Seismic Performance Assessment of Open Ground StoreyBuildings

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SUMMARY:
Open ground storey (OGS) buildings are vulnerable to earthquake excitation due to vertical irregularity caused by the differential stiffness distribution due to the presence of the infill. Conventionally, the infill is not modelled analytically. Also, the performance assessment studies implicitly take care of the failure modes through a set of performance criteria. This study attempts to simulate directly the sidesway collapse and thus, follows closely the methodology given in FEMA P695. It includes the representative design as per IS: 1893, nonlinear modelling of the building,nonlinear dynamic analysis to simulate collapse, assessment of uncertainties and probabilistic evaluation of collapse performance. Brittle failures like shear failures are incorporated indirectly in terms of limit state checks as required by the methodology for non-simulated failure modes. The collapse probability is estimated by the use of collapse fragility curves.

Keywords: seismic assessment, FEMA P695, soft storey, collapse probability

1. INTRODUCTION

Open Ground Storey (OGS) buildings are the popular building typology seen in the urban Indian scenario as they provide the much-needed parking space to the users. Due to their inherent seismic vulnerability because of vertical irregularity, much of the damage occurs in the first few stories of such buildings. This study attempts to investigate the vulnerability of OGS building from the context of collapse prevention. Current practice in design and codes relate the collapse safety through an acceptable value of storey or roof drift. Such approach undermines the redistribution of the damage throughout the structure during the earthquake duration.

Simulating structural collapse in sidesway mode due to earthquakes using nonlinear dynamic analysis, has been explored in the recent works of Haselton (2007) and Lignos(2008). Here the collapse is defined as the inability of the structure to withstand the gravity load due to dynamic instability triggered by large storey drifts. Incremental dynamic analysis (IDA),which precludes the numerical non-convergence, is usually employed as the tool to trace the dynamic instability. The direct simulation of collapse requires a robust structural model incorporating all possible deterioration modes and a robust solution algorithm that avoids the numerical non-convergence before the dynamic instability. Here a hybrid approach has been envisaged wherein the fibre model is utilized to capture the spread of the inelasticity with the shear capacity of the ground storey columns as the performance check and follows the FEMA P695 methodology.

2.FEMA P695 METHODOLOGY

Although developed as a tool to establish seismic performance factors for generic seismic-force-resisting systems, Appendix F of FEMA P695 proposes its use for collapse assessment of an individual building system. The methodology is based on the concept of collapse level ground motions, defined as the level of ground motions that cause median collapse. For a building to meet the collapse performance objectives, the median collapse capacity must be an acceptable ratio above the Maximum Considered Earthquake (MCE) ground motion demand level.

By starting with an acceptable collapse probability for MCE ground motions and working backwards, collapse margin ratio (CMR) can be calculated to determine the ground motion intensity corresponding to median collapse. By scaling the record set to this level, the individual building can be assessed for its intended performance. If the analytical model survives one-half or more of the records without collapse, then the building has a collapse probability that is equal to or less than the acceptable collapse probability for MCE ground motions and meets the collapse performance objectives of the methodology (FEMA P695, 2009).

3. NONLINEAR MODEL

Seismostruct has been used throughout the study for developing nonlinear analytical models for the example building (Figures 3.1, 3.2).In Seismostruct, fibre approach is made use of to represent the cross-sectional behavior, where each fibre is associated with a uniaxial stress-strain relationship; the sectional stress-strain state of beam-column elements is then obtained through the integration of the nonlinear stress-strain response of individual fibres with which the section has been discretized (Seismostruct, 2007). Both force-based (infrmFB) and displacement-based (infrmDB) formulations are available in the program to simulate inelastic behaviour of the beam-column elements. Here we have chosen displacement formulation for all the elements. Each element is assigned five integration points along its length where the nonlinear axial-flexural behaviour of the cross-section is monitored. The fibres in each cross-section are assigned material properties to represent unconfined concrete, confined concrete and the steel reinforcement. Here Mander's nonlinear model has been chosen to represent both confined and unconfined concrete whereas a bilinear model is assigned for steel reinforcement. The main advantages of the fibre include the ability to capture axial-flexural interaction and the effects of concrete tensile strength and tension stiffening along with user-friendly inputs without extensive calibration. However, it does not take into account the bond-slip flexibility and reinforcing bar buckling which may contribute to larger degradation in the case of direct simulation of collapse. Skyline solver method and Hilber-Hughes-Taylor integration scheme has been used for dynamic time history analysis. Seismostruct uses Crisafulli's model to represent nonlinear response of infill panels.

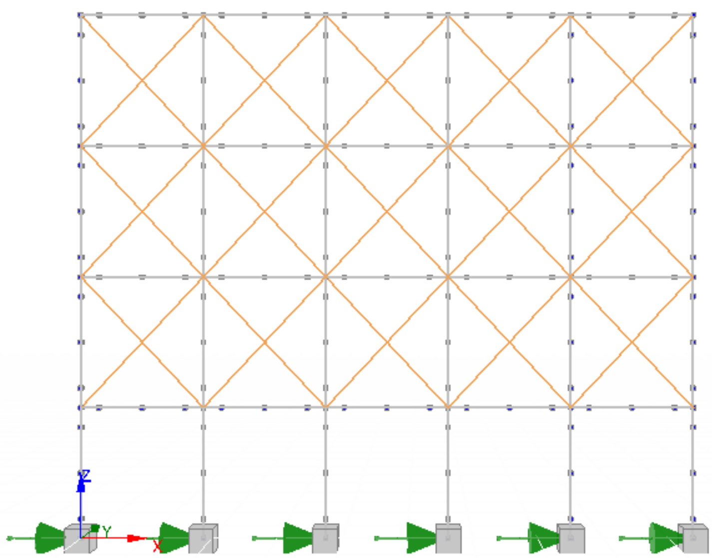

Figure 3.1 Analytical model with base excitation

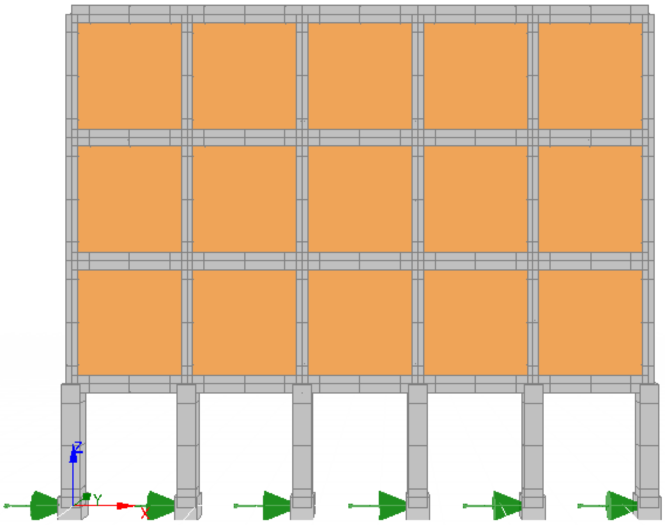

Figure 3.2 Example Building

Table 3.1 Example Building Details

Frame	Bay width(m)	Storey height(m)	Ground storey column size(mm)	% of reinforcement	Shear capacity of ground storey columns (kN)	Fundamental period (s)
4storey 5bay (case 1)	3.2	3.0	300 × 300	2.79	109.0	0.44
4storey 5bay (case 2)	3.2	3.0	500 × 500	1.51	345.0	0.25

4.LIMIT STATES AND ACCEPTANCE CRITERIA

A simulated collapse can be identified by a drift limit at which it is not possible for the building to be stable and this can be found from the pushover analysis results. In cases where it is not possible to directly simulate all major deteriorating modes leading to collapse, FEMA P695 prescribes to evaluate the non-simulated modes through alternative limit state checks. Since shear failure is not explicitly modelled, we consider the shear capacity of the ground storey columns as limit state check in addition to the roof drift limits.

In FEMA P695 methodology, it is suggested that the probability of collapse due to Maximum Considered Earthquake (MCE) ground motions be limited to 10%. Also, a limit of 20% is suggested as a criterion for the acceptability for potential outliers. However, it should be noted that these limits are based on judgment.

5. TOTAL SYSTEM UNCERTAINTY AND ACCEPTABLE COLLAPSE MARGIN RATIO

Many sources of uncertainty contribute towards the variability in collapse capacity. These include: (a) record-to-record variability, (b) design requirement uncertainty, (c) test data uncertainty, and (d) modelling uncertainty. Quality of test data has been assumed *fair* and the quality of design requirement is assumed *good*. Record-to-record variability and hence, the total system collapse uncertainty depends on period-based ductility. FEMA P695 specifies the value of total system uncertainty for various combination of quality ratings of test data and design requirements for period-based ductility values greater than 3.

Collapse margin ratio (CMR) is the ratio between median collapse intensity and the MCE intensity, which is the primary parameter used to characterize the collapse safety of the structure. Collapse capacity can be significantly influenced by the frequency content of the ground motion record set and hence, CMR is adjusted using a factor called Spectral Shape Factor (SSF). SSF is a function of fundamental building period, period based ductility and the seismic design category (SDC). Since the concept of SDC is unique to ASCE 7 and it is difficult to find the equivalence in IS 1893, it is assumed that the example building design confirms to SDC D_{min}category. Acceptable values of adjusted collapse margin ratio (ACMR) are based on total system uncertainty and are specified in FEMA P695. It is assumed there that the distribution of collapse level spectral intensities is lognormal.

6. PUSHOVER ANALYSIS

A nonlinear static (pushover) analysis is performed to quantify $V_{max}$and δ_u(Figure.6.1), which are then used to compute period based ductility, μ_T.

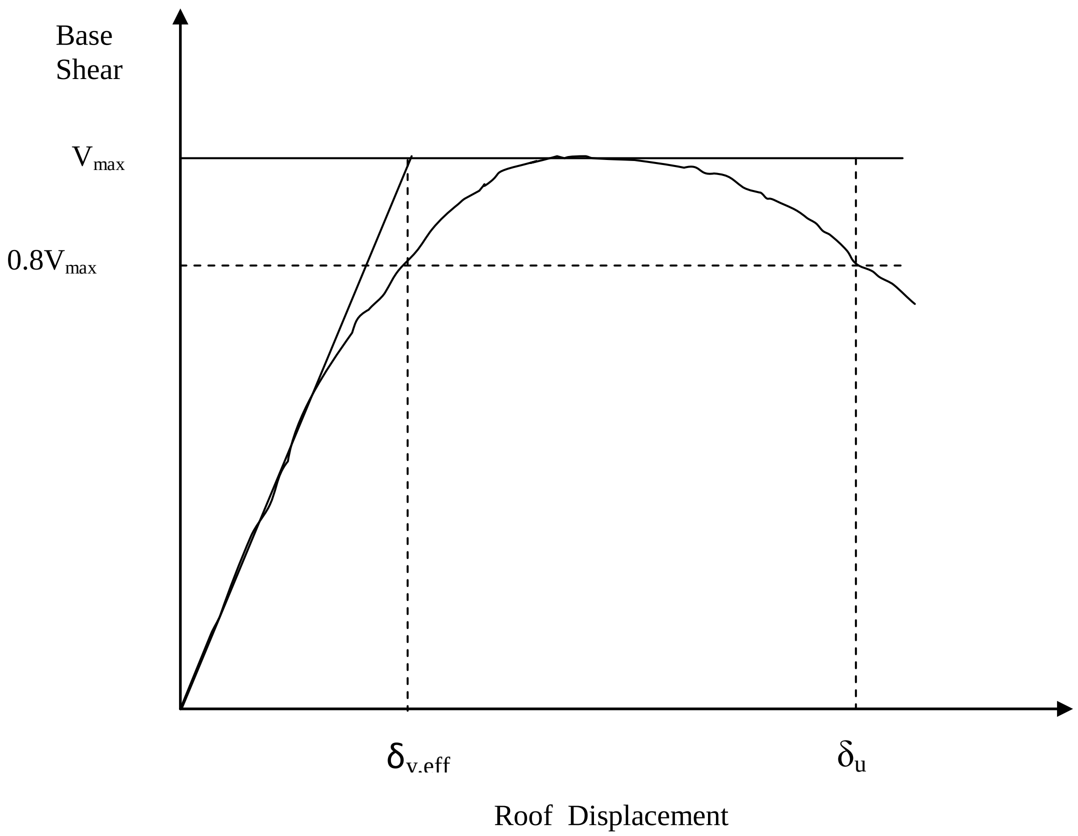

Figure 6.1FEMA P695 definitions

The period based ductility is defined as the ratio of ultimate roof drift displacement to the effective yield roof drift displacement, $\delta_{y,eff}$:

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{6.1}$$

The effective yield roof drift displacement is given by the following formula:

$$\delta_{y,eff} = C_0 \frac{V_{\max}}{W} \frac{g}{4\Pi^2} \left(\max(T, T_1)\right)^2 \tag{6.2}$$

where

$$C_0 = \phi_{1,r} \frac{\sum_1^N m_x \phi_{1,x}}{\sum_1^N m_x \phi^2_{1,x}} \tag{6.3}$$

where m_x is the mass at level x, $\phi_{1,x}$ is the ordinate of the fundamental mode at level x and N is the number of levels. T is the fundamental period defined in the Code and T_1 is the fundamental period found from eigen-value analysis.

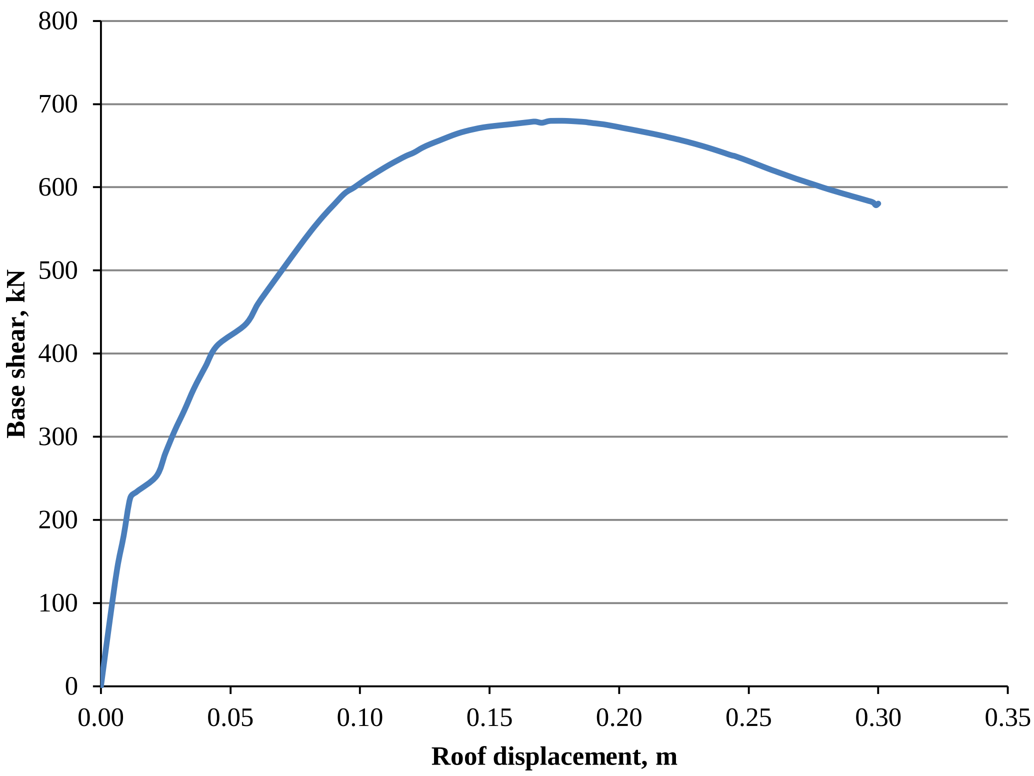

Figure 6.2 Pushover curve for example building (case 1)

7. GROUND MOTION RECORD SET AND SCALING

FEMA P695 requires a set of records that can be used for nonlinear dynamic analysis of buildings. We have used the Far Field record set of this methodology for the evaluation purpose. These records are broadly applicable to variety of structures and do not depend on the building-specific properties. Also the record set does not depend on the site condition and source mechanism.

Ground motion records are scaled to represent a specific intensity (here, collapse intensity). Individual records are normalized by their peak ground velocity to remove inherent variability between records without eliminating overall record-to-record variability. Then the normalized records are scaled up by multiplying with the scaling factor (SF) to the required collapse level intensity at which one-half of the records should not cause collapse of the building.

The scaling factor, SF is computed as follows:

$$SF = \frac{ACMR_{10\%}}{C_{3D}\ SSF}\ \frac{S_{MT}}{S_{NRT}} \tag{7.1}$$

where
$ACMR_{10\%}$ is the accepted value of the adjusted collapse margin ratio corresponding to an acceptable collapse probability of 10%.
C_{3D} is a coefficient taken 1.2 for three-dimensional analysis and 1.0 for two-dimensional analysis.
S_{MT} is the MCE, 5% damped, spectral acceleration at the fundamental period, *T* of the building.
S_{NRT} is the median value of the normalized record set, 5% damped, spectral acceleration at the fundamental period, *T*.

8. PERFORMANCE EVALUATION AND COLLAPSE FRAGILITY

Seismostruct treats incremental dynamic analysis as dynamic pushover analysis and hence, the output is similar to that of pushover analysis. Since the example building is first mode dominated structure (effective modal mass more than 90%), S_a is approximately estimated from (eq. 8.1):

$$V = \alpha_1 S_a W \tag{8.1}$$

where
α_1 = effective modal mass coefficient
S_a = spectral acceleration corresponding to first mode
W = inertial weight of the building
V = base shear

Shear failure in the ground storey columns is not modelledexplicitly, rather a performance criteria has been followed to check the intended failure. Since shear failure in any of the ground storey columns may lead to global instability, the OGS building is assumed to be a series-type of system wherein all or any column failure in the ground storey is defined as the collapse. It is found that in almost all cases, the collapse occurs due to the shear failure before reaching the ultimate roof drift. Also, it can be seen from the figure 8.1 that complete collapse occursat the MCE level ground motion, which is contrary to the objectives of the FEMA P695methodology (Case 1).

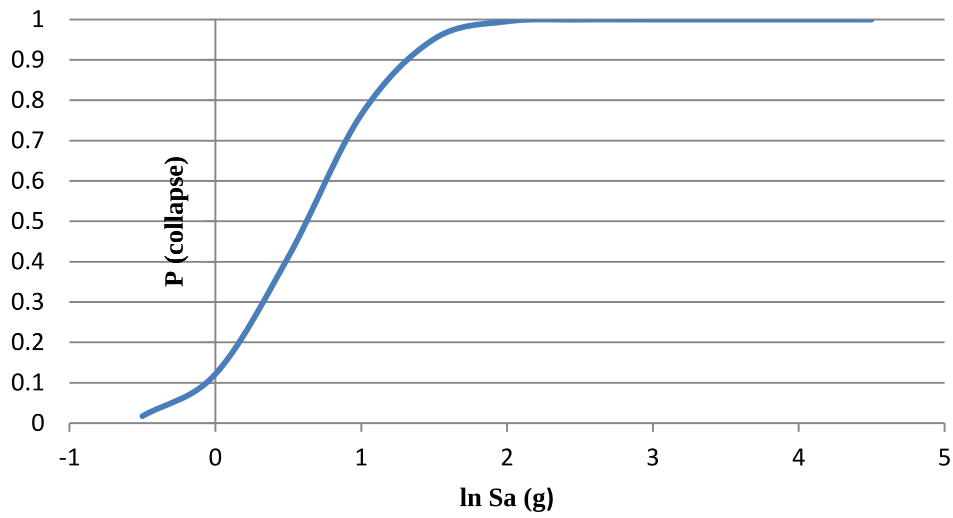

Figure 8.1 Collapse fragility curve, Case 1

IS 1893 recommends the capacity of columns in the soft storey be enhanced by 2.5 times of the seismic demand. Accordingly, the example building is modified and incremental dynamic analysis was performed till collapse. It is found that in very few cases, the upper storey columns fail due to the strengthening of the ground storey columns. It is observed (figure 8.2) that the collapse fragility curve becomes steeper with the mean shifted rightwards and the collapse probability at MCE level intensity improves significantly (Case 2).

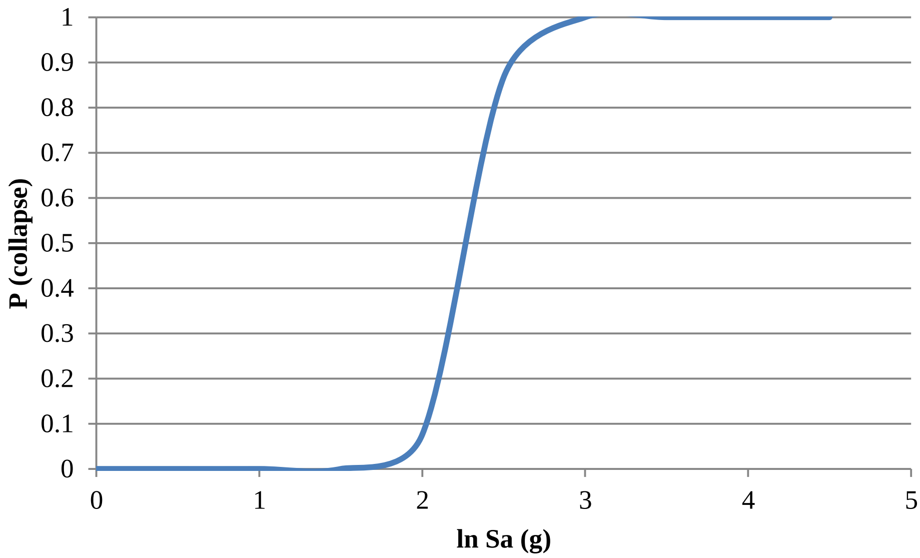

Figure 8.2 Collapse fragility curve, Case 2

9. CONCLUSIONS

The example building design as per IS 1893 (2002) does not meet the collapse probability objectives of FEMA P695. This could be due to the definition of collapse assumed or due to the analytical model used for this study. Further study is underway related to the detailed modelling aspects and for a different level of severity of the ground motion intensity.

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