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Seismic soil structure interaction analysis for asymmetrical buildings supported on piled raft for the 2015 Nepal earthquake

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ABSTRACT

Seismic damage surveys and analyses conducted on modes of failure of structures during past earthquakes observed that the asymmetrical buildings show the most vulnerable effect throughout the course of failures (Wegner et al., 2009). Thus, all asymmetrical buildings significantly fails during the shaking events and it is really needed to focus on the accurate analysis of the building, including all possible accuracy in the analysis. Apart from superstructure geometry, the soil behavior during earthquake shaking plays a pivotal role in the building collapse (Chopra, 2012). Fixed base analysis where the soil is considered to be infinitely rigid cannot simulate the actual scenario of wave propagation during earthquakes and wave transfer mechanism in the superstructure (Wolf, 1985). This can be well explained in the soil structure interaction analysis, where the ground movement and structural movement can be considered with the equal rigor. In the present study the object oriented program has been developed in C++ to model the SSI system using the finite element methodology. In this attempt the seismic soil structure interaction analysis has been carried out for T, L and C types piled raft supported buildings in the recent 25th April 2015 Nepal earthquake ($M = 7.8$). The soil properties have been considered with the appropriate soil data from the Katmandu valley region. The effect of asymmetry of the building on the responses of the superstructure is compared with the author's research work. It has been studied/observed that the shape or geometry of the superstructure governs the response of the superstructure subjected to the same earthquake load.

1. Introduction

Asymmetric plan buildings, i.e. buildings within the plan asymmetric mass and strength distributions, are systems characterized by a coupled torsional translational seismic response (Isbiliroglu et al., 2014). Asymmetric structures are almost unavoidable in modern construction due to various types of functional and architectural requirements.

Buildings with an asymmetric distribution of stiffness and strength in plan undergo coupled lateral and torsional motions during earthquakes. In many buildings the center of resistance does not coincide with the center of mass. The inelastic seismic behavior of asymmetric plan buildings is considered by using the histories of base torsion and the displacements. The behavior of buildings during earthquakes will be satisfactory only if all measures are taken to provide a favorable failure mechanism (Wegner et al., 2009). A special account must be taken so that torsional effect can be minimized. Torsion in buildings during earthquake shaking may be caused by a variety of reasons, the most common of which are non symmetric distributions of mass and stiffness. It is well known that the larger the eccentricity between the center of stiffness and the center of mass, the larger the torsional effects (Fig. 1). The equilibrium between inertial force and the resistance force depends upon the eccentricity (e), which is the distance between the center of mass (CM) and center of resistance (CR).

In structures, which remain elastic during an earthquake, torsional vibrations may cause significant additional displacements and forces in the lateral load resisting elements. However, the design of the majority of buildings relies on inelastic response. In that case torsional motion leads to additional displacement and ductility demands.

Modern codes deal with torsion by placing restrictions in the structural design with irregular layouts and also through the introduction of an accidental eccentricity that must be considered in the design. The lateral torsional coupling due to eccentricity between center of mass (CM) and center of resistance (CR) in asymmetric building structures generates torsional vibration even under purely

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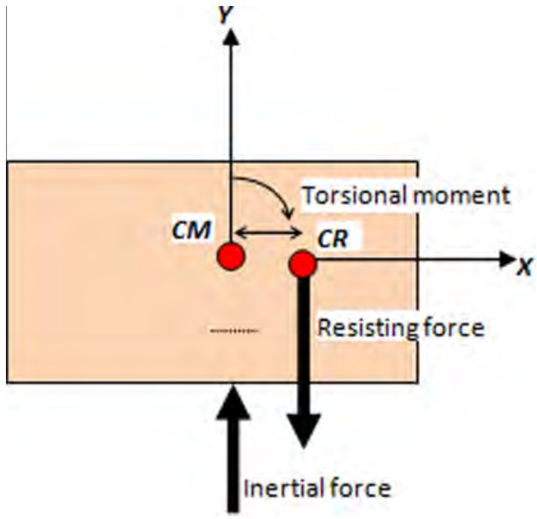

Fig. 1. Generation of torsional moment in asymmetric structures during seismic excitation.

translational ground shaking. During seismic shaking of the structural systems, inertia force acts through the center of mass while the resistive force acts through the center of resistance (Fig. 1).

Asymmetric buildings are more vulnerable to earthquake hazards compared to the buildings with symmetric configuration. The recognition of this sensitivity has led the researchers to concentrate their studies on earthquake characteristics, evaluation of the structural parameters and validity of the system models (Shakib and Fuladgar, 2004) among others. However, the destruction of numerous asymmetric buildings in the 1985 Mexico earthquake made researchers focus on soil–structure interaction effects and on the response behavior of such systems (Chopra, 2012). So far, several researchers have attempted to incorporate the flexibility of foundation in asymmetric system models. Mao and Wang (2009) used simple springs to represent frequency-independent values and to approximate the frequency-dependent foundation impedance functions in an asymmetric multistory building (Dutta and Roy, 2002). Subsequently Stewart et al. (1999) extensively investigated the steady-state response of flexibility supported torsionally coupled buildings subjected to harmonic ground motions by using frequency-independent springs and dashpots (Dutta and Roy, 2002). Using the same simple single-storey structure model. Wu and Finn (1997) presented a method of analysis to determine the seismic response of three-dimensional asymmetric multi-storey building foundation systems using approximate frequency independent foundation impedance functions (Kramer, 1996). Cai et al. (2000) incorporated the frequency-dependent foundation impedance functions in the frequency domain to assess the combined soil–structure interaction and torsional coupling effects on the asymmetric buildings. An accurate modeling of soil–structure interaction is expected to incorporate the major effects of soil–structure interaction in the response of complex systems such as torsionally coupled systems.

The response if the asymmetrical building has been investigated by Lin et al. (2013) and Olariu and Movila (2014) by analytical approaches like arithmetic sum method and spectral acceleration method to understand the behavior of shallow foundation by incorporating the interaction effect by spring and dashpot. Mason et al. (2013) and Yigit et al. (2015) carried out the experimental study with scaled down model of the asymmetrical dwarf building to study the soil structure interaction effect on the structural response under earthquake. Still the approaches not extended for the pile supported asymmetrical buildings. Chopra and Gutierres (1978) highlighted out that the numerical methods are most appropriate and accurate methods for soil structure interaction analysis. Followed by this several researchers, including Wegner (Wegner et al., 2009), Han (2009) and Sharma et al. (2014) carried out the study for SSI analysis of the asymmetrical building supported by the isolated, raft and shallow foundation system by considering the 3-D and the 2-D nonlinear analysis. Isbiliroglu et al. (2014) and Sharma et al. (2014) attempted to analyze the nonlinear dynamic SSI system of an asymmetrical building supported by shallow foundation and effect of interaction has been modeled by the spring and dashpot. As the asymmetrical buildings are one of common and unavoidable construction the more attention must be given toward the precise analysis which included the interaction effect. But once the interaction effect included in the numerical analysis the modeling becomes very complex and the time of analysis also increases exponentially due to consideration of soil element and up to the infinite domain.

2. Influence of plan geometry

The influence of the plan geometry of the building on its seismic performance is best understood from the basic geometries of the structures. Buildings with rectangular plans and straight elevation stand the best chance of doing well during an earthquake, because inertia forces are transferred without having to bend due to the geometry of the building. But, buildings with setbacks and central openings offer geometric constraint to the flow of inertia forces; these inertia force paths have to bend before reaching the ground (Murty et al., 2013). Fig. 2 shows the load transfer mechanism for the symmetrical and asymmetrical building.

Buildings with regular geometry like rectangle, square in plans have direct load paths for transferring seismic inertia forces to its base, while those with complex plans, including X, Y, L, T, V and irregular plan shape necessitate indirect load paths that result in stress concentrations at points where load paths bend (Fig. 3).

Thus, all asymmetrical buildings come under the category of complex plan system and significantly fails during the shaking events. It is really needed to focus on the accurate analysis of the building, including all possible techniques to improve the performance of the asymmetrical building during earthquakes.

In the fixed base analysis of any building the interaction effects are neglected as the analysis is costlier (time taken for analysis) and the modeling is very tedious. In this regard the dynamic nonlinear interactions analysis of the irregular/asymmetrical buildings, including the plan shapes of C, L and T type has been carried out to understand the building demands under the seismic.

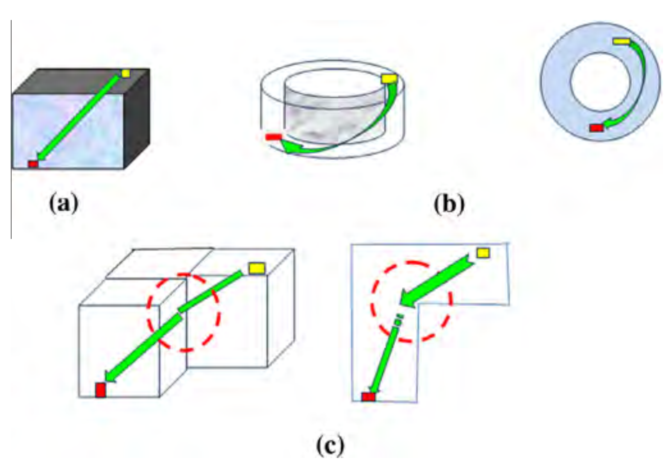

Fig. 2. Classification of buildings: (a) simple and (b) & (c) complex.

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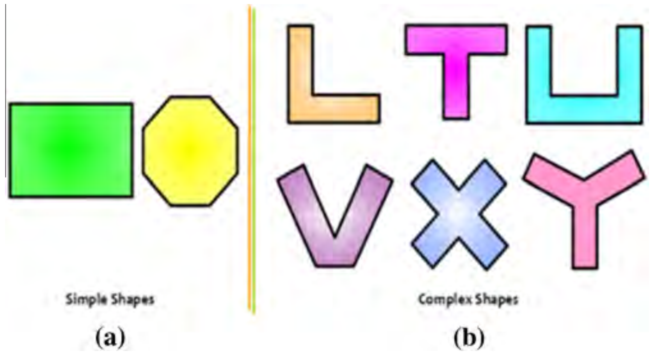

Fig. 3. Plan shapes of buildings: Buildings with (a) simple shapes undergo simple acceptable structural seismic behavior, while (b) those with complex shapes undergo complex unacceptable structural seismic behavior.

3. Details of 25th April 2015 Nepal Earthquake

The April 25, 2015 ($M = 7.8$) Nepal earthquake occurred as the result of thrust faulting on or near the main frontal thrust between the subducting India plate and the overriding Eurasia plate to the north. At the location of this earthquake, approximately 80 km to the northwest of the Nepal capital of Kathmandu, the India plate is converging with Eurasia at a rate of 45 mm/yr toward the north-northeast, driving the uplift of the Himalayan mountain range. The preliminary location, size and focal mechanism of the April 25 earthquake are consistent with its occurrence on the main subduction thrust interface between the India and Eurasia plates. Although a major plate boundary with a history of large-to-great sized earthquakes, large earthquakes on the Himalayan thrust are rare in the documented historical era. Just four events of magnitude 6 or larger have occurred within 250 km of the April 25, 2015 earthquake over the past century. One, a magnitude 6.9 earthquake in August 1988, 240 km to the southeast of the April 25 event, caused close to 1500 fatalities. The largest, magnitude 8.0 event known as the 1934 Nepal–Bihar earthquake, occurred in a similar location for the 1988 event. It severely damaged Kathmandu, and caused around 10,600 fatalities.

Taking a close view of this event, it is noticed that several buildings have been collapsed, which includes stone buildings, masonry structures, soft storey structures and monuments. Based on the state of buildings in Kathmandu and the surrounding areas the soft storey and asymmetrical building collapses are found to be more (PEER bulletin, 2015). Thus, it is primarily needed to analyze the buildings with most precision, including all possible reality in analysis like earthquake loading, soil structure interaction effect, wave propagation in soil and local soil behavior. In this concern the soil structure analysis is carried out for taking the standard asymmetrical shape like C, L and T.

4. Soil structure interaction analysis

The theory on soil structure interaction is established in the early 1960s. In 1985 Wolf, has given an understandable shape by introducing elastic half space theory for the soil structure interaction. Ground motions that are not influenced by the presence of structure are referred as free field motions. Structures founded on the rock are considered as fixed base structures. When a structure founded on solid rock is subjected to an earthquake, the extremely high stiffness of the rock constrain the rock motion to be very close to the free field motion and can be considered as a free field motion and fixed base structures.

Dynamic analysis of SSI can be done using Direct Method and Substructure Method. The direct approach is one in which the soil and structure are modeled together in a single step accounting for both inertial and kinematic interaction. Substructure method is one in which the analysis is broken down into several steps that is the principal of superposition is used to isolate the two primary causes of SSI (Wolf, 1985).

If the structure is supported on soft soil deposit, the inability of the foundation to conform to the deformations of the free field motion would cause the motion of the base of the structure to deviate from the free field motion. Also the dynamic response of the structure itself would induce deformation of the supporting soil. This process, in which the response of the soil influences the motion of the structure and the response of the structure influences the motion of the soil, is studied under the interaction effects and popularly known as soil structure interaction (Fig. 4). These effects are more significant for stiff and/or heavy structures supported on relatively soft soils. For soft and/or light structures founded on stiff soil these effects are generally small. It is also significant for closely spaced structure that may subject to pounding, when the relative displacement is large (Maheshwari et al., 2004).

When the seismic wave E_0 generated by an earthquake fault reaches the bottom of the foundation, they are divided into two types (Fig. 4). Transmission Waves which are entering into the building shown as E_1 and Reflection Waves which are reflected back into the ground shown as F_0 (Maheshwari et al., 2004).

When the transmission wave enters into the building it travels in upward direction due to which the structure subjects to vibration. And then the waves are reflected at the top and travel back down to the foundation of the structure. At this stage Soil–Structure Interaction phenomenon takes place. Again a part of the wave is transmitted into the ground, while the rest is reflected back again and starts to move upwards through the structure. The waves which transmitted to ground are known as Radiation Waves. When the radiated waves are in small amount, the seismic waves once transmitted into the structure continue to trapped in the building, and the structure starts to vibrate continuously for a long time, similar to the lightly damped structure.

The damping caused by radiation waves is known as Radiation Damping of the soil. The radiation damping results in increase of total damping of the soil–structure system in comparison to the structure itself. Also, under the influence of SSI the natural frequency of a soil structure system shall be lower than the natural frequency of the soil.

These interactions results not only in reducing the demands on the structure, but also increasing the overall displacement of the structure as due to these interactions, foundations can translate and rotate. Basically the dynamic soil structure interaction consists of two interactions, namely, kinematic interaction and inertial interaction.

4.1. Significance of seismic soil–structure interaction

The seismic response of an engineering structure is influenced by the medium on which it is founded. On the solid rock, a fixed base structural response occurs which can be evaluated by subjecting the foundation to the free-field ground motion occurring in the absence of the structure. However, on deformable soil, a feedback loop exists. In the other words, when the feedback loop exists, the structure responds to the dynamics of the soil, while the soil also responds to the dynamics of the structure. Structural response is then governed by the interplay between the characteristics of the soil, the structure and the input motion.

The Mexico City earthquake in 1985 and Christchurch-New Zealand earthquake in 2011 clearly illustrate the importance of local soil properties on the earthquake response of structures. These earthquakes demonstrated that the rock motions could be significantly amplified at the base of the structure. Therefore, there

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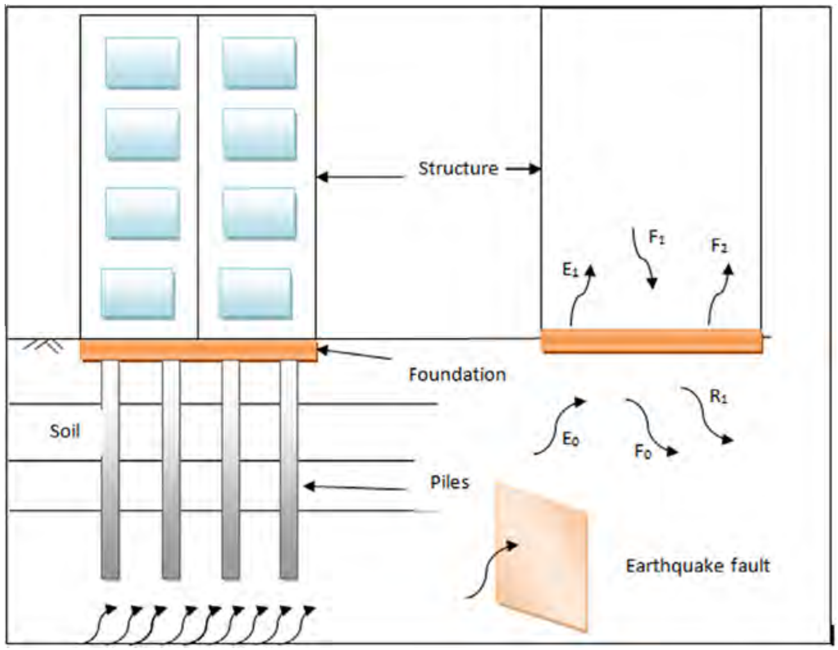

Fig. 4. General scenario of consideration of soil structure interaction effect and wave propagation (Maheshwari et al., 2004).

is a strong engineering motivation for a site-dependent dynamic response analysis for many foundations to determine the free-field earthquake motions. The determination of a realistic site-dependent free-field surface motion at the base of the structure can be the most important step in the earthquake resistant design of structures. For determining the seismic response of building structures, it is a common practice to assume the structure is fixed at the base. However, this is a gross assumption since flexibility of the foundation could be overlooked and underestimated in this case. This assumption is realistic only when the structure is founded on solid rock.

The main concept of site response analysis is that the free field motion is dependent on the properties of the soil profile including stiffness of soil layers. The stiffness of the soil deposit can change the frequency content and amplitude of the ground motion. Likewise, on the path to the structure, wave properties might be changed due to the stiffness of the foundation. In general, the subsoil foundation response subjected to seismic ground motion has been dictated by the soil attributes, the soil conditions, and the characteristics of the earthquake. Wave propagation theory denotes that soil layers; modify the attribute of the input seismic waves while passing through the soil, so that the acceleration record will be affected.

Soil–structure interaction, particularly for pile supported buildings resting on relatively soft soils may significantly amplify the lateral displacements and inter-storey drifts. Considering performance-base design approach, the amplification of lateral deformations due to SSI may noticeably change the performance level of the building frames. Consequently, the safety and integrity of the building would be degraded.

National and international design codes, e.g. Australian Standards (AS 1170.4-2007), International Building Code (IBC, 2012) and National Building Code of Canada (NBCC, 2010) permit the use of alternate methods of design to those prescribed in their seismic requirements with the approval of regularity agency having due jurisdiction (Roger and Frank, 2006). The ground motions in seismic regions in Asia–Pacific such as New Zealand, Indonesia, and some parts of Australia will most probably govern the design of lateral resisting systems of buildings. As a result, there is a strong need to develop design tools to evaluate seismic response of structures considering the foundation flexibility and sub-soil conditions.

In this study, numerical investigations are employed to study the effects of dynamic soil–structure interaction on seismic response of asymmetrical mid-rise building frames supported on the pile foundation system. To achieve this goal, a nonlinear soil structure interaction program is developed using C++ which has been verified by a series of test cases during execution time.

5. Finite element modeling of DSSI system for L, C and T-shape buildings

The finite element program using C++ is developed to analyze the SSI system. The Program can perform nonlinear static and dynamic analysis, including node to node contacts. The input need to be provided through the text files in the specified format.

The modeling of the DSSI system for 10 storey L, C and T shape asymmetrical building with generalized pile layout has been explained in detail. The soil structure interaction analysis for asymmetrical building has been considered for a homogenous soil condition. Table 1 explains the engineering properties of the various modeling parameters of superstructure, soil and the piles and interface/contact considered.

5.1. Superstructure modeling

The G+10 L, C and T shape superstructure components, including beams and column have been modeled with 2 noded 3-D beam elements. The joints between beam and column are considered to be rigid. The connection between the raft and first storey column is modeled as the rigid connections.

5.2. Soil modeling

Half space is modeled using as sandy silt and the engineering properties of the soil domain has been explained in detail in Table 1. The homogeneous soil of volume $20 \times 20 \times 20$ m

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Table 1
Properties of the soil domain considered for the analysis.

Engg. properties	Unit wt. (kN/m^3)	Friction angle, φ' (°)	Poisson's ratio	E (kN/m^2)	Vs (m/s)	Damping
Soil type: sand	18	35	0.35	445,872	300	Mass proportionate
Super structure	24	0	0.15	2.0×10^7	1200	Mass proportionate
Pile	24	0	0.15	2.0×10^7	1200	Mass proportionate
Raft	24	0	0.15	2.0×10^7	1200	Mass proportionate
Material model parameters	Poisson's ratio = 0.35		Friction angle = 35°		Cohesion C (kN/m^2) = 0	
Interface data	Friction angle (δ) = 1/3 φ' = 11.4°			Coefficient of friction = 0.7		

considered for the SSI analysis. The nonlinear behavior of the supporting soil is captured using Drucker–Prager material model (Drucker and Prager, 1963).

5.3. Raft and pile modeling

The 0.4 m thick raft with the 1.0 m offset from all the sides of the base of the superstructure have been modeled with the 3-D brick elements. The circular piles with 0.45 m and 9.0 m length have been modeled with the 3-D brick elements. The asymmetrical layout of C and T shape of piles accommodates the 23 and 21 number piles spaced at 1.5 m c/c. The joints between the raft and pile have been modeled with the rigid contacts.

5.4. Meshing

In the present study meshing of the finite element model has been created by using the GSA 2-D mesh tool. The coordinates of each model component like coordinates soil block, pile and the raft have to be given to the tool as an input to create the mesh in 2-D and which has been extruded to the 3rd dimension in order to get the 3D mesh of the model.

5.5. Material model

The nonlinear soil behavior under the earthquake load has been described by the Drucker–Prager elastoplastic material model. The model based on the three input parameters like cohesion, internal friction angle and dilatancy angle of the soil material. In the present study the model is implemented with the associative flow rule where yield and potential functions are the same and no hardening.

5.6. Boundary conditions

In the present study viscous boundary has been modeled in order to avoid the multiple reflections during the dynamic analysis span. A good measure of the ability of the viscous boundary to absorb impinging elastic waves is the energy ratio defined as the ratio between the transmitted energy of the reflected waves and the transmitted energy of the incident wave (Lysmer and Kuhlemeyer, 1969). This ratio can be computed from the wave amplitude ratios by considering the energy flow to and from a unit area of the boundary. In the present study all four sides of the sole domain has been modeled with the viscous boundary where the nodes of the extreme elements provided with the extra force which is equal to the force estimated at the of each time step to nullify the forces at the node. The bottom element of the soil domain is considered with the earthquake boundaries which provide the displacement in the same direction of earthquake given in the analysis and the rest of the DOF of the elements will be assigned as zero. In present study N–E (x-direction) component of the Nepal earthquake (2015, 0.18g) has been given to study the response of SSI system.

5.7. Interface modeling

The interface between the pile and soil has been modeled as a node to the node friction contact using the Lagrange multiplier method. In the finite element program, developed to analyze the DSSI system the node IDs which are in contact is needed to be provided explicitly through the input file of the interface data file. During the analysis for the node pair in contact the contact forces are estimated. The total displacement at the node, including its contact behavior can be estimated by adding the contact displacement which can be used to estimate the next time step response.

5.8. Analysis

The model is analyzed for both static and dynamic loading conditions. The system is modeled first statically to get the initial stress condition for the dynamic analysis. Once the static analysis has been completed the dynamic analysis of the system is being carried out. Following is the detailed procedure to carry out the static and dynamic analysis of the DSSI system.

5.8.1. Static analysis

The initially SSI system is analyzed for static load in order to get the initial stress condition which includes the self weight of the superstructure and the foundation system. The static analysis has been carried out by applying the fixed boundary condition in normal direction, i.e. constraining the displacements only in the normal direction to surface to the nodes of the extreme element of the soil volume considered. The effect of self weight of the slab of the 0.15 m thick slab of the superstructure has been included by applying the nodal forces at the corner nodes of each storey of the superstructure. The nonlinear response of the SSI system has been estimated using the iterative initial stiffness method.

5.8.2. Dynamic analysis

The stresses and displacement so obtained at the end of static analysis has been considered as the initial response for the dynamic analysis. The 25 April 2015 Nepal ground motion (M = 7.8, N–E component) has been applied at the bottom nodes of the soil domain and the analysis has been carried out for the peak response which lies in the 15 s (Fig. 5a). The dynamic responses have been predicted using explicit solver with the central difference method. The material nonlinearity has been considered by adopting the associative Drucker–Prager material model. The dynamic analysis has been carried out for the 15 s earthquake data which captures the peak of the ground motion (Fig. 5b).

In this study the SSI system is analyzed for the Nepal earthquake and the results are compared with the system response obtained for Bhuj, Uttarkashi and Chamba ground motions. The details of all earthquake considered in this study are given in Table 2.

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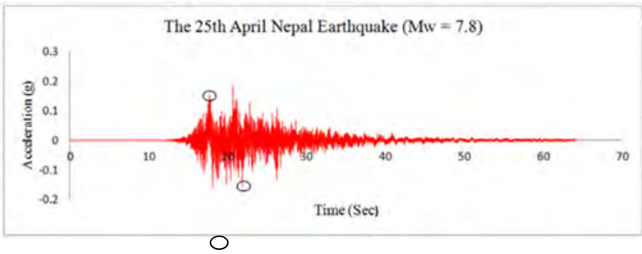
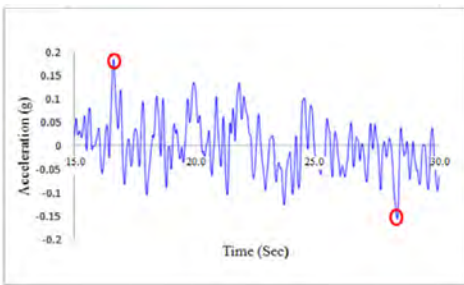


(a) Acceleration spectra for Nepal earthquake



(b) Part of ground motion considered for study

Fig. 5. Acceleration time history of the 25th April Nepal earthquake (M = 7.8).

Table 2
Details of earthquake considered for the present study.

Earthquake name	Magnitude	PGA (g)	Predominant frequency (Hz)
April 25, 2015 Nepal (NE)	7.8	0.19	1.02–4.18
January 26, 2001 Bhuj (NE)	7.7	0.31	1.32–4. 40
March 29, 1999 Uttarkashi (NW)	7.0	0.25	0.92–4.24
March 24, 1995 Chamba (NE)	4.9	0.13	0.35–3.53

6. DSSI modeling of asymmetrical buildings

The finite element model has been developed for the C shape pile supported building by implementing the method into the C++ program. The modeling of the SSI system includes the modeling of different parts like superstructure, soil, pile and the raft. The model is visualized with LS-PP tool. The G+10 asymmetrical assembly of C, L and T are modeled by incorporating the interaction effect by modeling the node to node interfaces between the pile and the soil. The effect of dynamic loading has been studied by observing the responses of the superstructure by altering the mentioned parameters during the analysis. The dynamic nonlinear analysis has been carried out including the material nonlinearity by including the Drucker and Prager soil material model. Figs. 6–8 show the generalized SSI model of pile supported C, L and T shape building.

7. Results and discussions

The different seismic event has its unique characteristics. In order to understand how a pile soil interaction takes place during earthquake it is required to estimate the predominant earthquake characteristics like frequency, peak acceleration and which ultimately reflects in the form of super structure responses when subjected to the ground shaking. The movement in the asymmetrical SSI system has been observed for 25th April Nepal Earthquake and results are compared with the other ground motions, including the 2001 Bhuj earthquake, the 1999 Uttarkashi earthquake, the 1995 Chamba Earthquake.

7.1. Dynamic analysis for C Type of building

The time history, displacement at each storey has been estimated through dynamic nonlinear interaction analysis. Figs. 9–11 show the floor wise response of C shape building in the principal directions, including X, Y and Z for the applied Nepal earthquake.

From the displacements profile observed in Figs. 9–11, it has been noted that the displacements obtained at the top of the structure have a higher value and goes on decreasing as for the lower floors till the bottom of the structure as observed by Chopra (2012). The reason for this is the inertia contribution of each floor under vibration. For the top storey the system experiences the least inertial resistance while at bottom storey it has its highest contribution.

7.2. Dynamic analysis for L shape of building

The dynamic analysis has been carried out for L shape buildings to understand the effect of shape on the building Response. To obtain this, the L shape SSI system is subjected to Nepal earthquake a external dynamic load and the response of the building is studied. Figs. 12–14 show the time history, displacement of the L shape building.

From the displacements profile observed in Figs. 12–14, it has been noted that the displacements obtained at the top of the structure have a higher value and goes on decreasing as for the lower floors due to its inertia effect. The study also noted that the displacement is found to be higher in case of C shape building than L shape for same dynamic loading in X, Y and Z directions.

7.3. Dynamic analysis for T shape of building

Also, the displacements have been estimated for T shape building at each floor. Figs. 15–17 show the time history of displacements studied at each floor, when T shape SSI system is subjected to Nepal earthquake.

Displacements profile perceived in Figs. 15–17, it has been noted that the displacements obtained at the top of the structure have a higher value and goes on decreasing for the lower stories due to the contribution of inertia. It is noted that the displacement is found to be least in case of T shape building than C and L shape building in X, Y and Z directions than L shape under same dynamic loading.

In order to compare the behavior of the asymmetrical buildings for different earthquakes, the assembly of C, L and T shape buildings are subjected to the other 3 earthquakes considered in the study, including, the 2001 Bhuj earthquake, the 1999 Uttarkashi earthquake and the 1995 Chamba Earthquake.

Fig. 18 shows the X direction time history, displacement for the 2001 Bhuj earthquake for C shape building.

Also, the displacements have been observed for the T and L shape building. Figs. 19 and 20 show the displacement at different storey height of the superstructure in X direction when L and T shape superstructure when the SSI system subjected under Bhuj earthquake.

The peak displacement noted during all 3 earthquakes considered for C, L and T shape buildings have been studied. The peak displacement in each direction, including X, Y and Z directions for each asymmetrical shape has been summarized in Table 3.

The percentage deviation in the responses of L, C and T shape building w.r.t. Bhuj earthquake has been estimated. Table 4 shows the deviation in responses in X, Y and Z direction for all considered earthquake w.r.t. Bhuj earthquake.

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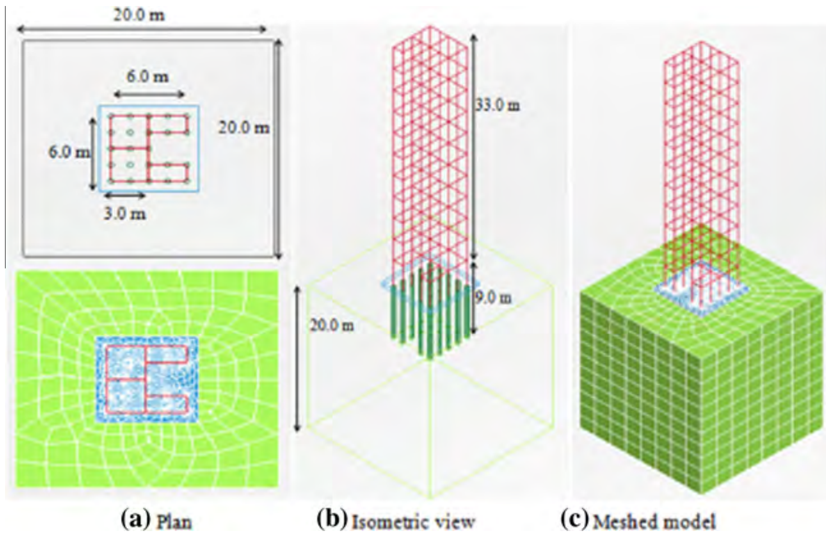

Fig. 6. Finite element model for C-shape G+10 building for DSSI system.

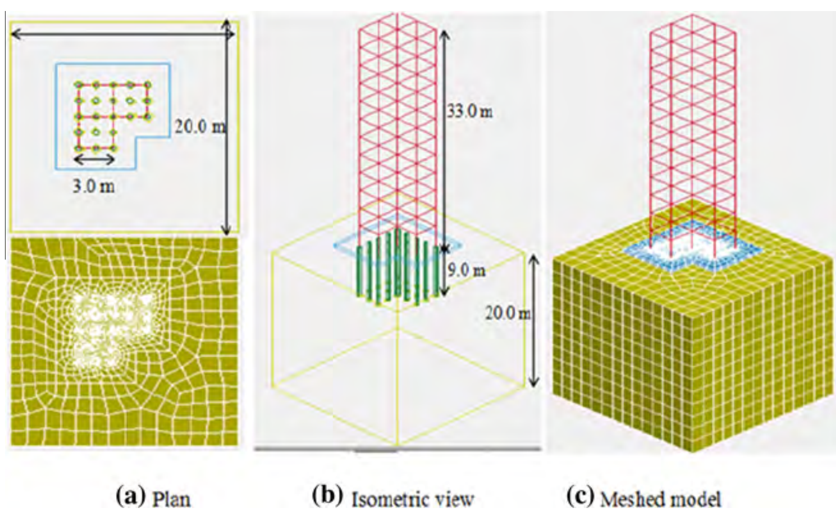

Fig. 7. Finite element model for L-shape G+10 building for DSSI system.

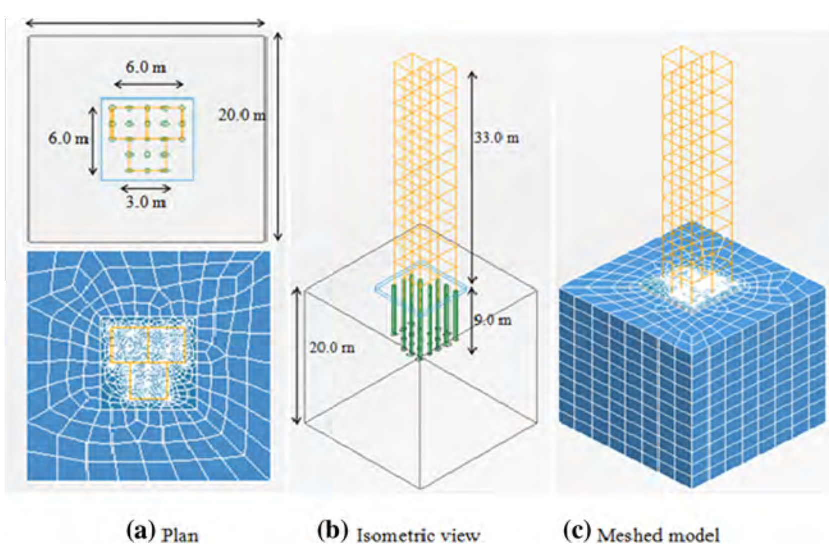

Fig. 8. Finite element model for T-shape G+10 building for DSSI system.

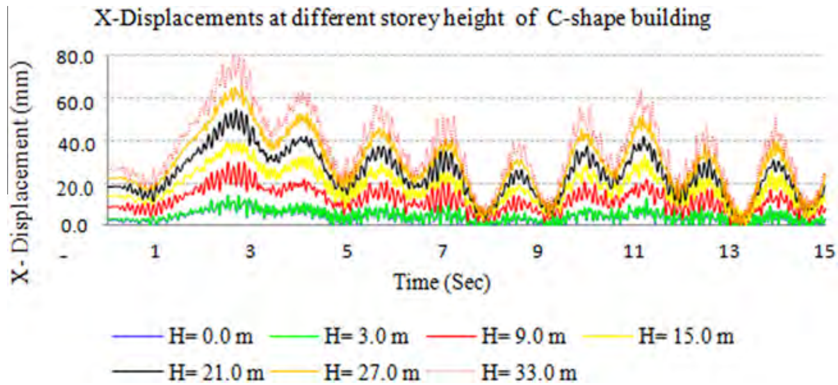

Fig. 9. *X*-direction response of C-shape G+10 building.

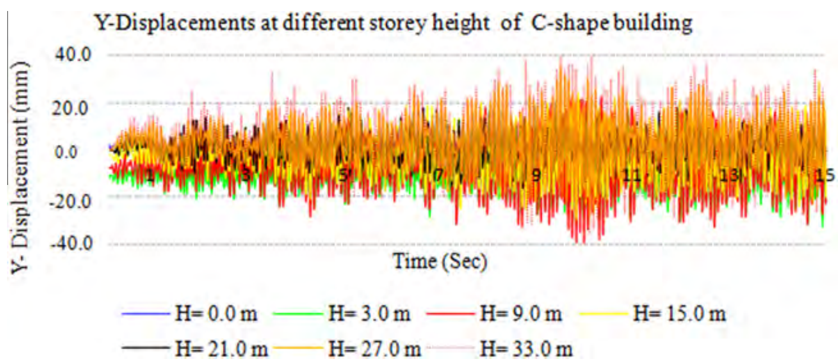

Fig. 10. *Y*-direction response of C-shape G+10 building.

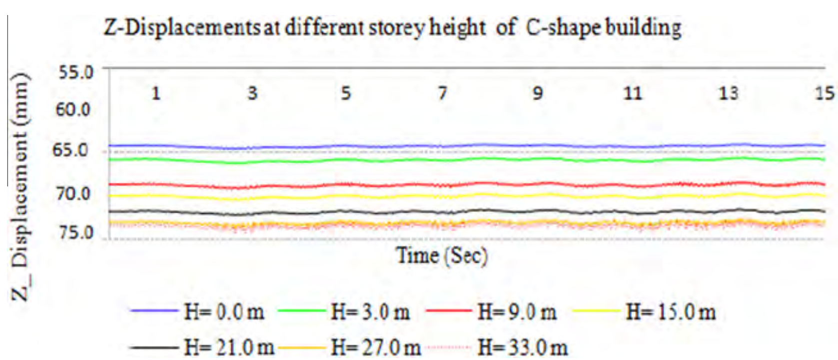

Fig. 11. *Z*-direction response of C-shape G+10 building.

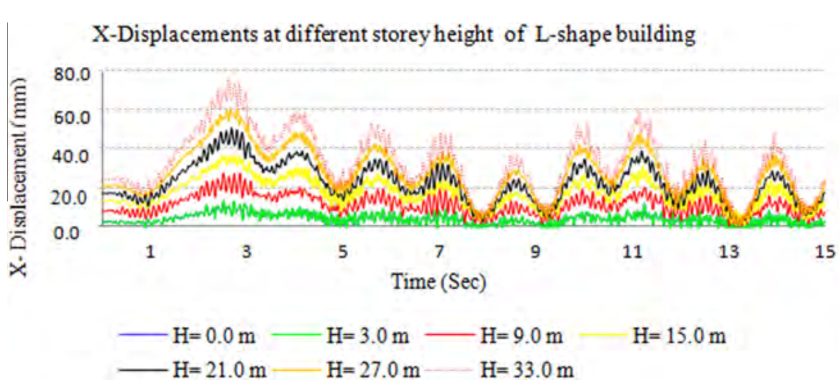

Fig. 12. *X*-direction response of L-shape G+10 building.

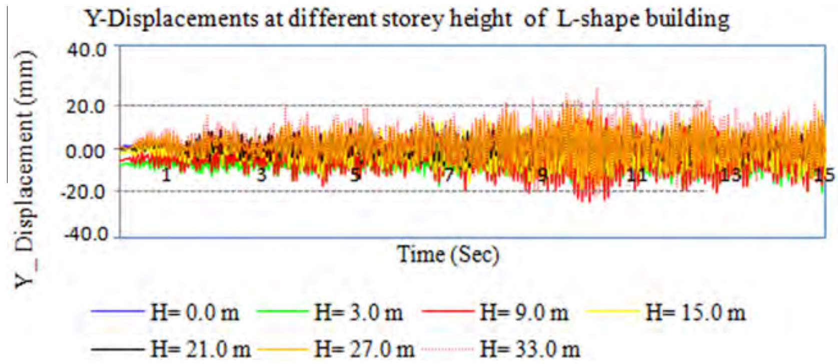

Fig. 13. *Y*-direction response of L-shape G+10 building.

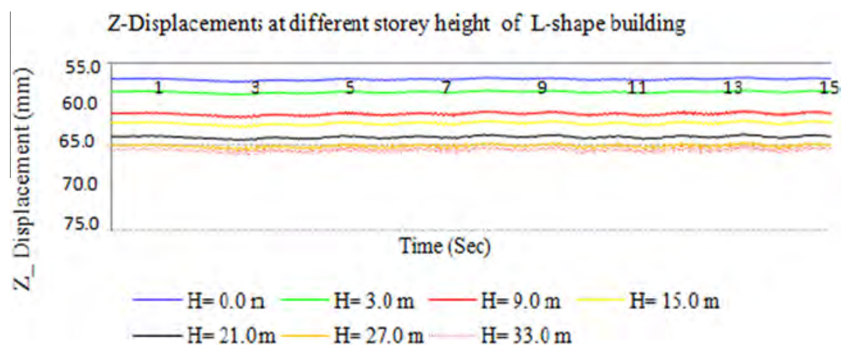

Fig. 14. *Z*-direction response of L-shape G+10 building.

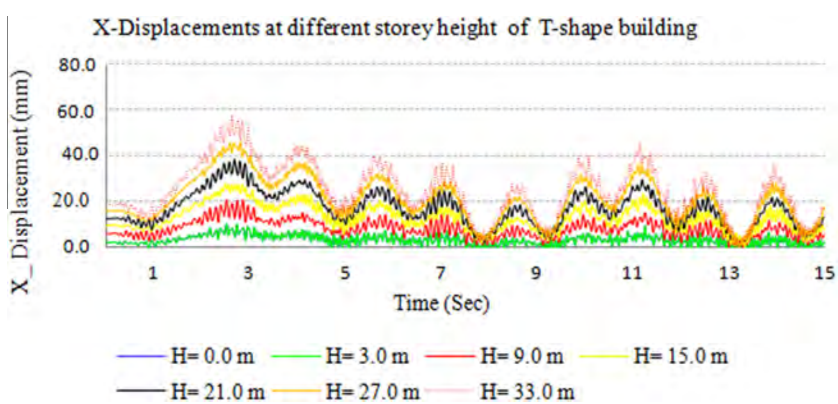

Fig. 15. *X*-direction response of T-shape G+10 building.

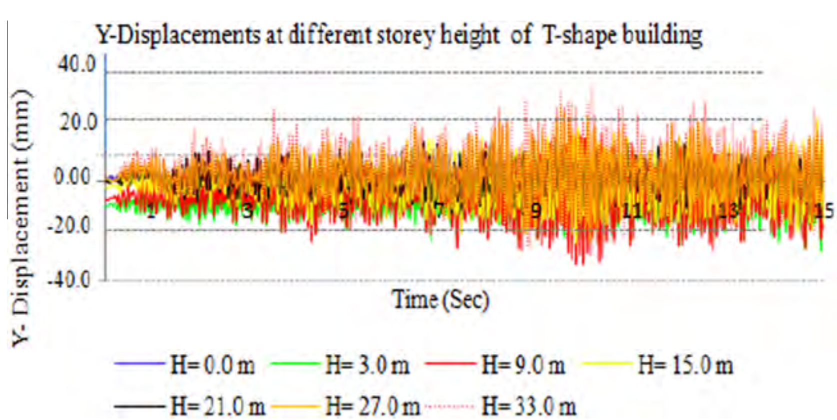

Fig. 16. *Y*-direction response of T-shape G+10 building.

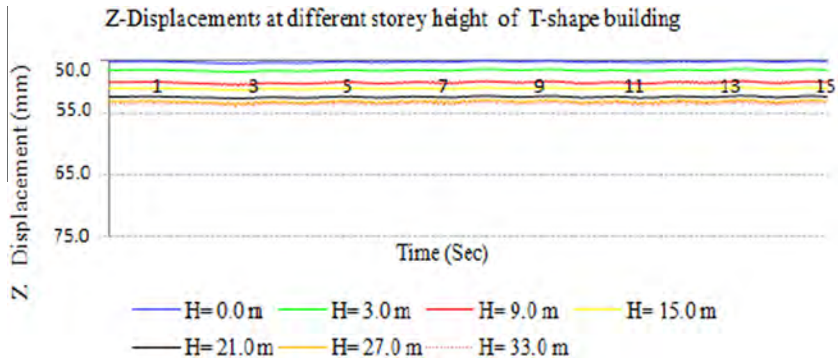

Fig. 17. *Z*-direction response of T shape G+10 building.

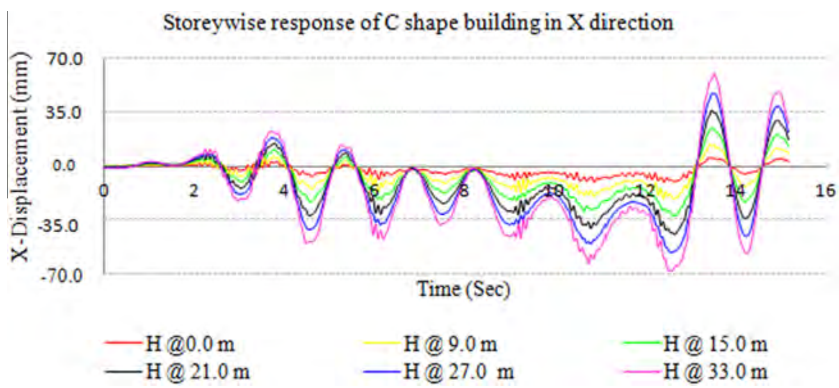

Fig. 18. The storey wise time history displacements in *X* direction for C-shape building.

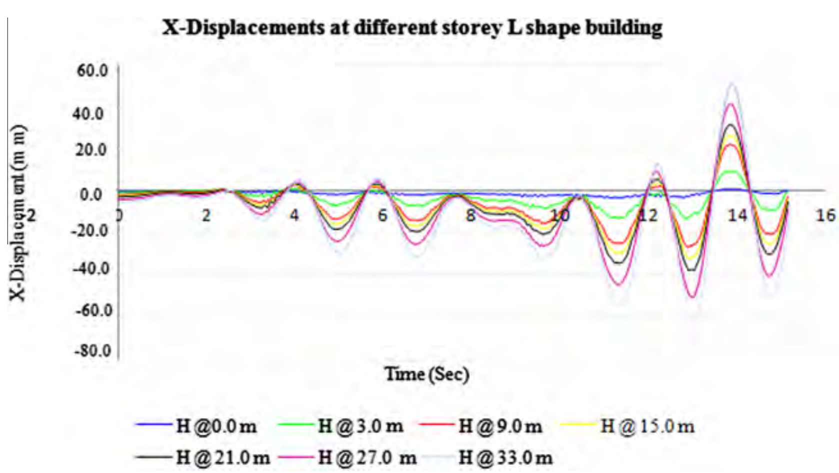

Fig. 19. The storey wise time history displacements in *X* direction for L-shape building.

8. Conclusions

In the present study, the dynamic soil structure interaction analysis for asymmetrical C, L and T shape G+10 pile supported building has been carried out to understand the behavior in terms of displacements at different storey heights. The soil structure interaction analysis is big size problem and computationally costlier. The finite element model of the integrated system, including superstructure, piled raft and soil has been developed using the Lagragian formulation by developing a self receptacle program in C language. The program includes the Drucker–Prager material model to include the material nonlinearity and node to node contacts to obtain the interaction between soil raft and soil pile. The G+10 storey C, L and T shape pile supported asymmetrical buildings have been analyzed during the considered ground motions. The effect different earthquake loading is studied in terms of superstructure responses.

The dynamic nonlinear analysis has been carried out for the April 25, Nepal earthquake and the results are compared with author's already carried out attempt, consist of the response of L, C and T type ground motions, including the 1995 Chamba, the 1999 Uttarkashi and the 2001 Bhuj. The behavior of the building has been studied under each earthquake event and comparative analysis has been discussed below in detail.

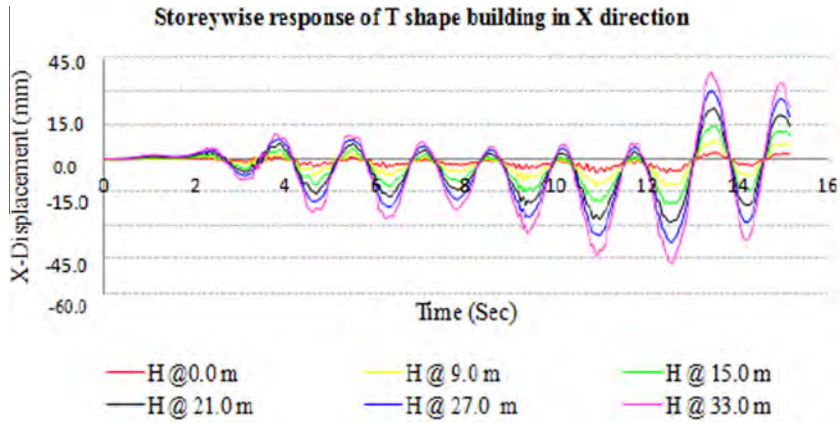

Fig. 20. The storey wise time history displacements in X direction for T-shape building.

Table 3
Peak response of superstructure system for different earthquakes.

Asymmetrical shape	Peak X-displacement (mm)			Peak Y-displacement (mm)			Peak Z-displacement (mm)		
	C shape	L shape	T shape	C shape	L shape	T shape	C shape	L shape	T shape
January 26, 2001 Bhuj (NE) (PGA = 0.31g)	68.39	62.9	46.5	49.5	49.1	45.1	92.1	88.7	69.82
April 25,2015 Nepal (NE) (PGA = 0.19g)	61.73	57.13	43.12	39.54	39.23	35.71	67.91	66.30	57.42
March 29, 1999 Uttarkashi (NW) (PGA = 0.25g)	63.93	59.35	45.76	42.56	41.98	40.10	62.50	67.28	62.55
March 24, 1995 Chamba (NE) (PGA = 0.13g)	25.53	18.13	15.83	22.5	16.41	13.12	36.70	26.06	22.75

Table 4
Percentage deviation in response w.r.t. 2001 Bhuj earthquake (PGA = 0.31g, M = 7.7).

Earthquake	C-shape building			L-shape building			T-shape building		
	X	Y	Z	X	Y	Z	X	Y	Z
April 25, 2015 Nepal (NE) (PGA = 0.19g, M = 7.8)	−9.74	−20.12	−20.10	−9.17	−20.10	−25.25	−7.27	−20.82	−17.76
March 29, 1999 Uttarkashi (NW) (PGA = 0.25g, M = 7.0)	−6.52	−14.02	−14.50	−5.64	−15.07	−24.14	−1.59	−11.09	−10.41
March 24, 1995 Chamba (NE) (PGA = 0.13g, M = 4.9)	−62.67	−54.55	−66.58	−71.17	−65.55	−70.62	−65.96	−70.91	−67.42

8.1. Effect of earthquake characteristics on the building responses

An earthquake is characterized by its magnitude and peak ground acceleration (PGA) value. The Magnitude indicates the amount of energy released at the source (or epicenter) and governs by the position of the fault and its characteristics. But peak ground acceleration depends on the local site characteristics, including dynamic soil properties, shear wave velocity, the arrangement of the soil strata and hydrostatic condition of the soil. Thus, PGA is more related to the actual earthquake force applied at the foundation level of the structure. Hence it is needed to correlate the PGA of different earthquake with the response of the structure. In this study, the peak response of the different asymmetrical structures like L, C and T have been observed for the Nepal earthquake (PGA = 0.19g) and compared with the responses studied for different earthquakes, including Bhuj (0.31g), Uttarkashi (0.25g) and Chamba earthquake (0.13g). It has been observed that the peak response shows the highest value in case of Bhuj ground motion than the Uttarkashi, Nepal and Chamba respectively. It has been noted that though the magnitude of the Nepal earthquake (7.8) is more than the Uttarkashi earthquake (7.0), the responses of C shape building, when it subjected to Nepal earthquake is found to be less by 20% (average) in all X, Y and Z direction. The reason behind this is studied as the peak ground acceleration (PGA) value is more for Uttarkashi earthquake (PGA = 0.25g) than Nepal Earthquake (PGA = 0.19g). The response of C shape superstructure during Nepal earthquake (PGA = 0.19g) is found to more by 60% (average) in all directions than the Chamba earthquake (PGA = 0.13g). The response of the C shape building when subjected to Bhuj Earthquake (PGA = 0.31g) is found to be 11% (average) more in all X, Y and Z directions than when it is subjected to Nepal earthquake (PGA = 0.19g).

The same observation has been made for L and T shape of buildings. Thus the Earthquake with greater peak ground acceleration (PGA) give more shaking to the superstructure. The PGA of the earthquake directly responsible for the response of the superstructure that the magnitude of the earthquake. The study concluded that the system response is more for the ground motion which carries, the more acceleration on the ground. The acceleration reaches to the ground mainly depends upon the local site conditions. Thus PGA and soil condition governs the dynamic response of the structure. Thus the peak ground acceleration of the earthquake plays very crucial role in the kinematic interaction in SSI analysis.

8.2. Effect of geometry of superstructure

The dynamic nonlinear analysis has been carried out for the 2015 Nepal ground motions for C, L and T shape of the superstructure and the building responses are compared with the 1995 Chamba (Mw = 4.9), the 1999 Uttarkashi (Mw = 7.0) and the 2001 Bhuj (Mw = 7.7) earthquake condition. The response of the superstructure founded on the homogenous soil condition has been studied for each ground motion.

In case of C building the response in the superstructure is found to be an average 30% more than L shape building and average 37% more than the T shape building under all earthquake loading conditions. It has been noted that the C shape building is found to be more critical as it experiences the highest displacement than T and L shape. The displacement demands are ranges increasingly from C, L and T. It has been noted that the C shape building is more complex as compared to L and T for the same plan area. Thus more the complex building, increases its chances of failure under earthquake. The same observation is made for all earthquake applications including 2015 Nepal, 1995 Chamba, the 1999 Uttarkashi and the 2001 Bhuj. Thus study concludes that the higher the degree of asymmetry of the superstructure, increases the chances of its failure under earthquake scenario.

In this study the dynamic nonlinear soil structure interaction analysis for asymmetrical building supported on piled raft foundation is analyzed. As the asymmetrical structure is one of the most unavoidable constructions in civil engineering practice and found to be most vulnerable under seismic event, it is really needed to adopt the preciseness in such analysis. In the soil structure interaction analysis, the system can be modeled nearest to the reality as the effect of supporting soil has been taken into consideration. In this attempt the three complex asymmetrical buildings consist of C, T and L are analyzed under the series of earthquakes to understand the vulnerability associated with the geometry of the building. It is concluded that the more the complex building shows high risk during an earthquake event and responses of the building governs by the peak ground acceleration of the particular earthquake rather than its magnitude.

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