IMPLICATIONS OF MAJOR INTERNATIONAL CODAL DESIGN PROVISIONS FOR OPEN GROUND STOREY BUILDINGS

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ABSTRACT
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. "Open Ground Storey" (OGS) buildings are those types of buildings in which the ground storey is free of any infill masonry walls. These types of buildings are very common in India for parking provisions. 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 the behaviour is different in the case of OGS framed building. OGS framed 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.
In the present study, a typical ten storied OGS framed building is considered and the building considered is located in Seismic Zone-II. The design forces for the ground storey columns are evaluated based as Indian and OGS frames are designed considering MF as 2.5 (Indian), The performance of building is studied by using stadd.pro analysis software. The computational models are developed in the program and observed in each case. The relative performances of building designed as per Indian code.

Keywords: *Fragility curves, Open ground storey (OGS), Multiplication Factor (MF), Peak Ground Acceleration (PGA), Probabilistic Seismic Demand Model (PSDM)*

INTRODUCTION
Need of space became very important in urban areas due to increase in population especially in developing countries like India. Need of parking space takes important vital role while planning a building. To provide adequate parking spaces, ground storey of the building is utilised. These types of buildings (Figure1) having no in filled walls in ground storey, but in-filled in all upper storeys, are called Open Ground Storey (OGS) buildings. The majority of apartments are of this type and the infill walls used are of mainly brick masonry.

Figure: Typical example of OGS building

The OGS framed building behaves differently as compared to that of a bare framed building (without any infill) or a fully infilled framed building under lateral load. Global lateral stiffness of a bare frame is much less than that of a fully infilled frame; it resists the applied lateral load through frame action and shows well-distributed plastic hinges at failure. When the frame is fully infilled, truss action is introduced.

A fully in filled frame shows less inter-storey drift, although it attracts higher base shear (due to increased stiffness). A fully in filled 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 implications of neglecting infill wall stiffness in OGS framed building may not be conservative. 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 Figure.

As reported by Davis (2009), Inclusion of stiffness and strength of infill walls in the OGS building frame decreases the fundamental time period compared to a bare frame and consequently increases the base shear demand and the design forces in the ground storey beams and columns. This increased design forces in the ground storey beams and columns of the OGS buildings are not captured in the conventional bare frame analysis. An appropriate way to analyse the OGS buildings is to model the strength and stiffness of infill walls. Unfortunately, no guidelines are given in IS 1893: 2002 (Part-1) for modelling the infill walls. As an alternative a bare frame analysis is generally used that ignores the strength and stiffness of the infill walls.

OPEN GROUND STOREY (OGS)

The presence of infill walls in the upper storeys of the OGS building increases the stiffness of the building globally, as seen in a typical infilled framed building. Due to the increase of global stiffness, the base shear demand on the building increases. In the case of typical infilled frame building, the increased base shear is shared by the both frames and infill walls in all the storeys. In OGS buildings, where the infill walls are not present in the ground storey (no truss action), the increased base shear is resisted entirely by the ground storey columns, without any load sharing possible by adjoining infill walls. The increased shear forces in the ground storey columns will induce increased bending moments and thereby higher curvatures, causing relatively larger drifts at the first floor level. The large lateral deflections further enhance the bending moments due to the P-effect. Plastic hinges develop at the top and bottom ends of the ground storey columns. The upper storeys would remain undamaged and move almost like a rigid body.

The damage is mostly concentrated in the ground storey columns, and this is termed as typical 'soft-storey collapse'. This is also called a 'storey-mechanism' or 'column mechanism' in the ground storey, as shown in Figure. These buildings are considered to be vulnerable due to the sudden lowering of stiffness or strength (vertical irregularity) in the ground storey compared to a typical infilled frame building. The presence of a soft story results in a localized excessive drift that causes heavy damage or collapse of the story during a severe earthquake. Most of the lateral deformations were found to be accumulated at the soft and weak ground storey because of the presence of heavy mass on upper stories and the absence of infills in the ground storey and plastic hinges will be formed.

MULTIPLICATION FACTOR (MF) PROVISIONS IN INDIAN STANDARDS IS-1893:2002

The OGS buildings can be considered as extreme soft-storey type of buildings in most of the practical situations, and shall be designed considering special provisions to increase the lateral stiffness or strength of the soft/open storey. Here we are ignoring the infill strength and stiffness of infill walls.

Indian standards IS-1893:2002

After the incident of the Bhuj earthquake, the IS 1893 code has been revised in 2002, incorporating new design recommendations to improve OGS buildings. Clause 7.10.3(a) states: "The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frame". The factor 2.5 can be called as a multiplication factor (MF). The prescribed multiplication factor (MF) of 2.5, applicable for all OGS framed buildings, is fairly high and suggests that all existing OGS framed buildings (designed to earlier codes) are highly vulnerable under seismic loading. The proposed MF does not account for dependence on number of storeys, number of bays, type and number of infill walls present, etc. The code proposal has also met with resistance in design and construction practice due to cost implications and congestion of heavy reinforcement in the ground storey columns.

As per IS 1893 (2002), a storey is called *soft-storey* (a type of vertical irregularity) if the lateral stiffness of a particular storey is less than 70% of stiffness of adjacent storey or less than 80% of the average lateral stiffness of three storeys above the storey under consideration. A storey is called *extreme soft-storey* if the lateral stiffness is less than 60% of that in the storey above or less than 70% of the average stiffness of the three storeys above. Stilts or open ground storey buildings fall under extreme soft-storey type of vertically irregular buildings.

If the stiffness ratio (k_0/k_1, where k_0 and k_1 are the lateral stiffness of ground storey and first storey respectively), is less than 0.7 then it is a weak in ground storey. Hence the shear forces and bending moments in the ground storey columns should be multiplied by a factor 2.5 for design purposes.

Functional designing of the building has become very important and requirements of buildings vary from building to building. Hence it is essential to finalize the program with reference to the people who will be using the buildings. So it is necessary that every Civil Engineer knows the basic principles involved in design of R.C.C. structures hence, this project is intended at DESIGN of a Multi-storey structure.

SCOPE OF STUDY

- The present study is limited to reinforced concrete multi-storey framed buildings that are regular in plan.
- The present study is based on a case study of ten storey six bays and the buildings with basement, shear wall and stiff plinth beams are not considered in this study.
- The infill walls are assumed to be non-integral, non-load bearing and made of brick masonry.
- Out-of-plane action of masonry walls is not considered in the study.
- Asymmetric arrangement of infill walls is ignored and window and door openings infill panels are neglected in the modelling.

LITERATURE REVIEW

SEISMIC BEHAVIOUR OF INFILL WALLS AND OPEN GROUND STOREY BUILDING

Under lateral loading, the frame and the infill wall stay intact initially. As the lateral load increases, the infill wall gets separated from the surrounding frame at the unloaded (tension) corner. However at the compression corners the infill walls are still intact. The length over which the infill wall and the frame are intact is called the length of contact. Load transfer occurs through an imaginary diagonal which acts like a compression strut. Due to this behaviour of infill wall, they can be modelled as an equivalent diagonal strut connecting the two compressive corners diagonally. The stiffness property should be such that the strut is active only when subjected to compression. Thus, under lateral loading only one diagonal will be operational at a time. This concept was first put forward by Holmes (1961).

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. Karisiddappa (1986) and Rahman (1988) examined the effect of openings and their location on the behaviour of single storey RC frames with brick infill walls. The behaviour of RC framed OGS building when subjected to seismic loads was reported by Arlekar et. al. (1997).

A four storied OGS building was analysed using Equivalent Static Analysis and Response Spectrum Analysis to find the resultant forces and displacements. It was shown that the behaviour of OGS frame is quite different from that of the bare frame. The effect of different parameters such as plan aspect ratio, relative stiffness, and number of bays on the behaviour of infilled frame was studied by Riddington and Smith (1997).

STUDIES ON THE DEVELOPMENT OF SEISMIC FRAGILITY CURVES

Kircil & Polat (2006) performed nonlinear dynamic analyses of representative RC buildings, designed with the 1975 code, using 12 artificial accelerograms with increasing intensity in order to define the parameters of lognormal vulnerability curves. Fragility curves for different steel grades were summed (sum weighted by the population of each sample) to provide a single curve for all buildings. A relationship was established between number of storeys and mean and standard deviation of the curves, so as to obtain curves for structures with number of storeys not in the examined range.

Erberik (2008) studied 28 RC frame buildings that were inspected after the Düzce earthquake. The buildings were constructed between 1973 and 1999. Pushover analyses were performed to obtain the bilinear capacity curves and the distribution of their characteristic properties. 2800 nonlinear dynamic analyses of randomly sampled SDOF structures were performed for a set of 100 recorded accelerograms.

Özer and Erberik (2008) developed vulnerability curves for RC frame structures in Turkey. 3, 5, 7 and 9-storey RC frames with poor, medium and good seismic designed were considered. Concrete and steel strength and modulus of elasticity were variables. Four damage states were introduces as slight or no damage (DS1), significant damage (DS2), severe damage (DS3) and collapse (DS4). The seismic demand statistics in terms of maximum inter storey drift ratio were obtained for different sets of ground motion records by performing non-linear time-history analyses.

Nagae et. al (2006) computed the annual frequency of maximum inter-storey drift ratios exceeding a specific value. The shapes of the curves of PGA and IDRmax are found to be significantly influenced by the type of the failure mechanism. Lagaros (2008) studied the effectiveness of the fragility curves in assessing the performance of RC buildings with soft storey designed to prescriptive code provisions.

Rota et. al (2010) proposed a new method for development of fragility curves for masonry buildings. The probability density functions are determined for selected damage state based pushover analysis and probability density functions of displacement demand obtained from nonlinear time history analysis.

Tavares et. al (2012) conducted a study to find the fragility curves for different bridge classes in eastern Canada. Bridge-system fragility curves are developed considering the vulnerability of critical components to assess the probability of bridge damage. The relationship between the bridge damage and the ground motion intensity is represented by power law proposed by Cornell et. al (2002).

Rajeev, P and Tesfamariam, S (2012) conducted a study on the Poor seismic performance of non-code conforming RC buildings, mainly designed for gravity loads prior to 1970s. Fragility based seismic vulnerability of structures with consideration of soft storey (SS) and quality of construction

(CQ) is demonstrated on three-, five-, and nine-storey RC frames designed prior to 1970s. Probabilistic seismic demand model (PSDM) for those gravity load designed structures is developed, using the nonlinear finite element analysis, considering the interactions between SS and CQ. The proposed approach of developing a predictive tool can enhance regional damage assessment tool, such as HAZUS, to develop enhanced fragility curves for known SS and CQ.

SEISMIC PERFORMANCE OF TYPICAL OPEN GROUND STORY FRAMED BUILDING

A typical ten-storey six-bay OGS RC frame that represents a symmetric building in plan is considered in the present study. Grades of concrete and steel are taken as M25 and Fe415 respectively. Typical bay width and column height are selected as 3m and 3.2m respectively. Slab thickness is of 150 mm. A live load of 3 kN/m^2 is considered at all floor levels except top floor, where it is considered as 1.5kN/m^2. Seismic load is taken according to IS 1893 (2002).

The selected building is assumed to be symmetric in both orthogonal directions in plan. The torsional response of the building is neglected and hence a single plane frame is considered to be representative of the building along one direction. The total width of building is of 18.0 m having 6 bays, width of each bay is 3.0 m. The total height of the building is 32.0 m, having 10 storeys, height of each storey being 3.2 m. Parapet wall of 0.6 m is considered. The size of typical columns and beams considered are 350mm x 350mm and 230mm x 350mm respectively.

Latin Hyper Cube Sampling (LHS)

To consider the uncertainty in the material properties, the characteristic strength of concrete, f_{ck} , the yield strength of the steel, f_y and the compressive strength of masonry f_m are taken as the random variable. The statistical details (Table 4.2) of the parameters, f_{ck} and f_y have been taken from Ranganathan (1999) and that for masonry is taken from Kaushik et. al. (2007). From the mean and STD deviations of each random variable, a set of 30 values of random variables are generated using LHS sampling method. This is carried out in MATLAB program. The sets of thirty statistically equivalent analytical models generated for the three random variables are tabulated in the Table below.

Table : Details of random variables used in LHS scheme

Material	Variable	Mean	COV (%)	Distribution	Remarks
Concrete	f_{ck} (MPa)	30.28	21.0	Normal	Uncorrelated
Steel	f_y (MPa)	468.90	10.0	Normal	Uncorrelated
Masonry	f_m (Mpa)	6.60	20	Normal	Uncorrelated

Modelling and Analysis

It is required to conduct nonlinear dynamic analysis for all the thirty building frames in order to capture the maximum inter-storey drift for corresponding PGA. Each Building frames are modelled in the program SeismoStruct (2007). SeismoStruct is a Finite Element package capable of predicting the large displacement behaviour of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity.

Seismo Structures fibre based spread plasticity elements for frame elements.

Number of Nodes	539	Highest Node
Number of Elements	1330	Highest Beam

Concrete

Concrete is modelled as per Mander et al. (1988). It is a uniaxial nonlinear constant confinement model. Five model calibrating parameters should be defined in order to fully describe the mechanical characteristics of the material:

(1) Compressive strength – f_c

It is the compressive stress capacity of the cylinder having a dimension of 100 mm x 200 mm and its values varies from 15 MPa to 45 MPa. The default value is 30 MPa.

(2) tensile strength - f_t

It is the tensile stress capacity of the material and it can usually be estimated as where k_t varies from 0.5 (concrete in direct tension) to 0.75 (concrete in flexural tension), as suggested by Priestley et al. [1996].The default value is 0 MPa.

(3) strain at peak stress - ε_c

This is the strain corresponding to the point of unconfined peak compressive stress (f_c). For normal strength plain concrete, this value is usually considered to lie within the range of 0.002 to 0.0022. The default value is 0.002 mm/mm.

(4) Confinement factor - k_c

This is the constant confinement factor, defined as the ratio between the confined and unconfined compressive stress of the concrete, and used to scale up the stress-strain relationship throughout the entire strain range. Its value usually fluctuates between the values of 1.0 and 1.3 for reinforced concrete members and between 1.5 and 4.0 for steel-concrete composite members.

(5) Specific weight – □

This is the specific weight of the material. The default value is 24 kN/m^3.

Reinforcements

Reinforcement bars are modelled as Bilinear steel model. This is a uniaxial bilinear stress-strain model with kinematic strain hardening, whereby the elastic range remains constant throughout the various loading stages, and the kinematic hardening rule for the yield surface is assumed as a linear function of the increment of plastic strain. This simple model is also characterised by easily identifiable calibrating parameters and by its computational efficiency. It can be used in the modelling of both steel structures, where mild steel is usually employed, as well as reinforced concrete models, where worked steel is commonly utilised. Five model calibrating parameters should be defined in order to fully describe the mechanical characteristics of the material:

(1) Modulus of elasticity – E_s

It is the initial elastic stiffness of the material. The value usually varies between 200 and 210 GPa

(2) Yield strength - f_y

It is the stress at yield and Its value varies from 230 MPa up to 650 MPa.

(3) strain hardening parameter – μ

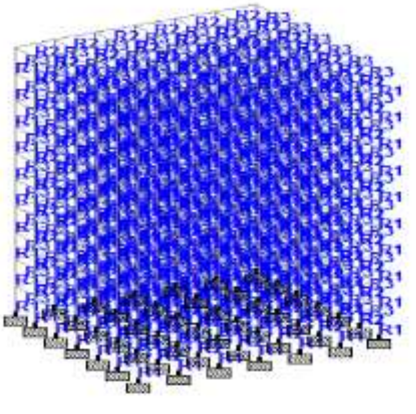
It is the ratio of the post-yield stiffness (E_{sp}) to the initial elastic stiffness (E_s) of the material. The former is defined as $E_{sp}=(f_{ult} - f_y)/(\varepsilon_{ult} - f_y/E_s)$, where f_{ult} and ε_{ult} represent the ultimate or maximum stress and strain capacity of the material, respectively. Its value commonly ranges from 0.005 to 0.015. fracture/ buckling strain - ε_{ult}

This is the strain at which fracture or buckling occurs. The default value is 0.1 .

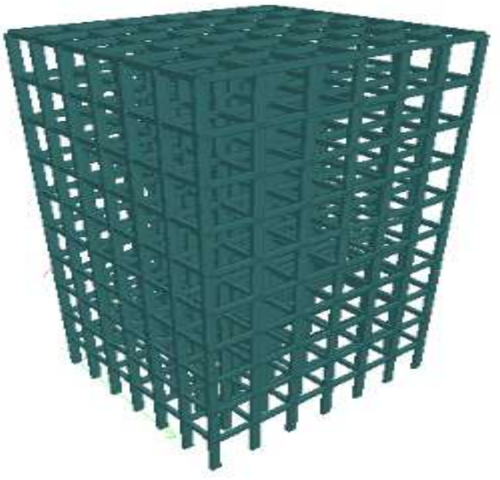
(5) specific weight – □

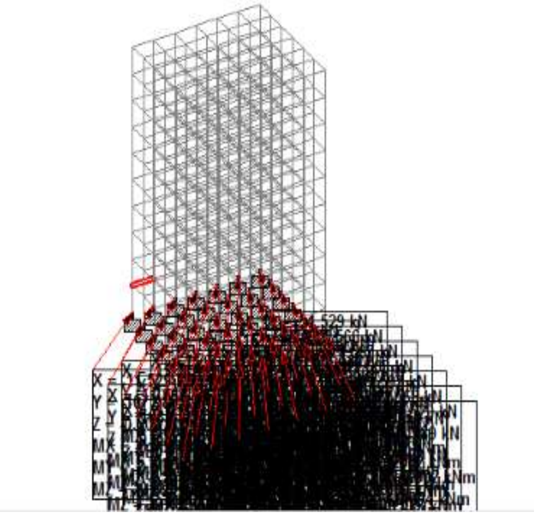
It is the specific weight of the material and the default value is 78 kN/m^3.

STADD MODEL



Whole structure





3D MODEL

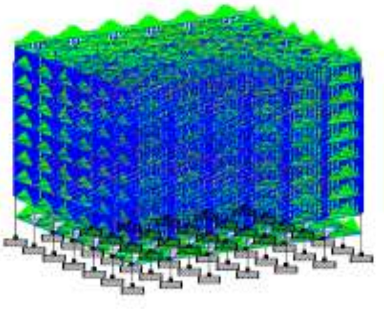
All / Summary / Envelope

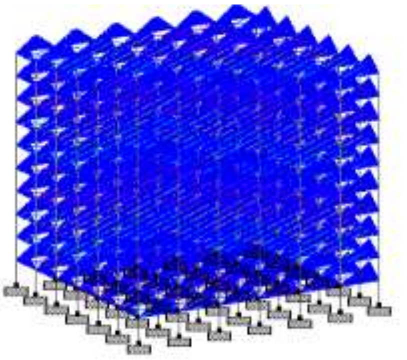
Node	L/C	Horizontal Fx kN	Vertical Fy kN	Horizontal Fz kN	Moment Mx kNm	My kNm	Mz kNm
1	1 EQ+X	-11.529	-107.989	-0.002	-0.002	-0.004	29.325
	2 EQ-X	11.529	107.989	0.002	0.002	0.004	-29.325
	3 EQ+Z	-0.003	-141.515	-12.236	-23.106	0.005	0.005
	4 EQ-Z	0.003	141.515	12.236	23.106	-0.005	-0.005
	5 DEAD LOA	1.199	873.991	0.528	0.576	-0.002	-1.416
	6 LIVE LOAD	0.776	128.881	0.389	0.418	0.001	-0.847
	7 WL+X	-9.883	-50.443	-0.004	-0.006	-0.056	19.065
	8 WL-X	7.451	50.402	0.003	0.005	0.040	-18.497
	9 WL+Z	-0.026	-75.466	-11.736	-17.456	0.108	0.053

G+8 PLAN - Statics Check Results

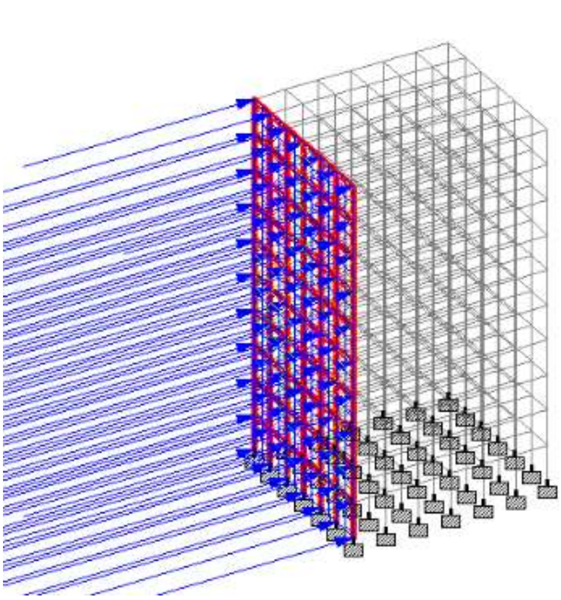
L/C		Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
1	Loads	702.037	0.000	0.000	0.000	5616.297	-16932.712
	Reactions	-702.037	-0.000	-0.000	0.000	-5616.297	16932.712
	Difference	0.000	-0.000	-0.000	0.000	0.000	-0.000
2	Loads	-702.037	0.000	0.000	0.000	-5616.297	16932.712
	Reactions	702.037	0.000	0.000	-0.000	5616.297	-16932.712
	Difference	-0.000	0.000	0.000	-0.000	-0.000	0.000
3	Loads	0.000	0.000	702.037	16932.712	-6318.335	0.000
	Reactions	-0.000	0.000	-702.037	-16932.712	6318.335	0.000
	Difference	-0.000	0.000	-0.000	-0.000	0.000	0.000
4	Loads	0.000	0.000	-702.037	-16932.712	6318.335	0.000
	Reactions	0.000	-0.000	702.037	16932.712	-6318.335	-0.000
	Difference	0.000	0.000	0.000	0.000	0.000	0.000

ANALYSIS<u>**Load cases**</u>





LOAD 1 **LOAD 2**



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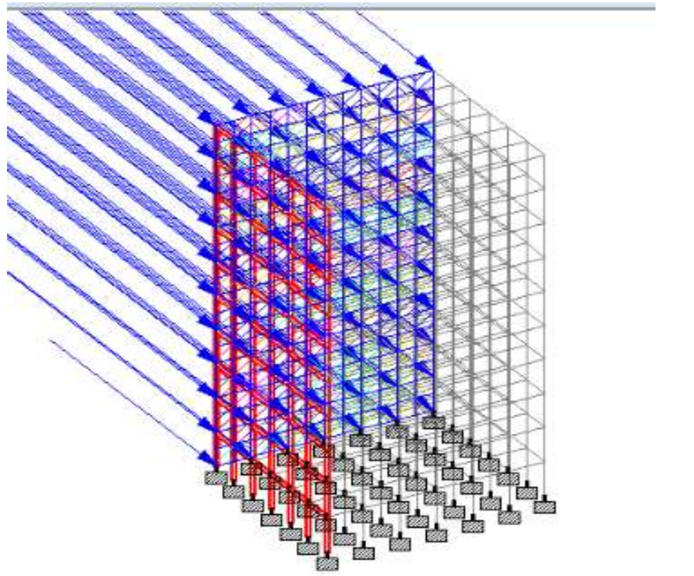

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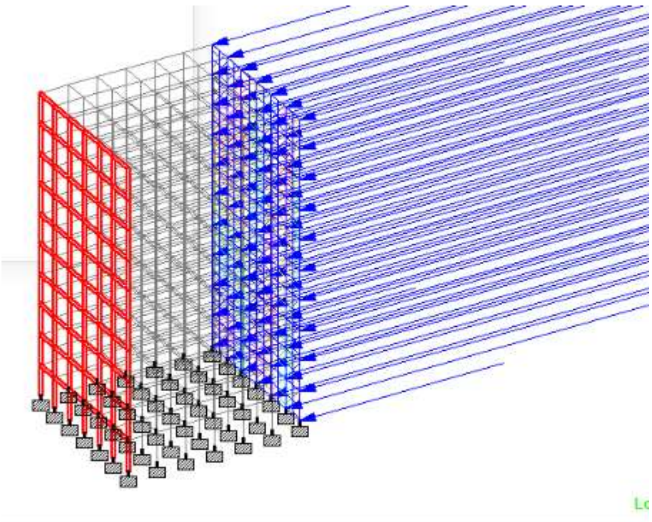

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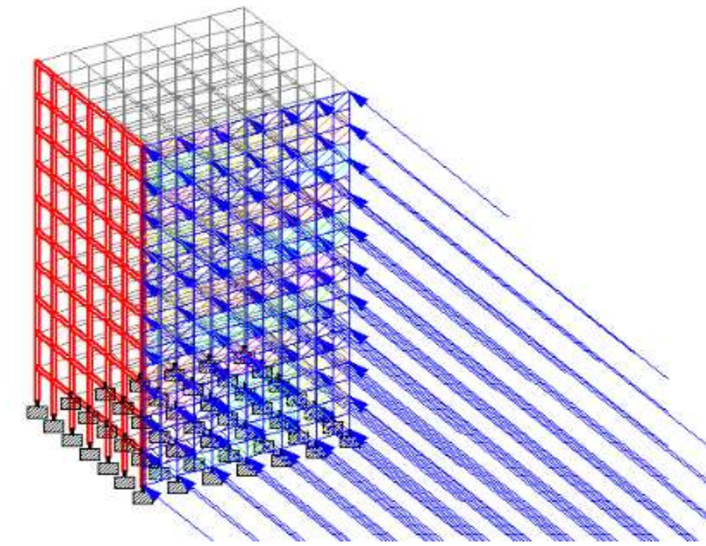
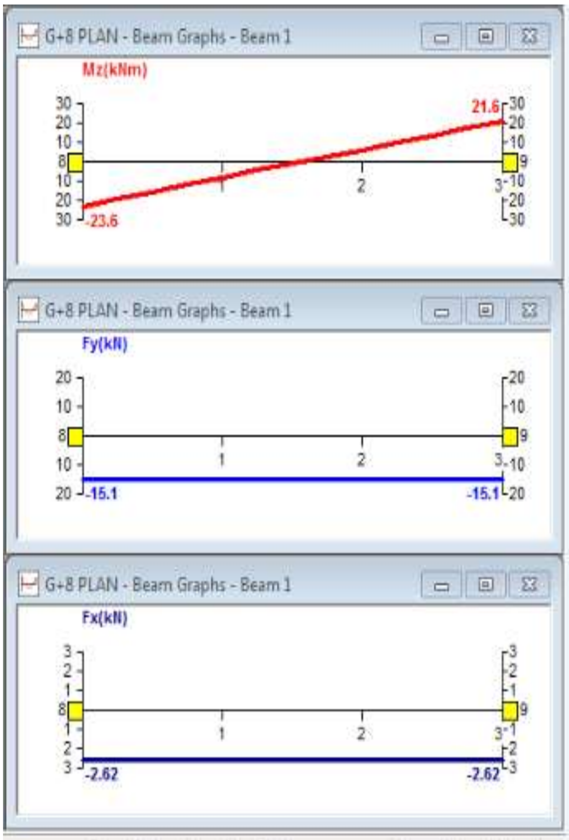
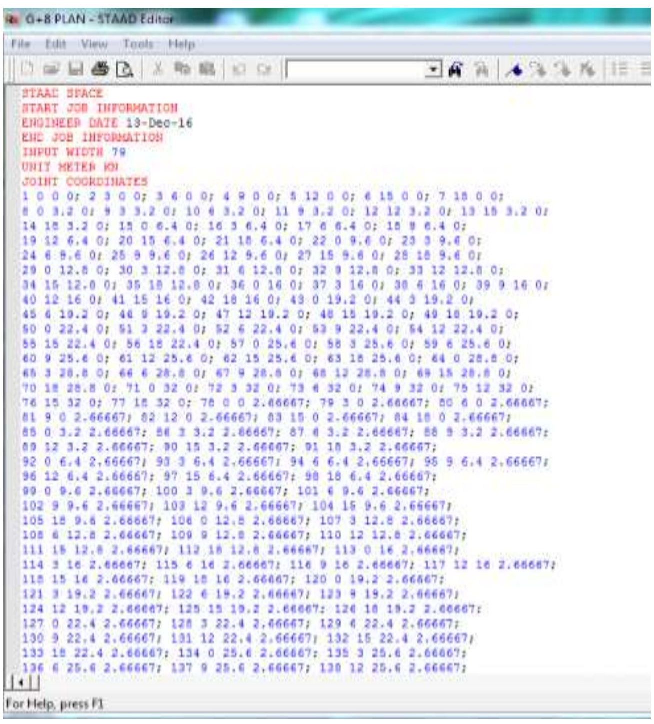


LOAD 6

Displacement



Beam formation



Final output view

CONCLUSIONS

Followings are the salient conclusions obtained from the present study:

- The performance of typical OGS buildings designed considering various magnification factors according to different codes are studied using fragility curves.
- Uncertainties in concrete, steel and masonry are incorporated using LHS scheme. It is found that the performances of the OGS frames, in terms of ground storey drift is increasing in the increasing order of magnification factors used by various codes for all the performance levels.
- In all the cases of the buildings designed using various codes, the first storey is about 80% more vulnerable than the ground storey except for Israel code.
- It is found that relative vulnerability of first storey increases due to strengthening of the ground storey.
- Except Israel code, no other code considers MF for first storey. In other words, the first storey of all the frames designed by codes other than Israel code remains same to yield same exceedance probability.
- Application of magnification factor only in the ground storey may not provide the required performance in all the other stories. It is found from the study that the OGS buildings designed using Israeli code, which considered the magnification factor in the adjacent storey, performed better compared to that of others. This indicates that the implementation of magnification factor in the adjacent storeys may be required to improve the performance of OGS buildings.

SCOPE FOR FUTURE WORK

- The present study is based on a case study of a ten storey six bay RC framed building that are regular in plan and elevation (with open ground storey). This study can be extended considering buildings having irregularity in plan and elevation. This involves analysis of three dimensional building frames that accounts for torsional effects.
- OGS buildings with basement, shear walls and plinth beams are not considered in this study. The present methodology can be extended to such buildings also.

- Soil - structure interaction effects are also ignored in the present study. It can also be extended to study the response of the OGS buildings considering the soil - structure interaction.

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