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Earthquake Resistant Design of Low-Rise Open Ground Storey Framed Building

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Abstract: *Presence of infill walls in the frames alters the behavior of the building under lateral loads. However, it is common industry practice to ignore the stiffness of infill wall for analysis of framed building. Engineers believe that analysis without considering infill stiffness leads to a conservative design. An existing RC framed building (G+3) with open ground storey located in Seismic Zone-V is considered for this study. This building is analyzed for two different cases: (a) considering both infill mass and infill stiffness and (b) considering infill mass but without considering infill stiffness. Two separate models were generated using commercial software SAP2000. Infill weights were modelled through applying static dead load and corresponding masses considered from this dead load for dynamic analyses. Infill stiffness was modelled using a diagonal strut approach. Two different support conditions, namely fixed end support condition and pinned end support condition, are considered to check the effect of support conditions in the multiplication factors. Linear and non-linear analyses were carried out for the models and the results were compared.*

Keywords: *infill walls, diagonal strut, open ground storey, equivalent static analysis, response spectrum analysis, pushover analysis, low rise building .*

I. Introduction

Due to increasing population since the past few years car parking space for residential apartments in populated cities is a matter of major concern. Hence the trend has been to utilize the ground storey of the building itself for parking. These types of buildings having no infill masonry walls in ground storey, but infilled in all upper storeys, are called Open Ground Storey (OGS) buildings. They are also known as ‘open first storey building’ (when the storey numbering starts with one from the ground storey itself), ‘pilotis’, or ‘stilted buildings’. There is significant advantage of these category of buildings functionally but from a seismic performance point of view such buildings are considered to have increased vulnerability. From the past earthquakes it was evident that the major type of failure that 2 occurred in OGS buildings included snapping of lateral ties, crushing of core concrete, buckling of longitudinal reinforcement bars etc. Due to the presence of infill walls in the entire upper storey except for the ground storey makes the upper storeys much stiffer than the open ground storey. Thus, the upper storeys move almost together as a single block, and most of the horizontal displacement of the building occurs in the soft ground storey itself. In other words, this type of buildings sway back and forth like inverted pendulum during earthquake shaking, and hence the columns in the ground storey columns and beams are heavily stressed. Therefore it is required that the ground storey columns must have sufficient strength and adequate ductility. The vulnerability of this type of building is attributed to the sudden lowering of lateral stiffness and strength in ground storey, compared to upper storeys with infill walls.

II. Objective Of The Study

The salient objectives of the present study have been identified as follows:

i) To study the effect of infill strength and stiffness in the seismic analysis of OGS buildings.

ii) To check the applicability of the multiplication factor of 2.5 as given in the Indian Standard IS 1893:2002 for design of low rise open ground storey building.

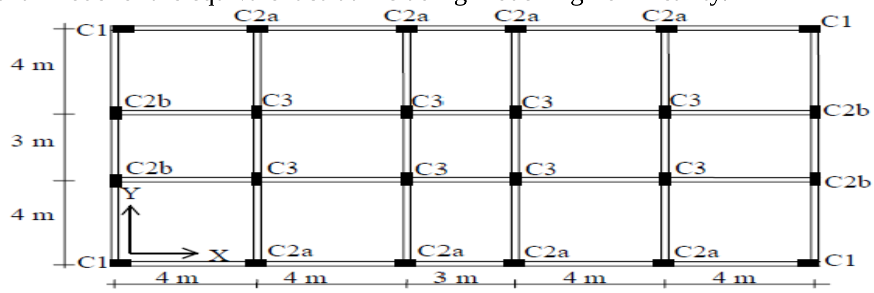
iii) To assess the effect of support condition on the seismic behaviour of OGS buildings.

III. Modeling

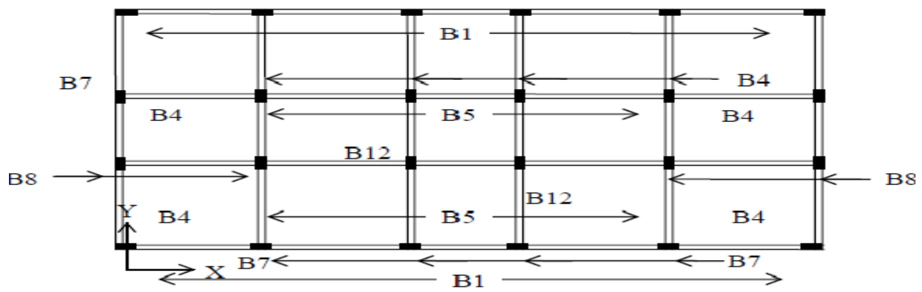
The methodology worked out to achieve the above-mentioned objectives is as follows:
(i) Review the existing literature and Indian design code provision for designing the OGS building
(ii) Select an existing building model for the case study.
(iii)Model the selected building with and without considering infill strength/ stiffness. Models need to consider two types of end support conditions as mentioned above.
(iv) Linear analysis of the selected building model and a comparative study on the results obtained from the analyses.
(v) Nonlinear analysis of the selected building model and a comparative study on the results obtained from the analyses.
(vi) Observations of results and discussions.

Structural Modeling
3.1 Overview

It is very important to develop a computational model on which linear / non-linear, static/ dynamic analysis is performed. The first part of this chapter presents a summary of various parameters defining the computational models, the basic assumptions and the geometry of the selected building considered for this study. Accurate modelling of the nonlinear properties of various structural elements is very important in nonlinear analysis. In the present study, frame elements were modelled with inelastic flexural hinges using point plastic model. A detailed description on the nonlinear modelling of RC frames is presented in this chapter. Infill walls are modelled as equivalent diagonal strut elements. The last part of the chapter deals with the computational model of the equivalent strut including modelling nonlinearity.



(a) Column Locations



(b) Beam Locations

Fig. 3.1: Typical floor plan of the selected building

3.2 Building Description

An existing OGS framed building located at Guwahati, India (Seismic Zone V) is selected for the present study. The building is fairly symmetric in plan and in elevation. This building is a G+3 storey building (12m high) and is made of Reinforced Concrete (RC) Ordinary Moment Resisting Frames (OMRF). The concrete slab is 150mm thick at each floor level. The brick wall thicknesses are 230 mm for external walls and 120 mm 24 for internal walls. Imposed load is taken as 2 kN/ m2 for all floors. Fig. 3.1 presents typical floor plans showing different column and beam locations. The cross sections of the structural members (columns and beams 300 mm×600 mm) are equal in all frames and all stories. Storey masses to 295 and 237 tonnes in the bottom storyes and at the roof level, respectively. The design base shear was equal to 0.15 times the total weight.

The amount of longitudinal reinforcement in the columns and beams is given in Table 3.2. Although the columns have equal reinforcement in all storey levels beam reinforcement in floor and roof are different. Refer Fig. 3.1 (a) and (b) for column and beam identification (ID).

3.3 Structural Modelling

Modelling a building involves the modelling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and structural elements used in the present study is discussed below.

3.3.1 Material Properties

M-20 grade of concrete and Fe-415 grade of reinforcing steel are used for all the frame models used in this study. Elastic material properties of these materials are taken as per Indian Standard IS 456: 2000. The short-term modulus of elasticity (E_c) of concrete is taken as:

$Ec = 5000\sqrt{f ck} M$

f_{ck} is the characteristic compressive strength of concrete cube in MPa at 28-day (20 MPa in this case). For the steel rebar, yield stress (f_y) and modulus of elasticity (E_s) is taken as per IS 456:2000. The material chosen for the infill walls was masonry whose compressive strength (f_m') from the literature was found out to be 1.5 MPa and the modulus of elasticity was stated as:

E_m = 350 to 800 MPa for table moulded brick

= 2500 to 5000 MPa for wire cut brick

According to FEMA 356:2000 elasticity of modulus of brick is taken as $E_m = 750 f_m'$. For the present study the modulus of elasticity of the masonry is taken as given in literature by Asokan (2006).

3.3.2 Structural Elements

Beams and columns are modelled by 3D frame elements. The beam-column joints are modelled by giving end-offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The beam-column joints are assumed to be rigid. Beams and columns in the present study were modelled as frame elements with the centrelines joined at nodes using commercial software SAP2000NL. The rigid beam-column joints were modelled by using end offsets at the joints (Fig. 3.2). The floor slabs were assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams.

The structural effect of slabs due to their in-plane stiffness is taken into account by assigning 'diaphragm' action at each floor level. The mass/weight contribution of slab is modelled separately on the supporting beams.

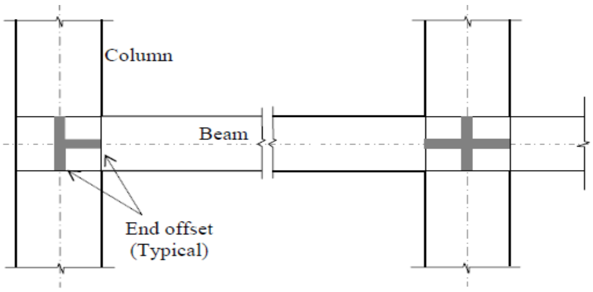


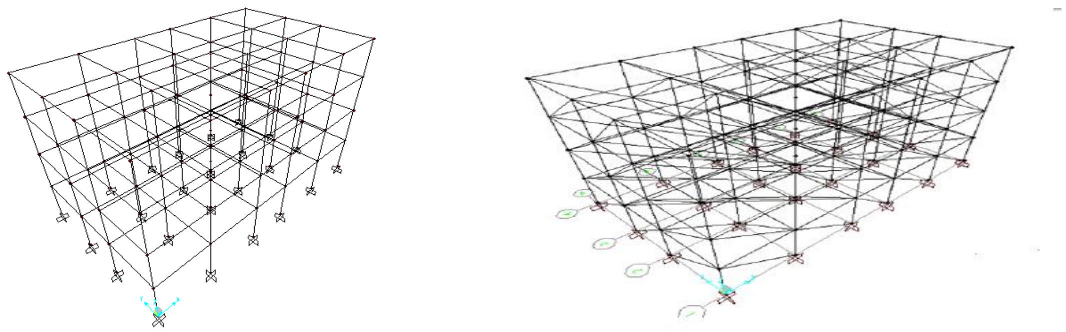
Fig. 3.2: Use of end offsets at beam-column joint

3.3.3 Modelling of Column Ends at the Foundation

The selected building is supported on a raft foundation. Therefore, the column ends are modelled as fixed at the top of the raft and analysed. To study how the response of the building changes with the support conditions, the same building model also analysed by providing a hinge in place of fixity.

3.3.4 Modelling Infill Walls

Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of buildings with infill wall. But the nonlinear modelling of a two dimensional plate element is not understood well. Therefore infill wall has to be modelled with a one-dimensional line element for nonlinear analysis of the buildings. Same building model with infill walls modelled as one-dimensional line element is used in the present study for both linear and nonlinear analyses. Infill walls are modelled here as equivalent diagonal strut elements. Section 3.5 explains the modelling of infill was as diagonal strut in detail. Fig. 3.3 presents a three-dimensional computer model of building without and with considering infill stiffness.



(a)Without infill (b) With infill

Fig. 3.3: 3D Computer model of building without and with considering infill stiffness respectively.

IV. Result & Discussions

As mentioned earlier the selected OGS building is analyzed for following two different cases and two end support conditions (fixed and pinned end support) (a) Considering infill strength and stiffness (with infill/infilled frame) (b) Without considering infill strength and stiffness (without infill/bare frame). Therefore there are a total of four building models: (a) building modelled without infill and fixed end support, (b) building modelled with infill and fixed end support, (c) building modelled without infill and pinned end support and (d) building modelled with infill and pinned end support. Equivalent static and response spectrum analyses of these four building models are carried out to evaluate the effect of infill on the seismic behaviour of OGS building for two different support conditions. Following sections presents the results obtained from these analyses.

4.1 Calculation of Time Period and Base Shear

The design base shear (V_B) was calculated as per IS 1893: 2002 corresponding to the fundamental period for moment-resisting framed buildings with brick infill panels as follows

$VB=AhW$

$Ah=Z/2.I/R.Sa/g$

where $W \equiv$ seismic weight of the building, $Z \equiv$ zone factor, $I \equiv$ importance factor, $R \equiv$ response reduction factor, $S_a/g \equiv$ spectral acceleration coefficient corresponding to an approximate time period (T_a) which is given by: $Ta=0.09h/\sqrt{d}$ for RC frame with masonry infill

	With infill		Without infill	
	V_x (kN)	V_y (kN)	V_x (kN)	V_y (kN)
Equivalent Static $(\overline{V_B})$	1566	1566	1566	1566
Response Spectra (V_B)	1427	1427	1300	1310
$\overline{V_B}/V_B$	1.10	1.10	1.20	1.19

Table 4.1: Comparison of fundamental time periods for with and without infill for pinned and fixed end support condition

4.2 Shift in Period

Fixed End	Empirical formula		Computational Value	
	With infill	Without infill	With infill	Without infill
Tx (s)	0.28	0.47	0.28	0.47
Ty (s)	0.33	0.47	0.33	0.47
(Sa/g)x	2.50	2.50	2.50	2.50
(Sa/g)y	2.50	2.50	2.50	2.50

Table 4.2 (a): Shift in period for fixed end support condition

Pinned End	Empirical formula		Computational Value	
	With infill	Without infill	With infill	Without infill
Tx (s)	0.28	0.47	0.52	0.61
Ty (s)	0.33	0.47	0.52	0.60
(Sa/g)x	2.50	2.50	2.50	2.23
(Sa/g)y	2.50	2.50	2.50	2.28

Table 4.2 (b): Shift in period for pinned end support condition.

From Table 4.2(a) and 4.2(b) we can see that there is not much considerable difference in the time periods of the building irrespective of the directions considered according to the empirical formula. From the computational value we can see that there is a considerable shift of period for buildings modelled with fixed end support conditions. But the period shift is found to be very little in case of buildings modelled with pinned end support conditions Hence it can be said that the IS 1893:2002 (Part-1) does not take into account the support conditions for the calculation of fundamental period. It always gives a lower bound solution to be conservative for force calculation.

4.3 Column Interaction Ratios

Table 4.3(a): Comparison of Ground Storey Column Interaction Ratio for Pinned End Case

Col. ID	IR (ESA)		Ratio of IR	IR (RSA)		Ratio of IR
	WI	WOI		WI	WOI	
C1	1.13	1.53	0.74	2.05	1.78	1.15
C2a	1.94	1.93	1.01	2.45	2.24	1.09
C2b	1.84	1.82	1.01	2.49	2.09	1.19
C3	1.84	1.91	0.96	3.9	3.34	1.17

Table 4.3(b): Comparison of Ground Storey Column Interaction Ratio for Fixed End Case

Col. ID	IR (ESA)		Ratio of IR	IR (RSA)		Ratio of IR
	WI	WOI		WI	WOI	
C1	0.89	0.93	0.96	1.24	1.19	1.04
C2a	1.04	1.13	0.92	1.26	1.23	1.02
C2b	1.01	1.07	0.94	1.24	1.93	1.04
C3	1.41	1.52	0.82	2.04	1.93	1.06

This table clearly shows that for a low rise OGS building model with fixed-end support the ground storey column forces actually reduced when infill stiffness is considered in Equivalent Static Analysis. It marginally increases (less than 10%) in the case of response spectrum analysis. This is because the forces applied to building model with infill stiffness is little more compared to that applied to building model without infill stiffness in Response Spectrum Analysis. But the applied forces to these two buildings are same in case of Equivalent Static Analyses. Therefore using a multiplication factor of 2.5 for ground floor columns of low rise OGS buildings as per Indian Standard IS 1893:2002 (Part-1) is not justified.

4.4 Beam Demand-to-Capacity Ratios

Table 4.4 (a): Comparison of Beam DCR (Pinned-End)

Beam ID	DCR (ESA)		Ratio of DCR	DCR (RSA)		Ratio of DCR
	WI	WOI		WI	WOI	
B1	1.88	2.81	0.67	1.32	1.65	0.80
B4	1.84	2.63	0.70	1.15	1.45	0.80
B5	1.03	1.53	0.67	0.62	0.81	0.77
B7	1.33	1.93	0.69	0.86	1.09	0.79
B8	1.77	2.52	0.70	1.24	1.52	0.82
Average			0.69			0.80
Standard Deviation			0.01			0.02

Table 4.4 (b): Comparison of Beam DCR (Fixed-End)

Beam ID	DCR (ESA)		Ratio of DCR	DCR (RSA)		Ratio of DCR
	WI	WOI		WI	WOI	
B1	1.04	1.74	0.60	0.65	0.95	0.68
B4	1.16	1.78	0.65	0.60	0.88	0.68
B5	0.76	1.08	0.70	0.34	0.52	0.65
B7	0.82	1.29	0.64	0.46	0.66	0.70
B8	1.01	1.59	0.64	0.64	0.89	0.72
Average			0.65			0.69
Standard Deviation			0.04			0.03

Table 4.4 presents results from both equivalent static analyses (ESA) and response spectrum analyses (RSA). The table presented above shows that the conclusion drawn for the columns hold good for beams also. Force demands in all first floor beams are found to be lower when infill stiffness modelled in OGS building. It can be concluded from this results that it is conservative to analyse low-rise OGS building without considering infill stiffness.

V. CONCLUSION

Followings are the salient conclusions obtained from the present study:

i) IS code gives a value of 2.5 to be multiplied to the ground storey beam and column forces when a building has to be designed as open ground storey building or stilt building. The ratio of IR values for columns and DCR values of beams for both the support conditions and building models were found out using ESA and RSA and both the analyses supports that a factor of 2.5 is too high to be multiplied to the beam and column forces of the ground storey. This is particularly true for low-rise OGS buildings.

ii) Problem of OGS buildings cannot be identified properly through elastic analysis as the stiffness of OGS building and Bare-frame building are almost same.

iii) Nonlinear analysis reveals that OGS building fails through a ground storey mechanism at a comparatively low base shear and displacement. And the mode of failure is found to be brittle.

iv) Both elastic and inelastic analyses show that the beams forces at the ground storey reduce drastically for the presence of infill stiffness in the adjacent storey. And design force amplification factor need not be applied to ground storey beams.

v) The linear (static/dynamic) analyses show that Column forces at the ground storey increases for the presence of infill wall in the upper storeys. But design force amplification factor found to be much lesser than 2.5.

vi) From the literature available it was found that the support condition for the buildings was not given much importance. Linear and nonlinear analyses show that support condition influences the response considerably and can be an important parameter to decide the force amplification factor.

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