Seismic Analysis of Vertical Geometric Irregular Building considering Soil Structure Interaction

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Abstract. Prevailing studies consider the design of reinforced concrete moment resisting frame having fixed base. When structure is subjected to earthquake, foundation undergoes three modes of deformation namely vertical, sliding and rocking. These deformations increase the force demand due to uncertainties in the characterization of soil. In the present study, a numerical modelling is carried out using the finite element software OpenSEES to evaluate the seismic response of a typical eight storey four-bay vertical geometric irregular building considering soil-structure interaction and the results are compared with a reference regular building. The numerical modelling is carried out considering the footings as beams on nonlinear Winkler foundation in which footing is modelled as elastic elements and the soil is modelled as discrete nonlinear springs in both vertical and horizontal direction. Fibre-based nonlinear element is used for modelling the reinforced concrete beams and columns. A suite of five earthquake ground motion records of different magnitude is considered for the dynamic analyses. Four types of soil-structure interactions are considered: Fixed Regular, Fixed VGI, Flexible Regular, and Flexible VGI. The effect of soil-structure interaction on the seismic response of framed building is demonstrated in terms of probabilistic seismic demand models and fragility curves.

Keywords: Numerical Modelling, Vertical Geometric Irregular, Soil Structure Interaction, Peak Ground Acceleration, Fragility Curve.

1 Introduction

Most of the design of structures consider its support as fixed against settlement, sliding and rotation. When a structure is subjected to earthquakes, inertial forces develop which introduce base shear and moment at the foundation. These base shear and moments will introduce foundation rotation and displacement unless foundation system and supporting soil are rigid. These rotation and displacement result in mischaracterization of dynamic properties of soil. The phenomenon in which motion of structure influence the response of soil and vice versa is known as soil-structure interaction.

A series of research outcomes have been reported in studies on seismic soil-structure interaction showing the behavior of the foundation subjected to earthquake [1-3]. Also, various studies were available showing the response of tall buildings when subjected to earthquake considering soil-structure interaction [4-8] and compared with the response of the buildings having fixed base. Stewart et al. [9] developed consensus guidance for implementing soil-structure interaction in response history analyses. They also provided a body of soil-structure interaction concept in a synthesis manner, distilled into a brief narrative and synchronized under a consistent set of units and variables. Anand and Kumar [10] summarized various approaches to include soil-structure interaction in the analysis of structures and outlined the guidelines in prominent seismic codes. They provided a clear understanding of soil-structure interaction phenomenon by tracking the historical development of the field and to understand the significance of soil-structure interaction in design practice.

Most of the studies consider only regular frames having a uniform distribution of mass, strength, stiffness and structural form. A few studies have considered irregular frames. One of the irregularities is the vertical geometric irregularity where the lateral force-resisting system in any storey is more than 150 percent of its adjacent storey and a very few studies have considered the seismic risk of these frames in terms of Probabilistic Seismic Demand Model (PSDM) Analysis and Fragility Curves with only having fixed base [11-13].

The present study, is an approach to investigate the role of soil beneath the superstructure, by analyzing a vertical geometric irregular frame considering soil-structure interaction subjected to a suite of ground motion using finite element software OpenSEES [14] and results were compared with a reference regular building having fixed base.

2 Computational Model

2.1 Modelling of Superstructure

Two types of frames have been used in the present study i.e., Regular frame and Vertical Geometric Irregular frame. Both the frames have eight storeys with a uniform storey height of 3.2 m and four bays in the direction of loading with a uniform bay width of 5 m (Fig. 1 and Fig. 2).

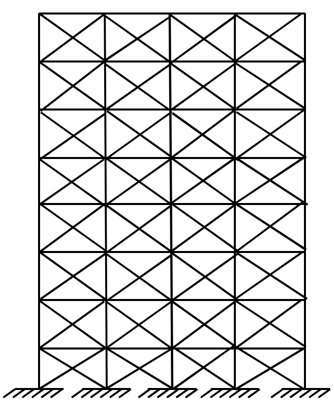

Fig. 1. Regular Building (2D Frame)

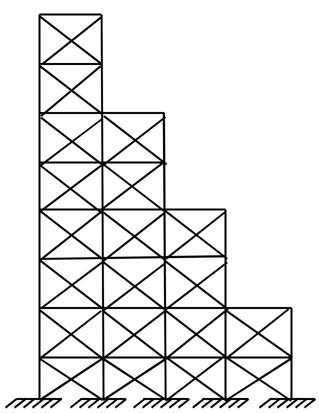

Fig. 2. Vertical Geometric Irregular Building (2D Frame)

A two-dimensional plane frame having 3 degrees of freedom per node in the direction of loading is considered. The selected frames have a beam c/s of 300 mm x 400 mm and a column c/s of 450 mm x 450 mm. A masonry infill wall of 230 mm thickness is considered uniformly in all storeys. The beams and columns of the superstructure have been modeled as force-based nonlinear beam-column element that considers spread of plasticity along the element. In a single element, five gauss integration points have been used. For modelling of infill wall, pin connected equivalent truss elements in both diagonals of each bay are considered. The concrete is modeled using Kent and Park [15] constitutive model having degraded linear unloading/reloading stiffness and linear tension softening whereas the steel rebars are modeled using uniaxial Menegotto and Pinto [16] steel material model having isotropic strain hardening properties. Mass of the entire structure is lumped at the nodes connecting the elements of beams and columns. The total mass considers entire dead load and 25 % of design live load in between floors and no loads on the terrace according to Indian Standard IS 1893 [17].

2.2 Modelling of Foundation

Five numbers of isolated square footing were considered and the mid node of the footing is connected to the support nodes of the superstructure and load is transmitted from superstructure to soil. Beams on Non-Linear Winkler Foundations (BNWF) model approach is implemented for modelling the entire foundation. The footing is modelled as elastic elements and the soil below is modelled as independent zero-length springs (Fig. 3). Various input parameters are used in a BNWF model such as type of soil (sand or clay), ultimate bearing capacity of the soil, Young's modulus (E), Poisson's ratio (¬), magnitude and distribution of vertical and lateral stiffness, spring spacing, radiation damping, tension capacity, etc. One dimensional zero-length springs having uniaxial material properties are used to simulate the load-settlement behaviour (q-z), horizontal passive load-displacement behaviour due to embedment along the side of the footing (p-x) and horizontal shear sliding behaviour due to base friction at the base of the footing (t-x) (Fig. 4).

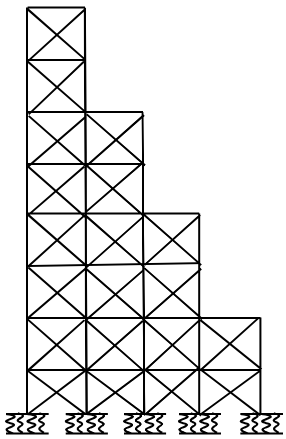

Fig. 3. Vertical Geometric Irregular Building considering Soil Structure Interaction

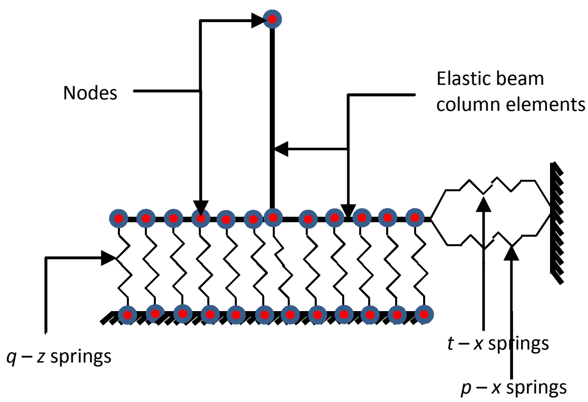

Fig. 4. BNWF Model (adapted after Raychowdhury, 2008)

The global stiffness of the springs (K_i) is presented by Gazetas [18] which depends upon dimensions of footing, Shear modulus (G) of soil and Poisson's ratio (¬) of soil. Eq. 1 and Eq. 2 represents surface stiffness for the vertical and horizontal direction.

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

$$K_x = \frac{GL}{2-\neg}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]+\frac{GL}{0.75-\neg}\left[0.1\left(1-\frac{B}{L}\right)\right] \quad (2)$$

where L and B represent length and width of the footing.

The ultimate load-carrying capacity of soil (q_{ult}) assigned to QzSimple2 material is calculated according to Meyerhof [19] whereas passive resistance (p_{ult}) assigned to PySimple1 material is obtained from Rankine [20]. The shear sliding capacity assigned to the TzSimple1 material represents the frictional resistance which is a function of angle of internal friction (¤) and shear strength of soil.

2.3 Selection and Application of Ground Motion Records

In the present study, five pairs of far-field natural ground motion records (Table 1) are collected from Haselton et al. [21]. The direction I and II in Table 1 represent the earthquake in both horizontal directions. By the use of a wavelet-based computer program WaveGen, developed by Mukherjee and Gupta [22], these records are scaled to match the design spectrum of IS 1893 [17]. In this program, a recorded accelerogram is disintegrated into a finite number of time histories having energy in non-intersecting frequency bands and these time histories are iteratively scaled up/down so that time history assembled gets compatible with a specified design spectrum. Selection of these five pairs is based on similarity of different conditions: (i) magnitude (M_w) > 6.5 in Richter scale, (ii) distance from site to source (R) > 10 km, (iii) peak ground velocity (PGV) > 15 cm/sec, (iv) peak ground acceleration (PGA) > 0.2g, (v) shear wave velocity (V_s) > 180 m/sec and (vi) lowest usable frequency (f) < 0.25 Hz. PGV is the Peak Ground Velocity which is the greatest shaking at a point during the event of an earthquake. Shear waves are the transverse waves in a medium. The velocity of shear wave is square root of shear modulus divided by square root of mass density of soil.

Table 1. Selected far-field ground motion records obtained from Haselton et al. (2012)

Sl.No.	Event	Magnitude	PGA, *g* Direction		Epicentral Distance (km)	Source (Fault Type)	Recording Station
			I	II			
1	Friuli, Italy 1976	6.5	0.35	0.31	20.2	Thrust	Tolmezzo
2	Northridge 1994	6.7	0.42	0.52	13.3	Thrust	Beverly Hills

3	Cape Mendocino 1992	7	0.39	0.55	22.7	Thrust	CHY101
4	Landers 1992	7.3	0.28	0.42	82.1	Strike - slip	Coolwater
5	Chi-Chi, Taiwan 1999	7.6	0.35	0.44	32	Thrust	CHY101

For nonlinear time history analysis, the Newmark Integrator method which considers average acceleration in one-time step of analysis was used having parameters and ¥ as 0.25 and 0.5 respectively. Detailed explanation of this integration method is already established which can be found in Newmark [23]. In time history analysis, a Rayleigh damping was used in which the damping matrix was calculated as a linear combination of the stiffness matrix scaled second coefficient. These coefficients are computed at two different periods by specifying equivalent fractions of critical modal damping. A displacement control analysis was used in which an incremental displacement was specified at each new step, and the integrator determines the associated load increment through the equation of equilibrium. Convergence tolerance of 10^{-8} and 10 iterations were used in the present study.

3 Results and Discussions

3.1 Validation

The selected approach of modelling and analysis is validated using PLAXIS 2D [24]. A static load equal to the ultimate load of foundation (Q_u) was applied to a foundation resting on dense sand and a graph is shown in between the load per unit area, q (kPa) vs normalized settlement, s/B (%). The results obtained by OpenSEES were compared with those obtained by PLAXIS 2D for the same boundary conditions. The OpenSEES results are obtained from the simulation of a numerical code based on TCL and C++ which is different from the result with which it is compared in terms of both algorithm (BNWF model) and working principle (stiffness distribution). Table 2 shows the material properties used in present study. Fig. 5 shows that both the methods have good agreement, which justifies that, OpenSEES has the ability to reasonably capture the behaviour of foundation.

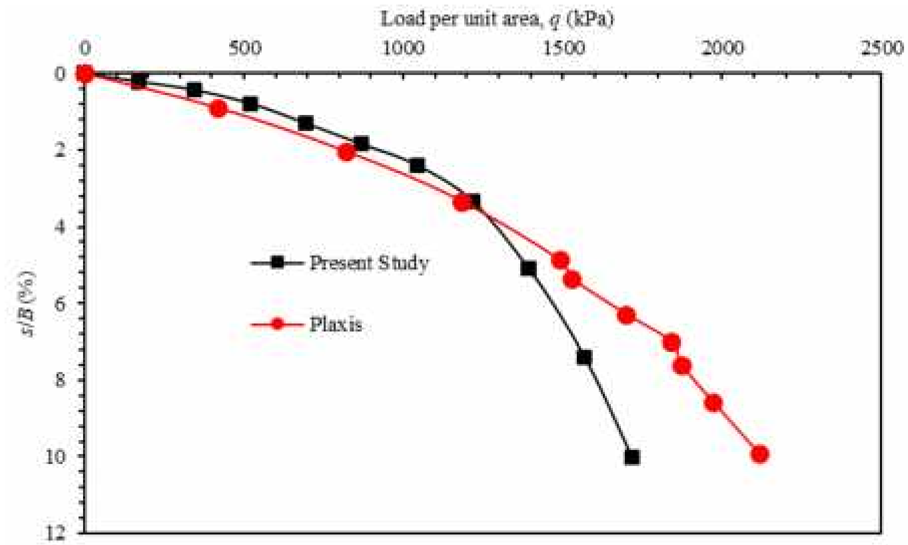

Fig. 5. Variation of Static Load Intensity (q) with normalized settlement [(s/B) %] for D_f/B = 0

Table 2. Material properties used in present study

Property	Soil	Footing
	Dense Sand	Concrete
Material Type	Uniaxial Material	Elastic
Element Type	Zero – Length Elements	Elastic Beam Column
Model Dimension (L x B x H) (m)	-	1.7 x 1.7 x 0.6
γ (kN/m^3)	18	25
Effective angle of internal friction, ϕ' (^{o})	41	-
Modulus of Elasticity, E (kPa)	62495	25000
Poisson's Ratio, μ	0.32	0.15

3.2 Probabilistic Seismic Demand Model (PSDM) Analysis

It represents the relationship between Engineering Demand Parameter (ISD_{max}) and Intensity Measure (PGA) of selected ground motion. The probabilistic representation of demand parameter can be obtained by performing a series of nonlinear time history analysis of selected building subjected to a suite of ground motion of various intensity measure. An analytical approximation of this representation is considered as per Cornell et al. [25] that says given the level of *PGA*, the predicted median drift demand (ISD_{max}) can be approximately represented by the form (see Eq. 3)

$$ISD_{\max} = a(PGA)^b \tag{3}$$

where, ISD_{max} is the maximum inter storey drift ratio (%), *PGA* is the peak ground acceleration (g), *a* and *b* are constant coefficients.

The drift demands are assumed to be distributed log normally [26] about the median, ISD_{max} with a standard deviation, $_{(D|PGA)}$ (dispersion in PGA, considering natural logarithm at a given *PGA* level) (see Eq. 4)

$$_{(D|PGA)} = \sqrt{\frac{\sum\left(\ln(ISD_{\max}) - \ln\left(a(PGA)^b\right)\right)^2}{N-2}} \tag{4}$$

where, $_{(D|PGA)}$ = dispersion in peak ground acceleration and *N* = number of ground motions

The three parameters *a*, *b* and $_{(D|PGA)}$ can be obtained by performing a number of nonlinear analysis and then plotting an exponential trend line of $ln(ISD_{max})$ on $ln(PGA)$ (Table 3). Fig. 6 shows PSDM analysis for both regular and VGI frames resting on dense sand and results were compared with structures having a fixed base.

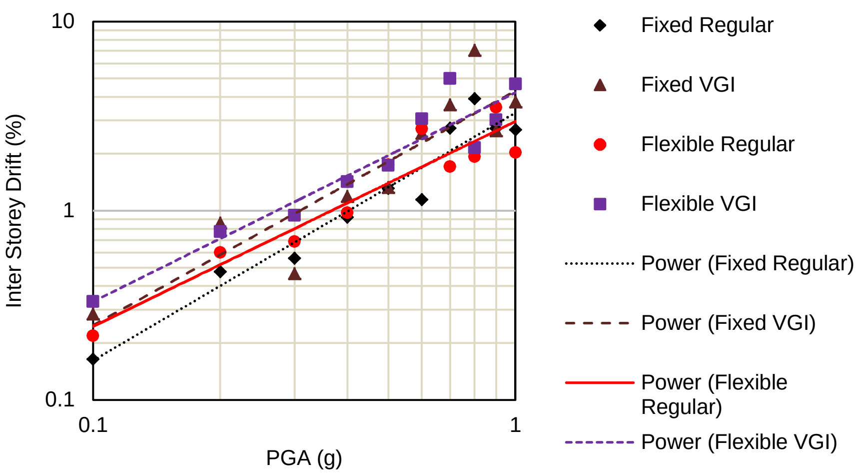

Fig. 6. Plot of PGA (g) vs inter storey drift (%) for different support conditions

Table 3. PSDMs for different frames

Frame	$a(PGA)^b$	R^2	$_{(D\|PGA)}$
Fixed Regular	$3.296(PGA)^{1.309}$	0.9365	0.265
Flexible Regular	$2.967(PGA)^{1.084}$	0.901	0.279
Fixed VGI	$4.302(PGA)^{1.240}$	0.8183	0.454
Flexible VGI	$4.212(PGA)^{1.104}$	0.8979	0.289

From Fig. 6, it was observed that vertical geometric irregular buildings considering soil-structure interaction shows highest inter storey drift (%) for all values of PGA whereas, regular buildings having fixed support shows lowest inter storey drift (%) up to 0.6g PGA and regular buildings considering soil-structure interaction shows lowest inter storey drift (%) above 0.6g PGA.

3.3 Fragility Curves

A fragility function represents the probability of exceedance of the seismic drift demand under a specific PGA beyond a selected performance level. The fragility curve presents a cumulative probability distribution as a function of PGA, which indicates that probability of a building will be damaged to a given state or more a severe one. Three different types of performance levels have been selected i.e., Collapse Prevention (CP), Significant Damage (SD) and Damage Limitation (DL). Figs. 7-9 show fragility curves for selected frames for different performance levels. Fragility curves can be obtained separately for each damage state and can be expressed in closed form as follows (see Eqs. 5-7).

$$P(D \mid PGA) = \Phi\left(\frac{\ln(PGA) - \ln(IM_m)}{{}_{comp}}\right) \tag{5}$$

$$IM_m = \exp\left(\frac{\ln C - \ln a}{b}\right) \tag{6}$$

$${}_{comp} = \frac{\sqrt{{}^{2}_{D|PGA} + {}^{2}_{c}}}{b} \tag{7}$$

where, $P(D|PGA)$ = Probability of Exceedance, IM_m = Median of drift at chosen performance level c = Normal Distribution, C = Median ISD capacity and ${}_{c}$ = Dispersion in material = 0.25.

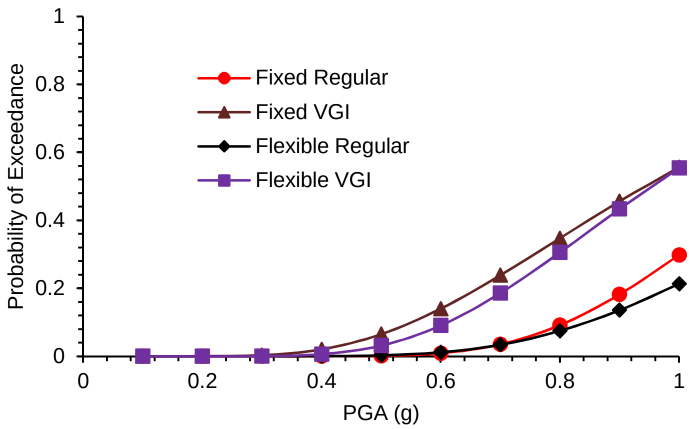

Fig. 7. Plot of PGA (g) vs probability of exceedance considering Collapse Prevention (CP) performance level for different support conditions

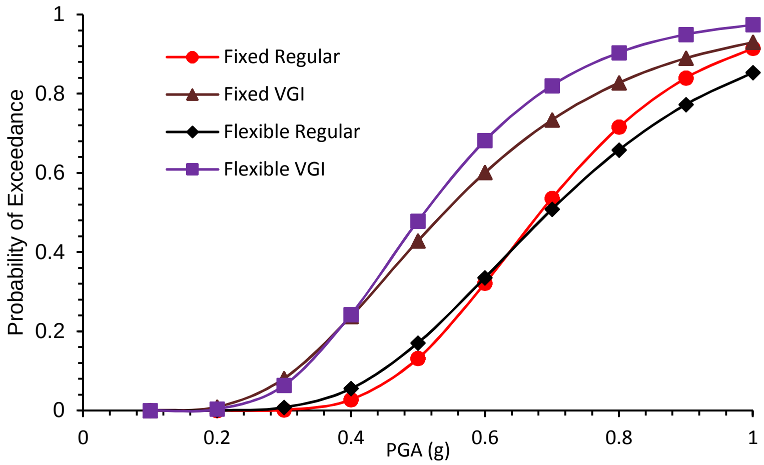

Fig. 8. Plot of PGA (*g*) vs probability of exceedance considering Significant Damage (SD) performance level for different support conditions

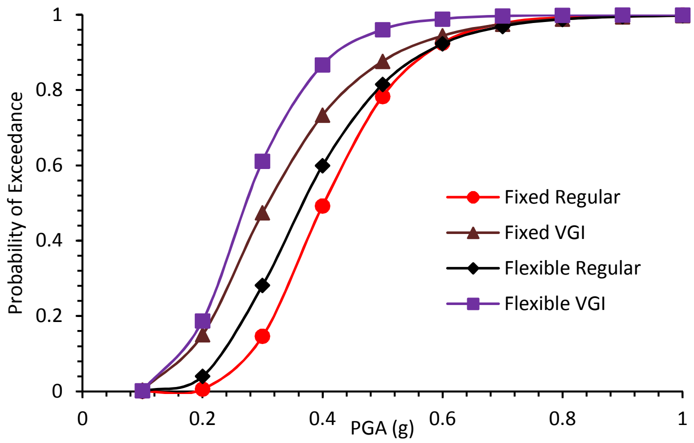

Fig. 9. Plot of PGA (*g*) vs probability of exceedance considering Damage Limitation (DL) performance level for different support conditions

From Fig. 7-9 it was observed that vertical geometric irregular buildings considering soil-structure interaction shows the highest probability of exceedance for all values of PGA for all performance levels whereas, regular buildings having fixed support shows lowest probability of exceedance up to 0.6g PGA and regular buildings considering soil-structure interaction shows lowest probability of exceedance above 0.6g PGA for all performance levels.

4 Conclusions

In this present study, the seismic performances of vertical geometric irregular buildings considering soil foundation superstructure interaction have been investigated. The present study represents seismic risk of regular and vertical geometric irregular building resting on dense sand in terms of PSDM analysis and fragility curves and results were compared with structures having fixed base. Based on the analysis of results, the probability of failure of structure was suggested in following points:

- Structures considering soil-structure interaction have a higher probability of failure than structures without considering soil-structure interaction.
- Vertical geometric irregular buildings have a higher probability of failure than regular structures.
- If the soil is stiff enough, the probability of failure of regular structures with and without considering soil-structure interaction shows almost similar trends whereas, a significant difference was observed in case of vertical geometric irregular structures, which shows the importance of irregularity of structures as well as soil-structure interaction.

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