra of the input motion at bed rock level and output motion at the surface of various borehole locations, obtained for 5% damping are as shown in Fig. 6. It is noted that the maximum spectral acceleration corresponds to 0.55 sec. for input motion. The maximum spectral acceleration for the free field motion at BH5, BH10, BH12, BH18 and BH19 corresponds to 0.14sec, 0.26sec, 0.55sec, 0.10sec and 0.08sec respectively due to the influence of local site conditions.

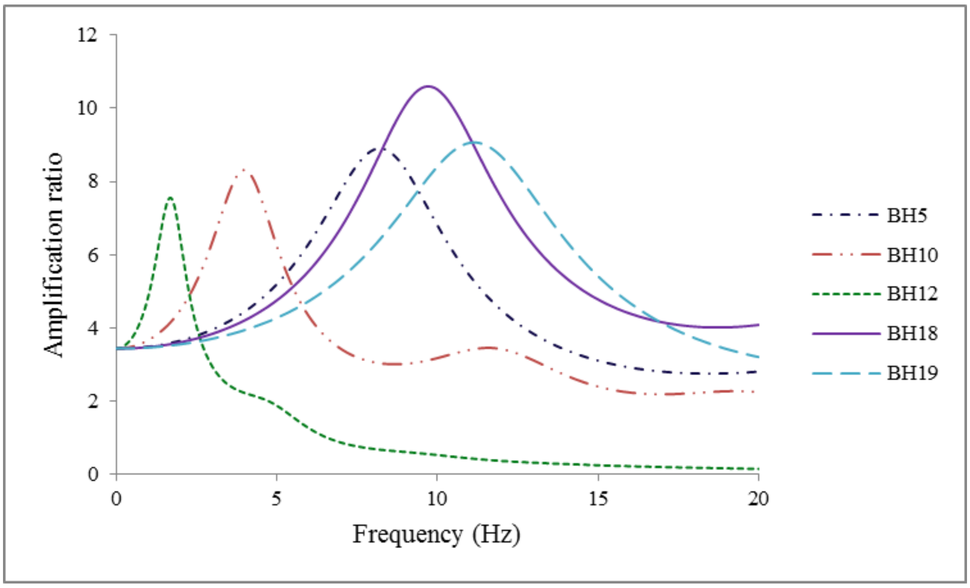


Fig. 8 Amplification ratio between surface and base motion

Table 1 Geometric properties of building components

Aspect Ratio	Columns (m)		Shear wall thickness (m)
	Up to 3 story	Above 3 story	
1	0.32×0.32	0.32×0.32	0.15
2	0.40×0.40	0.35×0.35	0.20
3	0.50×0.50	0.40×0.40	0.20
4	0.60×0.60	0.50×0.50	0.25
Raft foundation slab:	0.3 m		
Roof and floor slab:	0.15 m		
Beams:	0.23×0.23 m		

In general, the ground motions were amplified as the waves travel through the soil strata depending on the soil type, layer thickness and soil stiffness. The maximum variation in accelerations with depth is as shown in Fig. 7. Significant amplification in acceleration values from 0.101 g to 0.158 g, 0.154 g, 0.150 g, 0.184 g and 0.159 g were observed at BH5, BH10, BH12, BH18 and BH19 respectively. It is seen that the top layer soil causes the maximum variation in acceleration, especially at BH18.

The amplification among the surface motion and base motion for varying frequencies are shown in Fig. 8. Amplification ratios of 9.0, 8.3, 7.6, 10.6 and 9.1 were seen at frequencies 8.2 Hz, 3.8 Hz, 1.6 Hz, 9.4 Hz and 10.9Hz respectively at BH5, BH10, BH12, BH18 and BH19 bearing the fundamental natural frequency of soil deposit as 10.04 Hz, 5.10 Hz, 3.89 Hz, 10.69 Hz and 13.01 Hz respectively. As the average shear wave velocity of top layer of soil at BH18 is less compared to other borehole locations, maximum amplification is observed at BH18. It is observed that soft soil amplifies the low frequency content of ground motion.

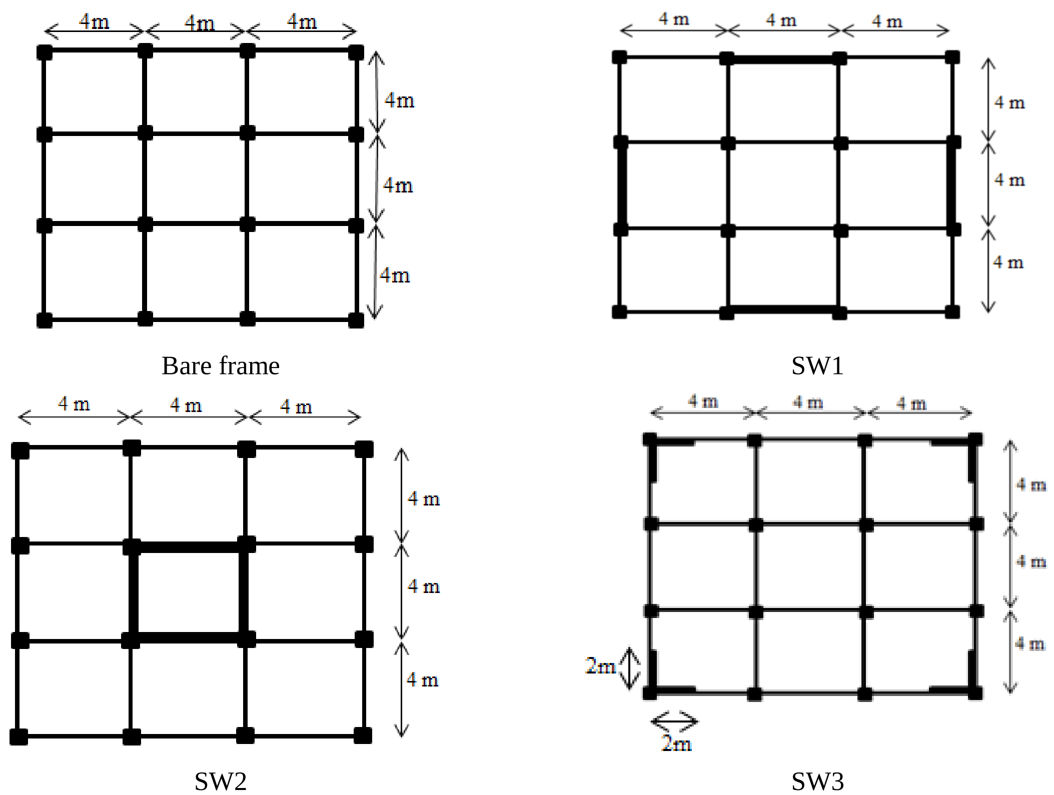


Fig. 9 Plan of bare frame and frames with various locations of shear wall

4.2 Seismic SSI analysis

Idealization of structure

Multi-storey reinforced concrete framed buildings with aspect ratio (AR) of 1, 2, 3 and 4 with and without shear walls resting on raft foundation were considered for the transient analysis. Symmetric plan buildings constituting ordinary moment resisting frames with 3 bays equal in length along each direction were considered neglecting the effects of infill. To study the effect of position of shear wall, shear walls of same size and mass were placed symmetrically on both directions of building at the middle bay of exterior frames, at the core and at all four corners of the exterior frames. Considering the buildings to be for domestic or small office building use, the storey height was chosen as 3 m and length of each bay of frames as 4m. Thickness of shear walls was varied from 150-250 mm depending on the building height. Openings in shear walls were not considered presuming that extra strengthening and stiffening were provided around the openings. Dimensions of all building elements were as per the design standards of IS 456 (2000) and IS13920 (1993). Details of geometric properties of building components are as given in Table 1.

Idealized 3 bay×3 bay frame having plan dimensions of 12 m×12 m with shear walls at various positions are presented schematically in Fig. 9. Moment resisting frames without shear walls are denoted as ‘bare frame’ (BF) and frames with shear walls placed at middle bay of exterior frames are denoted by SW1, at core by SW2 and at corners by SW3.

Table 2 Details of soil parameters

Soil description	Shear wave velocity (V_s) (m/sec)	Poission's ratio μ	Unit weight (ρ) (kN/m^3)	Young's modulus (E_s) (kN/m^2)
Sandy silt(Layer1)	212	0.35	18	2.23E+05
Sandy silt(Layer2)	299	0.35	18	4.43E+05

Idealization of infinite soil

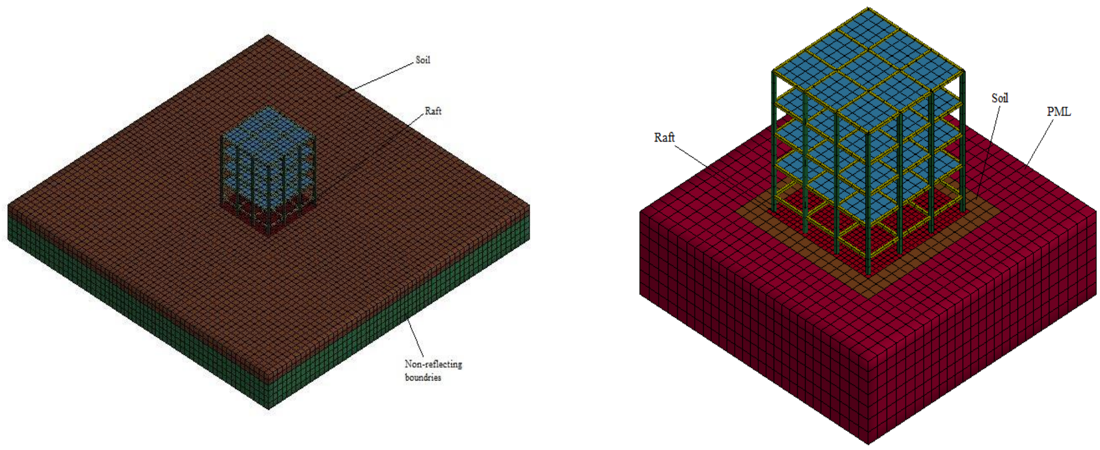
From the site specific ground response analysis carried out, the results show that the free field motion at BH18 has the maximum ground acceleration. Hence the geotechnical data of this location was used for the investigation to determine the effect of local soil in varying the seismic response of structures in this study area. Elastic properties of soil classified based on shear wave velocity were evaluated and are as given in Table 2. To represent the soil, elastic continuum finite element model was adopted.

In numerical treatment of soil-structure interaction problem, both the structure and soil need to be introduced into the computational domain and then discretised. Size of soil domain is practically of infinite extent as compared to structure. To represent the seismic response of unbounded soil domain in an efficient and accurate manner, comparison between the two absorbing boundary conditions (i) non-reflecting boundaries and (ii) Perfectly Matched Layer (PML) concept was made for computational costs as a part of this study.

In the soil model with non-reflecting boundaries, the boundaries of soil are placed at a sufficient distance away from the structure such that the static response dies out Wolf (1985). In this study, the lateral boundary of soil was placed at a distance of 1.5 times the least width of the raft foundation (Maharaj, Amruthavalli *et al*. 2004) and the bottom boundary was assumed at a depth at the bedrock level as per the local soil profile. The bottom boundaries of soil were restricted from all translations while the lateral vertical soil boundaries were modelled with non-reflecting boundaries. However, in the soil model with PML, a layer of PML material was placed at the boundary of a bounded domain of soil close to the structure to simulate unboundedness of the domain at the boundary. PML layer form a cuboid box around the bounded soil domain, with the axes of box aligned with coordinate axes. PML layer was made of 5 elements through its depth as the excitation source was assumed to be at reasonable distance from the layer. The size of PML element was kept similar to that of elements in the bounded domain near the layer and the nodes on outer boundary of layer were fully constrained.

Finite element modeling

Finite element modeling and analysis were carried out using finite element software LS DYNA. In modelling of 3D space frames, Belytschko-Schwer resultant beam element having three translational and three rotational degrees of freedom at each node was considered. Belytschko-Tsay shell elements with both bending and membrane capabilities were used in modelling of slab at various storey levels, shear wall and raft foundation slab. Belytschko-Tsay shell elements possess all six degrees of freedom at each node. In modelling the three dimensional soil stratum, fully integrated selectively reduced (S/R) solid having three translational degrees of freedom at each node was considered. Node compatibility problem occurs at interface of structure and soil due to different number of degrees of freedom at these nodal points. This node incompatibility is overcome by INTERFACE_SSI card which describes the soil-structure interface in LS DYNA. INTERFACE_SSI card creates a tied-contact interface amid two defined segment sets, the master



Elastic continuum with non-reflecting Boundaries   Elastic continuum with PML

Fig. 10 Finite element model of idealized soil-structure system of typical 4 storey building

Table 3 Comparison of computational time of the soil models

Model	Elements		Computation time
Elastic continuum with PML	Beams	704	3 hrs, 36 mins, 18 sec.
	Shells	722	
	Solid	5,103	
	Total	6,561	
Elastic continuum with non-reflecting boundaries	Beams	704	6 hrs, 1 min, 14 sec.
	Shells	722	
	Solid	20,886	
	Total	22,367	

surface forming on the soil side and the slave on the structure side. A tied surface to surface contact between the soil surface and base of the structure is employed such that the translational motion of soil due to bending of raft is imposed and the raft and soil are coupled effectively for the analysis of the entire soil-structure system. A three dimensional finite element model of idealized soil-foundation-structure system with non-reflecting boundaries and PML are as shown in Fig. 10.

Investigation on FE model of 4 storey bare frame building with supporting unbounded soil medium represented in the form of elastic continuum with (i) non-reflecting boundaries and (ii) PML were carried out to determine the computational costs of these two soil models and to select the efficient approach for further study. Soil characteristics at BH18 and the corresponding free field motion were considered in the seismic analysis. Comparison of computational time to determine the efficiency of these two models to represent infinite soil in seismic analysis is represented in Table 3. It is noticed that the number of elements and computational time required for SSI analysis with PML soil model are significantly lesser than the elastic continuum model with non-reflecting boundaries. The total number of elements employed in analysis using elastic continuum model with non-reflecting boundaries was 22,367 against 6,561 in PML model and the

Table 4 Comparison of structural seismic responses

Seismic response	Elastic continuum soil model	
	Non-reflecting boundaries	PML
Natural period (sec)	0.9919	0.9935
Base shear (kN)	198.73	195.78
Roof deflection (m)	0.0487	0.0487
Axial force of ground floor corner column (kN)	174.39	174.6
Shear force of ground floor corner column (kN)	16.04	16.09
Bending moment of ground floor corner column (kN-m)	4.299	4.359

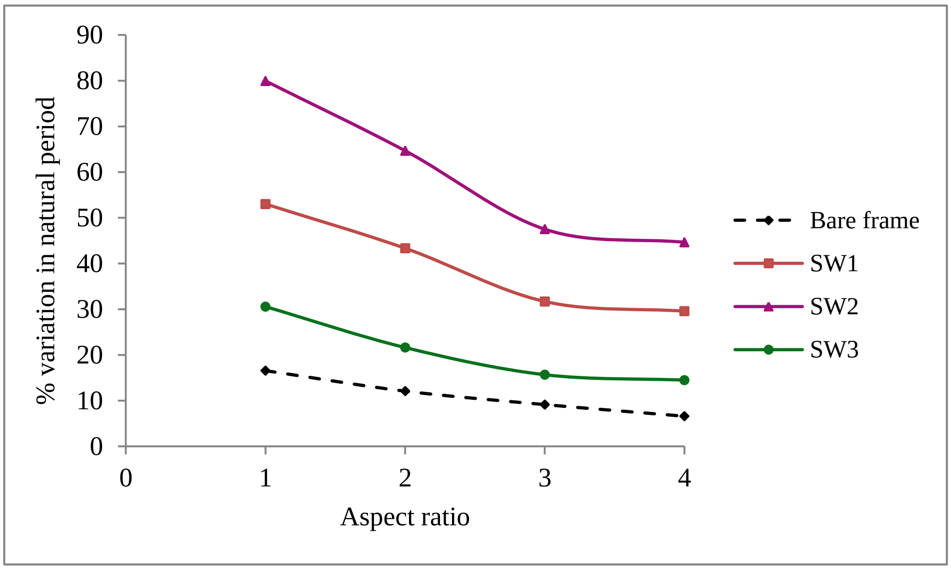


Fig. 11 Variation of natural period of buildings due to local site effects

computation time taken by these models were 360 minutes and 216 minutes respectively. The values of seismic response obtained from the two soil models are tabulated in Table 4. From Table 4 it is evident that seismic response values obtained from both the models are identical except for base shear where PML soil model gives slightly less value (<2%) than elastic continuum with non-reflecting boundaries. By comparing the computational costs of the two models PML model was found to be more efficient, hence it was adopted for the site specific SSI analysis

5. Results

Site specific SSI analyses were carried out on three-dimensional finite element integrated soil-foundation-structure models to determine the response of the system during ground motion. Multistorey buildings of different aspect ratios with three different shear wall positions were considered. Effects of SSI were evaluated by considering the soil profile at the study area. Responses due to the effect of soil flexibility and varying positions of shear wall were evaluated in terms of variation in natural period, base shear, roof deflection and stress resultants in exterior column of ground floor.

Lateral natural period

A significant role is played by fundamental natural period in determining the seismic response of structures. Percentage variation in natural period obtained from free vibration analysis of buildings with various aspect ratios due to the effect of supporting soil and position of shear walls are shown in Fig. 11.

The natural period increases with increase in aspect ratio due to the increase in flexibility of structure. The natural period obtained by consideration of soil flexibility is higher than conventional fixed base condition on account of reduced stiffness by the inclusion of soil in the dynamic system. Further, with the introduction of shear wall in framed buildings, natural period is reduced due to the increased stiffness of structure. By varying the location of shear walls, the values of natural period vary despite of total mass of structure being unchanged. The fundamental natural period of fixed base structures considered are in the range of 0.85 to 3.00sec for bare frame buildings and 0.18 to 1.98sec for shear wall buildings. Among the various locations of shear walls considered, the fundamental natural periods are highest in shear wall buildings with SW3 configuration i.e., shear wall placed at corners of exterior frame and lowest for buildings with SW2 configuration i.e., shear wall placed at core.

Differences in the values of natural period obtained between the conventional fixed base condition and SSI, decreased with increase in aspect ratio showing that the flexibility of soil has more impact on lower aspect ratio buildings as compared to higher aspect ratio buildings. The variation in natural periods between fixed base and SSI is lower (6.6% in *AR*=4) in case of bare frame building and higher (80% in *AR*=1) in case of shear wall buildings with shear wall placed at core. The effect of soil flexibility caused 80% increase in natural period of low rise building with shear wall placed at core.

Seismic base shear

One of the primary inputs in seismic design of structures is seismic base shear (VB). Seismic base shear reflects the seismic vulnerability of structures. It is the maximum expected lateral force that is probable to occur at the base of a structure due to seismic ground motion. The seismic base shear ($\tilde{V}$B) of frame buildings with shear walls at various positions over raft foundation expressed in terms of the total seismic weight (W), of the building are shown in Fig. 12.

From Fig. 12 it is observed that base shear in bare frame buildings are lesser than shear wall buildings due to lower seismic weight of building which forms the crucial parameter in base shear calculation. It is also evident that the base shear obtained by conventional fixed base condition is higher than the SSI analysis making conventional analysis results more conservative. Base shear values increased with increase in aspect ratio. The values of base shear vary in the range of 0.01W to 0.04W for shear wall buildings resting on soil. However, for the conventional fixed base condition it varies in the range of 0.05W to 0.21W, where W is the seismic weight of the structure. As per the conventional fixed base condition, the base shear is highest for shear wall buildings with shear wall at core (SW2) and least for shear wall buildings with shear wall placed at corners of exterior frame (SW3). However, considering the local site conditions, the highest and lowest values of base shear are observed in SW3 and SW2 configuration respectively. This could be due to the effect of high amplitude frequency content of the ground motion corresponding to the natural frequency of SW3 building configuration. The fundamental natural frequency for SW3

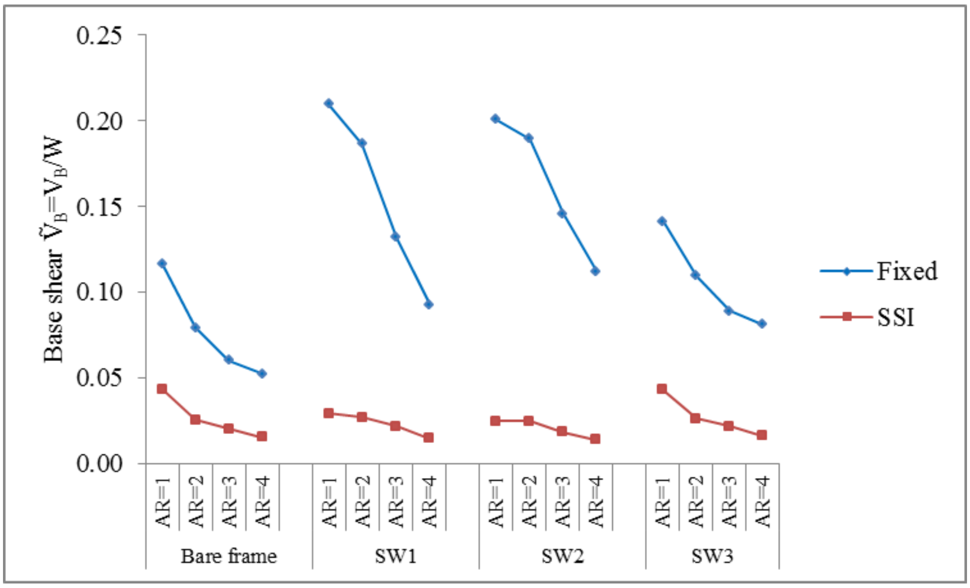


Fig. 12 Seismic base shear of buildings

configuration with SSI ranges from 0.44 Hz to 2.17 Hz. It is seen that some higher amplitude peaks lie at 1.15 Hz to 2.16 Hz range in the surface ground motion generated from ground response analysis. Hence SW3 shear wall configuration shows higher base shear unlike in fixed base case and as compared to SW2.

As evident from these results, considering the structure to be fixed at base is highly conservative in the analysis and design of structures for seismic load. The percentage reduction in seismic base shear due to the inclusion of soil flexibility are lowest in bare frame buildings and highest in shear wall buildings with shear wall at core (SW2). High rise buildings with shear wall placed at core experience the highest percentage reduction of 88% in seismic base shear. Considering only the conventional fixed base analysis, the probable selection of location of shear wall would be according to SW3 configuration resulting only slight increase in base shear as compared to bare frame. But the actual base shear in buildings is less than 5% of seismic weight (considering site conditions) and the least in SW2 configuration, hence represents the ideal location of shear walls in buildings at this site.

Roof deflection

Roof deflection of bare frames and buildings with shear walls at varying positions with varying aspect ratio are as shown in Fig. 13.

Roof deflection of buildings observed for the conventional fixed base condition is lower than the values obtained from SSI analysis. This is due to the inclusion of more flexibility in the SSI system. From Fig. 13, it is observed that roof deflection values are considerably reduced by the addition of shear wall in building. The amount of roof deflection varies by varying the location of shear wall. It is least in shear wall buildings with shear wall at core (SW2) and highest in shear wall buildings with shear wall at corners of exterior frame (SW3). Variations in the value of roof deflection obtained from conventional analysis and SSI analysis are more prominent in bare frame

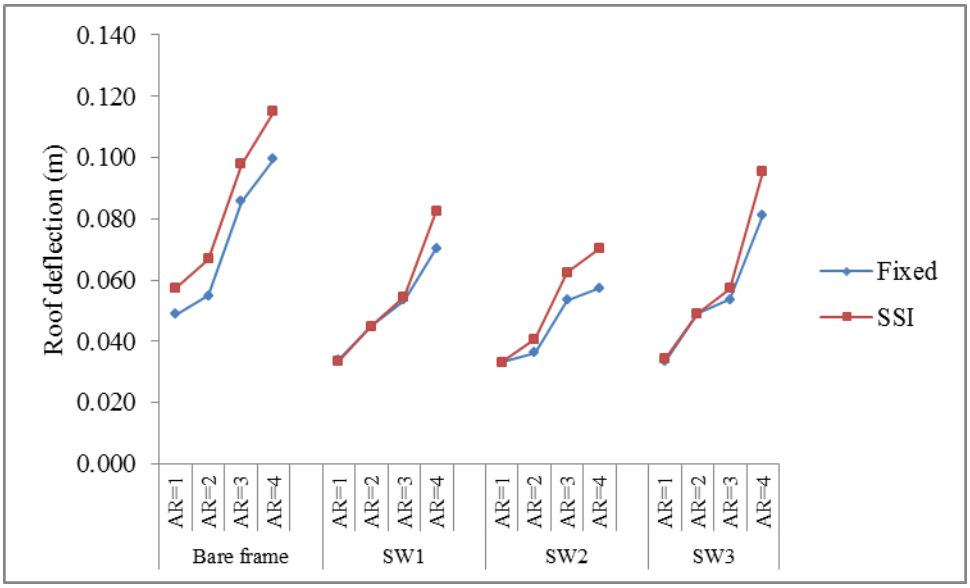


Fig. 13 Roof deflection of buildings

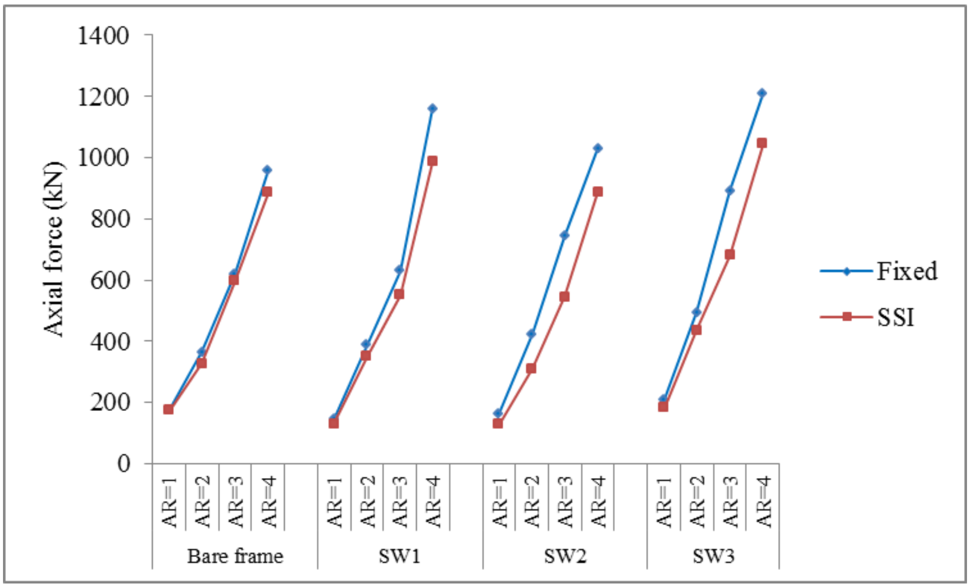


Fig. 14 Axial force in exterior column of ground floor

buildings. In shear wall buildings, the variation in roof deflection due to SSI is very less for buildings with low aspect ratio. It increased with increase in aspect ratio. The highest percentage increase of 23% is observed in roof deflection for shear wall building with shear wall placed at core for an aspect ratio of 4 due to the effect of soil flexibility, although this configuration of shear wall causes the least roof deflection for the site considered.

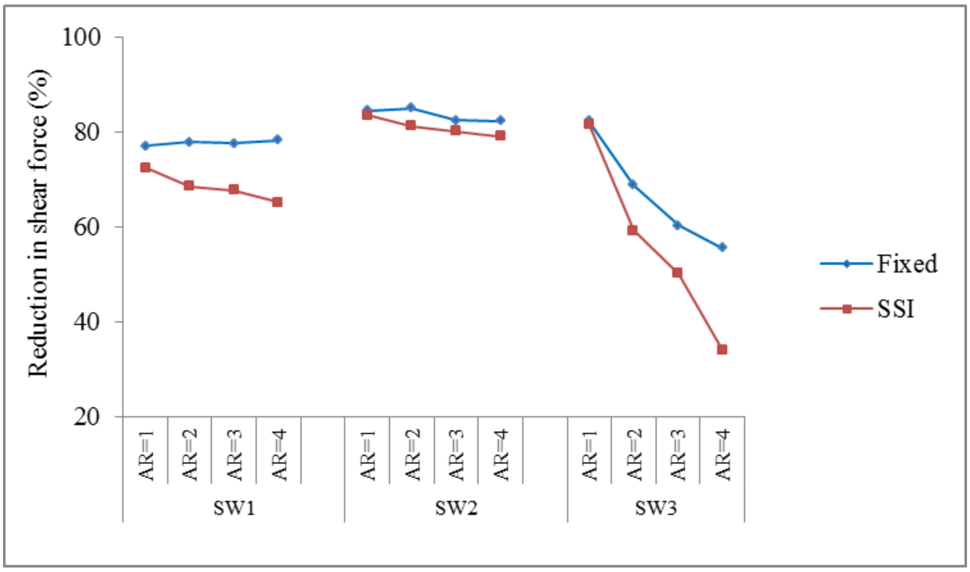


Fig. 15 Variation in shear force in exterior column of ground floor

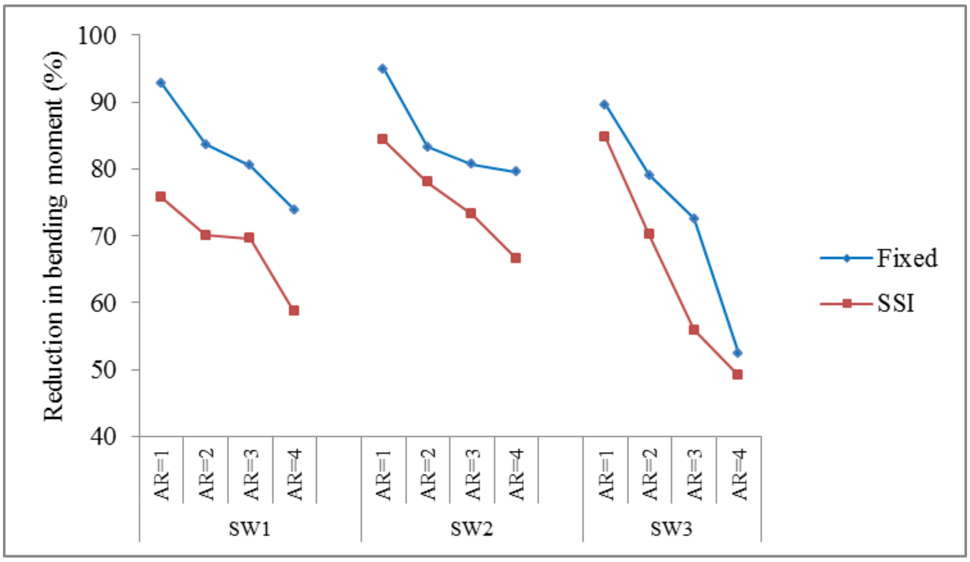


Fig. 16 Variation in bending moment in exterior column of ground floor

Axial force in corner column

Axial force variations in ground floor columns due to earthquake may cause the global failure of structure. Axial forces in exterior corner column of ground floor of buildings with shear walls at various positions are shown in Fig. 14. It is observed that axial force of exterior ground floor column is higher in fixed base condition than with consideration of SSI. Only negligible variation is seen in axial force obtained from SSI analysis and conventional fixed base analysis. With the effect of soil flexibility, highest reduction of 29.4% is observed in SW3 configuration with aspect ratio 3. Among the shear wall configurations considered, SW2 shows the least and SW3 shows the highest axial force for buildings of all aspect ratios.

Shear force and bending moment in corner column

Variation in shear force and bending moment values of ground floor corner column due to the effect of local soil conditions and inclusion of shear wall at various locations with respect to bare frame building are as shown in the Figs. 15 and 16. From Figs. 15 and 16 it is observed that shear force and bending moment values are greatly reduced due to the inclusion of shear wall in the framed buildings. The percentage reduction in shear force and bending moment is the highest in conventional fixed base condition and it decreased with inclusion of soil flexibility. Highest and lowest percentage reduction in value of shear force and bending moment are observed in building configurations SW2 and SW3 respectively. With SSI, the highest and the lowest percentage reduction of 83.5% and 34.1% in shear force and 84.3% and 49.2% in bending moment are observed in shear wall buildings SW2 and SW3 respectively. From this site specific SSI analysis it is concluded that the provision of shear walls at core (SW2) is advantageous.

6. Conclusions

Ground response analysis to determine the effect of local soil conditions was implemented followed by SSI analysis of multi-storey reinforced concrete shear wall buildings with shear walls placed at various locations. Geometric properties of super structure were altered and material properties of soil at site were incorporated to realize the implication of SSI. Variation in natural period, seismic responses of building such as base shear, roof deflection, axial force, shear force and bending moment in ground floor column were considered to evaluate the effect of local soil conditions.

Following conclusions are drawn from present study,

• Soil layers significantly amplify the input motion. Maximum amplification is observed at locations where the top layers of soil have low shear wave velocity. Soft soil amplifies the low-frequency motion

• Shear wall buildings with shear wall placed at corners of exterior frame (SW3) have the highest natural period and buildings with shear wall placed at core (SW2) have the least natural period.

• Base shear obtained by conventional fixed base condition is higher than SSI values and hence conservative. Incorporating the site effects, the least value of base shear is seen in shear wall building with shear wall placed at core.

• Roof deflection of building increases with inclusion of soil flexibility in system. Roof deflections are observed to be the least in shear wall buildings with shear wall at core and the highest in shear wall buildings with shear wall at corners of exterior frame.

• Axial forces in columns are reduced by incorporating SSI effect. The percentage reduction in shear force and bending moment of shear wall buildings with respect to bare frame buildings are observed to be the highest in shear wall buildings with shear wall at the core and the least in shear wall buildings with shear wall at the corner of exterior frames.

From this three dimensional seismic SSI analysis of buildings it is concluded that providing shear walls at the core is advantageous for the site under consideration and incorporating SSI effect is economical.

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