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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 filled 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 in filled 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-V. The design forces for the ground storey columns are evaluated based on various codes such as Indian, Euro, Israel, Bulgarian codes and Kaushik et.al (2009) suggested approach. Various OGS frames are designed considering MFas1.0, 2.1 (Israel), 2.5 (Indian), 3.0 (Bulgarian), 3.79 (Kaushik et. al, 2009) and 4.68 (Euro). The performance of each building is studied using the fragility analysis method introduced by Cornell et. al (2002). Uncertainty in concrete, steel and masonry walls are accounted. Thirty computational models are developed in the program Seismo struct (2012) for nonlinear dynamics analysis for each case. For the analysis, a set of thirty natural time histories is selected and modified to match the Response spectrum as per Indian code (IS 1893-2002). In the present study, fragility curves are generated for each building, by developing a Probabilistic Seismic Demand Model (PSDM) according to power law. The relative performances of each building designed as per various codes are compared using fragility curves. It is found that as MF increases the exceedance probability of inter-storey drift at the ground of OGS buildings decreases. Out of all the OGS frames considered, OGS frames designed using MF as 1.0 is found to be the most vulnerable. It is also found that the application of magnification factor only to the ground storey may lead to increase in the inter- storey drift demand in the adjacent storey.

INTRODUCTION

Overview

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 (Figure 1.1) 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.

Upper stories of these buildings are stiff and the inter-storey drifts will be small, resulting in large curvatures, shear force sand bending moments of the ground storey columns. Hence, the strength demand on the columns in the ground storey of the buildings is very high. The majority of this type of buildings had collapsed in the past earthquakes in many countries. The failure of OGS buildings is observed to be due to storey mechanism in the ground storey. The sudden reduction in lateral stiffness and mass in the ground storey results in higher stresses in the ground storey columns under seismic loading. In most cases, ground-story columns were either damaged severely or failed completely, there by damaging the buildings. 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. Figure 1.2 distinguishes the behaviour of a full in filled frame and a OGS building during the Bhuj earthquake (2001). It can be seen that the building which is on the left has survived with minor cracks in the infill walls in the ground storey. The building on the right side is an OGS frame, completely collapsed due to soft-storey mechanism in the ground storey due to the absence of infill walls.

The OGS framed building behaves differently as compared to that of a bare framed building (without any infill) or a fully in filled framed building under lateral load. Global lateral stiffness of a bare frame is much less than that of a fully in filled frame; it resists the applied lateral load through frame action and shows well-distributed plastic hinges at failure. When the frame is fully in filled, 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 in filled 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 in filled 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 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 in filled framed building. Due to the increase of global stiffness, the base shear demand on the building increases. In the case of typical in filled 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-Δ 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 1.3. 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 in filled 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 infill's in the ground storey and plastic hinges will be formed.

MULTIPLICATION FACTOR (MF) PROVISIONSIN VARIOUS CODES

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. The various code recommendation is to magnify the bending moments and shear forces of bare frame for the columns in the soft/open storey by MF

Indian standardsIS-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 2.5times 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 underseismic 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 storey's 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 storey's above. Stilts or open ground storey buildings fall under extreme soft-storey type of vertically irregular buildings.

Euro Code 8 EN 1998-1:2003

Euro Code have not suggested to check criteria of vertical irregularity, as in of other codes. *Eurocode 8* (2003) recommends increasing the resistance of columns in the less in filled storey in proportion to the amount of deficit in strength of masonry infill (MI). If there is a drastic reduction of infill walls in any storey compared to the adjoining storeys, seismic forces in the less in filled storey (ground storey of OGS building) shall be increased by a multiplication factor (MF). However, further research (Fardis and Panagiotakos 1997) has shown that increasing the beam resistance would further increase the seismic demands on the columns, thus seismic design forces in only columns are increased by a factor as follows,

$$\left[1+\frac{\Delta V_{RW}}{\sum V_{Ed}}\right] \leq q \tag{1.1}$$

where ΔV_{RW} is the total reduction of the lateral resistance of MI in the ground storey compared to that in the upper storey, As there is no infill wall in the ground storey of an OGS building , ΔV_{RW} is equal to the resistance of masonry in the first storey itself and $\sum V_{Ed}$ is the sum of seismic shear forces acting on all structural vertical elements of the storey concerned. The term q is called behaviour factor, which accounts for energy dissipation capacity of the structure and the value varies from 1.5 to 4.68 depending upon the type of building systems, ductility classes, and plan regularity in the building. The maximum vertical irregularities allowed by *Eurocode 8* (2003) in buildings are such that q is never more than 4.68, which is larger than the factor 2.5 given in the Indian code (IS-1893:2002). Also, q is applied only to columns of the soft story, whereas in the Indian code, both beams and columns of the soft story are required to be designed for increased forces. *Eurocode 8* (2003) does not clearly mention whether the buildings with open ground story are permitted; it only restricts the value of q.

Bulgarian Code(1987)
According to the Bulgarian code (1987), members of the soft stories (story stiffness less than half the stiffness of the adjacent stories) are required to be designed for increased forces by introducing a coefficient while calculating the design forces. The value of coefficient for *regular* RC frames with MI is 0.3 as compared to a value of 0.2 for the bare frames, and the coefficient for the RC frames with a soft story is 0.6. Therefore, it recommends the seismic design forces for soft storey in MI-RC frames are required to be increased by two times the corresponding design forces for a regularly in filled frame, and by three times the design seismic forces for a regular bare frame.

Israel CodeSII-413(1995)
According to Israel code SII-413 (1995) a storey is considered as a soft storey, if the lateral stiffness is less than 70% of that of the storey above, or less than 80% of average stiffness of three storeys above, and which contains less than half the length of the infill walls, as compared to the storey above it, in at least one of its principal directions. A weak storey is defined as a storey if the lateral shear capacity in any direction is less than 80% of that of the storey above in the same direction.

This code allows soft or weak storey, including open ground storey, only in buildings with low or medium ductility levels. The design forces for flexible or weak storey members, and for the members in the storey above and below, are required to be increased by a factor 0.6R, where R is the response reduction factor. For masonry in filled RC frame buildings, R is 3.5 for low ductility level, and 5.0 for medium ductility level. Therefore, the beams and columns of the soft/weak storey, and also the adjacent storeys are required to be designed for at least 2.1-3.0times the design forces for regular storey, depending upon the level of ductility.

NEED FOR THE PRESENT STUDY

The multiplication factors proposed by selected international codes and recent research works are not consistent as discussed in previous sections. The performance of the buildings designed by the various MFs proposed by the international codes may be different.

Objective
To study the seismic performance of typical OGS buildings designed as per applicable provisions in international codes in a Probabilistic Frame Work

- Indian
- Euro
- Bulgarian
- Israel

1. To develop Probabilistic Seismic Demand Model for the designed buildings
2. To develop fragility curves for the designed OGS buildings

METHODOLOGY

Various steps to be followed to achieve the objectives are given below.

Step 1 : Select a ten storey six bay frame
Step2:Design the frame asperIS456 andIS1893
Step3:Develop Fragility curves for the designed frame sasper Cornellet. al(2002)
Step 4 : Building performance levels are considered using FEMA – 356
Step5:Analyse the fragility curves obtained to draw the conclusions

ORGANIZATION OF THE THESIS

This introductory chapter (Chapter 1) gives a brief introduction to the inconsistency in MF values in International Codes with regard to design of OGS buildings. Chapter 2discussesthe literature review on various studies conducted on OGS building and also in filled frame buildings in general and some studies on Fragility curves. Chapters 3 discuss the development of fragility curves. Chapter 4 presents the performance assessment of the selected building using fragility curves. Finally, a summary of the present study and the conclusions are given in Chapter 5.

LITERATURE REVIEW

INTRODUCTION

The literature review is divided into two parts. The first part of this Chapter deals with an overview of seismic behaviour of infill walls and open ground storey building. The second part of this chapter deals with the Previous Studies on the development of Seismic Fragility Curves.

SEISMIC BEHAVIOUR OF INFILLWALLS 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 in filled 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 in filled 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 in filled frame was studied by Riddington and Smith (1997).

Scarlet (1997) studied the qualification of seismic forces in OGS buildings. A multiplication factor for base shear for OGS building was proposed. This procedure requires modelling the stiffness of the infill walls in the analysis. The study proposed a multiplication factor ranging from 1.86 to 3.28 as the number of storey increases from six to twenty.

Deodhar and Patel (1998) pointed out that even though the brick masonry in in filled frame are intended to be non-structural, they can have considerable influence on the lateral response of the building.

Davis and Menon (2004) concluded that the presence of masonry infill panels modifies the structural force distribution significantly in an OGS building. The total storey shear force increases as the stiffness of the building increases in the presence of masonry infill at the upper floor of the building. Also, the bending moments in the ground floor columns increase (more than two fold), and the mode of failure is by soft storey mechanism (formation of hinges in ground floor columns).

Das and Murthy (2004) concluded that infill walls, when present in a structure, generally bring down the damage suffered by the RC framed members of a fully in filled frame during earthquake shaking. The columns, beams and infill walls of lower stories are more vulnerable to damage than those in upper stories.

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 approach along with the proposed hinge property provides a better estimate of the inelastic drift of the building.

Kaushik (2006) indicated that the multiplying factor 2.5 given in the IS 1893 (2002) is specified for all buildings with soft-storeys, irrespective of the extent of irregularities and the proposal is quite empirical.

A study was carried out, based on various proposed strengthening schemes for the ground storey of OGS building frames, followed by pushover analysis.

Hashmi and Madan (2008) conducted non-linear time history and pushover analysis of OGS buildings. The study concludes that the MF prescribed by IS 1893 (2002) for such buildings is adequate for preventing collapse.

Sattar and Abbie (2010) in their study concluded that the pushover analysis showed an increase in initial stiffness, strength, and energy dissipation of the in filled frame, compared to the bare frame, despite the wall's brittle failure modes. Likewise, dynamic analysis results indicated that fully-in filled frame has the lowest collapse risk and the bare frames were found to be the most vulnerable to earthquake-induced collapse. The better collapse performance of fully-in filled frames was associated with the larger strength and energy dissipation of the system, associated with the added walls.

Patel(2012) conducted both linear(Equivalent Static Analysis and Response Spectrum Analysis) and nonlinear analyses (Pushover Analysis and Time History Analysis) for Low-rise open ground storey framed building with infill wall stiffness as an equivalent diagonal strut model. And, the analysis results shows that a factor of 2.5 is too high to be multiplied to the beam and column forces of the ground storey of low-rise open ground storey buildings. Their study concluded that the problem of open ground storey buildings cannot be identified properly through elastic analysis as the stiffness of open ground storey building and a similar bare-frame building are almost same.

STUDIES ON THE DEVELOPMENT OF SEISMIC FRAGILITY CURVES

Fragility curves are the conditional probability of excee dance of response of a structure for a given ground motion intensity. Fragility curves are used commonly for the estimation of probability of structural damage due to earthquakes as a function of ground motion indices or other design parameters. Some of these studies, based on analytical methods, are presented in the following section.

Singhal & Kiremidjian [1995] developed the vulnerability curves and damage probability matrices for low, mid and high-rise RC framed structures using the Park and Ang damage index. The ground motion characterization parameters used are the spectral acceleration and the root mean square acceleration. Nonlinear dynamic analyses were performed. Constrained Monte Carlo simulation techniques were used for evaluating the fragility curves.

Mosalam et al. (1997) developed vulnerability curves for low-rise bare and in filled RC frames designed for gravity loads. Pushover analyses were performed, assuming variability of concrete, steel and masonry properties, in order to obtain trilinear capacity curves. The characteristic values were assumed to follow a lognormal distribution with assumed coefficients of variation. Nonlinear analysis of the trilinear SDOF systems were performed for 800 artificial accelerogramsThe Monte Carlo technique was employed to sample 200 capacity curves foreach accelerogram. Reasonable agreement was found with fragility curves obtained from the ATC-13 damage probability matrices (better agreement for low levels of damage).

Masanobu Shinozuka et. al (2000) has studied the fragility curves of a bridge by two different analytical approaches; one of them is the time-history analysis and the other uses the capacity spectrum method. The latter approach is one of the simplified nonlinear static procedures recently developed for buildings. In this respect, a sample of 10 nominally identical but statistically different bridges and 80 ground-motion time histories are considered to account for the uncertainties related to the structural capacity and ground motion, respectively. The comparison of fragility curves by the nonlinear static procedure with those by time-history analysis indicates that the agreement is excellent for the state of at least minor damage, but not as good for the state of major damage where nonlinear effects clearly play a crucial role. Overall, however, the agreement is adequate even in the state of major damage considering the large number of typical assumptions under which the analyses of fragility characteristics are performed.

Cornell et. al (2002) developed a probabilistic framework for seismic design and assessment of structures and applied to steel moment-resisting frame buildings. This framework was based on realizing a performance objective, expressed as the probability of exceeding a specified performance level for the structure. Performance levels described the desired level of structural behavior in terms of generic structural variables, demand and capacity. Demand and capacity were represented by an explicitly nonlinear, dynamic, and displacement-based structural response, the maximum inter storey drift ratio. This provided an analytical expression for the probability of exceeding the performance level as the primary product of framework development.

Christiana Dymiotis (2001) has focussed on the probabilistic assessment of reinforced concrete (RC) frames in filled with clay brick walls and subjected to earthquake loading is carried out dynamic, inelastic time-history analyses of 2D frame models using DRAIN-2D/90. The vulnerability and seismic reliability of two 10-storey 3 bay in filled frames (a fully in filled one and one with a soft ground story) are derived and subsequently compared the values with corresponding to the bare frame. It is found that failure probabilities, especially at the ultimate limit state, are highly sensitive to the structural stiffness; hence, bare frames benefit from lower spectral ordinates than in filled ones.

Kappos et al. (2003), within the RISK-UE project, applied the capacity spectrum method on several configurations of regular RC buildings with and without in fills (the case of soft ground storey was also examined) and with different levels of seismic design. As the capacity spectrum method assumes a bilinear response, when the displacement demand is higher than the capacity of regularly in filled frames buildings with good seismic design, it was suggested to use the capacity curve for the bare frame. The uncertainty in the definition of damage state and the variability of the capacity were taken from HAZUS. The dispersion for all damage states of a given structural class was the mean of the dispersions for each damage state, so as to avoid intersecting fragility curves. Reference was made to the cost of replacement and to a damage index. The vulnerability curves were developed following the hybrid method, where analytical and observational capacity curves are combined.

Vacareanu et al (2004) focused on the seismic vulnerability assessment of representative residential RC buildings in Bucharest using HAZUS and ATC-40 methodology. The buildings were designed with low-level and medium-level seismic codes. A relationship was established between inter storey drift and Park & Ang damage index. The demand corresponded to a single recorded accelerogram. Monte-Carlo simulation was used to calibrate the fragility function parameters.

Akkar et al (2005) presented vulnerability curves for low-rise and mid-rise in filled frame RC buildings. Pushover analyses of 32 existing buildings in Duzce were performed to define the intervals of base shear capacity, period and ultimate drift of 2, 3, 4 and 5-storey buildings with low-level of seismic design. Nonlinear dynamic analyses were then performed for 82 recorded accelero grams and bilinear structures with properties within the identified intervals. The number of storeys was found to have a significant effect on the probability of exceeding the moderateand the severe damage limit states. Spectral displacement correlated better with peak ground velocity (PGV) than peak ground acceleration (PGA) , particularly for higher levels of damage. There was good agreement of the vulnerability curves with observed damage after the 1999 Duzce earthquake.

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 asto 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 studyto 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.

DEVELOPMENT OF FRAGILITY CURVES

INTRODUCTION

The first part of this chapter deals with developing the fragility curves using Cornell et. al (2002) by taking the Engineering Demand Parameter (*EDP*) versus intensity measure (*IM*). This has been done by considering 30 models of 10storey6bay,each model have different material properties like concrete f_{ck} ,steel f_y, masonry f_m. This can be done by sampling and to confirm results obtained from dynamic time history analysis for thirty selected models are carried out by selecting 30 different ground motions.

The second part of this chapter deals with the selection of ground motions and convertinginto Indian Spectrum and all of these are far field have been selected which is explained in this chapter. Next building performance levels have been considered according to FEMA- 356.

DEVELOPMENT OF FRAGILITY CURVES

It is imperative to resort to the analytical fragility curves, with the scarcity of post earthquake reconnaissance data available for the reliable estimate of the vulnerability. The fragility function represents the probability of exceedance of the selected Engineering Demand Parameter (*EDP*) for a selected structural limit state (*DS*) for a specific ground motion intensity measure (*IM*). Fragility curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state (DS) or a more severe one, as a function of a particular demand.

PROBABILISTIC SEISMIC DEMAND MODEL(PSDM)

It has been suggested by Cornellet.al(2002)that the estimate of the median engineering demand parameter (*EDP*) can be represented bya power law model as given in Eq. 3.4.

$$\widehat{EDP}=(IM)^b \qquad (3.4)$$

In this present study, inter-storey drift (δ) at the first floor level (ground storey drift) is taken as the engineering damage parameter (*EDP*) and peak ground acceleration (*PGA*) as the intensity measure (*IM*) , ‘a’ and ‘b’ are regression coefficents.

SAMPLING

Sampling is concerned with the selection of a subset of individuals from within a population to estimate characteristics of the whole population. Material Properties of concrete, steel and masonry used in the construction are not going to be the same. It’s going to be different in nature because of its making process, environment condition, workman ship and etc.So, whileanalysing the structure it is not good way to consider same Compressive strength throughout the study. Hence to incorporate the uncertainties in concrete, steel and masonry sampling is required. Broadly sampling is divided into two parts:(i) Probability Sampling Method and (ii)Non-Probability Sampling Method

LATIN HYPER CUBE SAMPLING (LHS)

The techniques of random sampling are more powerful and useful for performing probabilistic analyses. However, in some case, the problem being analyzed is extremely complex, and the time needed to evaluate the problem for a single trial (N=1) may be very long. As a result, the time needed to perform hundred or thousand of simulation may be unfeasible.

In 1979 McKay, Beckman and Conover proposed Latin hypercube sampling as an attractive alternative to simpler and omsampling in computer experiments. Latin hypercube sampling(LHS) is astatistical method for generating a distribution of plausible collections of parameter values from a multidimensional distribution. The Latin hypercube method is one technique for reducing the number of simulations needed to obtain reasonable results. In this method, the range of possible values of each random input variable is partitioned into “strata” and a value from each stratum is randomly selected as a representative value. The representative values for each randomare then combined so that each representative value is considered once and only once in the simulation process. In this way, all possible values of the random variables are represented in the simulation.

GROUND MOTION DATA

The number of ground motions required for anunbiased estimate of the structural response is 3 or 7 as per ASCE 7-05. However, ATC 58 50% draft recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. ASCE/SEI 41 (2005) suggests 30 recorded ground motions to meet the spectral matching criteria for NPP infrastructures. A set of thirty Far-Field Ground Motion Sets are collected from Haselton and Deierlein (2007).

Selected ground motion consists of strong motions that may cause structural collapse of modern buildings. This typically occurs at extremely large levels of ground motion, so this ground motion set was selected to represent these extreme motions to the extent possible. To ensure that the records represent strong motion that may cause structural collapse, we imposed minimum limits on event magnitude, as well as peak ground velocity (PGV) and acceleration. The limits were chosen to balance selection of large motions, while ensuring that enough motions will meet the selection criteria.

i. Magnitude>6.5 in Richter Scale
ii. Distance from source to site > 10km
iii. Peak ground acceleration (PGA)>0.2gand peak ground velocity(PGV)>15 cm/sec.
Soil shear wave velocity,in upper 30m of soil,greaterthan180m/s
iv. Limit of six records from a single seismic event, if more than six records pass the initial criteria, then the six records with largest PGV are selected, but in some cases a lower PGV record is used if the PGA is much larger.
v. Lowest useable frequency<0.25Hz,to ensure that the low frequency content was not removed by the ground motion filtering process
vi. Strike-slip and thrust faults(consistent with California)

Operational Level

Operational level is the lowest level of overall damage to the building (highest performance). The structure will retain nearly all of its pre-earthquake strength and stiffness. Expected damage includes minor cracking of facades, partitions, and ceilings, in addition as structural components. All mechanical, electrical, plumbing, and other systems necessary for normal operation of the buildings are expected to be functional, possibly from standby sources. Negligible damage to non structural components is expected. Under very low levels of earthquake ground motion, most buildings ought to be able to meet or exceed this performance level. Typically, however, it will not be economically practical to design for this level of performance under severe levels of ground shaking, except for buildings that house essential services.

Immediate Occupancy Level(IO)

Overall damage to the building is light. Damage to the structural systems is similar to the Operational Performance Level. However, somewhat more damage to non-structural systems is expected. Non-structural components such as cladding and ceilings, and mechanical and electrical components remain secured; however, repair and cleanup may be needed. It is expected that utilities necessary for normal function of all systems will not be available, although those necessary for life safety systems would be provided. Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake ground motion. In addition, some owners may desire such performance for very important buildings, under severe levels of earthquake ground shaking. This level provides most of the protection obtained under the Operational Building Performance Level, without the associated cost of providing standby utilities and performing rigorous seismic qualification to validate equipment performance.

Life Safety Level(LS)

Structural and non-structural damage is significant. The building may lose a substantial amount of its pre-earthquake lateral strength and stiffness, but the gravity-load bearing elements function. Out-of-plane wall failures and tipping of parapets are not expected, but there will be some permanent drift and select elements of the lateral-force resisting system may have substantial cracking, spalling , yielding, and buckling. Non-structural components are secured and not presenting a falling hazard, but many architectural, mechanical, and electrical systems are damaged. The building may not be safe for continued occupancy until repairs are done. Repair of the structure is feasible, but it may not be economically attractive to do so. This performance level is generally the basis for the intent of code compliance.

Collapse Prevention Level or Near Collapse Level(CP)

The structure sustains severe damage. The lateral-force resisting system loses most of its pre- earthquake strength and stiffness. Load-bearing columns and walls function, but the building is near collapse. Substantial degradation of structural elements occurs, including extensive cracking and spalling of masonry and concrete elements, and buckling and fracture of steel elements. In fills and unbraced parapets may fail and exits may be blocked. The building has large permanent drifts. Non-structural components experience substantial damage and maybe falling hazards. The building is unsafe for occupancy. Repair and restoration is probably not practically achievable. This building performance level has been selected as the basis for mandatory seismic rehabilitation ordinances enacted by some municipalities, as it results in mitigation of the most severe life-safety hazards at relatively low cost.

Performance Indicators

Inter-story drift ratio is considered in the present study as a measure of structural demands because it can be related to performance levels of reinforced concrete buildings as per FEMA356:2000. In the Present study three performance levels are adopted to derive fragility curves, namely Immediate Occupancy (IO), Life safety (LS) and collapse Prevention (CP) levels.

These three performance levels have been widely used in the earthquake engineering community and can be compared or calibrated with various other sources.Table3.1 shows the Damage limits for Reinforced concrete frames for various structural performance levels as per FEMA 356 (2000).

DEVELOPMENT OF FRAGILITY CURVES FLOW CHART

The flow chart represents show the fragility curves are drawn using Cornellet. al(2002) method how the procedure.

SUMMARY

In this chapter the procedure of development of fragility curves as per Cornell et. al(2002) is explained. This method includes sampling the random variables (characteristic strength of materials), selection of ground motions and modification of the ground motion to a spectrum compatible data. This chapter also describe about the building performance levels considered in the study. A flowchart is also shown that illustrate the entire procedure for developing the fragility curves using the method proposed by Cornell et. al (2002).

SEISMIC PERFORMANCE OF TYPICAL OPEN GROUND STORY FRAMED BUILDING

INTRODUCTION

This chapter deals with the seismic performance of typical open ground storey 2-D frame using fragility analysis. First part presents design details selected frame, multiplicationfactors adopted for various codes and sections and reinforcement detailing. Second part deals with the development of fragility curve which includes sampling of material strengths, selection and modification of ground motions, development of 30 frames models fornonlinear dynamics analysis, Probabilistic Seismic Demand Model.

DETAILS OF CASE STUDY BUILDINGS

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 3mand 3.2mrespectively. 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 building considered is located in seismic zone Vhaving Z = 0.36 and medium soil is considered and in the analysis R value considered as 3 for ordinary RC moment resisting frame (OMRF).

DEVELOPMENT OF FRAGILITY CURVES

Fragility curves are developed as per the methodology explained in Chapter 3. The following sections explain the details of the process.

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 variables, 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.

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 Seismo Struct (2007).
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 from15 MPa to 45MPa. 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
$f_t = k_t\sqrt{f_c}$, 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. The default is 1.2.

(5) specific weight–□
This is the specific weight of the material. The default value is 24kN/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 andItsvaluevariesfrom230 MPa upto 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)/(ε_{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. The default value is 0.005.
(4) 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 is78kN/m^3.

Brick masonry

The infill walls were modelled as equivalent diagonal strut introduced by Crisafulli (1997). Brick masonry is modelled as inelastic infill panel element. A four-node masonry panel element, developed and initially programmed by Crisafulli (1997) and implemented in Seismo Struct by Blandon (2005), for the modelling of the nonlinear response of infill panels in framed structures. Each panel is represented by six strut members, each diagonal direction features two parallel struts to carry axial loads across two opposite diagonal corners and a third one to carry the shear from the top to the bottom of the panel. This latter strut only acts across the diagonal that is on compression. Hence its" activation" depends on the deformation of the panel. The axial load struts use the masonry strut hysteresis model, while the shear strut uses a dedicated bilinear hysteresis rule.

Also as can be observed in the Figure.4.4, four internal nodes are employed to account for the actual points of contact between the frame and the infill panel (i.e. to account for the width and height of the columns and beams, respectively), whilst four dummy nodes are introduced with the objective of accounting for the contact length between the frame and the infill panel. All the internal forces are transformed to the exterior four nodes (which, as noted here, need to be defined in anti-clockwise sequence) where the element is connected to the frame.

Following parameters are to be defined for this type of element:

(1) Infill Panel Thickness-t
(2) Equivalent contact length-h_z
(3) Strut Areas-A
(4) Specific weight-□
(5) Horizontal and Vertical offsets–Xoi and Yoi
(6) Proportion of stiffness assigned to shear-□$_s$

Analysis

Dynamic analysis is commonly used to predict the nonlinear inelastic response of a structure subjected to earthquake loading. In nonlinear dynamic analysis (NDA), a numerical direct integration scheme must be employed in order to solve the system of equations of motion. In Seismo Struct, such integration can be carried out by means of two different implicit integration algorithms. (i) Newmark integration scheme and (ii) Hilber-Hughes-Taylor integration algorithm. Here, Hilber-Huges-Taylor integration method is used to solve the non-linear dynamic analysis and the solver used is Skyline solver. Rayleigh damping model is used with 3% damping in the first mode and 5% damping in the third mode.

PROBABILISTIC SEISMIC DEMAND MODEL (PSDM)

It has been suggested by Cornell et. al (2002) that the estimate of the median engineering demand parameter (*EDP*) can be represented by a power law model as given in Eq. 4.1 .

$$\widehat{EDP}=(IM)^b \tag{4.1}$$

In this present study, inter-storey drift (δ) at the first floor level (ground storey drift) is taken as the engineering damage parameter (*EDP*) and peak ground acceleration (*PGA*) as the intensity measure (*IM*).

The nonlinear dynamic analyses are used to build the PSDM. Nonlinear time history analyses of all the thirty statistically equivalent analytical models have been performed to obtain a set of thirty inter-storey drifts(□) for the corresponding PGAs. The parameters '*a*'and '*b*' of the Eq. 4.1 are determined for the set of thirty values by performing a regression analysis using power-law. The demand models for each frame is obtained using linear regression analysis and the generated model are as shown in Figure 4.5. The parameters 'a' and 'b' of PSDM models of all the frames are shown in the Table 4.4.The inter-storey drift at the ground storey is more for the OGS 1.0 and as the MF increases the inter-storey drift also reduces. The inter-storey drift follow the order, OGS 1.0,Israel(MF=2.1), OGS 2.5, Bulgarian(MF= 3.0), Kaushik et. al, 2009 (MF = 3.97) and Euro (MF =4.68), with the OGS 1.0 having the highest inter-storey drift.

It is found that the inter-storey drift at ground storey of OGS frame designed using MF = 2.5 is reduced by 80% compared to that of OGS frame designed using MF=1.0.Similarly, with reference to OGS frame designed using MF =1.0, the inter-storey drift at ground storey is reduced by 66% for frame designed using MF = 2.1, 83.3% for frame designed with MF = 3.0, 94..6 % for frame with MF = 3.97 and 96% for frame designed with MF = 4.68.

COMPARISON OF FRAGILITY CURVES

The PSDM models are used for generating fragility curves of each building frame as per the methodology discussed in Chapter 3. The PSDM models and corresponding fragilities are presented in the Figures 4.6 to 4.11.

The application of multiplication factors increases the strength and stiffness of the ground storey columns. It is observed from Figure. 4.6 that the exccedance probability for a PGA of 3gofthe OGS frame designed with MF= 1.0is 77% for IO performance level, about 9% for LS level and close to 0 % for CP level. Ground storey columns have been multiplied by 2.5 times of B.M and S.F of b are fare and the ground storey columns have increased their column sections. It can be seen from Figure. 4.7, that the performance of the frame (probability of exceedance of inter-storey drift decreased) is increased when compared to the building designed with MF=1.

COMPARISON OF FRAGILITY CURVE FOR EACH STOREYS FOR DIFFERENT CODES

A Comparison of fragility curve for each storey for different codes is made to understand the behaviour further more. Figure.4.18 represents the fragility curve of ground storey for various codes. As the Israel code uses the MF factor of 2.1, the resulting fragility is more at ground storey compared to that of other codes.

Figure.4.19 represents the fragility curve of first storey which shows that the probability of exceedance of inter-storey drift is same for all the codes except for Israel code. 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.

SUMMARY

The seismic performance assessment of typical open ground storey2-D frames designed with Multiplication factors as per various codes is carried out with the help of fragility curves. A method introduced by Cornell et. al (2002) is used in the present study for fragility curve development. The PSDM models are developed for each frames selected. It is found that as MF increases the inter-storey drift at the ground storey reduces.

The inter-storey drift for OGS 1.0 is found to be the largest. The inter-storey drift decreases for the building frames in the order, OGS 1.0, Israel (MF =2.1), OGS 2.5, Bulgarian (MF = 3.0), Kaushik et. al, 2009 (MF = 3.97) and Euro (MF =4.68).

SUMMARY AND CONCLUSIONS

SUMMARY

Open ground storey buildings are considered as the vertically irregular buildings as per IS 1893: 2002 .In the present study, a typical ten storied OGS framed building is considered and the building considered is located in Seismic Zone-V. The design forces for the ground storey columns are evaluated based on various codes such as Indian, Euro, Israel, Bulgarian codes and Kaushik et. al(2009) suggested approach. Various OGS frames are designed considering MF as 1.0, 2.1 (Israel), 2.5 (Indian), 3.0 (Bulgarian), 3.79 (Kaushik et. al, 2009) and 4.68 (Euro).

The performance of each building is studied using the fragility analysis method introduced by Cornell et. al (2002). Uncertainty in concrete, steel and masonry walls are accounted. Thirty computational models are developed in the program Seismo struct (2012) for nonlinear dynamics analysis for each case.

For the analysis, a set of thirty natural time histories is selected and modified to match the Response spectrum as per Indian code (IS 1893-2002). In the present study, fragility curves are generated for each building, by developing a Probabilistic Seismic Demand Model (PSDM) according to power law. The relative performances of each building designed as per various codes are compared using fragility curves.

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.
- Un certainties 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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