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

G. V. Shabareesh
M.Tech Student
SVR Engineering College, India

Abstract— The behaviour of the building with infill wall when the lateral loads are acting on it will be different when compared to the building without infill wall. But in the common practice in the analysis of the building the stiffness of the infill wall is ignored. In compensation for the discontinuity of the stiffness IS 1893: 2002 allows analysis of the building without considering infill stiffness but with a multiplication factor of 2.5. The code says that the beams and columns of ground floor without infill wall must be designed for 2.5 times the moments and shears calculated for a bare frame under seismic load. So the main objective of this paper is to check whether the multiplication factor 2.5 is realistic for the low rise buildings and to compare the seismic analysis of the building with and without considering the infill strength and stiffness.

Key words: Earthquake Resistant Low-Rise Building

I. Introduction

Due to the increase in population in recent years, the parking space of cars for residential apartments in populated cities is a matter of great concern. Therefore, the tendency was to use the ground floor of the building for parking. This type of buildings (Fig.1) that do not have masonry walls on the ground floor, but which are filled on all the upper floors, are called open-ground storey buildings (OGS). They are also known as "open first storey building" (when the numbering of the floor begins with one on the ground floor).

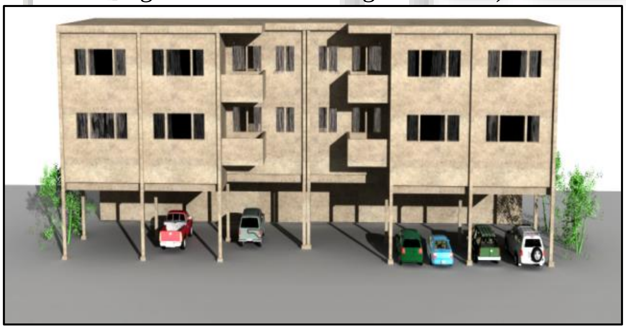


Fig. 1: Open Ground Storey

There is a significant advantage of this category of buildings from a functional point of view, but from the point of view of seismic performance it is considered that these buildings have increased their vulnerability. From the earthquakes of the past, it was evident that the main type of failure that occurred in the buildings of the OGS included the breaking of the lateral nodes, the crushing of the concrete core, the instability of the reinforcing longitudinal bars, etc. Due to the presence of filling walls throughout the upper floor, with the exception of the ground floor, the upper floors are much stiffer than the open ground floor. Therefore, the upper floors move almost together as a single block and most of the horizontal movement of the building occurs on the same ground level. In other words, these types of buildings swing back and forth like an inverted pendulum during the shaking of the earthquake and, therefore, the columns and the beams of the ground floor should have adequate strength.

II. Literature Review

Rao et. al. (1982) conducted theoretical and experimental studies on infilled frames with opening strengthened by lintel beams. It was concluded that the lintel over the opening does not have any influence on the lateral stiffness of an infilled frame [4].

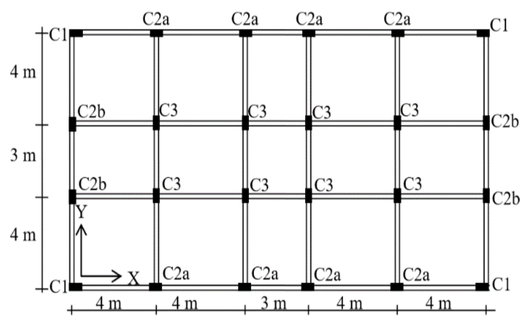
Arlekar et.al (1997) reported the behaviour of RC framed OGS building when subjected to seismic loads. A four storeyed OGS building was analysed using Equivalent Static Analysis and Response Spectrum Analysis to find the resultant forces and displacements. This paper shows that the behaviour of OGS frame is quite different from that of the bare frame [5].

Asokan (2006) studied how the presence of masonry infill walls in the frames of a building changes the lateral stiffness and strength of the structure. This research proposed a plastic hinge model for infill wall to be used in nonlinear performance based analysis of a building and concludes that the ultimate load (UL) approach along with the proposed hinge property provides a better estimate of the inelastic drift of the building [6].

III. Modelling the Structure

A. Building Description

The building is quite symmetrical in plan and height. This building is a G + 3 (12 m high) and is made of Reinforced Concrete (RC) Ordinary Moment Resisting Frames (OMRF). The reinforced concrete slab has a thickness of 150 mm on each level of the floor. The thickness of the brick walls is 230 mm for the external walls and 120 mm for the internal walls. The imposed load is taken as 2 KN / m2 for all the floors. The cross-sections of the structural elements (columns and beams 300 mm × 600 mm) are the same in all frames and in all stories. Storey masses to 295 and 237 tonnes in the bottom storyes and at the roof level, respectively. Fig. 2 presents typical floor plans showing different column and beam locations. Table 1 shows the beam and column reinforcement details.



(a) Column Locations

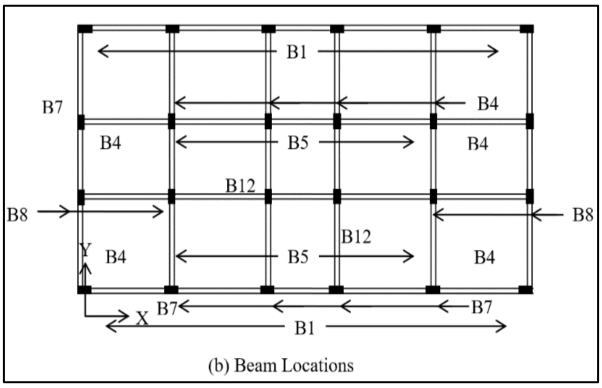


Fig. 2: Floor Plan of the Building

Column ID	Longitudinal Reinforcement
C1	12Y16
C2(a)	8Y20
C2(b)	8Y20
C3	8Y16

Beam ID	Top steel	Bottom steel
B1	4Y16	3Y16
B4	3Y16	2Y16
B5	2Y16, 1Y12	2Y16
B7	3Y16	3Y16
B8	3Y16	3Y16
B12	3Y16	2Y16, 1Y12
Roof Beams	2Y16	2Y16

Table 1: Longitudinal Reinforcement Details of Frame Sections

B. *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 (Ec) of concrete is taken as:

$$Ec = 500\sqrt{fck}\,MPa \quad (1.1)$$

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

Em = 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 Em = 750 fm'.

For the present study the modulus of elasticity of the masonry is taken as given in literature by Asokan (2006).

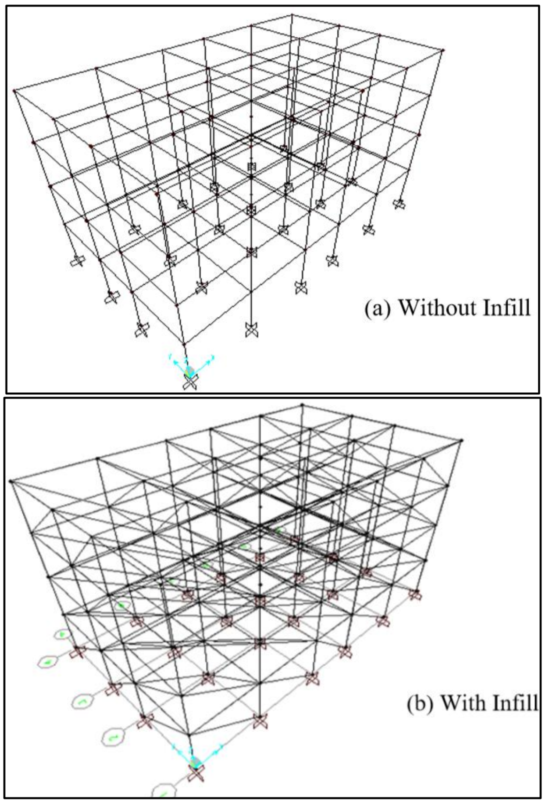


Fig. 3: Model of Building Without & with Considering Infill Stiffness Respectively

IV. RESULTS FROM LINEAR ANALYSIS

Equivalent Static Analysis (Seismic Coefficient Method) and Response Spectrum Analysis are considered for the analysis of buildings studied here. 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.

A. *Calculation of Time Period & Base Shear*

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

$$V_B = A_h * W \quad (1.2)$$

$$A_h = (Z/2)*(I/R)*(S_a/g) \quad (1.3)$$

Where W ≡ seismic weight of the building, Z ≡ zone factor, I ≡ importance factor, R ≡ response reduction factor, S_a/g ≡ spectral acceleration coefficient

The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in meters) and height of the building from the support is represented as h (in meters). Same base shear were applied in the two building models. The equivalent lateral forces at each storey level are applied statically at the design centre of mass locations for equivalent static analysis (ESA). The building models also analyzed using Response Spectrum analysis (RSA). The first five modes were considered in the dynamic analysis, which give more than 90% mass participation in both of the horizontal directions. The base shears for the equivalent static method and the response spectrum methods are given in Table 2. This table indicates that there is no considerable difference between two models with regards to the global stiffness and design forces.

	With infill		Without infill	
	V_x(KN)	V_y(KN)	V_x(KN)	V_y(KN)
Equivalent Static (V'_B)	1566	1566	1566	1566
Response Spectra (V_B)	1427	1427	1300	1310
V'_B/ V_B	1.1	1.1	1.2	1.19

Table 2 Comparison of Time Periods for With & Without Infill for Pinned & Fixed End Support Condition

B. *Shift in Period*

When the infill stiffness is considered in the OGS building model the global stiffness is bound to increase, reducing the fundamental period of the building. This reduction may attract additional seismic force and this is one of the factors that make difference between buildings modeled with and without infill stiffness. Therefore shift in fundamental period can be considered as an important parameter to describe how much the infill stiffness contributes to the global stiffness of the OGS building. The fundamental time periods in the predominant direction of vibration and the spectral acceleration coefficients corresponding to medium soil for the building for various cases are given in Table 3(a) and 3(b) for building models with fixed and pinned end supports respectively.

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	2.5	2.5	2.5	2.5
(Sa/g)y	2.5	2.5	2.5	2.5

Table 3(A): Shift in Period for Fixed End Support Condition

From Table 3(a) and 3(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.

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	0.6
(Sa/g)x	2.5	2.5	2.5	2.23
(Sa/g)y	2.5	2.5	2.5	2.28

Table 3(B): Shift in Period for Pinned End Support Condition

C. *Column Interaction Ratios*

All four building models were analyzed with lateral force associated with 'with infill' case for linear static analyses. However, for response spectrum analyses the base shear is a function of the respective structural natural periods. The demands (moments, axial forces) obtained at the critical sections from the linear (static and dynamic) analyses are compared with the capacities of the individual elements. For a column, the moment demand due to bi-axial bending under axial compression is checked using the P-Mx-My interaction surface, generated according to IS 456: 2000. The demand point is plotted in the P-Mx-My space and a straight line is drawn joining the demand point to the origin. This line (extended, if necessary) will intersect the interaction surface at the capacity point. The ratio of the distance of the demand point (from the origin) to the distance of the capacity point (from the origin) is termed as the interaction ratio (IR) for the column.

Table 4(a) and Table 4(b) presents the interaction ratio (IR) for all ground storey columns for buildings modelled with pinned end and fixed end support conditions respectively. These tables also show the ratio of IR for similar columns to show how the ground floor column forces increases for modelling infill stiffness at the upper storeys.

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

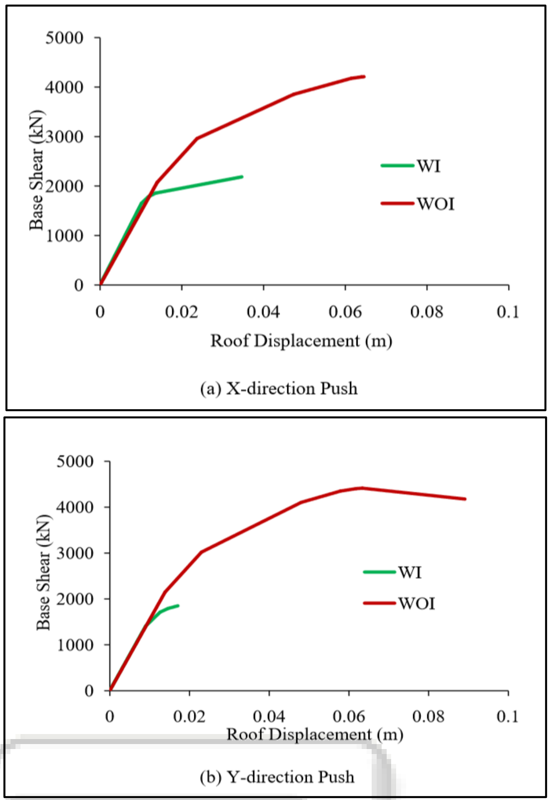
Table 4(A): Comparison of Ground Storey Column Interaction Ratio for Pinned End Case

Col. ID	IR(ESA)		Ratio of IR	IR(RSA)		Ratio of IR
	WI	WOI		WI	WOI	
C1	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

Table 4(B): Comparison of Ground Storey Column Interaction Ratio for Fixed End Case

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.

V. RESULTS FROM NON-LINEAR ANALYSIS



(a) X-direction Push

(b) Y-direction Push

Fig. 4: Pushover Curves for Pinned-End Building

The above figures are the results from the pushover analysis for pinned end support condition in both X- and Y-direction respectively. It is found that both of the maximum base shear and roof displacement capacity for without-infill case is higher than that of with-infill case.

Figs. 5(a) and (b) show plastic hinge distribution in a typical X-Z frame at collapse under X-direction push. It is clear that without-infill model utilizes the full capacity of the building before collapse as the hinges are evenly distributed in all storeys of the building. Whereas the plastic hinges, for with-infill model, are concentrated only in the ground storey columns and building model fails by storey mechanism. There is a major difference in mode of failure for the two building models. Similar conclusion can be made from the Y-direction pushover analysis.

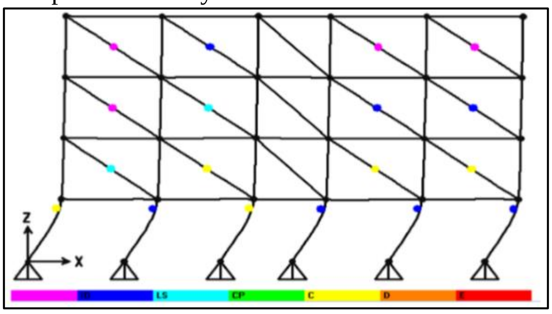


Fig. 5(a): Distribution of Plastic Hinges for WI Building Model

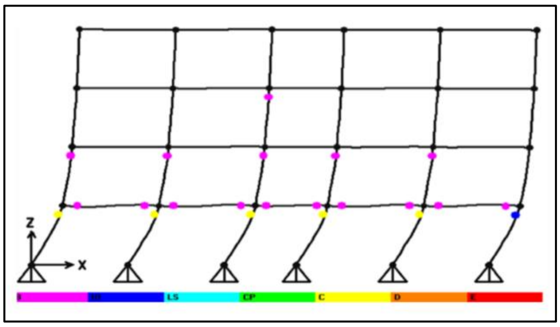
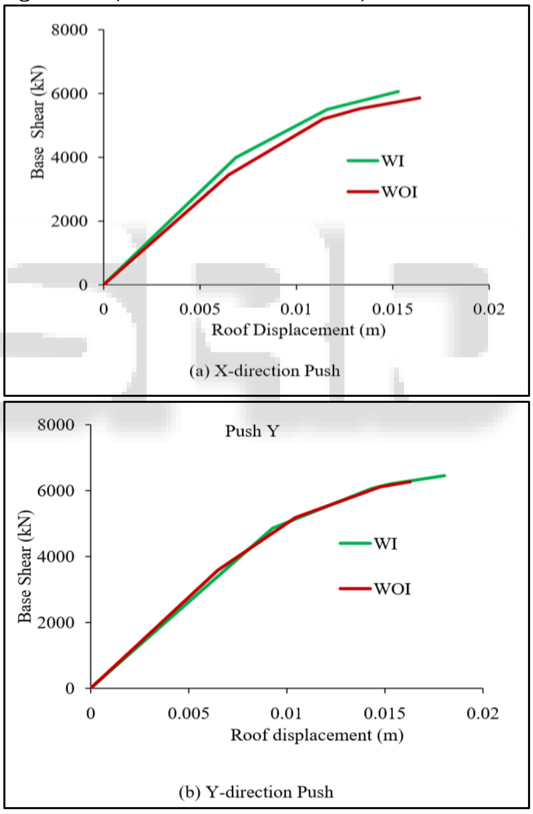


Fig. 5(b): Distribution of Plastic Hinges for WOI Building Model

Fig. 6(a) and 6(b) show the results of the building modelled with fixed-end support condition. The pushover curves presented in these figures indicates similar results. WOI model has higher base shear capacity compared to WI model. Obviously in case of fixed end condition the maximum base shear capacity (around 6900kN in X direction) is much higher compared to that of pinned-end building model (4300kN in X direction).

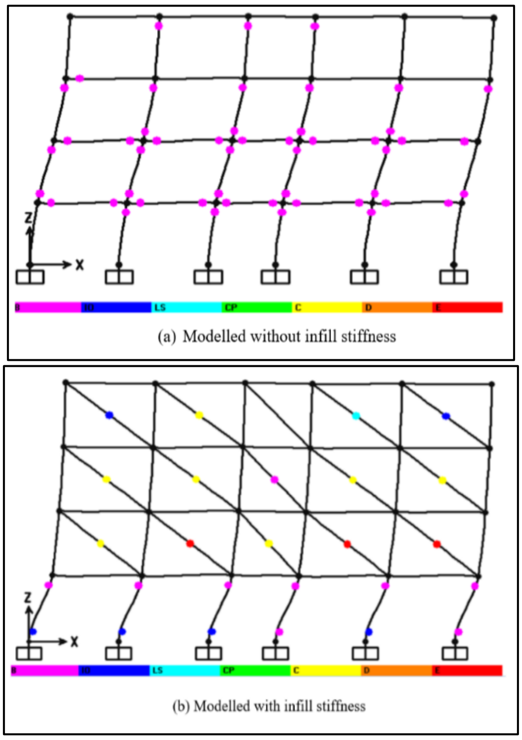


(a) X-direction Push

(b) Y-direction Push

Fig. 6: Pushover Curves for Fixed-End Building Model

Figs. 7(a) and 7(b) show the distribution of plastic hinges formed during collapse for fixed end support condition. Similar to the previous case WOI model found to utilize the full capacity of the building as plastic hinges are distributed almost equally in all storeys. Also, in WI model, the compressive struts fail prior to the column which may be advantageous from seismic point of view.



(a) Modelled without infill stiffness

(b) Modelled with infill stiffness

Fig. 7. Hinge Distribution at Collapse for Fixed-End Building Model (X-Direction Push)

ACKNOWLEDGMENT

According to the IS code when the building has to be designed as an open ground storey the beams and columns forces at the ground storey must be multiplied by 2.5. But the analysis is done by using equivalent static analysis and response spectrum analysis for both fixed and pinned end conditions and column interaction ratio values shows that the factor 2.5 is too high for low rise open ground storey buildings.

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.

And the analysis shows that the with infill model performance better than the without infill model

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