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

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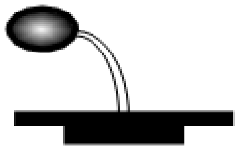
Abstract- *Presence of infill walls in the frames alters the behaviour of the building under lateral loads. However, it is common industry practice to ignore the stiffness of infill wall for analysis of framed building. Engineers believe that analysis without considering infill stiffness leads to a conservative design. But this may not be always true, especially for vertically irregular buildings with discontinuous infill walls. Hence, the modelling of infill walls in the seismic analysis of framed buildings is imperative. Indian Standard IS 1893: 2002 allows analysis of open ground storey buildings without considering infill stiffness but with a multiplication factor 2.5 in compensation for the stiffness discontinuity. As per the code the columns and beams of the open ground storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frames (i.e., without considering the infill stiffness). However, as experienced by the engineers at design offices, the multiplication factor of is not realistic for low rise buildings. This calls for an assessment and review of the code recommended multiplication factor for low rise open ground storey buildings. Therefore, the objective of this thesis is defined as to check the applicability of the multiplication factor of 2.5 and to study the effect of infill strength and stiffness in the seismic analysis of low rise open ground storey 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 (Fig. 1.1) 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 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.



The OGS framed building behaves differently as compared to a bare framed building (without any infill) or a fully infilled framed building under lateral load. A bare frame is much less stiff than a fully infilled frame; it resists the applied lateral load through frame action and shows well-distributed plastic hinges at failure. When this frame is fully infilled, truss action is introduced. A fully infilled frame shows less inter-storey drift, although it attracts higher base shear (due to increased stiffness). A fully infilled frame yields less force in the frame elements and dissipates greater energy through infill walls. The strength and stiffness of infill walls in infilled frame buildings are ignored in the structural modelling in conventional design practice. The design in such cases will generally be conservative in the case of fully infilled framed building. But things will be different for an OGS framed building. OGS building is slightly stiffer than the bare frame, has larger drift (especially in the ground storey), and fails due to soft storey- mechanism at the ground floor as shown in Fig.. Therefore, it may be unconservative

to ignore strength and stiffness of infill wall while designing OGS buildings.

II. STRUCTURAL MODELLING

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 for internal walls.

Imposed load is taken as 2 kN/ m^2 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 300mm×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.

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:

$$E_c=5000f_{ck}$$

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 (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

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.

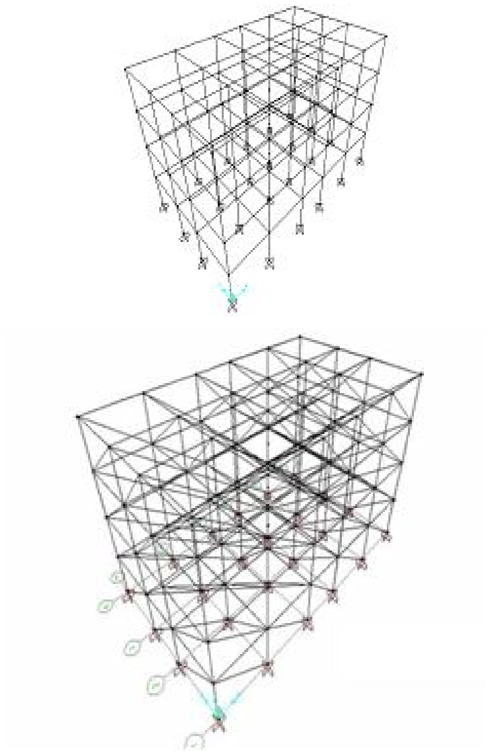
Beams and columns in the present study were modelled as frame elements with the centrelines joined at nodes using commercial software SAP2000NL. The rigid beam- column joints were modelled by using end offsets at the joints (Fig. 3.2). The floor slabs were assumed to act as diaphragms, which ensure integral 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.

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.

Modelling Infill Walls

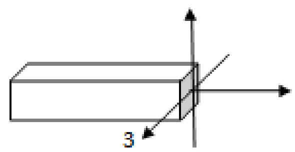
Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of buildings with infill wall. But the nonlinear modelling of a two dimensional plate element is not understood well. Therefore infill wall has to be modelled with a one-dimensional line 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.



3D Computer model of building without and with considering infill stiffness respectively.(a) without infilla

MODELLING OF FLEXURAL PLASTIC HINGES

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. In the present study, a point- plasticity approach is considered for modelling nonlinearity, wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. Beam and column elements in this study were modelled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns). Refer Fig. 3.4 for the local axis system considered. Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load. In the present study the plastic hinge properties are calculated by SAP 2000. The analytical procedure used to model the flexural plastic hinges are explained below.



The coordinate system used to define the flexural and shear hinges

Flexural hinges in this study are defined by moment-rotation curves calculated based on the cross-section and reinforcement details at the possible hinge locations. For calculating hinge properties it is required to carry out moment–curvature analysis of each element. Constitutive relations for concrete and reinforcing steel, plastic hinge length in structural element are required for this purpose. The flexural hinges in beams are modelled with uncoupled moment (M3) hinges whereas for column elements the flexural hinges are modelled with coupled P-M2-M3 properties that include the interaction of axial force and bi-axial bending moments at the hinge location. Although the axial force interaction is considered for column flexural hinges the rotation values were considered only for axial force associated with gravity load.

Modelling of Moment-Curvature in RC Sections Using the Modified Mander model of stress-strain curves for concrete (Panagiotakos and Fardis, 2001) and Indian Standard IS 456 (2000) stress-strain curve for reinforcing steel, for a specific confining steel, moment curvature relations can be generated for beams and columns (for different axial load levels). The assumptions and procedure used in generating the moment-curvature curves are outlined below.

MODELLING OF EQUIVALENT STRUT

For an infill wall located in a lateral load-resisting frame, the stiffness and strength contribution of the infill has to be considered. Non-integral infill walls subjected to lateral load behave like diagonal struts. Thus an infill wall can be modelled as an equivalent ‘compression only’ strut in the building model. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts to the beam-to-column junctions. This section explains the procedure based on Smith and Carter (1969) to calculate the modelling parameters (effective width, elastic modulus and strength) of an equivalent strut.

III. RESULTS FROM LINEAR ANALYSIS

Thus broadly we can say that linear analysis of structures to compute the earthquake forces is commonly based on one of the following three approaches.

1. An equivalent lateral procedure in which dynamic effects are approximated by horizontal static forces applied to the structure. This method is quasi-dynamic in nature and is termed as the Seismic Coefficient Method in the IS code.
2. The Response Spectrum Approach in which the effects on the structure are related to the response of simple, single degree of freedom oscillators of varying natural periods to earthquake shaking.
3. Response History Method or Time History Method in which direct input of the time history of a designed earthquake into a mathematical model of the structure using computer analyses.

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.

Calculation of Time Period and 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:

$V_B = A_h W$
$A_h = Z\ I\ S_a/2\ R\ g$

where W □ seismic weight of the building, Z □ zone factor, I □ importance factor, R □ responsereduction factor, S_a/g □ spectral acceleration coefficient corresponding to an approximatetime period (T_a) which is given by:

$$T_a = 0.09h/D^{1/1s}$$

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 4.1. 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	1.10	1.10	1.20	1.19

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 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	2.50	2.50	2.50	2.50
Sa/g) y	2.50	2.50	2.50	2.50

IV. SUMMARY AND CONCLUSIONS

Open ground storey buildings are considered as vertically irregular buildings as per IS 1893: 2002 that requires dynamic analysis considering strength and stiffness of the infill walls. IS 1893: 2002 also permits Equivalent Static Analysis (ESA) of OGS buildings ignoring strength and stiffness of the infill walls, provided a multiplication factor of 2.5 is applied on the design forces (bending moments and shear forces) in the ground storey columns and beams. The objective of the present study is to review the rationality of this approach. An existing RC framed building (G+3) with open ground storey located in Seismic Zone-V is analyzed for two different cases:

(a) considering infill strength and stiffness and

(b) without considering infill strength and stiffness (bare frame).

Infill weights (and associated masses) were modelled in both the cases through applying static dead load. Non-integral infill walls subjected to lateral load behave like diagonal struts. Thus an infill wall can be modelled as an equivalent ‘compression only’ strut in the building model. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts to the beam-to-column junctions. Infill stiffness was modelled using a diagonal strut approach as per Smith and Carter (1969).

V. CONCLUSIONS

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 of2.5 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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