EVALUATION OF MAGNIFICATION FACTORS FOR OPEN GROUND STOREY BUILDINGS USING NONLINEAR ANALYSES

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ABSTRACT :

Open Ground Storey (OGS) framed buildings, with soft-storey at ground storey are commonly analysed in practice, ignoring the infill wall stiffness (linear 'bare frame' analysis). Design codes impose a magnification factor (MF) on the design forces in the columns of the ground storey, based on such analysis. The present study attempts to estimate typical variations in MF, by modelling infill walls using Smith and Carter (1961) for linear analysis and Crisafulli (1999) for nonlinear analysis, accounting for the variability of compressive strength and modulus of elasticity of infill walls. Response Spectrum Analysis (RSA) and Nonlinear Dynamic Analysis (NDA) are carried out on a four storeyed and a seven storeyed building, for various infill wall arrangements. The results from RSA (linear analysis) indicate MF values in the range 1.04-1.13, for the four storeyed building and 1.11-2.39 for the seven storeyed building. However the results of NDA (nonlinear analysis) including hysteresis effects in frame and infill, suggest that there is no need for applying MF to low-rise building frames. In the case of the seven-storeyed building frame, values of MF in the range 1.14 to 1.29 were observed, applicable to the base shear. However, this is not applicable to column bending moments, where MF values were found to be less than unity.

KEYWORDS: Open ground storey buildings, Magnification factor, Infill walls, Bare frame, Response spectrum analysis, Nonlinear dynamic analysis

1. INTRODUCTION

Open Ground Storey (OGS) framed buildings are generally considered to be extremely vertical irregular buildings (with soft storey at ground storey), and hence require rigorous linear/ nonlinear dynamic analysis for the estimation of design forces. However, such methods are not adopted in practice because they are computationally intensive, and also because of uncertainties associated with the modelling of infill frames under seismic loading. Design codes address this issue by permitting simplified analysis of soft storey buildings. Linear analysis of the 'bare frame' (neglecting infill wall stiffness) is allowed, but the column forces (bending moments and shear forces) in the soft storey need to be magnified by specified factors in design. For example, IS 1893 (2002) recommends a magnification factor of 2.5 to be applied on bending moments and shear forces calculated for the bare frame under seismic loads. Similarly other national codes, such as EC-8, UBC and Israeli codes etc specify magnification factors of similar orders. The conservative nature of these empirical recommendations has been pointed out by Subramanian (2004), Kanitkar and Kanitkar (2004) and Kaushik et. al. (2006). These recommendations have met with some resistance in design and construction practice, due to the need for heavy reinforcement in the column, leading to congestion.

Detailed linear analysis of OGS buildings requires modelling of infill wall stiffness. Studies based on such analysis (Davis et. al. (2007) and Fardis and Panagiotakos (1997)) have shown that, in comparison with the 'bare frame' analysis, the OGS building frame has following implications.

- The lateral stiffness of the building frame increases
- The fundamental time period decreases
- The base shear demand increases
- The fundamental mode shape is significantly altered
- Higher curvatures are induced in ground storey columns

- Shear forces and bending moments in the ground storey columns increases

However, as it is well known, the OGS building frame (designed with response reduction factor) is expected to behave nonlinearly in the event of the design earthquake. At ultimate loads this involves formation of plastic hinges in the frame. For collapse prevention, the OGS building is expected to have adequate capacity to dissipate the input energy by undergoing large inelastic deformations in the frame members and infill walls. Magnification factors for the ground storey columns in OGS building should ideally be based on the results of such nonlinear analysis. However, as mentioned earlier this is computationally difficult and needs considerable research. On the basis of detailed parametric studies, a proper basis for the magnification factor can be explored. At present not much research is reported in the literature to validate the accuracy of the magnification factors given in various codes regarding OGS buildings. The present paper summarises the results of recent study carried out at IIT Madras.

This paper reports the assessment of MF by both linear response spectrum analysis and nonlinear dynamic analysis for two typical OGS building frames, (i) four storeyed building with five bays and (ii) a seven storeyed building with three bays, with various infill arrangements. The frames are designed for both vertical and lateral loads.

2. EXAMPLE BUILDING FRAMES

The five storeyed-five bay OGS frame (plane frame) considered is located in Indian seismic zone IV. The PGA of this zone is specified as 0.24*g* for medium soil conditions. The building is designed and detailed as per IS 1893 (2002) and IS 456 (2000) code provisions. The grade of steel used is Fe415 and that of concrete is M25. The building is symmetric in plan, and hence a single plane frame may be considered to be representative of the building along one direction. The design base shear (V_B) is calculated as per IS 1893 (2002),

$$V_B = \left(\frac{z \times I \times {S_a}/{g}}{2 \times R} \right) W \tag{1}$$

where seismic zone factor, z = 0.24, Importance factor I =1.0, $\left({S_a}/{g} \right)$ = 2.5 for time period $T = 0.075H^{0.75} = 0.51s$, building height H = 12.8 m and seismic weight, W=1540 kN. The dead load of the slab, including floor finishes, is taken as 2.5 kN/m^2 and live load as 3 kN/m^2. The unit weight of brick masonry infill is taken as 18 kN/m^3. The response reduction factor in this case is taken as 3. Accordingly, the design base shear works out to 10% of seismic weight of the building frame, i.e. V_B.= 154 kN.

The elevation of the bare frame and the regular OGS frames are shown in Figs. 1a and 1b respectively. All the upper storeys may not have full infill walls owing to the presence of large openings. The stiffness of infill walls shall be neglected if the opening percentage exceeds 40% (Murty and Jain 2005). Figure 1c, labelled OGS-A1, refers to a frame with three of the upper storey panels having area of opening of 50%. Similarly, Fig. 1d shows OGS-A2, having five panels with area of opening of 50%.

The seven storeyed-three bay OGS frame (plane frame) considered is taken from Hashmi and Madan (2008). The building is located in Indian seismic zone IV. The PGA of this zone is specified as 0.24*g* for medium soil conditions. The grade of steel used is Fe415 and that of concrete is M35. More details of the frame are given in Hashmi and Madan (2008).The elevations of the seven storeyed bare frame and OGS frames are shown in Fig. 2 using the same nomenclature as in Fig. 1.

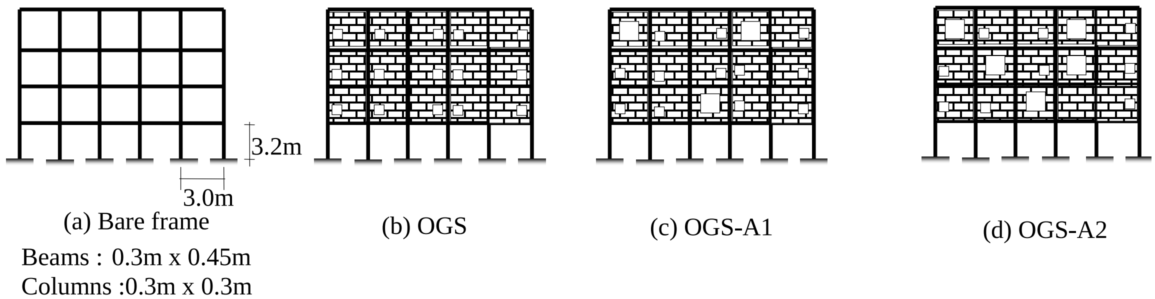

Figure 1 Details of the four storey bare frame and OGS frames

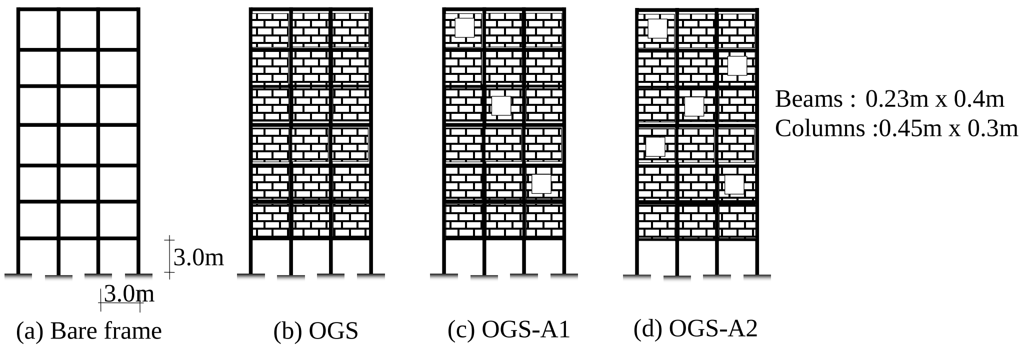

Figure 2 Details of the seven storey bare frame and OGS frames

3. MODELLING OF INFILL WALLS FOR LINEAR AND NONLINEAR ANALYSES

3.1. Modelling for Linear Analysis

Based on the study of available literature on various infill wall models for linear analysis, conducted by Asokan (2006), it can be seen that the model introduced by Smith and Carter (1961), based on the concept of equivalent diagonal strut, is fairly accurate for linear analysis of the infill wall. The same model enables the estimation of the strut width for the elastic analysis, considering failure modes in shear cracking and corner crushing.

3.2. Modelling for Nonlinear Analysis

Modelling of infill walls for nonlinear analysis (dynamic) has generated research interest for quite some time, and there are many models available in literature. Fardis and Panagiotakos (1997), Madan et. al. (1997), Crisafulli (1999) are some of the common infill wall hysteretic models reported. These models are different from one another and use equivalent strut approach. The model introduced by Crisafulli (1999) is used in the present study. It is based on a multi-strut model, comprising four diagonal truss elements to model diagonal compression and two shear springs to account for shear. Strength and stiffness degradation effects are considered in both the diagonals, along with shear springs.

Nonlinear analysis requires parameters such as modulus of elasticity, compressive strength, tensile strength, strain at maximum stress, ultimate strain, shear bond strength, maximum shear resistance, friction coefficient, equivalent contact length, strut area reduction factor (to account for decrease in width of the strut as the load in the strut increases).

The parameters given by Crisafulli (1999) model are modified based on various studies for the bricks available in Indian conditions. The ultimate strain of the brick infill walls is taken based on the experiments conducted by Kaushik (2006), in which various types of brick masonry units are tested to find the stress - strain curves. Although the strength and ultimate strain of brick masonry available in India shows wide variation, average value of strain is considered for the nonlinear analysis in this study.

The width of the equivalent strut for the model proposed by Crisafulli (1999) uses simplified expressions. But the width of the equivalent strut for the nonlinear analysis shall be based on ultimate load approach as per the studies conducted by Asokan (2006). Hence an ultimate load based model, Saneinjad and Hobbs (1995), is used to find out the width of the strut.

3.3. Validation of Infill Wall Model for Monotonic Load

Validation of nonlinear infill wall model and the frame element nonlinear model is done by simulating the experiments done by Achintya et.al (1991), Mehrabi et. al. (1996), Alchar et. al. (2002), Choubay and Sinha (1994). Monotonically increasing lateral load is applied at the top left point of the portal frame fixed at the bottom, until failure. Comparison of experimental and computational results shows that the model can predict the collapse load and deformation reasonably accurately. Details are given in Davis (2008).

3.4. Validation of Infill Wall Model using Psuedo-Dynamic Tests

The hysteretic model for infill wall and the frame elements is validated by simulating the pseudo dynamic test conducted at ELSA, test reported in literature. A uniformly infilled four storeyed frame (3D) reported by Negro and Verzeletti (1996) and a bare frame (plane) reported by Pinho and Elnashai (2000) are modelled using the program 'Seismostruct'. The RC frames are modelled using fibre element, in which the sectional state of stress-strain is arrived by integration of uniaxial material responses of the individual fibres, which takes into account the spread of plasticity. Constitutive model introduced by Mander et. al.(1988) is used for RC elements and the hysteretic rule is based on Rueda and Elnashai (1997).
The same input displacement history, as applied in the pseudo dynamic experiment, is applied to all floors. Static time history analysis of the computational model is carried out and the base shear versus the time is plotted. The base shear time history plot from the computational model is compared with experimentally obtained base shear for the two four-storeyed uniformly infilled frame and bare frame, as shown in Fig. 3a and 3b respectively. The comparison shows that the computational model for both infill walls and the frame elements could predict the behaviour fairly well and hence the same models are used for nonlinear dynamic analysis of OGS buildings. Further information on the validation of nonlinear models for dynamic analysis is reported by Smyrou (2006).

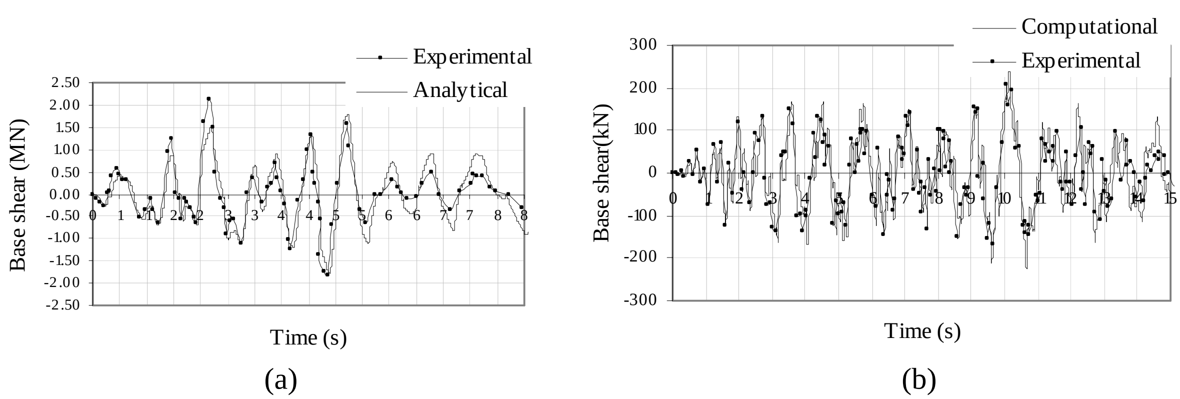

Figure 3 Comparison of base shear versus time

4. RESPONSE SPECTRUM ANALYSIS (RSA)

Response spectrum analysis of the all building models is carried out to find the magnification factors. Stiffness of infill walls in each bay is calculated as discussed in section 3.1. Truss elements are provided diagonally as equivalent struts wherever infill wall is present in the OGS models. RSA is done for the response spectrum corresponding to medium soil for 5% damping as per IS 1893 (2002), for all the frames.

The modulus of elasticity for the bricks found India varies between 350 MPa (table moulded) and 5000 MPa (wire-cut), as reported by Asokan (2006). To represent the extreme cases of strong and weak infill walls, two combinations of infill walls are considered for modelling. The thicker wall, (230mm thick) is combined with strong infill wall (*E* = 5000 MPa) and thinner wall, (115mm thick) is combined with weak infill wall (*E* = 350 MPa). The designations, OGS-230-5000 and OGS-115-350, are thus refer to the OGS frame model corresponding to these two cases.

The base shears from the RSA for the two building frames for various models (Bare, OGS-230-5000, OGS-A1-230-5000, OGS-A2-230-5000, OGS-115-350, OGS-A1-115-350, and OGS-A2-115-350) are given in Table 1. The base shear demands from RSA for OGS frames are found to be higher than that of the bare frame in both the cases. The magnification factors in each case are also listed (shown in parenthesis).

Table 1 Base shears and magnification factors from RSA

(a) four storey building frame

Model		Bare	OGS-230-5000	OGS-A1 230-5000	OGS-A2 230-5000	OGS-115-350	OGS-A1 115-350	OGS-A2 115-350
Base shear (kN)		136.22	152.44 (1.12)[a]	152.40 (1.12)	152.04 (1.12)	141.82 (1.04)	141.72 (1.04)	141.26 (1.04)
Maximum BM in GS (kNm)	Interior Column	36.66	39.58 (1.08)	39.72 (1.08)	39.62 (1.08)	37.83 (1.03)	37.87 (1.03)	37.72 (1.03)
	Exterior Column	33.09	37.30 (1.13)	37.30 (1.13)	37.19 (1.12)	34.56 (1.04)	34.52 (1.04)	34.36 (1.04)
Fundamental Time period (s)		0.56	0.41	0.42	0.42	0.50	0.53	0.53

(b) seven storey building frame

Model		Bare	OGS-230-5000	OGS-A1 230-5000	OGS-A2 230-5000	OGS-115-350	OGS-A1 115-350	OGS-A2 115-350
Base shear (kN)		111.22	266.42 (2.39)[a]	257.44 (2.31)	254.32 (2.29)	134.30 (1.21)	131.38 (1.18)	129.58 (1.17)
Maximum BM in GS (kNm)	Interior Column	60.85	118.16 (1.94)	114.06(1.87)	112.57 (1.85)	68.63 (1.13)	67.14 (1.10)	66.21 (1.09)
	Exterior Column	53.63	118.67 (2.21)	106.85(1.99)	105.52 (1.97)	61.59 (1.15)	60.30 (1.13)	59.47 (1.11)
Fundamental Time period (s)		1.48	0.69	0.71	0.71	1.22	1.25	1.27

[a] magnification factor, value (Base shear/BM etc) divided by corresponding value for the bare frame

The base shear demand for the model with 230mm thick infill walls is higher than that with 115mm thick infill wall for a given infill wall arrangement by only 7.5% in the four storey case and by as much as 50% for seven storey case. It can be seen that as the thickness of infill walls decreases, the base shear also decreases for a given infill wall arrangement. The OGS frame with infill wall arrangement A2, having relatively less number of effective infill walls, compared to that of A1, attracts less base shear. The magnification factor is found to increase as the thickness and strength of the infill increases. The maximum magnification factor is found to be in the range, 1.04-1.13 for the four storey building, 2.39-1.11 for the seven storey building, both for base shear and ground storey column bending moments. The magnification factor is primarily attributable to the shift in fundamental time period of the OGS building frame compared to a similar bare frame (Davis et. al. 2007). The higher magnification in the seven storey building is due to higher shift in time period due to stiffening effect of infill walls in seven storey building compared to four storeyed building.

5. NONLINEAR DYNAMIC ANLYSIS

Nonlinear dynamic analysis (NDA) is conducted for the same frame models (four storeyed and seven storeyed) to estimate the magnification factor considering nonlinearities in the infill walls and frame elements. Four number of spectrum compatible time histories are generated which matches with IS 1893 (2002) response spectra for medium soil as shown in Fig. 4. Nonlinear dynamic analysis is done for the PGA of 0.12*g*, which is three times more than the PGA at the expected yield. The average of the maximum values of base shear and bending moments in the ground storey columns for four time history data (A, B, C and D) are generated, as depicted in Table 2.

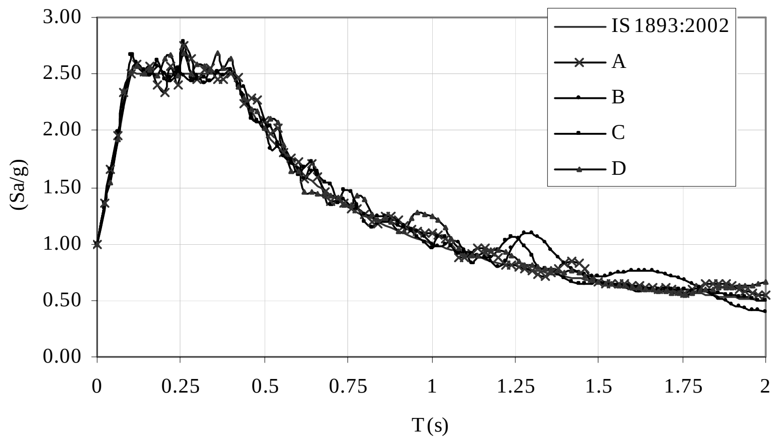

Figure 4 IS 1893-2002 Response spectrum compatible time histories

Table 2 Base shears and magnification factors from NDA
(a) four storey building frame

Model		Bare	OGS-230-5000	OGS-A1 230-5000	OGS-A2 230-5000	OGS-115-350	OGS-A1 115-350	OGS-A2 115-350
Base shear (kN)		130.00	71.42 (0.55) [a]	72.47(0.56)	73.67 (0.57)	93.41 (0.72)	94.02 (0.72)	95.12 (0.73)
Maximum BM in GS (kNm)	Interior Column	35.25	18.47 (0.52)	18.77 (0.53)	19.12 (0.54)	26.59 (0.75)	25.84 (0.73)	26.15 (0.74)
	Exterior Column	31.31	15.99 (0.51)	16.32 (0.52)	16.66 (0.53)	23.44 (0.75)	22.89 (0.73)	23.00 (0.73)

(b) seven storey building frame

Model		Bare	OGS-230-5000	OGS-A1 230-5000	OGS-A2 230-5000	OGS-115-350	OGS-A1 115-350	OGS-A2 115-350
Base shear (kN)		74.30	95.92 (1.29)[a]	94.42(1.27)	93.3 (1.26)	92.04 (1.24)	87.48 (1.18)	84.89 (1.14)
Maximum BM in GS (kNm)	Interior Column	71.98	47.45 (0.66)	46.23 (0.64)	48.22 (0.67))	55.97 (0.78)	65.72 (0.91)	62.90 (0.87)
	Exterior Column	82.22	56.81 (0.69)	56.94 (0.69)	58.71 (0.71)	70.10 (0.85)	81.35 (0.99)	78.88 (0.96)

[a] magnification factor, value (Base shear/BM etc) divided by corresponding value for the bare frame

The results of the NDA reveal that the magnification factors are much less, compared to the results of the linear RSA. Infact, surprisingly, it is observed that for the four storeyed building frame the MF is less than unity, implying that when the nonlinear and hysteretic effects are accounted for at ultimate loads, the OGS frame attracts less base shear than the bare frame.

In the case of seven storeyed building, it is seen that the MF applicable for base shear is in the range 1.14 to 1.29, the lower values being applicable when large openings in the infill walls are provided. However, the corresponding MF, applicable for bending moments in the ground storey column turn out to be less than unity.

6. CONCLUSIONS

The OGS building frame is generally considered to be more vulnerable than a corresponding 'bare frame' under seismic loads, and this can be established by carrying out linear dynamic analysis. Design codes, such as IS 1893 (2002), recommend a magnification factor of 2.5 to be applied on the calculated column shear forces and bending moments in the ground storey, based on simple bare frame analysis. In the present study it is shown that the MF based on linear dynamic analysis is in the range 1.04-1.13 for a four storeyed OGS building and 1.11-2.39 for seven storeyed building frame. The MF increases with the height of the building, primarily due to the higher shift in the time period. Also it is seen that when large openings are present and thickness of infills are less, there is a reduction in MF. However the results of NDA, including hysteresis effects in frame and infill, suggest that there is no need for applying MF to low-rise building frames. In the case of the seven-storeyed building frame, values of MF in the range 1.14 to 1.29 were observed applicable to base shear. However, this is not applicable to column bending moments, where MF values were found to be less than unity. The MF obtained from nonlinear analysis is much less than that generated from linear dynamic analysis. Further studies based on NDA of OGS frames are required to validate the results.

REFERENCES

1. Subramanian N. (2004). Discussion on seismic performance of conventional multi-storey building with open ground floors for vehicular parking by Kanitkar and Kanitkar. *The Indian Concrete Journal.* **78:4**, 11-13.
2. Kanitkar, R., and Kanitkar, V. (2004) Seismic performance of conventional multi-storey buildings with open ground floors for vehicular parking. *The Indian Concrete Journal.* **78:2**, 99-104.
3. Kaushik, H. (2006). Strengthening schemes for open ground storey buildings. *PhD. Thesis* Indian Institute of Technology, Kanpur.
4. IS 1893 (Part I): 2002 — Criteria for Earthquake Design of Structures – Part I: General Provisions and Buildings (Fifth revision). *Bureau of Indian Standards*. New Delhi.
5. Smith, S.B. and Carter, C. (1969). A method of analysis for infilled frames. *Proceedings of Institution of Civil Engineers* Part 2. 44, 31-48.
6. Asokan A. (2006) Modelling of masonry infill walls for nonlinear static analysis of buildings under seismic loads. *MS Thesis.* Indian Institute of Technology Madras. Chennai.
7. Fardis, M.N. and Panagiotakos, T.B. (1997). Seismic design and response of bare and masonry-infilled reinforced concrete buildings. *Journal of Earthquake Engineering*. **1:3**, 475-503.
8. Madan, A., Reinhorn, A.M., Mander, J.B. and Valles, R.E. (1997). Modelling of Masonry Infill Panels for Structural Analysis. *ASCE Journal of Structural Engineering.* **123**, 1295-1301.
9. Crisafulli, F.J. (1997). Seismic behavior of reinforced concrete structures with masonry infills. *PhD. Thesis.* University of Canterbury. New zealand.
10. SeismoSoft [2007] SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures [online]. Available from URL: http://www.seismosoft.com.
11. Saneinejad, A., and Hobbs, B. (1995). Inelastic design of infilled frames. *Journal of Structural*

Engineering. **121**, 634-650.
12. Achintya, P., Dayaratnam and Jain, S.K. (1991). Behaviour of brick infilled RC frame under lateral load. *Indian Concrete Journal*. **65,** 453–457
13. Mehrabi, A.B., Shing, P.B., Schuller, M.P. and Noland, J.L. (1996). Experimental evaluation of masonry infilled RC frames. *ASCE Journal of Structural Engineering*. **122,** 228-237
14. Al-Chaar, G., Issa, M. and Sweeney, S. (2002). Behaviour of masonry infilled non-ductile reinforced concrete frames. ASCE Journal of Structural Engineering. **128,** 1055-1063.
15. Choubey, U.B. and Sinha, S. N. (1994). Cyclic response of infilled frames. *Journal of Structural Engineering*. Structural Engineering Research Centre. **21,** 203-211.
16. Mander J.B., Priestley M.J.N. and Park R. (1988). Theoretical stress-strain model for confined concrete, *Journal of Structural Engineering.* **114:8,** 1804-1826.
17. Negro, P. and Verzeletti, G. (1996). Effect of infills on the global behaviour of R/C frames: energy considerations from pseudodynamic tests. *Earthquake Engineering and Structural Dynamics*, **25,** 753-773.
18. Pinho, H. and Elnashai, A.S. (2000) Dynamic collapse testing of a full-scale four storey RC frame. *ISET Journal of Earthquake Technology.* Paper No.406. **37:4**, 143-164.
19. Smyrou E. (2006) Implementation and verification of a masonry panel model for nonlinear dynamic analysis of infilled RC frames. *MSc Dissertation.* European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), Pavia, Italy
20. Rueda, M.J.E. and Elnashai, A.S. (1997). Confined concrete model under cyclic load. *Materials and Structures.* **30:197**, 139-147.
21. Crisafulli, F.J. (1997). Seismic behavior of reinforced concrete structures with masonry infills. *PhD. Thesis*, University of Canterbury, New zealand.
22. Davis, R., Menon, D. and Prasad, A.M. (2007) Alternate lateral load profile for aseismic design of open ground storey buildings. *Proceedings of 9th Canadian Conference on Earthquake Engineering.* Ottawa. Canada. Paper No. 1209.
23. Murty C.V.R. and Jain S.K. (2005) Proposed draft provisions and commentary on Indian seismic code IS 1893 (Part I): 2002. *IITK-GSDMA project on Building Codes*, Indian Institute of Technology. Kanpur.
24. Davis, R. (2008) Seismic resistant design of open ground storeyed building. *PhD. Thesis*, IIT Madras, Chennai.