Earthquake Response of 3D Asymmetric Building with Infill wall under Soil-Structure Interaction

Abhijit Chakraborty[1[0000-0003-1493-6566]], Kamal Bhattacharya[2] and V. A. Sawant[3[0000-0002-6730-4311]]

[1]Research Scholar, Civil Engineering Department, IIT, Roorkee, India
[2]Professor, Civil Engineering Department, NIT Durgapur, India
[3]Associate Professor, Civil Engineering Department, IIT, Roorkee, India
abhijitchakraborty114@gmail.com

Abstract. Due to encroachment of Soil-Structure interaction effect, behaviour of structural system response is greatly influenced. The variation in structural response with the variation of soil stiffness during earthquake is the subject of present study. An attempt of studying seismic response behaviour with the inclusion of SSI effect, of an asymmetric 3D multi-storey building frame is done in finite element software LUSAS in frequency domain as well as time domain. Moreover, infill wall as diagonal strut is considered to study the comparison of base shear, torsional base shear of in-filled frame with bare frame. Site specific input earthquake motion is obtained through de-convolution using DEEPSOIL, by 1D wave propagation. Natural frequency of both the models (bare frame & in-filled frame) with different storey height due to SSI effect is obtained and found that irrespective of storey height in-filled frames are stiffer than the other as usual because of which for low rise building resonance occurs in stiff soil with maximum base shear whereas, for high rise building peak response occurs in flexible soil. It is observed that due to infill wall though 'base shear' was increased but 'torsional shear' a common phenomenon of asymmetric building, was reduced. When the soil is soft 'torsional shear' appears lesser than fixed base structure. Therefore during a site selection, for high rise building soft soil and for low rise building stiff soil should be avoided as much as possible.

Keywords: SSI, Asymmetric building, infill wall, de-convolution, Torsional Shear.

1. Introduction:

Soil-Structure Interaction (SSI) analysis evaluates the collective response of Structure, Foundation and Soil to a specified motion. SSI was traditionally and conveniently been neglected with the assumption that such interaction makes a system lesser stiff with increase in damping. This conservative simplification is valid for certain class of structures and soil conditions, such as light structures in relatively stiff soil. But in reality SSI can have a detrimental effect on the structural response [1], and neglecting SSI in the analysis may lead to unsafe design for both the superstructure and the foundation. When a structure is subjected to an earthquake excitation, it interacts with the foundation and the soil, and thus changes the motion of the ground. Earthquake ground motion causes soil displacement known as free-field motion. However, the foundation embedded into the soil will not follow the free field motion. This inability of the foundation (due to its stiffness) in matching the free field motion causes 'Kinematic interaction'. On the other hand, the mass of the superstructure transmits the inertial force to the soil, causing further deformation in the soil, which is termed as ‘Inertial interaction’. Using rigorous numerical analyses, Mylonakis and Gazetas (2000) [1] have shown that soft soil sediments can significantly elongate the period of seismic waves and the increase in natural period of structure may lead to the resonance with the long period ground vibration. Due to the eccentricity between Centre of mass (CM) and Centre of rigidity (CR) in asymmetric building, torsional moment induces in the structure which is the product of lateral force with the eccentricity. The torsional effect is measured by estimating mass participation factor in torsional modes of any building in all three directions. There are reports of extensive damages to buildings that are attributed to excessive torsion responses caused by asymmetry in earthquakes such as the 1972 Managua earthquake (Pomares Calero 1995), the 1985 Michanocan earthquake (Esteva 1987) and the 1989 Loma Prieta earthquake (Mitchell et. al. 1990). Moreover, soil flexibility changes the torsional response. However, the destruction of numerous asymmetric buildings in 1985 Mexico earthquake made researchers focus on SSI effects and on the response behaviour of such systems. So far, several researchers have attempted to incorporate the flexibility of foundation in asymmetric system models. Among them, Balendra et al. [2] used simple springs to represent frequency-independent values and to approximate the frequency-dependent foundation impedance functions in an asymmetric multi-storey building. In most of the cases in-fill wall is not considered in the analysis of a multi-storey building. Murty and Jain (2000) [10] deals with the study of effect of infill masonry wall in building. Their study comprises with some experimental results on cyclic tests of RC frames with masonry in-fills. It is seen that the masonry in-fills contribute significant lateral stiffness and strength. Llera and Chopra (1996) [5] studied the inelastic seismic behaviour of asymmetric building and different structural characteristics are considered in order to study the torsional response of building. Authors have suggested that, two important guidelines like, increasing the torsional capacity by providing resisting planes in orthogonal directions and modifying the distribution of stiffness and strength to localize yielding in selected resisting planes can be very much effective design solution for asymmetric buildings. Stefano and Pintucchi (2006) [6] presented an overview of progress of research on irregular buildings and three main aspects are mainly focused in this study and these are plan irregularity, mitigation of torsional effect of buildings and lastly the vertical irregularity and it has been observed from this state of art that vertical irregularity is still a where the less number of research is devoted. Ferhi and Truman (1996) [7] carried out study on effect of stiffness and strength eccentricities on the inelastic behaviour of asymmetric building. It has been observed that the elastic deformations are mainly dependent on stiffness eccentricity but the inelastic deformation was strongly influenced by both stiffness and strength eccentricities. Abdelkareem et al. (2013) [3] carried out the study on equivalent strut width for modelling R.C. in-filled frames. The basic parameter of equivalent struts is their equivalent width, which affects the stiffness and strength. Their study presents a general review of several expressions proposed by researchers to calculate this equivalent width. The comparative study of different expressions shows that the Paulay and Priestley (1992) equation is the most suitable choice for calculating the diagonal equivalent strut width, due to its simplicity and because it gives an approximate average value among those studied in this work. It shows that the ratio of the estimated equivalent strut width to the diagonal length of infill (w/d_{inf}) are ranging between about 0.1 to 0.33 except the result calculated by using Stafford Smith and Carter (1969) method equation which generate large value for the equivalent strut width.

In this paper a comparative study on multi-storey building with plan irregularity with and without in-fill is studied. Infill wall is been modelled as equivalent diagonal strut. Moreover different parameters such as fundamental frequency variation with different soil stiffness, base shear variation with soil flexibility and torsional base shear are also studied. For earthquake analysis de-convoluted earthquake data is used as within motion. De-convolution procedure is elaborately discussed later on.

2. Theory and analytical steps:

As per IS 1893:2002 [12], total design lateral force or design seismic base shear (V_b) along any principal direction shall be determined by the following expression:

$$V_b = A_h \cdot W \tag{1}$$

Where, A_h = design horizontal seismic coefficient, expression for A_h is given by,

$$A_h = \frac{Z.I.S_a}{2R.g} \tag{2}$$

Where,
Z = Zone factor for the maximum considerable earthquake.
I = Importance factor.
R = Response Reduction factor, S_a/g = Average response acceleration coefficient.
W = Seismic weight of building.

The design base shear, V_b computed above shall be distributed along the height of the building as per the following expression,

$$Q_i = \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} V_b \tag{3}$$

Where,
Q_i = design lateral force ati^{th} floor.
W_i= seismic weight of i^{th} floor.
h_i= height of i^{th} floor measured from the base.
n = numbers of the storey in the building is the number of the levels at which masses are located.

Torsional Shear force in X & Y direction along any column line V_x and V_y are;

$$V_x = \frac{T}{I_{xy}} . \bar{y} . K_{xx} \tag{4-a}$$

$$V_y = \frac{T}{I_{xy}} . \bar{x} . K_{yy} \tag{4-b}$$

Where, K_{xx} and K_{yy} are the total stiffness of the columns under consideration in the X and Y directions, $\bar{x}$ and $\bar{y}$ are the distances from the column line in X and Y directions respectively. I_{xy} is the rotational stiffness.

The equations of motion for MDOF system is given by:

$$[m]\{\ddot{x}(t)\} + [c]\{\dot{x}(t)\} + [k]\{x(t)\} = -[m]\{r\}\ddot{x}_g(t) \tag{5}$$

Where,
$[m]$ = Mass matrix ($n \times n$), $[k]$ = Stiffness matrix ($n \times n$)
$[c]$ = Damping matrix ($n \times n$), $\{r\}$ = Identity matrix ($n \times n$)
$x(t)$ = relative displacement vector, $\dot{x}(t)$ = relative velocity vector
$\ddot{x}(t)$ = relative acceleration vector, $\ddot{x}_g$ = earthquake ground acceleration.

3. Characterization,simulation of earthquake data

Frequency domain (FD) equivalent linear (EQL) and time domain (TD) nonlinear (NL) analyses are the most common approaches used for performing 1D seismic site response analysis. In this study frequency approach is followed by 1D wave propagation. This study performs a site response analyses that consider i) input motion of synthetic time history using (TARSTCH) from response spectra of IS 1893:2002 (Fig.-3) [12], ii) de-convolution by DEEPSOIL (Hashash et al., 2012) [14] for a certain depths of soil column, damping property of soil (considered to be very low e.g. 0.1%), shear wave velocity of soil etc. For hard, medium and soft soil shear velocities are considered as 600 m/s, 350 m/s and 200 m/s respectively [17]. Operation input and output is not in terms of the upward and downward propagating wave trains, but in terms of the motions at (a) boundary between two layers, referred to as a 'within' motion (b) at a free rock-surface, referred to as an 'outcrop' motion [8]. This synthetic time history response is given as an input to DEEPSOIL (soil column depth 45 m) as an Outcrop motion and the output is obtained at the bed-rock as a within motion. This bed rock motion (at depth 45 m) is re-convoluted to get target ground motion. Moreover, for further verification of de-convolution, secondary ground acceleration is obtained for free field condition by using finite element software LUSAS. The time histories and response spectra for de-convoluted and re-convoluted data vis-a-vis codal (IS 1893:2002) [12] spectra for medium soil are given below.

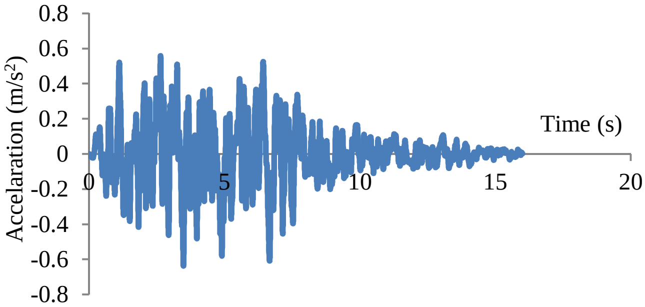

Fig. 1: Simulated ground time history (TARSCTHS-code).

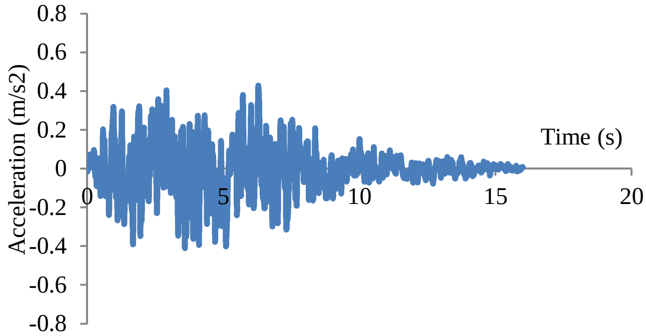

Fig. 2: De-convoluted time history at 45m deep bedrock (DEEPSOIL).

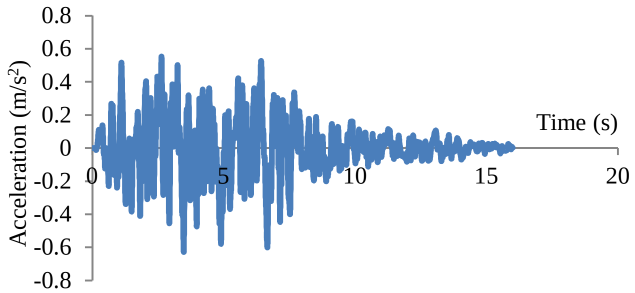

Fig. 3: Re-convoluted time history at ground (DEEPSOIL).

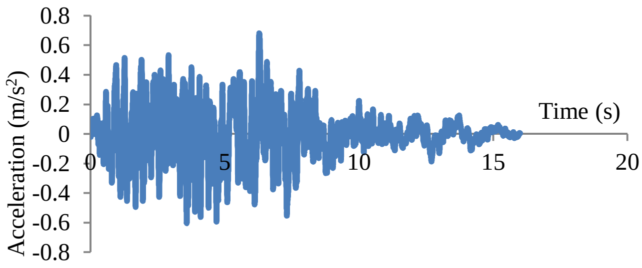

Fig. 4: Secondary time history at ground surface (LUSAS).

The codal (IS 1893:2002) [12] spectra is multiplied by $\frac{Z.I}{2R}$ (Z = 0.36 for zone V, I = 1 and R =3) to get the factored response spectra.

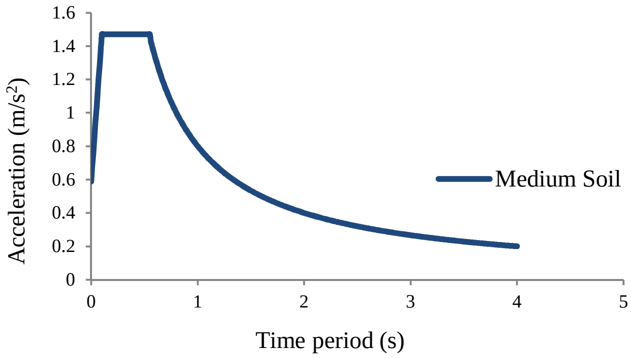

Fig. 5: Target Response Spectra (TRS) (IS: 1893-2002).

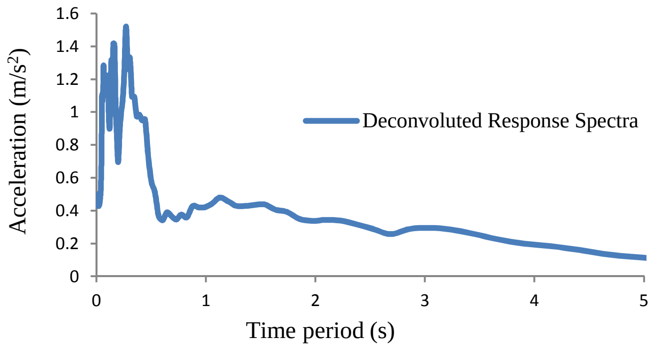

Fig. 6: De-convoluted response spectra (DRS) at Bed-rock.

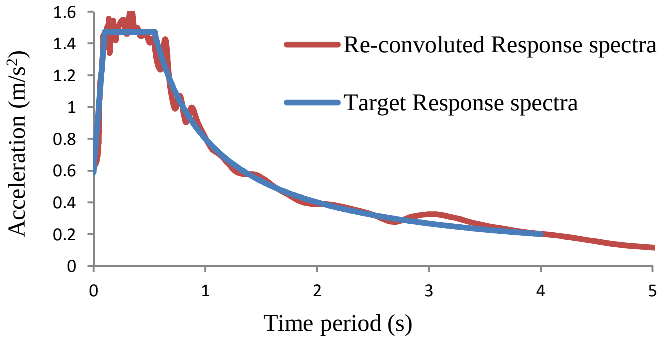

Fig. 7: Re-convoluted and Target Response Spectra (TRS).

Seeing the closeness of two spectra as in Fig.7 it is clear that the de-convolution practice is very much accurate. We should use the de-convoluted data at any depth below the ground surface rather than using the approximation as proposed by code at the ground surface.

4. Geometric modelling

The plan of asymmetric building model is taken from SP 22 (S & T)-1982 [15]. Plan of the building is given below in Fig. 8. Plan area of the building is taken as 30 m × 22.5 m and height of each storey is 3 m. Column and Beam dimensions are taken as 0.6 m × 0.4 m and 0.5 m× 0.4 m and modelled as thick beam element. Slab thickness is taken as 0.15 m and modelled as thick shell element. Wall thickness is 0.2 m. Diagonal strut is used in the modelling in lieu ofwall as thin beam to neglect the moment of inertia. The width of diagonal strut is taken as 1 m (width to diagonal length ratio, aspect ratio [3] = 0.135) and depth is taken as wall thickness 0.2m. In Table-1, geometric properties of different elements used for modelling of the building are given.

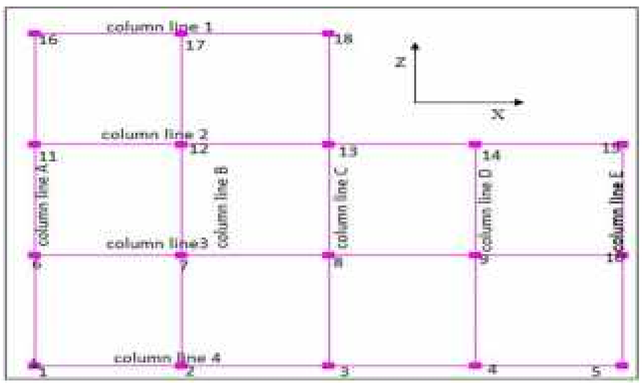

Fig. 8: plan of asymmetric building.

Table-1: Geometric properties of different structural elements of the building.

	Column	Beam	Footing	Tie Beam	Strut
A	0.24 m^2	0.2 m^2	0.8 m^2	0.01 m^2	0.2 m^2
I_{yy}	3.2×10^{-3} m^4	4.167×10^{-3} m^4	0.010667 m^4	8.3333×10^{-6} m^4	0.667×10^{-3} m^4
I_{zz}	7.2×10^{-3} m^4	2.67×10^{-3} m^4	0.26667 m^4	8.3333×10^{-6} m^4	0.01667 m^4
J_{xx}	7.51×10^{-3}	5.47×10^{-3}	0.03729	14.08×10^{-6}	2.330×10^{-3}
A_s	0.2 m^2	0.167 m^2	0.667 m^2	8.33×10^{-3} m^2	_

SSI models for bare frame and frame with in-fill are analysed. In case of bare frame, load of the non-structural elements like, in-fill and slab are given as non-structural point mass in this study. In both the model mass is conserved. For modelling of soil, different models with dimension 9 times, 6 times, 3 times (570 m × 427 m, 390 m × 292.5 m, 210 m × 157.5 m) of building plan in two lateral directions are considered and depth of soil is taken as 2 times (45 m) of the least plan dimension of structural model [4].

5. Finite element meshing

Soil is modelled as Solid element (Stress element) of HX8M element. For meshing of soil in vertical *Y*-direction, for first 15 m meshing is taken as equal 3 divisions and last 30 m is divided in 4 divisions with bias ratio 0.5 (first to last element) and in two lateral directions besides building plan first 15 m is taken as 4 equal elements. After that in *X*-direction number of division is kept 18 with bias ratio 0.2222 and in *Z*-direction number of division is kept 14 with bias ratio 0.125. Below building footing (tie beam) in *Z*-direction meshing is considered same as footing and in *X*-direction bay distance 7.5 m is divided into 2 elements. Beams are considered as 1 element and columns as 2 elements. The footing is divided into 2 elements.

6. Material properties

Mainly two materials are modelled in this study, i.e. reinforced concrete material and soil mass. In Table-2 material properties of concrete are given.

Table-2: Material properties of concrete.

Young Modulus (E)	2.5×10^{10} N/m^2
Poisson's Ratio (μ)	0.25
Unit weight (γ)	24 kN/m^3

However, diagonal struts are attributed with Young's modulus of 1.25×10^{10} N/m^2 but with negligible density. The mass of infill walls are given as lumped masses at beam-column junctions. To model the soil for different stiffness the elastic modulus [17] is varied between 1×10^7 N/m^2 to 2.56×10^9 N/m^2. Soil mass unit weight (18 kN/m^3) and poisson's ratio (0.25) is kept constant for this wide range of variation of soil stiffness. From the above reference SP: 22 (S & T) [15] the walls are considered of 0.20 m thick with unit weight 20 kN/m^3, this wall mass is given as non-structural point mass to the desired points of the wall location at the beam column junction. Floor slabs are 0.15 m thick and unit weight is 24 kN/m^3. Reduced Live Load is 25% of 2.0 kN/m^2, given as lumped mass in each floor (except roof), equally distributed at the points of beam column junction.

7. Validation

As per IS 1893:1984 [13] fundamental natural time period of a building can be estimated as $T = 0.1n$; where n is number of storey. For validation of our model three natural frequency models (G+3: $n = 4$) are analysed under free vibration and fundamental natural frequency values are compared with LUSAS in Fig. 9. Staad results are given as added information. Where, model 1 is bare frame, model 2 is frame with slab, model 3 is frame with slab and in-fill. For capturing the behaviour of frequency changes with the changes in the structural elements of a building model, 3 different models are analysed. It can be seen that as the stiffness of models increases fundamental frequency increased and the model 3 gives fundamental frequency value as 2.49 Hz which resembles to codal (IS 1893:1984) [13] value 2.5 Hz (as T is 0.4 sec). SSI effect on fundamental frequency with varying soil stiffness is studied and presented in Fig.10.

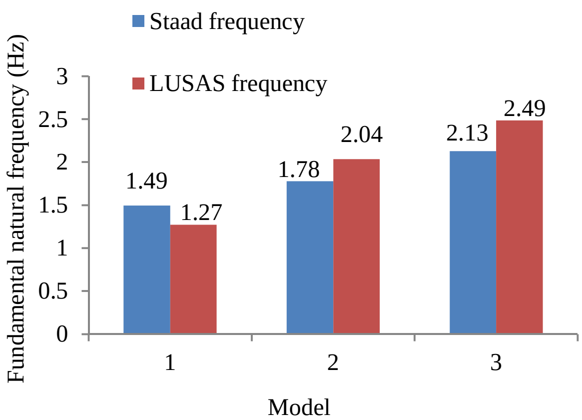

Fig. 9: Fundamental natural frequency comparison.

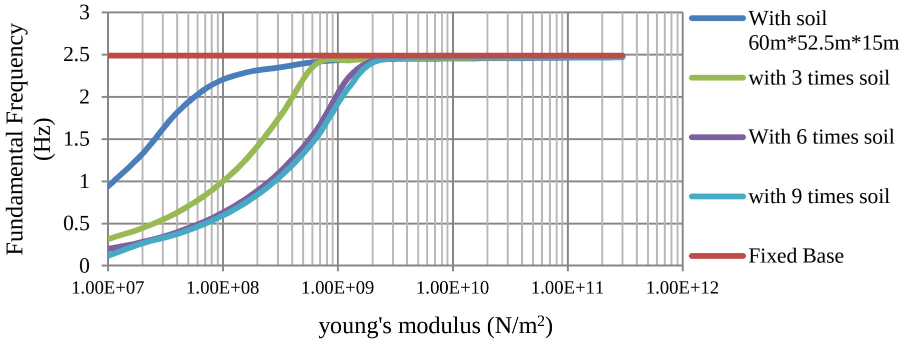

Fig. 10: Natural frequency variation with SSI.

It has been observed that at high soil stiffness fundamental natural frequency approaches to the fixed base fundamental frequency. It can also been seen that the for smaller soil dimensions are giving lower interaction behaviour, however lateral soil extend up to 6 times to base dimension and that of 9 times are giving almost same frequencies for a wide range of variation of Young's modulus.

8. Result and discussion

8.1 Base Shear. In Fig.11 comparison of base shear of G+3 building (with infill wall) for fixed base condition (IF) as well as under soil-structure interaction (IS) for codal (IS 1893: 2002) and de-convoluted spectra ('*Z*'-direction) is done. It is seen that in all soil models peak response of base shear is obtained at soil young's modulus 6.4×10^{8} N/m^{2} i.e. for stiff soil. In case of 6 and 9 times soil extent, for de-convoluted response spectra it is seen that base shear value is almost 36% lesser than that of base shear for target response spectra.

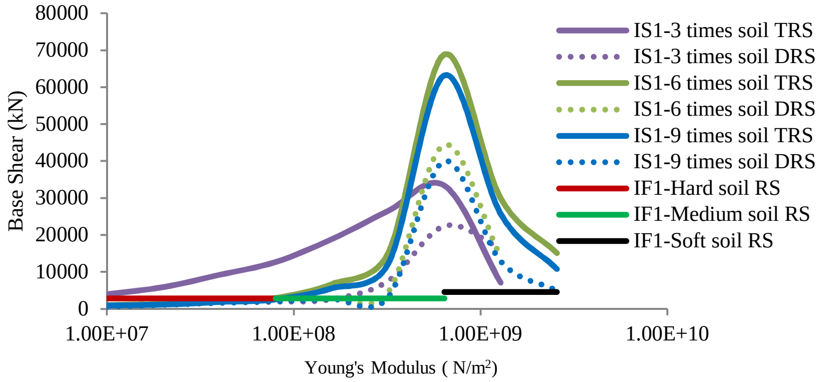

Fig. 11: Base shear for Target Response Spectra and De-convoluted Response Spectra.

Hence, it is desired to use the de-convoluted motion of earthquake for any soil-structure interaction problem. Moreover, it is observed that in case of fixed base condition soft soil spectra is giving peak value but G+3 in-filled frames under SSI, peak base shear occurred at stiffer soil. Peak response obtained at soil Young's modulus 6.4×108 N/m^2; it is giving 9 times more value than fixed base condition (soft soil spectra).

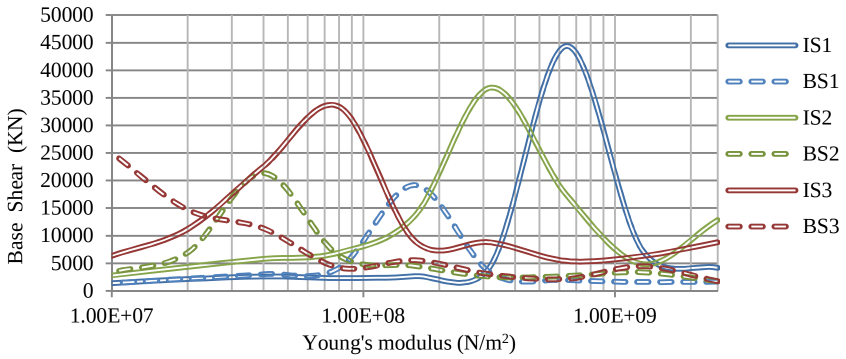

Fig. 12: Base shear comparison for bare frame and in-filled frame.

From time history response recorded at ground (Fig. 4) of a bare soil model, it is found that maximum acceleration obtained from the motion is 0.103059 m/s^2 and maximum velocity is 0.07693 m/s and from this data the calculated frequency content is coming to be 1.3265 Hz, which is very close to the frequency of combined soil-structure system (1.591 Hz) at soil young's modulus 6.4×10^8 N/m^2 and for this reason resonance occurred and peak base shear occurred at stiffer soil. In Fig.12 the comparison of base shear for different storey height like; G+3, G+6, G+10 with varying soil young's modulus for bare frame and frame with infill is shown. Where I, B, S imply in-filled frame, bare frame, soil respectively and 1, 2, 3 imply the number of story 1 for G+3, 2 for G+6 and 3 for G+10 building. It is seen that, irrespective of storey height bare frame is having less base shear than frame with in-fill. Moreover, it is also seen that due to less stiffness of bare frame system with respect to frame with infill, peak base shear is found to be at lesser stiff soil than the frame with infill. In case of bare frame maximum base shear is found in G+10 building; whereas in case of in-filled frame maximum base shear is found in G+3 building.

8.2 Torsional base shear. Torsional base shear variation with soil stiffness for G+3, G+6 and G+10 stories building frames with and without infill are shown in Fig.13 and corresponding numbering of 1, 2 and 3 are given, respectively. It is observed that with the increase in storey height, torsional base shear increases. The peak response for

G+3 building is found to be at soil young's modulus 6.4×10^8 N/m^2 and as the building height increases, peak torsional base shear is also found to be at flexible soils. It is also seen in Fig. 13 that for G+10 building, peak torsional base shear is shifted towards the soft soil than other two building models. It is also found that, in softer soil torsional shear is lesser than fixed base condition i.e. soft soil in account of interaction with structure reduces the torsional shear.

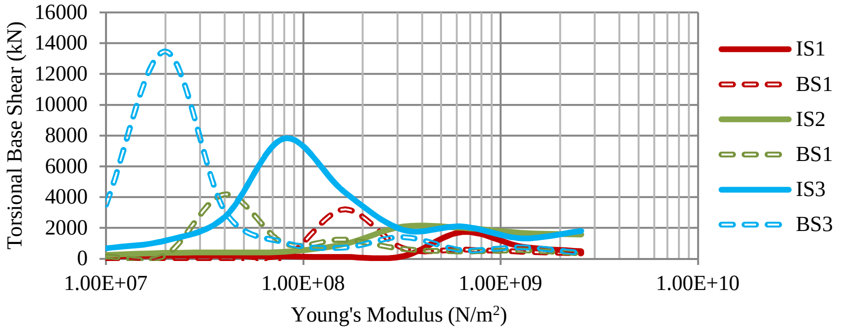

Fig. 13: Torsional base shear comparison for bare frame and in-filled frame.

It is also observed that due to infill wall torsional shear reduced. In case of G+3 model, peak torsional shear for bare frame is approximately 1.85 times more than frame with infill. Whereas, for G+6 this is coming 2 times and for G+10 it is 1.723 times. So it can be said that infill wall has reduced torsional shear considerably.

9. Conclusions

In this study a comparison of responses of bare frame and frame with in-fill is done. Following conclusions can be drawn from this study-

1. Smaller buildings resonate at stiffer soil whereas taller buildings resonate at softer soil.
2. Due to infill wall, stiffness of building increases irrespective of storey height which gives higher frequency than bare frame and for that reason in-filled frame building resonates at relatively stiffer soil. Due to addition of infill wall there is considerable increase in base shear for a particular building height than the bare frame.
3. Torsional shear has been reduced due to infill wall. It is seen that, consideration of infill is beneficial for multi-storey asymmetric building as it reduces the torsional response. With an increasing storey height (or storey number) torsional shear increases. In softer soil torsional shear is considerably reduced than stiffer soil.

4. De-convoluted motion results in accurate estimation of response of building rather than considering the ground motion (overestimation) applied at soil base in numerical modelling.

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