

PROBABILISTIC EVALUATION OF COLUMN OVERDESIGN FACTOR FOR
STEEL FRAME STRUCTURES

MOHAMMAD SHARFUDDIN

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In seismic design concept, the design philosophy of “weak beam strong column” has been widely accepted by researchers and designers. In this concept, it is assumed that yielding of all beams in flexure will occur prior to possible yielding of columns which is considered to be the preferable failure mode, because of its large capacity to absorb earthquake energy before the structure actually collapses. To ensure that a frame structure collapses according to the preferable beam-hinging pattern, the columns of the structure that receive forces from the beams of the structure are generally designed with a column overdesign factor (COF) greater than one to make the columns relatively stronger than the beams.

Various COF requirements have been addressed by different structural codes. However, recent major earthquake disasters and case studies by researchers have shown that the formation of plastic hinges in columns cannot be avoided even though the structures were designed to present provisions. One major reason incurring such a phenomenon is the uncertainties existing in the structural members and earthquake loads. Therefore, It is difficult to absolutely ensure that the structure will collapse according to the preferable failure mode; a better strategy may be the probabilistic approach, i.e., to ensure an occurrence probability of the preferable failure mode larger than the probabilities of the undesirable modes. The COF ensuring probabilistic priority of the beam hinging failure mode to story mechanisms needs to be determined. Target values of COF that probabilistically ensure the preferable entire beam hinging failure mode of frames have been evaluated in this study following some code prescribed load distributions. This will guide the engineers to select the minimum values of COF for frame structures under specific reliability level to probabilistically avoid the undesirable story collapse modes during earthquakes.

The thesis consists of five chapters. Chapter 1 describes the background of the present study as described above. The remaining part of the thesis is briefly described below.

Chapter 2 deals with the probabilistic investigation on the story mechanisms of moment resisting steel frame structures. The failure modes of the multistory ductile frame structures grouping into three types: upper story collapse, middle story collapse

and lower story collapse are investigated probabilistically applying first order reliability method (FORM). It is observed that under any specific reliability level, the mean value of the load is generally a linear function of COF. The failure probabilities of the middle story and upper story collapse modes follow some specific pattern but the failure probabilities of the lower story collapse modes do not follow any specific pattern. In case of upper story collapse modes, it is observed that the failure probability steadily increase with the increase of the number of failure stories. In case of middle story collapse modes, it is observed that the failure probability with higher n_b (number of unbroken story at the bottom of the frame) is less than that with lower n_b . The value of COF has a significant effect on the probabilistic order of the lower story failure modes. Each lower story mechanism has a special COF region in which the failure probability of that mode is the largest. Among all the failure modes of a multi-story ductile frame structure the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability, i.e., these modes are the most likely failure modes of a multi-story ductile frame structure. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006. That means, most likely failure modes are independent of the type of the distribution applied in the evaluation.

Chapter 3 deals with the probabilistic evaluation of the target COF that probabilistically ensures the preferable entire beam hinging failure mode and probabilistically avoids the undesirable story collapse modes of the frames, applying First order reliability method (FORM). The target COF is the minimum value of COF that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame. The target values of COF for three to seven story frames under reliability levels ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) based on A_i distribution and the distribution of UBC-94 and IBC-2006 are presented in chapter three. Under the same reliability level the target COF requirement increases with the increase of the number of stories and decreases with the increase of the reliability level. It is observed that the rate of change of target COF with number of story and reliability level is almost linear. Target COF requirement for frames with height and mass irregularity are evaluated. It is observed that higher COF value has to be provided for frames with higher floor height in first story. The higher the height irregularity due to story height variation in the first story, the higher is the target COF requirement. The target COF requirement also increases with the increase of the mass

irregularity, i.e. the higher the mass irregularity, the higher the target COF requirement. The target COF requirement further increases when both the height and mass irregularity is combined in the same frame, i.e., COF requirement of this frame is higher than the frame with only height irregularity or only mass irregularity.

Chapter 4 introduces the evaluation of target COF considering system reliability. The system reliability of the frames are evaluated applying Dimension Reduction Integration (DRI) method. The method directly calculates the reliability indices (and associated failure probabilities) based on the first few moments of the system performance function of a structure. It does not require the reliability analysis of the individual failure modes; also, it does not need the iterative computation of derivatives, or the computation of the mutual correlations among the failure modes, and does not require any design points. Thus, this method should be more effective for the system reliability evaluation of complex structures than currently available methods. The accuracy of results obtained with DRI method has been thoroughly examined by comparisons with large sample Monte Carlo simulations (MCS). It is observed that the results of DRI show good agreement with that of MCS. The target values of COF for three to seven story frames under reliability levels 2 and 3 ($\beta_T=2$, and $\beta_T=3$) based on A_i distribution of Japan are presented in chapter four. It is observed that under the same reliability level the target COF requirement increases with the increase of the number of stories and decreases with the increase of the reliability level. However, target COF based on likely modes and that of system reliability is different. That means, target COF requirement is affected by the evaluation methods.

In chapter five a brief summary of the present research is presented.

PROBABILISTIC EVALUATION OF COLUMN OVERDESIGN FACTOR FOR STEEL FRAME STRUCTURES

ABSTRACT

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Chapter 1

INTRODUCTION

1.1 Research Background

Uncertainties are ubiquitous in structural engineering. Civil engineering structures are to be designed for loads created by environmental actions like earthquakes and wind. These actions are exceptionally uncertain in their manifestations. Materials used in civil engineering constructions also display wide scatter in their engineering properties. As an engineer, it is therefore important to recognize the presence of all major source of uncertainty in engineering. The source of uncertainty may be classified into broad types:

- Those that is associated with natural randomness. For example randomness of loads such as wind, earthquake, snow, ice, water pressure, or live load. This type of uncertainty is usually considered as the *aleatory* type of uncertainty.
- Those that is associated with inaccuracies in our prediction and estimation of reality. For example approximations during design phase, calculation errors, omissions, lack of knowledge. This type of uncertainty is usually considered as the *epistemic* type of uncertainty.

The subject of structural reliability offers a rational framework to quantify uncertainties mathematically. The subject combines theories of probability, random variables and random processes with principles of structural mechanics and forms the basis on which modern structural design codes are developed. Examples include the American Institute of Steel Construction Load and Resistance Factor Design (LRFD), code for steel buildings (AISC, 1986; 1994), American association of State Highway and Transportation Officials LRFD code (AASHTO, 1994; 1998), and many European codes (e.g., CEC, 1984).

Structural reliability analysis enable to perform more rational risk evaluations: they are an alternative approach to traditional deterministic structural design for taking account of all the uncertainties affecting the parameters characterizing the physical state of the structure and its environment. It makes it easier to achieve either of the following goals:

- For a given cost, design a more reliable structure.
- For a given reliability, design a more economic structure.

It is, therefore, considered as a promising research area, both for theoretical developments and for practical applications. In the last decades it has been increasingly applied in many aspects of engineering.

In the present research the concept of reliability is applied to evaluate the column overdesign factor, i.e., beam to column strength ratio at a node of moment resisting steel frame structures. In seismic design concept, the design philosophy of “weak beam strong column” has been widely accepted by researchers and designers. In this concept, it is assumed that yielding of all beams in flexure will occur prior to possible yielding of columns which is considered to be the preferable failure mode, because of its large capacity to absorb earthquake energy before the structure actually collapses (Anderson and Gupta 1972; Park and Pauley 1975; Clough and Penzien 1982; Lee 1996). In fact, this mode provides higher ductility, and a better distribution of inelastic deformation and energy dissipation among the structural elements and thus before the collapse, the building can absorb a large amount of energy. To ensure that a frame structure collapses according to the preferable beam-hinging pattern, the columns of the structure that receive forces from the beams of a building structure are generally designed with a column overdesign factor (COF) greater than one to make the columns relatively stronger than the beams. However, it is difficult to specify the exact value of COF for a structure due to large uncertainty in the member strength and the earthquake loads.

Various COF requirements have been addressed by different structural codes. The American Concrete Institute’s (ACI) “Building Code Requirement for Structural Concrete” is the leading code used throughout the U.S. for concrete building design. ACI 318-71 (ACI Committee 318 1971) was the first version of this code to include special provisions for seismic design. In Section A.6.2 for special ductile frames, the sum of the moment strengths of the columns at the design axial load were required to be greater than the sum of the moment strengths of the beams along each principle plane at any beam-column connection. In ACI 318-83 (ACI Committee 318 1983) provisions, the design flexural strengths of the columns were required to exceed the design flexural strengths of the beams at the beam-column joint centers by at least 20%. The intent of the increased column strength requirement was to reduce the likelihood of yielding in the column. Although some other values were proposed by some recommendations, but still this is prevailing in this code. The same ratio has been provided for the general use frames by the code for seismic design of buildings (GB50011 2001) of China. Seismic Provision of Structural Steel Building (ANSI/AISC 341-05) suggested 1.0 for steel structures. A minimum COF of 1.5 is required for cold-formed square tube structures in Japan according

to corresponding design provision (BCJ 2004), and in other countries, such as New Zealand and Mexico, a COF ranging from 1.5 to 2.0 is needed (Dooley and Bracci 2001).

Although structural codes have provided different minimum COF requirements for various kinds of structures, recent major earthquake disasters and case studies by researchers have shown that the formation of plastic hinges in columns cannot be avoided even though the structures were designed to present provisions (Bertero and Zagajeski 1979; Park and Paulay 1975). One major reason incurring such a phenomenon is the large uncertainties existing in the structural members and earthquake loads (Kumamura *et al.* 1989). Therefore, It is difficult to absolutely ensure that the structure will collapse according to the preferable failure mode; a better strategy may be the probabilistic approach, i.e., to ensure an occurrence probability of the preferable failure mode larger than the probabilities of the undesirable modes. The COF ensuring probabilistic priority of beam hinging failure mode to story mechanism needs to be determined. Target values of COF that probabilistically ensure the preferable entire beam hinging failure mode of frames have been evaluated in this study following some code prescribed load distribution. This will guide the engineers to select the minimum values of COF for frame structures under specific reliability level to probabilistically avoid the undesirable story collapse modes during earthquakes.

1.2 Review of the Previous Researches

In recent years, many studies have been conducted by researchers in search of the dominant collapse modes of frames and design of the strong column weak beam frames. This section gives an overview of those studies. The review is not only limited to the researches performed for steel structures, but also includes those for reinforced concrete structures or concrete filled tube structures.

Studies related to steel frames

Hibino and Ichinose (2005a) presented a numerical study on the effect of column-to-beam strength ratio on the seismic energy dissipation of beams and columns in fish-bone-type steel moment frames. The major parameters considered were the number of stories, the strengths of the columns, the strengths of the beams and the ground motion. The seismic energy dissipation was classified into two categories: energies contributing to story mechanism and energies contributing to total mechanism. Findings of the study show that with the increase of the beam to column strength ratio, the energy contributing to the story mechanism decreases. The dynamic responses of structures subjected to 50 artificial earthquake waves were obtained, in

most cases of which the energies contributing to story mechanism decreases significantly as the column-to-beam strength ratio increases from 1.0 to 1.3. Another study of the effect of COF on the ductility ratios of structures was also presented (Hibino and Ichinose 2005b).

Nakashima and Swaizumi (1999) presented a numerical study on the column-to-beam strength ratio required for ensuring beam-hinging responses in steel moment frames. The major parameters considered were the type of frames, number of stories, type of beam hysteresis, and type and amplitude of ground motions. It was found that the column-to-beam strength ratio that ensures beam-hinging responses increases steadily with the increase of the ground motion amplitude, and the maximum story drift angle is about 1.7 to 2.0 times larger than the maximum overall story drift angle, which indicates that this level of fluctuation in the story drift is present along the stories even for frames in which beam-hinging behavior is ensured. In another study by Sawaizumi and Nakashima (1999), the effects of the column-to-beam strength ratio were examined. A series of numerical analysis was conducted for column-to-beam strength ratios successively decreased from the ratio for ensuring the column-elastic response. It was observed that the change in response is not abrupt with the decrease of the ratio and quantitative information was provided for the degree of change in the maximum story drift and beam and column maximum rotations with respect to the ratio.

Choi and Park (2009) presented an optimum seismic design algorithm for preventing plastic hinges on the columns of special steel moment frames. The proposed algorithm was then applied to a two dimensional steel moment frame to evaluate the minimum column to beam moment strength ratio required for prevention of plastic hinges in column.

In an analytical investigation by Roeder and Schneider (1993) the inelastic response of some moment resisting steel frame are analyzed for a range of different earthquake acceleration condition. The analysis shows that the strong column weak beam frames result in much smaller story drifts and better distribution of inelastic deformation than weak column strong beam frames.

Ogawa and Tomozawa (2005) carried out an investigation on COF required for controlling damage concentration in steel moment frames. Quantitative information was provided for the degree of change in the maximum story drift angle with respect to column to beam strength ratio. It was found that the COF to avoid the concentration of displacement should be 1.2 for velocity of ground motion of 1.5m/s, 1.4 for 2.25m/s, and about 1.5 for 3.0m/s. The COF requirements obtained were independent of the number of stories. The stiffness ratio of column and beam has insignificant effect on COF.

In an analytical investigation by Lee (1996), a two bay six story moment resisting steel frame was designed according to ATC 3-06 and its behavioral characteristics in inelastic range were observed through the inelastic analyses. It was reported that the design concept of strong column and weak girder, which is usually implemented by the design rule that the sum of column plastic moments should be larger than that of girder plastic moments at the joint by same margin, cannot actually prevent the occurrence of plastic hinges in columns. The reason was clarified by close investigation inelastic behaviors around joints up to the point of formation of a collapse mechanism.

Studies related to concrete frames

Kuwamura *et al.* (1989) conducted Monte Carlo simulations on the static and dynamic performance of a six-story rigid plane frame in which the randomness of the member strengths were taken into account. The simulation results showed that the randomness in the yield strengths has a predominant influence on the failure mechanism and consequently on the system ductility. It was also implied that weak-beam-strong-column structures could be realized only when the randomness in the yield strengths was reduced by means of a higher quality control in manufacturing and construction process, otherwise a considerably high margin should be provided to column strengths.

Dooley and Bracci (2001) investigated the influence of the COF at the joints of two RC frame structures under seismic excitation using inelastic time-history dynamic analyses. The frames were assessed with a COF value ranging from 0.8 to 2.4. Additionally, the influence of changing the column to beam stiffness ratio was also investigated. Findings suggest that a minimum COF of 2.0 is more appropriate to prevent the formation of a story mechanism under design seismic loading. The results also shown that, it is more effective to increase the COF without increasing the strength and stiffness ratio simultaneously.

Kawano *et al.* (1998) presented basic information on the COF for forming the weak-beam type of plastic mechanisms in steel reinforced concrete frames. In order to evaluate the requirement of the COF, a nonlinear dynamic response analysis was carried out, in which the degradation of inter-story rigidities was rigorously taken into account. It was found that, the large COF prevents a frame model from forming the inter-story plastic mechanism and also disperses the plastic hinges in the frame. It was also observed that, the encased steel shapes and the lateral reinforcements in columns are effective to reduce the requirement of COF.

Medina and Krawinkler (2005) studied a family of regular frames to evaluate the strength demands relevant to the seismic design of the columns and indicated that the potential of

plastic hinging in the columns is high for frames designed according to the strong column weak beam requirements of current code provisions.

Some other studies related to the behavior of beam column joint of RC frame with floor slab can be found in Durrani and Zerbe (1987), Durrani and Wight (1988; 2000), Ehsani and Wight (1985) which mainly focused on the effect of floor slab in joint.

Studies related to CFT frames

In an experimental study by Azizinamini and Schneider (2004), the behavior of circular concrete filled tube (CFT frame) under seismic loads was studied through testing six columns which were subjected to a constant axial load in addition to a cyclic lateral load. Failure modes for through beam connection detail were identified by testing seven, two-thirds scale connection specimens. The experimental results showed that column failure was prevented when the column-to-beam flexural strength ratio was approximately 1.5 for full penetration weld and approximately 2.0 for fillet weld. It was recommended that these conservative values should be used as lower limits on the column-to-beam strength ratio for through beam connection detail until additional experimental data become available to justify lower values.

Saisho, Katsuki and Ota (2001) investigated the seismic response and damage of concrete filled steel tube frame (CFT frame) in relation with its column overdesign factor. Ground motion recorded in Kobe (1995) and El Centro (1994) are considered in the study. Based on the findings of the investigation COF value equal to 2.0 is proposed as the critical value in the earthquake resistant design of multi-story CFT frames.

Most of the studies described above concluded that the COF has great effect on the failure probabilities of structures, and the existing code provisions of COF are not sufficient to avoid story mechanism. In order to assure the preferable entire beam-failure mode, a higher COF value is required. Moreover, most of these studies described above used deterministic approach for specific structures and the occurrence probability of the undesirable failure mode and the risk of failure of the structure remain unknown. Although the probabilistic approach has been applied by Dooley and Bracci (2001) and Kuwamura *et al.* (1989), however, the studies have been conducted for specific structure with specific earthquake input.

1.3 Objectives and Organization of the Present Research

In the present study, considering the uncertainties of earthquake load and strengths of structural members, the failure modes of the multistory ductile frame structures are investigated probabilistically. Load distribution along the height of the building frames

prescribed by some building codes, such as A_i distribution of Building Standard law of Japan, the distribution of the Uniform Building Code (UBC-1994) and International Building Code (IBC-2006) are taken into account in this study. Based on the investigations, the target values of COF that ensure probabilistically the preferable entire beam hinging failure mode prior to story collapse are evaluated. Although the system reliability is very complicated, but an initiative has also taken to consider the system reliability in the evaluation of target COF.

The remaining part of the thesis consists of four chapters.

Chapter 2 deals with the probabilistic investigation on the story mechanisms of moment resisting steel frame structures. The story failure modes are categorized as the upper story mechanism, the middle story mechanism, and the lower story mechanism according to the location of failure stories. The most likely story mechanisms of frame structures are then investigated. The failure modes of the frames are investigated considering A_i distribution of the Building Standard law of Japan, the distribution of the Uniform Building Code (UBC-1994) and International Building Code (IBC-2006). The basic assumptions applied in this study are also presented here.

Chapter 3 deals with the probabilistic evaluation of the target COF that probabilistically avoids the undesirable story mechanisms during earthquakes. A_i distribution and the load distribution of UBC-1994 and IBC-2006 are taken into consideration for this purpose. The effects of height and mass irregularity of the frames on target COF based on A_i distribution of load are also presented in this chapter.

Chapter 4 introduces the evaluation of target COF considering system reliability. The Dimension Reduction Integration (DRI) with fourth moment standardization is mainly applied in the investigation. The results of DRI are also investigated with Monte Carlo Simulation (MCS).

Chapter 5 presents the summarized conclusions of the present research.

Chapter 2

INVESTIGATION ON THE STORY MECHANISMS OF THE FRAME STRUCTURES

2.1 Introduction

Many sources of uncertainty are inherent in the structural design. Despite what we often think, the parameters of the loading and the load carrying capacities of the structural members are not deterministic quantities (i.e., quantities which are perfectly known). They are random variables, and thus absolute safety (or zero probability of failure) cannot be achieved. Conceptually, we can design the structure to reduce the probability of failure, but increasing the safety (or reducing the probability of failure) beyond a certain optimum level is not always economical. Consequently, structures must be designed to serve their function with a finite probability of failure.

Frame structure may collapse in different failure modes. It depends upon many factors such as combination of applied loads, the strength of the various elements etc. Identification, evaluation and description of all these failure modes are really a difficult and time consuming task.

Among all the failure modes the story collapse of one or more stories is the most dangerous. This is because story collapse leads to the collapse of the columns which is obviously more dangerous compared to beam failure or other minor failures. Therefore, the present study is based on this type of failure mode. For convenience, the story collapse modes are defined before the probabilistic evaluation so that the investigation can be carried out sequentially for each type. Story mechanisms are classified as the upper story mechanism, the middle story mechanism, and the lower story mechanism, depending on the location of failure stories.

Many studies concerned with failure modes of frames have been performed so far. Nafday (1987) developed a systematic approach based on linear programming model for the identification, enumeration and description of multiple failure modes for structural frames. Ang and Ma (1981) developed a method to find the stochastically relevant mode directly by solving a nonlinear optimization problem, which was performed to find the minimal reliability index. Ohi (1991) developed the stochastic limit analysis method, which is one of the mathematical programming techniques to obtain the likely failure modes in relatively short computation time.

2.2 Definitions and Basic Assumptions

2.2.1 Definition of node COF

The column overdesign factor (COF) is defined for each beam-column node as the ratio of the sum of the moment capacity of columns to the sum of moment capacity of beams at that node as:

$$COF(k) = \frac{\sum_i \mu_{mci}}{\sum_i \mu_{mbi}} \quad (2.1)$$

where, μ_{mci} = the mean plastic moment strength of column connected in the k th node and μ_{mbi} = the mean plastic moment strength of beam connected in the k th node.

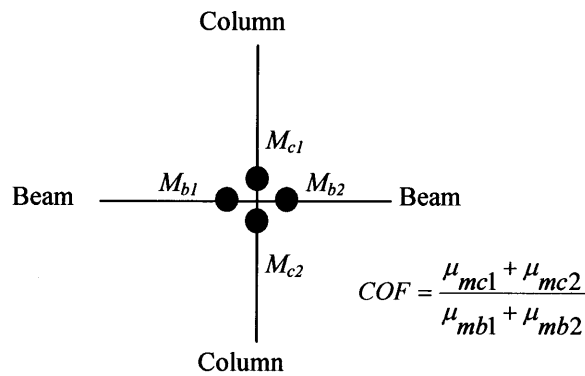


Fig. 2.1 Definition of node COF

2.2.2 Basic Assumptions

For the ductile frame structures considered in this study the following basic assumptions are applied:

- Elastic-plastic frame structures are considered. The failure of a section means the imposition of a hinge and an artificial moment at that section.
- The structural uncertainties are represented by considering only the moment capacities as random variables. The coefficient of variation of the material strength is considered to be 0.1 (AIJ 1990).
- Plastic moment capacities are statistically independent to one another and independent of the applied loads. All the random variables are assumed to follow the lognormal distribution.
- The external load considered is only the lateral earthquake load. The A_i distribution of

Building Standard Law of Japan, the load distributions of the Uniform Building Code (UBC-1994), and International Building Code (IBC-2006) are taken into account for this purpose. Based on some studies (Kanda 1993; AIJ 1990) and considering other uncertainty the coefficient of variation of the earthquake load is considered to be 0.8.

- The geometrical second-order and shear effects are neglected. The effect of the axial forces on the reduction of moment capacities is also neglected.
- All beam-column nodes have identical COFs, i.e., there is only one value of COF for a structure.

2.3 A Brief Description of FORM

The fundamental problem in structural reliability theory is the computation of the multi-fold probability integral

$$P_f = Prob[Z = G(\mathbf{X}) \leq 0] = \int_{G(\mathbf{x}) \leq 0} f(\mathbf{X}) d\mathbf{X} \quad (2.2)$$

where $\mathbf{X} = [X_1, \dots, X_n]^T$, in which the superposed T =Transpose, is a vector of random variables representing uncertain structural quantities, $f(\mathbf{X})$ denotes the joint probability density function of \mathbf{X} , $G(\mathbf{X})$ is the performance function defined such that $G(\mathbf{X}) \leq 0$, the domain of integration, denotes the failure set, and P_f is the failure probability. Difficulty in computing this probability has led to the various approximate methods, of which first order reliability method (FORM) is one of them.

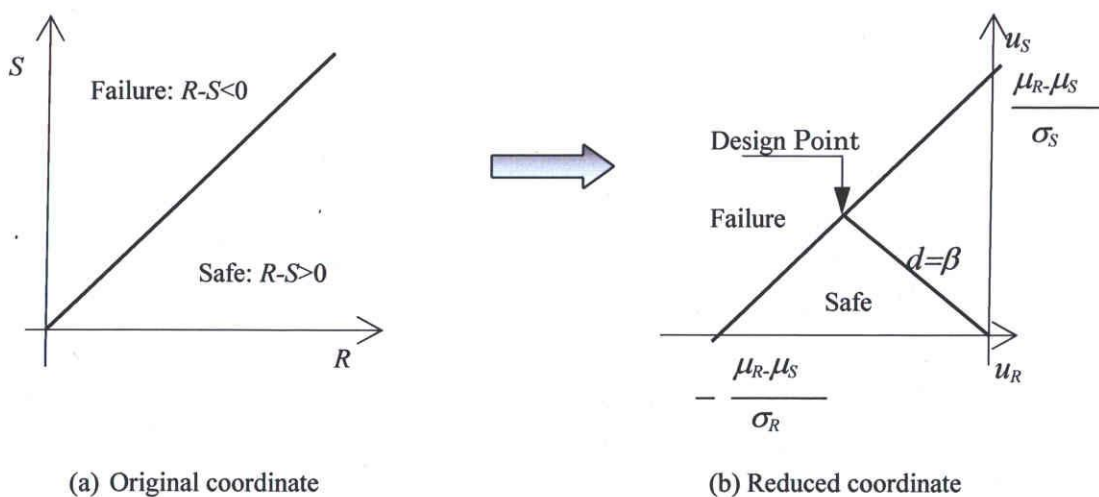


Fig. 2.2 Limit state surface in original and u -space

FORM is an analytical approximation in which the reliability index is interpreted as the minimum distance from the origin to the limit state surface in standardized normal space (u -space) and the most likely failure point (design point) is searched using mathematical programming method.

For some situation, especially when there are large numbers of basic variables and for complex limit state equation the following algorithm (Hohenbichler and Rackwitz, 1981) is used.

1. Assume an initial checking point \mathbf{x}_0
2. Using the Rosenblatt transformation, obtain the corresponding checking point in u -space, that is, \mathbf{u}_0
3. Determine the Jacobian Matrix

$$\mathbf{J} = \frac{\partial \mathbf{x}}{\partial \mathbf{u}} = \begin{bmatrix} \frac{\partial x_1}{\partial u_1} & \frac{\partial x_1}{\partial u_2} & \dots & \frac{\partial x_1}{\partial u_n} \\ \frac{\partial x_2}{\partial u_1} & \frac{\partial x_2}{\partial u_2} & \dots & \frac{\partial x_2}{\partial u_n} \\ \vdots & \vdots & \ddots & \vdots \\ \frac{\partial x_n}{\partial u_1} & \frac{\partial x_n}{\partial u_2} & \dots & \frac{\partial x_n}{\partial u_n} \end{bmatrix} \quad (2.3)$$

evaluated at \mathbf{x}_0

4. Evaluate the performance function and gradient vector at \mathbf{u}_0

$$G_u(\mathbf{u}_0) = G(\mathbf{x}_0) \quad (2.4)$$

$$\nabla G(\mathbf{u}_0) = \mathbf{J}^T \nabla G(\mathbf{x}_0) \quad (2.5a)$$

The term gradient generally refers to the derivative of vector functions as,

$$\nabla G(\mathbf{u}) = \left(\frac{\partial G(\mathbf{u})}{\partial u_1}, \frac{\partial G(\mathbf{u})}{\partial u_2}, \dots, \frac{\partial G(\mathbf{u})}{\partial u_n} \right) \quad (2.5b)$$

5. Obtain a new checking point

$$\mathbf{u}^{(k+1)} = \frac{1}{\nabla^T G(\mathbf{u}^{(k)}) \nabla G(\mathbf{u}^{(k)})} \left[\nabla^T G(\mathbf{u}^{(k)}) \mathbf{u}^{(k)} - G(\mathbf{u}^{(k)}) \right] \nabla G(\mathbf{u}^{(k)}) \quad (2.6)$$

and in the space of original variables, the checking point is

$$\mathbf{x}^{(k+1)} = \mathbf{x}^{(k)} + \mathbf{J}(\mathbf{u}^{(k+1)} - \mathbf{u}^{(k)}) \quad (2.7)$$

6. Calculate the reliability index

$$\beta = \sqrt{\mathbf{u}^{*T} \mathbf{u}^*} \quad (2.8)$$

7. Repeat step 2 to 6 using the above \mathbf{x}^* as the new checking point until convergence is achieved.

2.4 Ai Distribution of Load and Determination of Load Level

2.4.1 A Brief Description of the Ai Distribution

In this section, Ai distribution of the Building Standard Law of Japan is taken into account.

The mean values of the load of the upper floors μ_{pj} are obtained from mean value of load acting on the first floor of structure μ_p as follows:

$$\mu_{pj} = C_j \mu_p \quad (2.9)$$

where μ_p is the mean value of load acting on the first floor of structure and C_j is the lateral load coefficient for j th story obtained from the distribution of load. C_j is related to the seismic lateral forces F_i at various levels which is calculated as:

$$F_i = Q_i - Q_{i+1} \quad (2.10)$$

The lateral shear Q_i is calculated as:

$$Q_i = C_i W_i \quad (2.11)$$

where W_i is the weight at and above level i . The seismic shear coefficient C_i for i th level is determined by:

$$C_i = Z R_i A_i C_0 \quad (2.12)$$

where Z is the seismic zone coefficient, R_i is the design spectral coefficient, C_0 is the standard shear coefficient, A_i is the horizontal shear distribution factor calculated as:

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2T}{1 + 3T} \quad (2.13)$$

where T is the fundamental time period equal to $0.03 \times$ total height in meter, α_i is the ratio of the weight carried by the i^{th} floor to the total weight of the structure.

2.4.2 Determination of Load Level

In this study, the investigation of the column overdesign factor (COF) was conducted under a specific reliability level, which means that for a given reliability index of the entire beam-hinging failure mode, the load levels are adjusted to ensure that the first order reliability index becomes equal to the target reliability index β_T for frame structures designed with various COFs. The first order reliability method (FORM) and target reliability level 2, 3 and 4 ($\beta_T=2$, $\beta_T=3$ and $\beta_T=4$) are considered in this study.

Based on the principle of virtual work, performance function for the entire beam-hinging failure mode as shown in Fig. 2.3 can be established as follows:

$$G(\mathbf{X}) = 2\sum_{i=1}^m M_{bni} + 2\sum_{j=1}^{n-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^2 M_{csl} + \sum_{l=1}^{m-1} M_{cl} - \sum_{j=1}^n jhP_j \quad (2.14)$$

where M_{bni} is the moment strength of the beam of the top story, M_{bij} is the moment strength of the beam of the i th span and j th story, M_{cl} is the moment strength of an interior column, M_{csl} is the moment strength of an exterior column, P_j is the load acting on the j th story of the structure, n is the number of stories, m is the number of spans and h is the story height.

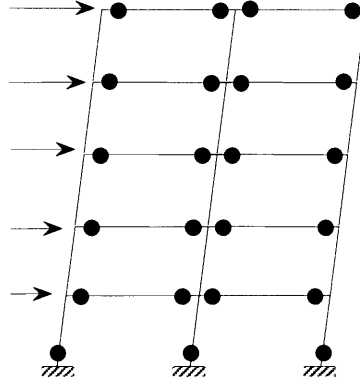


Fig. 2.3 Entire beam hinging failure mode

The moment strength of the members and the load acting on the structure are assumed as random variable. The mean strengths of structural members are determined through Eq. 2.15. Eq. 2.16 gives the coefficient of variation of each random variable.

$$\mu_{bni} = \mu_b, \quad \mu_{bij} = 2\mu_b, \quad \mu_{csl} = COF * \mu_b, \quad \mu_{cl} = 2COF * \mu_b, \quad (2.15)$$

$$V_{bni} = V_{bij} = V_{cl} = V_{csl} = V_1, \quad V_{pj} = V_2 \quad (2.16)$$

where μ_b = mean value of M_{bni} , μ_{bij} = mean value of M_{bij} ($j < n$), μ_p = mean value of load acting on the first floor of structure and A_{ij} is the lateral load coefficient for j th story. V_1 and V_2 are coefficients of variation of member strength and load respectively. The mean value of the strength of the top beam is assumed and mean value of the other members are obtained from the above relation of Eq. 2.15.

The mean value of moment strength of the beam of the top story is assumed to be half of the mean value of moment strength of other beams of the lower stories. This is because the top beam has to sustain a lower load than the beams of the lower stories due to absence of walls and some other loads. The mean values of the moment strength of columns are obtained by multiplying respective COF value. For example when COF=1.1 and the mean strength of the top beam is 104.15 KN.m, the mean strength of other beams, exterior columns and interior column will be 208.30 KN.m, 114.565 KN.m and 229.13 KN.m. respectively. Since, this study has been conducted for low-rise structures of up to seven stories; therefore, columns of all the stories are considered to have the same strengths in order to simplify the calculation. In practical construction of low-rise structures also, the column of all stories are usually constructed with same strength sections in order to simplify construction work.

In order to understand the relationships between μ_p -COF, μ_p - μ_b , and μ_p - m , consider the reliability index of the entire beam-hinging mode with the performance function described in Eq. (2.14).

Decompose the performance function of entire beam-hinging failure mode into three parts as:

$$x_b = 2 \sum_{i=1}^m M_{bni} + 2 \sum_{j=1}^{n-1} \sum_{i=1}^m M_{bij} \quad (2.17a)$$

$$x_c = \sum_{l=1}^2 M_{csl} + \sum_{l=1}^{m-1} M_{cl} \quad (2.17b)$$

$$x_p = \sum_{j=1}^n jhP_j \quad (2.17c)$$

So, the performance function is expressed as:

$$G(\mathbf{X}) = x_b + x_c - x_p \quad (2.18)$$

Following Eq. (2.15) & Eq. (2.16), the mean value and standard deviation of x_b , x_c , and x_p are given by:

$$\mu_{xb} = 2m(2n-1)\mu_b \quad (2.19a)$$

$$\sigma_{xb} = 2V_1\mu_b\sqrt{m(4n-3)} \quad (2.19b)$$

$$\mu_{xc} = 2mCOF\mu_b \quad (2.19c)$$

$$\sigma_{xc} = V_1\mu_bCOF\sqrt{2(2m-1)} \quad (2.19d)$$

$$\mu_{xp} = \mu_p h \sum_{j=1}^n j^2 \quad (2.19e)$$

$$\sigma_{xp} = V_2\mu_p h \sqrt{\sum_{j=1}^n j^4} \quad (2.19f)$$

Then, the mean and the standard deviation of the performance function are obtained as:

$$\mu_G = 2m(2n-1)\mu_b + 2mCOF\mu_b - \mu_p h \sum_{j=1}^n j^2 \quad (2.20)$$

$$\sigma_G = \sqrt{\sigma_{xb}^2 + \sigma_{xc}^2 + \sigma_{xp}^2} = \sqrt{V_1^2\mu_b^2[2COF^2(2m-1) + 4m(4n-3)] + V_2^2\mu_p^2h^2\sum_{j=1}^n j^4} \quad (2.21)$$

The second moment reliability index is then obtained as:

$$\beta_{SM} = \frac{\mu_G}{\sigma_G} = \frac{2m(2n-1)\mu_b + 2mCOF\mu_b - \mu_p h \sum_{j=1}^n j^2}{\sqrt{V_1^2\mu_b^2[2COF^2(2m-1) + 4m(4n-3)] + V_2^2\mu_p^2h^2\sum_{j=1}^n j^4}} \quad (2.22)$$

As V_1 is much smaller than V_2 , so it can be ignored and the second moment reliability index can be approximately given as:

$$\beta_{SM} \approx [2m\mu_b(2n-1 + COF) - \mu_p h \sum_{j=1}^n j^2] / (V_2\mu_p h \sqrt{\sum_{j=1}^n j^4}) \quad (2.23)$$

From Eq. (2.23) the mean value of load applied to the first story of structure is obtained as:

$$\mu_p \approx 2m\mu_b(2n-1 + COF) / h(\beta_{SM}V_2\sqrt{\sum_{j=1}^n j^4} + \sum_{j=1}^n j^2) \quad (2.24)$$

As β_{SM} is unknown, Eq. (2.24) cannot be directly used to determine μ_p under a specific reliability level of FORM. The distribution type of performance function has a direct relationship with its moment, and the reliability index or the failure probability corresponding

to $Z = G(\mathbf{X}) < 0$ is also the function of moment of the performance function. The standardized random variable $x_s = (Z - \mu_G) / \sigma_G$ is assumed to have a relationship with a standard normal random variable u as expressed by the following equation:

$$u = N(x_s, \{\alpha_{Gk}\}) \quad (2.25)$$

where N is the function describing the relationship between x_s and u , and α_{Gk} is the k -order dimensionless center moment of $Z = G(\mathbf{X})$:

$$\text{prob}[G \leq 0] = \text{prob}\left[x_s \leq -\frac{\mu_G}{\sigma_G}\right] = \text{prob}[x_s \leq -\beta_{SM}] \quad (2.26)$$

The reliability index can be given by:

$$\beta = -N(-\beta_{SM}, \{\alpha_{Gk}\}) \quad (2.27)$$

Because V_1 is far smaller than V_2 , the dimensionless k -order center moment of $Z = G(\mathbf{X})$ can be expressed approximately as:

$$\alpha_{Gk} = \frac{\alpha_{xbk} \sigma_{xb}^k + \alpha_{xck} \sigma_{xc}^k + \alpha_{xpk} \sigma_{xp}^k}{\alpha_G^k} \approx \alpha_{xpk} \quad (2.28)$$

$$\alpha_{xpk} = \frac{\alpha_{pk} \sum_{j=1}^n j^{2k}}{\left(\sum_{j=1}^n j^4\right)^{k/2}} \quad (2.29)$$

where α_{xbk} = the dimensionless k -order center moment of x_b , α_{xck} = the dimensionless k -order center moment of x_c , α_{xpk} = the dimensionless k -order center moment of x_p , and α_{pk} = the dimensionless k -order center moment of load applied on the first story.

According to the above equations, it can be found that generally α_{Gk} is not related to COF. Since the reliability index β remains to be the same level even though the value of COF changes, β_{SM} will also remain at the same level in spite of the increase of COF.

Then one can easily understand that μ_p is basically a linear function of COF. Furthermore, β_{SM} is also not affected by the mean value of the member strength or the number of bays, so μ_p also changes linearly with respect to μ_b and m .

The relation of mean value of load acting on the first floor of structure, μ_p and COF is shown in Fig. 2.4. From this Fig. it is observed that the mean value of the load is generally a linear function of COF. A similar observation is also observed in earlier studies by Zhao *et al.* (2002)

and Pu and Zhao (2007) for triangular load.

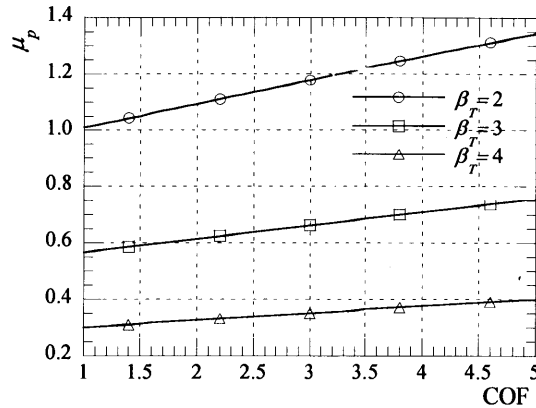


Fig. 2.4 Load-COF curve

2.5 Probabilistic Investigation on Story Collapse Modes Considering Ai Distribution

In this study, the story failure modes are classified into three patterns: upper story failure pattern, middle story failure pattern and lower story failure pattern, each of which depends on the location of the failure stories, as shown in Fig. 2.5. The upper story failure pattern is characterized by continuous collapsed stories from the top story of the frame; the lower story failure pattern is characterized by the continuous collapse of stories from the first story of the frame; in the middle story failure pattern, the mechanism occurs in the middle stories of the frame and the stories at the top and bottom remain elastic.

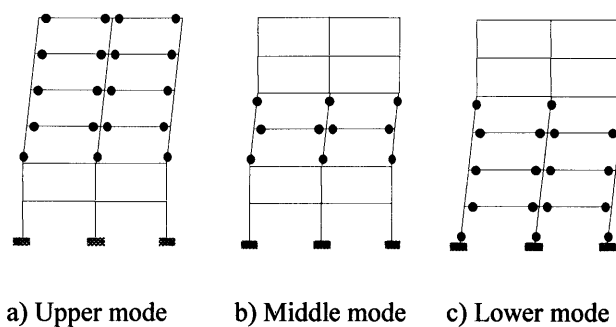


Fig. 2.5 Story collapse modes

2.5.1 Investigation on Upper Story Mechanisms

Based on the principle of virtual work, the performance function for the upper collapse modes can be established as:

$$G_U(\mathbf{X}) = 2 \sum_{i=1}^m M_{bmi} + 2 \sum_{j=n-n_c+1}^{n-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^2 M_{csl} + \sum_{l=1}^{m-1} M_{cl} - \sum_{j=n-n_c+1}^n (j+n_c-n)hP_j \quad (2.30)$$

where, n_c = the number of failure stories.

Following the same procedure as described in sec 2.4.2 the approximate expression of the second moment reliability index is obtained as:

$$\beta_{SM-U} \approx \frac{2m(2n_c - 1 + COF)\mu_b - \mu_p h \sum_{j=n-n_c+1}^n j(j+n_c-n)}{V_2 \mu_p h \sqrt{\sum_{j=n-n_c+1}^n j^2 (j+n_c-n)^2}} \quad (2.31)$$

Let us now consider a six story two bay frame having equal bay width of 8m and equal story height of 4m. For this six story frame, there are five upper collapse modes. These collapse modes are shown in Fig. 2.6.

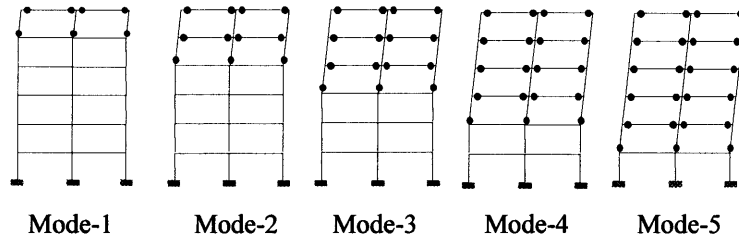


Fig. 2.6. Upper story collapse modes of a six story frame

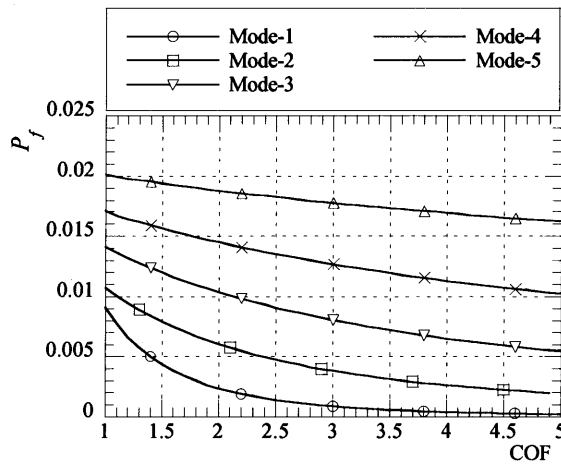


Fig. 2.7. Failure probability of upper story collapse modes

The failure probabilities of the upper story failure modes are shown in Figure 2.7. It is observed that the failure probabilities of the upper story failure modes steadily increase with the increase of the number of failure stories. It is also observed that with the increase of COF the failure probabilities decrease.

2.5.2 Investigation on Middle Story Mechanisms

Based on the principle of virtual work, the performance function for the middle collapse modes can be established as:

$$G_M(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^4 M_{csl} + \sum_{l=1}^{2m-2} M_{cl} - \sum_{j=1}^{n_c} jhP_{j+n_b} - \sum_{j=n_c+1}^{n-n_b} n_c hP_{j+n_b} \quad (2.32)$$

where, n_b = number of unbroken stories at the bottom, n_c = number of failure stories.

Following the same procedure as described in sec 2.4.2 the approximate expression of the second moment reliability index is obtained as:

$$\beta_{SM-M} \approx \frac{4m\mu_b(n_c - 1 + COF) - \mu_p h \left[\sum_{j=1}^{n_c} j(j + n_b) + n_c \sum_{j=n_c+1}^{n-n_b} (j + n_b) \right]}{V_2 \mu_p h \sqrt{\sum_{j=1}^{n_c} j^2 (j + n_b)^2 + n_c^2 \sum_{j=n_c+1}^{n-n_b} (j + n_b)^2}} \quad (2.33)$$

For six story frame, there are ten middle collapse modes. These collapse modes are shown in Fig. 2.8

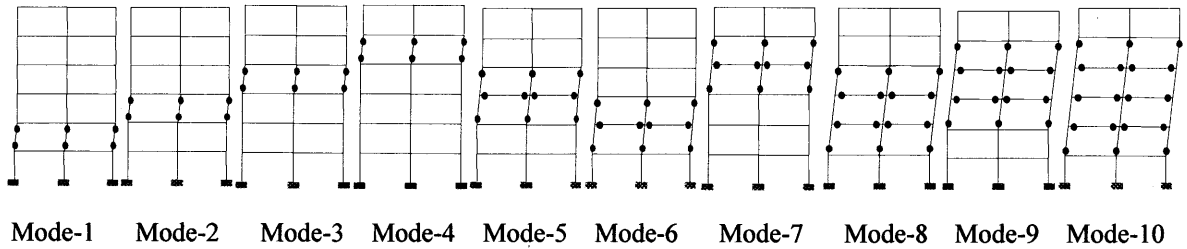


Fig. 2.8. Middle story collapse modes of a six story frame

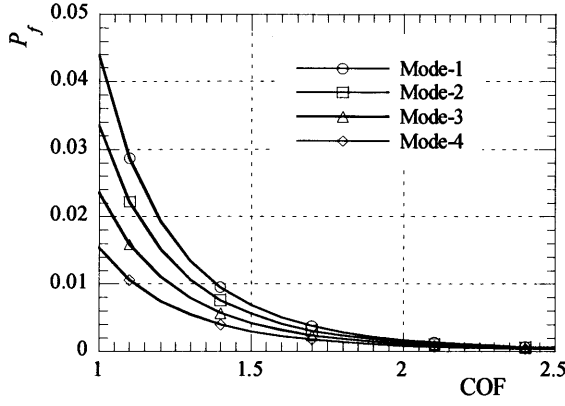


Fig. 2.9. Failure probability of middle modes ($n_c = 1$)

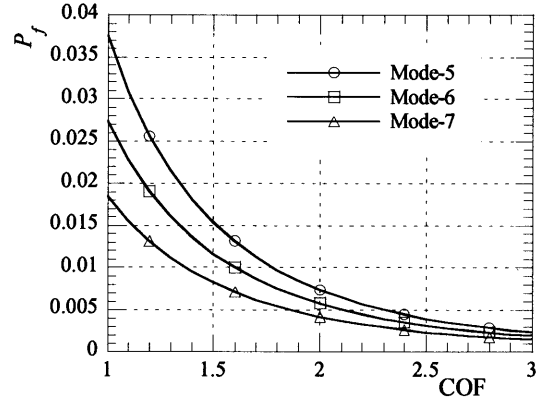


Fig. 2.10. Failure probability of middle modes ($n_c = 2$)

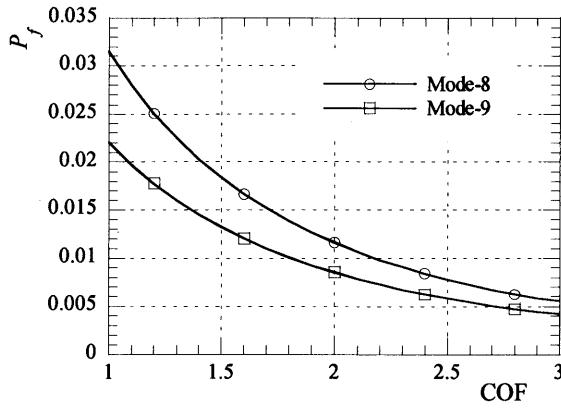


Fig. 2.11. Failure probability of middle modes ($n_c = 3$)

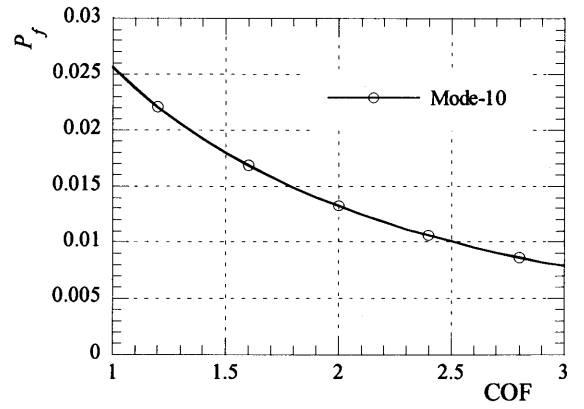


Fig. 2.12. Failure probability of middle modes ($n_c = 4$)

The failure probabilities of the middle story failure modes are shown in Figure 2.9 to Figure 2.12 for number of collapse stories equal to 1 to 4 respectively. It is observed that in all cases the failure probability with higher n_b is less than that with lower n_b . That means the number of unbroken stories at the bottom n_b , has dominant effect on failure probabilities of the middle story failure modes. It is also observed that with the increase of COF the failure probabilities decrease.

2.5.3 Investigation on Lower Story Mechanisms

Based on the principle of virtual work, the performance function for the upper collapse modes can be established as:

$$G_L(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^4 M_{csl} + \sum_{l=1}^{2m-2} M_{cl} - \sum_{j=1}^{n_c} jhP_j - \sum_{j=n_c+1}^n n_c hP_j \quad (2.34)$$

where, n_c = the number of failure stories.

Following the same procedure as described in sec 2.4.2 the approximate expression of the second moment reliability index is obtained as:

$$\beta_{SM-L} \approx \frac{4m(n_c - 1 + COF)\mu_b - \mu_p h \left[\sum_{j=1}^{n_c} j^2 + n_c \sum_{j=n_c+1}^n j \right]}{V_2 \mu_p h \sqrt{\sum_{j=1}^{n_c} j^4 + n_c^2 \sum_{j=1}^n j^2}} \quad (2.35)$$

For this six story frame, there are five lower collapse modes. These collapse modes are shown in Fig. 2.13.

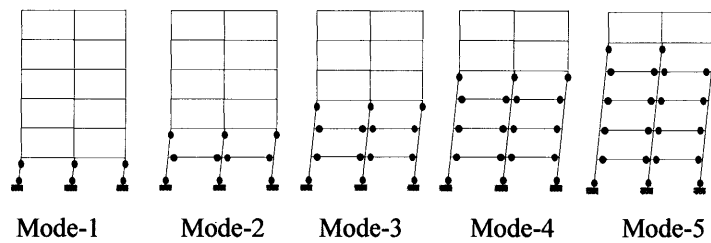


Fig. 2.13. Lower story collapse modes of a six story frame

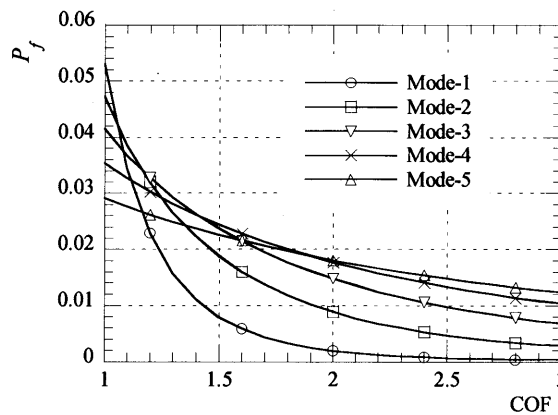


Fig. 2.14. Failure probability of lower story collapse modes

The failure probabilities of the lower story failure modes are shown in Figure 2.14. It is observed that the failure probabilities of the lower story collapse modes do not follow any specific patterns. The value of COF has a significant effect on the probabilistic order of the lower story failure modes. Each lower story mechanism has a special COF region in which the

failure probability of that mode is the largest. It is also observed that with the increase of COF the failure probabilities decrease.

2.5.4 Likely Story Mechanisms

From the investigation, it is observed that, among all the failure modes of a multi-story ductile frame structure the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability. It can be clearly observed from the following example of a five story frame. The failure modes of this frame are shown in Fig. 2.15.

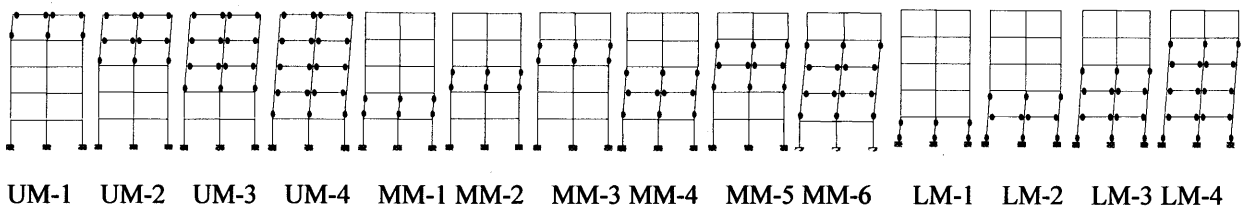


Fig. 2.15. Story collapse modes of a five story frame

The failure probabilities of all the modes are shown in Fig. 2.16. From this figure it is clearly observed that the lower failure modes and the upper failure mode with the maximum failure stories are the most likely failure modes.

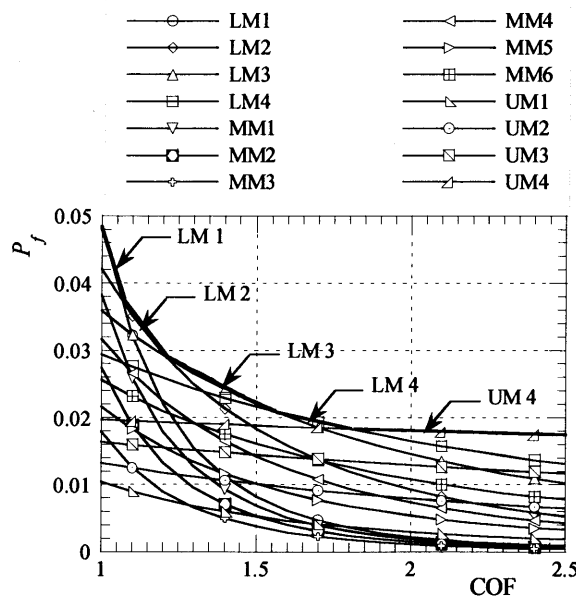


Fig. 2.16. The evaluation of likely failure modes

An earlier study by Zhao *et al.* (2007) on the story failure modes of the frame structures based on triangular distribution of load along the height of the frame also showed that all the lower story collapse modes and the upper story collapse modes with maximum failure stories are the most likely failure modes. Therefore, these most likely failure modes are considered in the target COF evaluation.

2.6 Load Distribution of UBC-94 and Determination of Load Level

2.6.1 A Brief Description of the Load Distribution of UBC-94

In this section, the load distribution of the Uniform Building Code (UBC-94) is taken into account.

The mean values of the load of the upper floors μ_{pj} are obtained from mean value of load acting on the first floor of structure μ_p as shown in Eq. 2.3. In this Eq. C_j is the lateral load coefficient for j th story obtained from the distribution of load.

The static equivalent base shear is defined in the Uniform Building Code (UBC-1994), as:

$$V = \frac{ZIC}{R}W \quad (2.36)$$

where W is the total dead load, Z is the seismic zone coefficient, I is the importance factor, R is the response modification coefficient, and C is numerical coefficient given by the relation:

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \quad (2.37)$$

where S is the site soil coefficient and T is the fundamental time period calculated as:

$$T = C_t(h_n)^{3/4} \quad (2.38)$$

where h_n is the height (ft) and C_t is equal to 0.035 for steel moment resisting frame. The base shear will be distributed along the height according to the relation:

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (2.39)$$

where F_t is the concentrated force acting at the top (roof) of the structure in addition to the F_x force at that level. For T greater than 0.7 second; $F_t = 0.07TV \leq 0.25V$, otherwise it is equal to zero.

It should be noted here that, the seismic code of some countries such as, Bangladesh National Building Code (BNBC), Taiwanese Building Code (TBC) etc. have followed the similar distribution of base shear that is described by UBC.

2.6.2 Determination of Load Level

It is already mentioned that in this study, the investigation of the column overdesign factor (COF) was conducted under a specific reliability level, which means that for a given reliability index of the entire beam-hinging failure mode, the load levels are adjusted to ensure that the first order reliability index becomes equal to the target reliability index β_T for frame structures designed with various COFs.

The relation of mean value of load acting on the first floor of structure, μ_p and COF is shown in Fig. 2.17. From this Fig. also it is observed that the mean value of the load is generally a linear function of COF. A similar observation is also observed in case of Ai distribution of Japan as described in sec 2.4.2 and in earlier studies by Zhao *et al.* (2002) and Pu and Zhao (2007) for triangular load.

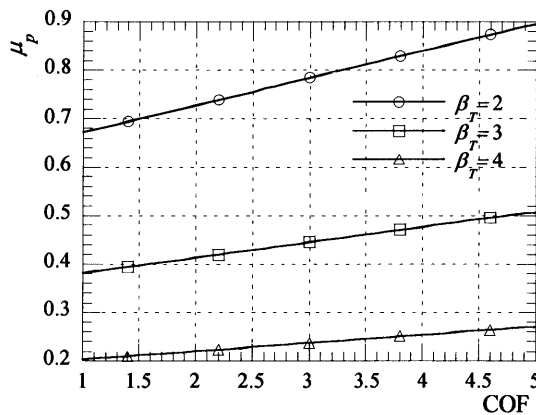


Fig. 2.17 Load-COF curve (UBC-94)

2.7 Probabilistic Investigation on Story Collapse Modes Considering UBC-94

The story failure modes are classified into three patterns: upper story failure pattern, middle story failure pattern and lower story failure pattern, each of which depends on the location of the failure stories, as shown in Fig. 2.5.

2.7.1 Investigation on Upper Story Mechanisms

The performance function for the upper collapse modes based on the principle of virtual

work is similar to Eq. (2.30). The approximate expression of the second moment reliability index is also similar to Eq. (2.31). It should be noted that the mean value of the load which is determined applying FORM considering the load distribution of UBC-94 is different from that of A_i distribution.

Let us now consider a six story two bay frame having equal bay width of 8m and equal story height of 4m. For this six story frame, there are five upper collapse modes. These collapse modes are same as shown in Fig. 2.6.

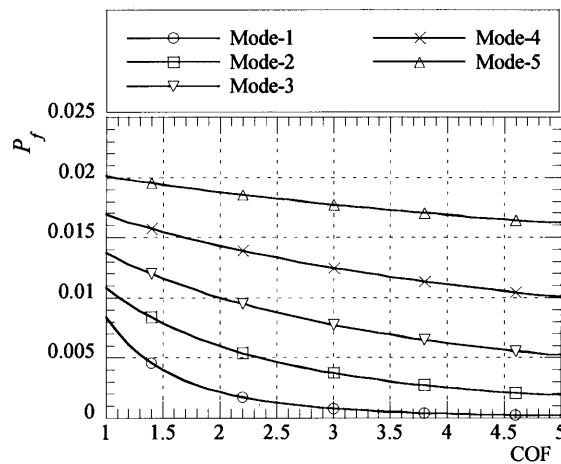


Fig. 2.18. Failure probability of upper story collapse modes (UBC-94)

The failure probabilities of the upper story failure modes are shown in Figure 2.18. It is observed that the failure probabilities of the upper story failure modes steadily increase with the increase of the number of failure stories. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of A_i distribution also.

2.7.2 Investigation on Middle Story Mechanisms

The performance function for the middle collapse modes, based on the principle of virtual work is similar to Eq. (2.32). The approximate expression of the second moment reliability index is also similar to Eq. (2.33). However, the mean value of the load which is determined applying FORM considering the load distribution of UBC-94 is different from that of A_i distribution.

For six story frame, there are ten middle collapse modes. These collapse modes are same as shown in Fig. 2.8.

The failure probabilities of the middle story failure modes are shown in Fig. 2.19 to Fig.

2.22 for number of collapse stories equal to 1 to 4 respectively.

It is observed that in all cases the failure probability with higher n_b is less than that with lower n_b . That means the number of unbroken stories at the bottom n_b , has dominant effect on failure probabilities of the middle story failure modes. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of Ai distribution also.

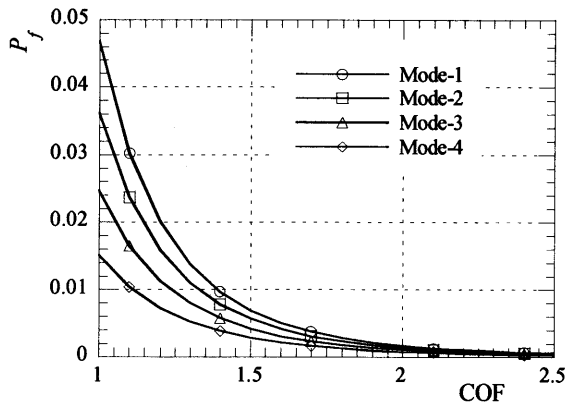


Fig. 2.19. P_f of middle modes with $n_c = 1$ (UBC-94)

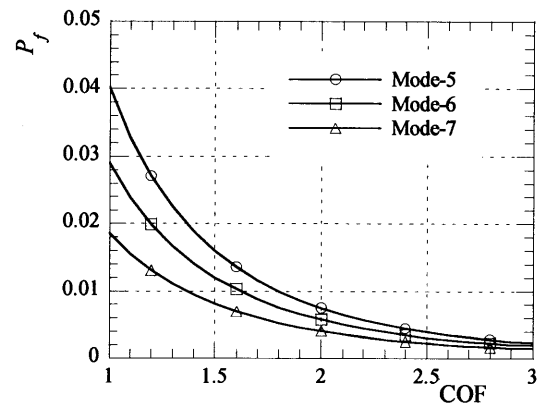


Fig. 2.20. P_f of middle modes with $n_c = 2$ (UBC-94)

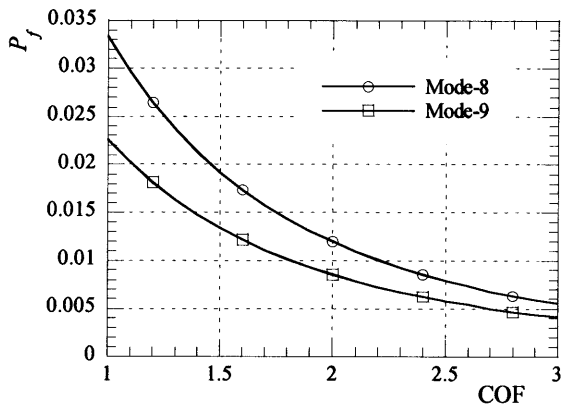


Fig. 2.21. P_f of middle modes with $n_c = 3$ (UBC-94)

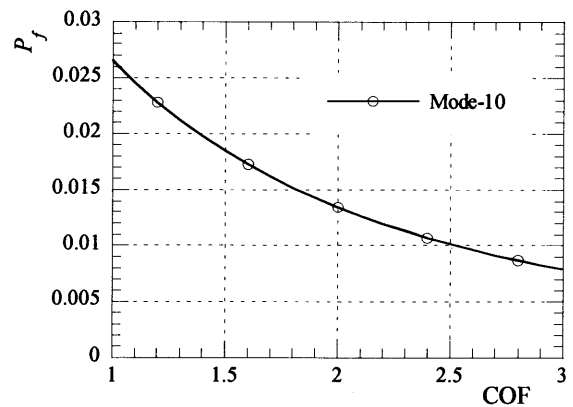


Fig. 2.22. P_f of middle modes with $n_c = 4$ (UBC-94)

2.7.3 Investigation on Lower Story Mechanisms

The performance function for the lower collapse modes, based on the principle of virtual work is similar to Eq. (2.34). The approximate expression of the second moment reliability index is also similar to Eq. (2.35). However, the mean value of the load which is determined applying FORM considering the load distribution of UBC-94 is different from that of Ai distribution.

For this six story frame, there are five lower collapse modes. These collapse modes are same as shown in Fig. 2.13.

The failure probabilities of the lower story failure modes of the frame considered are shown in Fig. 2.23.

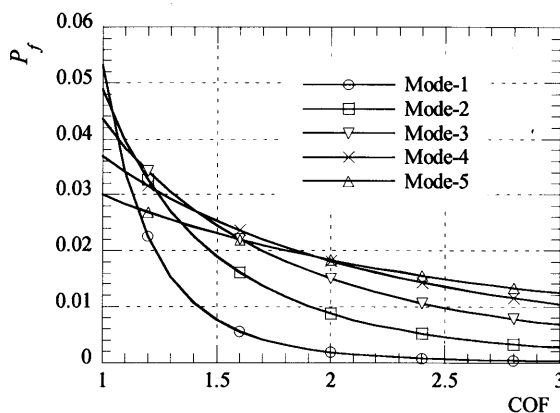


Fig. 2.23. Failure probability of lower story collapse modes (UBC-94)

It is observed that the failure probabilities of the lower story collapse modes do not follow any specific pattern. The value of COF has a significant effect on the probabilistic order of the lower story failure modes. Each lower story mechanism has a special COF region in which the failure probability of that mode is the largest. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of Ai distribution also.

2.7.4 Likely Story Mechanisms

From the investigation, it is observed that, in case of the load distribution of UBC -94 also, the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability among all the failure modes of a multi-story ductile frame structure.

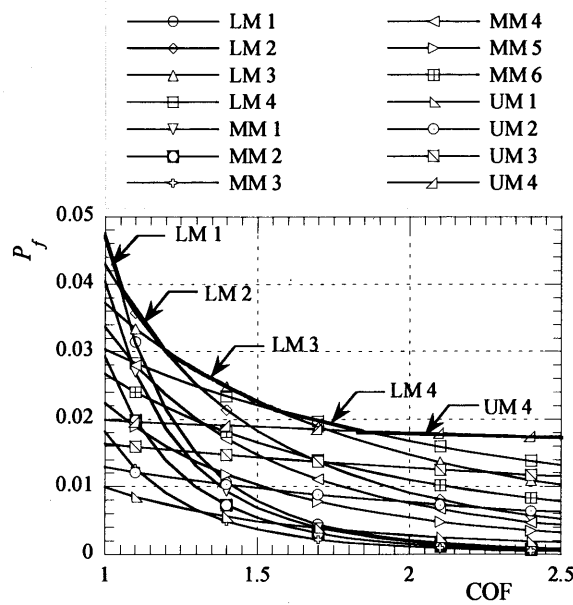


Fig. 2.24. The evaluation of likely failure mode (UBC-94)

It can be clearly observed from the following example of a five story frame. The failure modes of this frame are same as shown in Fig. 2.15.

The failure probabilities of all the modes are shown in Fig. 2.24. From this figure it is clearly observed that the lower failure modes and the upper failure mode with the maximum failure stories are the most likely failure modes.

2.8 Load Distribution of IBC-2006 and Determination of Load Level

2.8.1 A Brief Description of the Load Distribution of IBC-2006

In this section, the load distribution of the International Building Code (IBC-2006) is taken into account.

The mean values of the load of the upper floors μ_{pj} are obtained from mean value of load acting on the first floor of structure μ_p as shown in Eq. 2.3. In this Eq. C_j is the lateral load coefficient for j th story obtained from the distribution of load.

The seismic base shear specified by ASCE 7-05 and adopted by the International Building Code (IBC-2006) is determined according to the following equation:

$$V = C_s W \quad (2.40)$$

where W is the effective seismic weight and C_s is the seismic response coefficient defined as:

$$C_s = \frac{S_{DS}}{R/I} \quad (2.41)$$

where R is the response modification factor, I is the important factor and S_{DS} is the design spectral response acceleration in the short period range.

The seismic response coefficient need not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I)} \quad \text{for } T \leq T_L \quad (2.42)$$

$$C_s = \frac{S_{D1}T_L}{T^2(R/I)} \quad \text{for } T > T_L \quad (2.43)$$

The value of C_s shall not be less than 0.01. T_L is the long period transition period(s), S_{D1} in the above formula is the design spectral response acceleration at a period of 1 s. S_{DS} and S_{D1} are defined as follows:

$$S_{DS} = \frac{2}{3} F_a S_s \quad (2.44)$$

$$S_{D1} = \frac{2}{3} F_v S_1 \quad (2.45)$$

where F_a and F_v are the site coefficients. S_s is the mapped maximum considered earthquake spectral response acceleration at short period and S_1 is the mapped maximum considered earthquake spectral response acceleration at a period of 1 s.

T is the approximate fundamental period calculated as:

$$T = C_t h_n^x \quad (2.46)$$

where h_n is the height above base (ft) and C_t is equal to 0.028 and x is equal to 0.8 for steel moment resisting frame.

The lateral seismic force (F_x) at any level shall be determined from following two equations as:

$$F_x = C_{vx} V \quad (2.47)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (2.48)$$

where C_{vx} is the vertical distribution factor, V is the total shear at the base of the structure, w_i and w_x are the portion of the total effective load located or assigned to level i or x , h_i and h_x are the height from the base to level i or x and k is an exponent related to structure period. For structures having a period of 0.5 seconds or less, $k = 1$ and for structures having a period of 2.5 seconds or more, $k = 2$. For structures having a period between 0.5 and 2.5, k shall be determined by linear interpolation.

2.8.2 Determination of Load Level

The relation of mean value of load acting on the first floor of structure, μ_p and COF is shown in Fig. 2.25. From this Fig. also it is observed that the mean value of the load is generally a linear function of COF. A similar observation is also observed in case of Ai distribution of Japan as described in sec 2.4.2, in case of the load distribution of UBC-94 as described in sec 2.6.2 and in earlier studies by Zhao *et al.* (2002) and Pu and Zhao (2007) for triangular load.

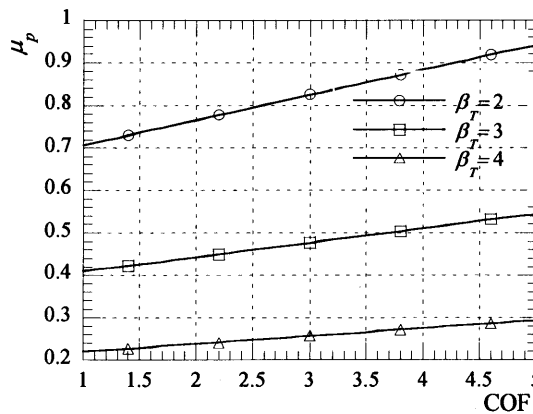


Fig. 2.25 Load-COF curve (IBC-2006)

2.9 Probabilistic Investigation on Story Collapse Modes Considering IBC-2006

The story failure modes are classified into three patterns: upper story failure pattern, middle story failure pattern and lower story failure pattern, each of which depends on the location of the failure stories, as shown in Fig. 2.5.

2.9.1 Investigation on Upper Story Mechanisms

The performance function for the upper collapse modes based on the principle of virtual work is similar to Eq. (2.30). The approximate expression of the second moment reliability index is also similar to Eq. (2.31). It should be noted that the mean value of the load which is

determined applying FORM considering the load distribution of IBC-2006 is different from that of Ai distribution.

Let us now consider a six story two bay frame having equal bay width of 8m and equal story height of 4m. For this six story frame, there are five upper collapse modes. These collapse modes are same as shown in Fig. 2.6.

The failure probabilities of the upper story failure modes are shown in Fig. 2.26. It is observed that the failure probabilities of the upper story failure modes steadily increase with the increase of the number of failure stories. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of Ai distribution and in case of the load distribution of UBC-94 also.

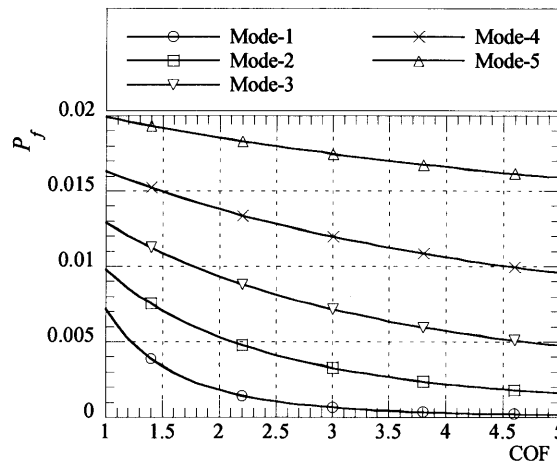


Fig. 2.26. Failure probability of upper story collapse modes (IBC-2006)

2.9.2 Investigation on Middle Story Mechanisms

The performance function for the middle collapse modes, based on the principle of virtual work is similar to Eq. (2.32). The approximate expression of the second moment reliability index is also similar to Eq. (2.33). However, the mean value of the load which is determined applying FORM considering the load distribution of IBC-2006 is different from that of Ai distribution.

For six story frame, there are ten middle collapse modes. These collapse modes are same as shown in Fig. 2.8.

The failure probabilities of the middle story failure modes are shown in Fig. 2.27 to Fig. 2.30 for number of collapse stories equal to 1 to 4 respectively.

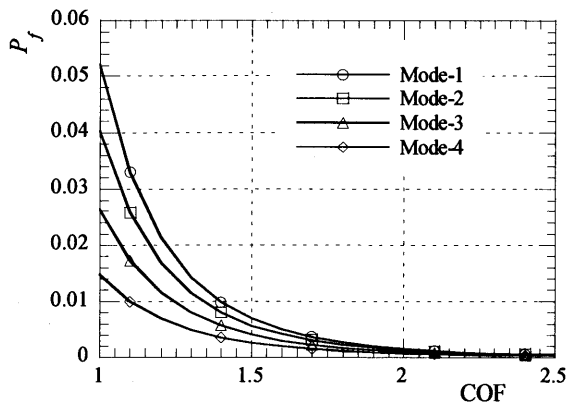


Fig. 2.27. P_f of middle modes with $n_c = 1$ (IBC-2006)

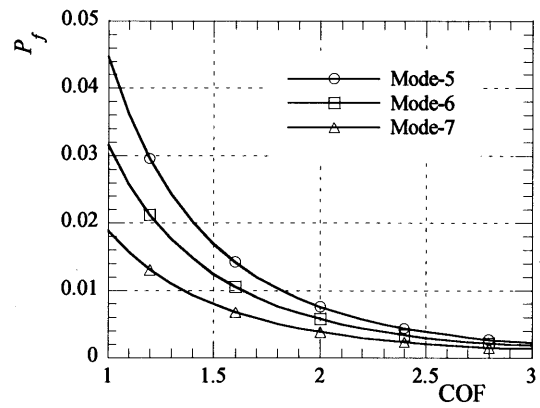


Fig. 2.28. P_f of middle modes with $n_c = 2$ (IBC-2006)

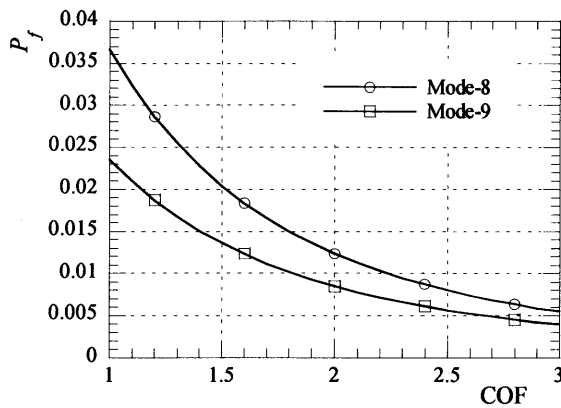


Fig. 2.29. P_f of middle modes with $n_c = 3$ (IBC-2006)

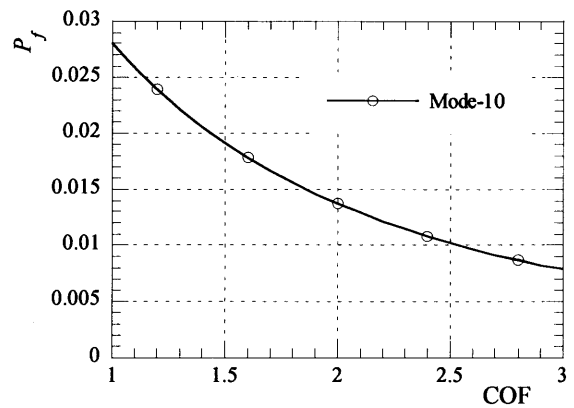


Fig. 2.30. P_f of middle modes with $n_c = 4$ (IBC-2006)

It is observed that in all cases the failure probability with higher n_b is less than that with lower n_b . That means the number of unbroken stories at the bottom n_b , has dominant effect on failure probabilities of the middle story failure modes. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of Ai distribution and in case of the load distribution of UBC-94 also.

2.9.3 Investigation on Lower Story Mechanisms

The performance function for the lower collapse modes, based on the principle of virtual work is similar to Eq. (2.34). The approximate expression of the second moment reliability index is also similar to Eq. (2.35). However, the mean value of the load which is determined applying FORM considering the load distribution of UBC-94 is different from that of Ai distribution.

For this six story frame, there are five lower collapse modes. These collapse modes are same as shown in Fig. 2.13.

The failure probabilities of the lower story failure modes of the frame considered are shown in Fig. 2.31.

It is observed that the failure probabilities of the lower story collapse modes do not follow any specific pattern. The value of COF has a significant effect on the probabilistic order of the lower story failure modes. Each lower story mechanism has a special COF region in which the failure probability of that mode is the largest. It is also observed that with the increase of COF the failure probabilities decrease. The similar observations are found in case of Ai distribution and in case of the load distribution of UBC-94 also.

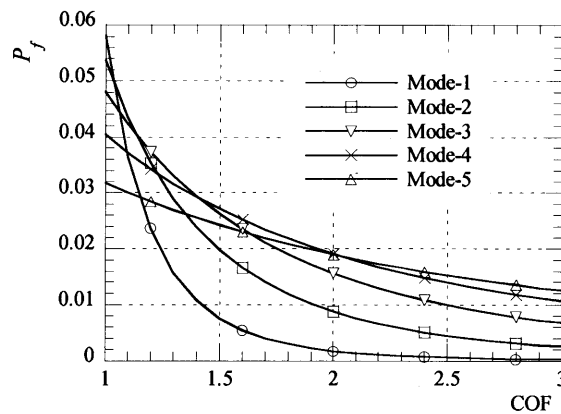


Fig. 2.31. Failure probability of lower story collapse modes (IBC-2006)

2.9.4 Likely Story Mechanisms

From the investigation, it is observed that, in case of the load distribution of IBC -2006 also, the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability among all the failure modes of a multi-story ductile frame structure.

It can be clearly observed from the following example of a five story frame. The failure modes of this frame are same as shown in Fig. 2.15.

The failure probabilities of all the modes are shown in Fig. 2.32. From this Fig. it is clearly observed that the lower failure modes and the upper failure mode with the maximum failure stories are the most likely failure modes.

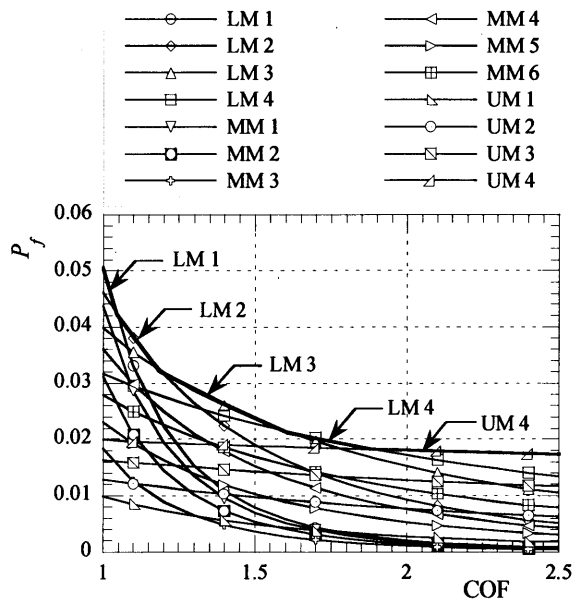


Fig. 2.32. The evaluation of likely failure mode (IBC-2006)

2.10 Conclusions

In this chapter the failure modes of multi-story ductile frame structures grouping into three categories; upper collapse, middle collapse and lower collapse are investigated probabilistically considering the A_i distribution of load and the distribution of UBC-94 and IBC-2006 along the height of the frame. From the investigation it is observed that

1. Under any specific reliability level, the mean value of the load is generally a linear function of COF. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006.
2. The failure probabilities of the middle story and upper story collapse modes follow some specific pattern but the failure probabilities of the lower story collapse modes do not follow any specific pattern. It is observed that in all cases of middle story collapse modes the failure probability with higher n_b (number of unbroken story at the bottom of the frame) is less than that with lower n_b . In case of upper story collapse modes, it is observed that the failure probability steadily increase with the increase of the number of failure stories. The value of COF has a significant effect on the probabilistic order of the lower story failure modes. Each lower story mechanism has a special COF region in which the failure probability of that mode is the largest. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006.

3. With the increase of the COF the failure probabilities of all modes decrease. A similar observation is found in case of Ai distribution of load and the distribution of UBC-94 and IBC-2006.
4. Among all the failure modes of a multi-story ductile frame structure the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability, i.e., these modes are the most likely failure modes of a multi-story ductile frame structure. A similar observation is found in case of Ai distribution of load and the distribution of UBC-94 and IBC-2006. That means, most likely failure modes are independent of the type of the distribution applied in the evaluation.

Chapter 3

ESTIMATION OF TARGET COLUMN OVERDESIGN FACTORS AVOIDING STORY MECHANISM

3.1 Introduction

The column overdesign factor (COF) requirement that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame structure has been evaluated in this chapter. Although there are many code provisions regarding the values of COF, but in reality we observe that many structures collapse unexpectedly according to some undesirable failure modes, because of the uncertainties associated with the member strength and the earthquake loads. Therefore, in the present research the probabilistic approach has been adopted. Considering the uncertainties of earthquake load and strengths of structural members, the probabilistic priority of the preferable beam hinging mode has been evaluated.

Target values of COF that ensure probabilistically the preferable entire beam hinging failure mode of frames have been evaluated by Ono *et al.* (2000) and Zhao *et al.* (2002). Using this probabilistic evaluation method Yoshihara *et al.* (2004) showed that the probabilistic evaluation of COF is not affected by the vertical loads of the frame and Pu and Zhao (2007) showed that the probabilistic evaluation of COF is not affected by the number of bays of the frame. However, all the investigations were conducted based on triangular distribution of load along the height of the frame. The triangular distribution is simple and convenient for analytical investigation, but nowadays this distribution is not commonly used. In Japan the Ai distribution of load is used, but the probabilistic evaluation of COF based on this distribution and the effect of many parameters on target COF have not been investigated sufficiently so far. In the present chapter the target COF requirement has been investigated based on Ai distribution of the Building Standard Law of Japan. The effects of some given parameters on target COF also have been investigated. The parameters considered are number of story, reliability index, height irregularity and mass irregularity etc. The target COF requirements based on the base shear distribution of UBC-1994 and IBC-2006 have also been investigated in the present chapter.

3.2 Target COF Evaluation

3.2.1 Evaluation Index

To probabilistically avoid the story mechanisms, the probabilities of the story mechanisms should be controlled at least lower than that of the entire beam hinging failure mode. In the target COF evaluation, following evaluation index is used:

$$\gamma = P_{f2} / P_{f1} \quad (3.1)$$

where P_{f1} = the occurrence probability of the beam hinging failure mode and P_{f2} = the occurrence probability of the story mechanisms.

In the target COF evaluation, the reliability index of the entire beam hinging mode β_T should be given first to indicate the safety requirement of the structure. P_{f1} is the probability corresponding to the reliability index, namely

$$P_{f1} = \Phi(-\beta_T) \quad (3.2)$$

The method used in this research is to assume a reliability index such as $\beta_T=2$ or $\beta_T=3$ for the entire beam hinging failure mode first to specify the safety level of the structure and then to compute the mean value of the earthquake load using first order reliability method (FORM) to ensure that the first order reliability index becomes equal to the target reliability index β_T for frame structures designed with various COFs. The obtained load is then applied to compute the probabilities of the undesirable story mechanisms.

The probabilities of the preferable collapse mode and the undesirable collapse mode are calculated under the same load conditions; otherwise the evaluation index γ is meaningless.

After obtaining the aforementioned evaluation index, to ensure probabilistically that the designed structure collapses according to the designed preferable failure mode, the relative occurrence rate of the most likely story mechanism γ should be controlled lower than a specific allowable level γ_0 as follows:

$$\gamma = P_{f2} / P_{f1} \leq \gamma_0 \leq 1 \quad (3.3)$$

By conducting the failure mode analysis and the reliability analysis using a different COF for a frame structure, a γ -COF curve can be obtained and the target value of the COF for which Eq. 3.3 is satisfied can be determined. The larger the value of COF, the smaller the value of the relative occurrence rate of the undesirable failure modes.

When $\gamma_0=1$, the undesirable failure mode and the preferable entire beam-hinging mode have the same likelihood of occurrence, i.e., both probabilities are equal. A COF value lower than

the value corresponding to $\gamma_0 = 1$ enhances the story collapse, i.e., the probability of the story collapse is higher than that of the entire beam-hinging mode, therefore this value is not allowed. The threshold value of COF when $\gamma_0 = 1$ is defined here as the target or basic COF. When the COF of the frame is higher than the target COF, the occurrence probability of the undesirable story collapse failure modes considered, i.e., most likely failure modes is less than that of the expected entire beam hinging failure mode.

The P_f -COF and γ -COF curves of a six story frame are presented in Fig. 3.1 and Fig. 3.2 respectively.

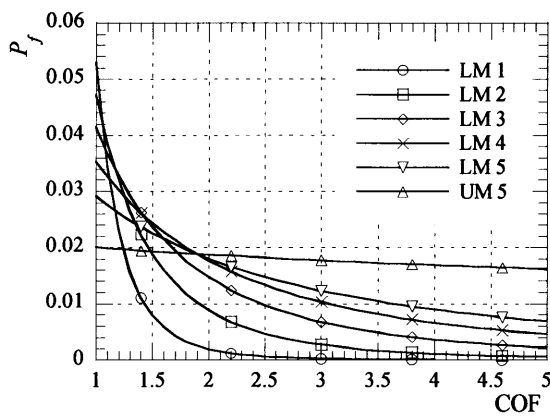


Fig. 3.1. P_f -COF curve of a six story frame ($\beta_T=2$)

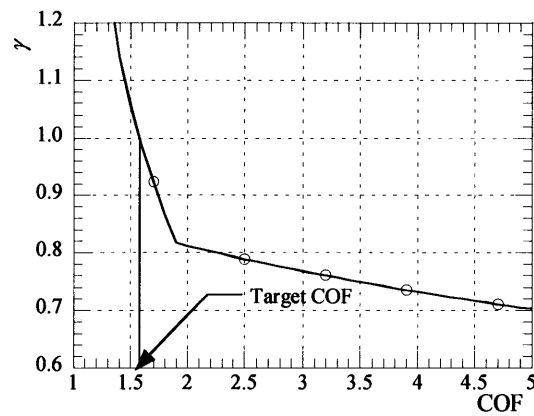


Fig. 3.2. γ -COF curves of a six story frame ($\beta_T=2$)

3.2.2 Evaluation of Target COF for Multi-story Frames

In this section the target values of COF for three story to seven story frame under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$ and $\beta_T=4$) have been evaluated based on aforementioned evaluation index.

Fig. 3.3, Fig. 3.4, and Fig. 3.5 show the γ -COF for three story to seven story frame under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$ and $\beta_T=4$) respectively. From these Figs. it is observed that target COF increases with the number of story and decreases with the increase of reliability levels.

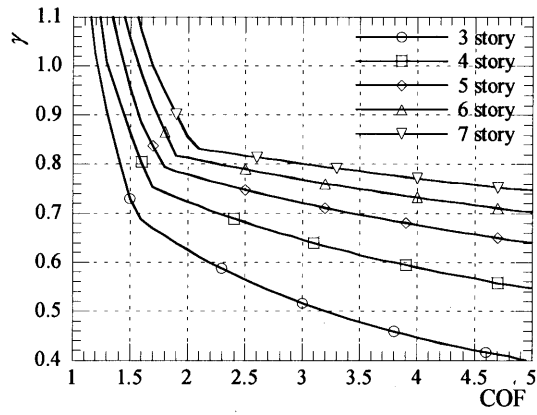


Fig. 3.3. γ -COF curves for three to seven story frame ($\beta_T=2$)

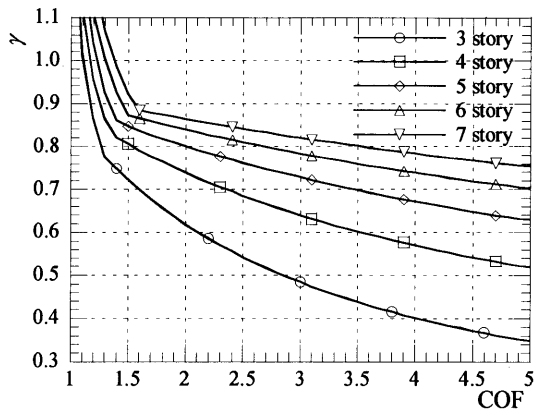


Fig. 3.4. γ -COF curves for three to seven story frame ($\beta_T=3$)

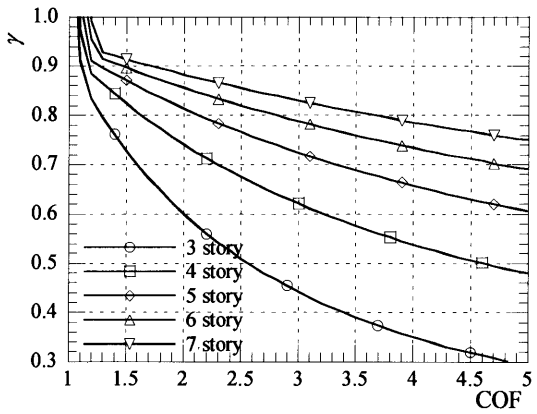


Fig. 3.5. γ -COF curves for three to seven story frame ($\beta_T=4$)

3.2.3 Basic and Optimum COF for Multi-story Frame

If we observe the γ -COF curve we will easily notice a remarkable point at which γ -COF curve changes abruptly. The COF value corresponding to this point is defined here as optimum COF. The optimum COF has been clearly shown in the Fig. 3.6.

It is clear from the Fig. 3.6 that in the COF range left to this point, the evaluation index decreases rapidly with the increase of COF; in the COF range right to this point the evaluation index decreases very slowly with the increase of the COF. This is because the shape of the evaluation curve is determined by the maximum failure probabilities of the undesirable story collapse modes. The left portion of the curve is controlled by the lower story failure modes and the right portion of the curve is controlled by the upper story failure mode with highest failure story. It is obvious that increasing the COF in the left region of the controlling point is very effective in controlling the undesirable story collapse modes and to ensure probabilistically the desirable beam hinging failure mode. The COF range between the basic COF and optimum COF can be used for design purpose.

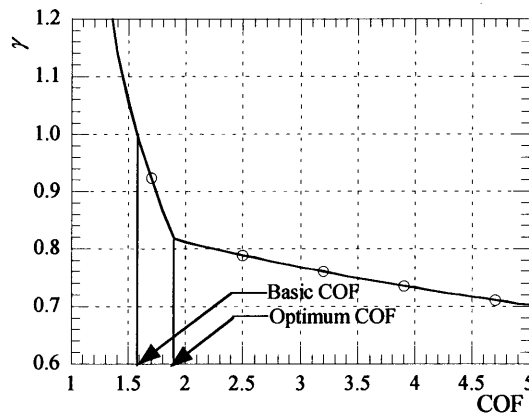


Fig. 3.6. Basic and Optimum COF

The basic and optimum COF for three to seven story building frames based on A_i distribution of load under reliability levels 2, 3 and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) have been presented in Table 1.

Table 3.1 Basic and optimum COF for multi-story frames

		3story	4story	5story	6story	7story
$\beta_T=2$	Basic.	1.22	1.31	1.45	1.58	1.70
	Optimum.	1.61	1.71	1.80	1.90	2.1
$\beta_T=3$	Basic.	1.11	1.17	1.23	1.31	1.38
	Optimum.	1.30	1.31	1.40	1.50	1.60
$\beta_T=4$	Basic.	1.07	1.09	1.13	1.16	1.19
	Optimum.	1.11	1.19	1.20	1.30	1.31

In this research, the main interest is the evaluation of target COF. Therefore all the following study and discussion are related to only target COF.

3.2.4 Target COF with Number of Story and Reliability Indices

Fig. 3.7 shows the change of target COF with number of story under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$). It is observed that under the same reliability level, the target COF requirement increases with the increase of the number of stories. The rate of change of target COF with number of story is almost linear.

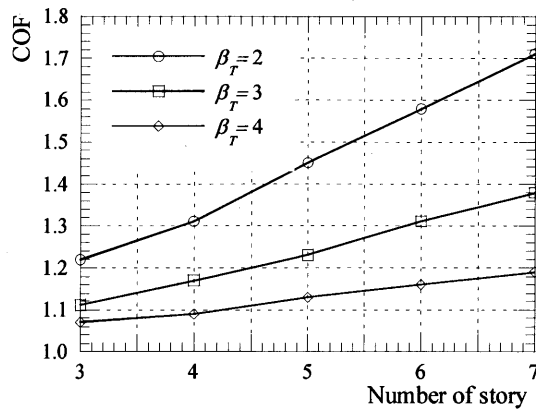


Fig. 3.7. Target COF with number of story

Fig. 3.8 shows the change of target COF with reliability levels ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) for three to seven story frames. It is observed that for any frame, the target COF requirement decreases with the increase of the reliability level. The rate of change of target COF with reliability indices is also almost linear.

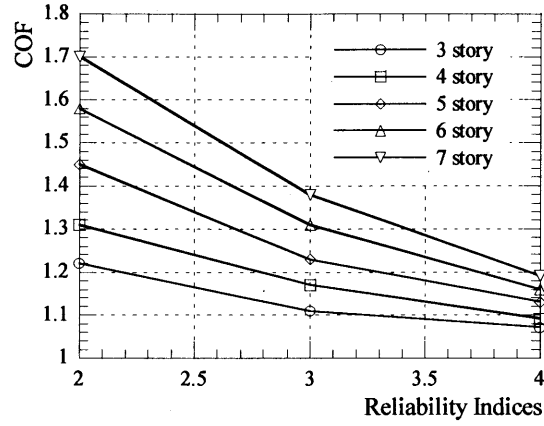


Fig. 3.8. Target COF with reliability indices

Target COF with some other parameters such as COV of material strength and load are also investigated. It is observed that the COV of material strength merely affects the failure probability of the structure and the target COF requirement while that of load has some effect.

3.2.5 Empirical Formula for Target COF

The trial and error method is applied to form equation that reasonably estimates the COF values obtained from FORM analysis. After trial and error the following Eq. 3.4 has been obtained for the evaluation of the least COF for regular frames in terms of number of story and reliability index. This can approximately estimate the least COF for multi-story frames.

$$COF = (a + b)n \quad (3.4)$$

The factor a is equal to 0.17, 0.105 and 0.04 for $\beta_T=2$, $\beta_T=3$, and $\beta_T=4$ respectively. These values can also be estimated by the following Eq. 3.5 in terms of reliability index, β_T as follows:

$$a = 3/10 - 6.5/100 * \beta_T \quad (3.5)$$

The factor b can be estimated by the following Eq. 3.6. The factor c used in this equation is equal to 1.29, 1.17 and 0.98 for $\beta_T=2$, $\beta_T=3$, and $\beta_T=4$ respectively, which can also be estimated by the following Eq. 3.7 in terms of reliability index, β_T as follows:

$$b = 1/\beta_T^{7.5*10^{-2}} * 1/n^c \quad (3.6)$$

$$c = 1.32 + 5.5/100 * \beta_T - 3.5/100 * \beta_T^2 \quad (3.7)$$

The target values of COF for three to seven story frames have been computed using the aforementioned equations and compared with that obtained from FORM. From Table 3.2 we can see that the values obtained from FORM and that from the proposed equation are very close.

Table 3.2 Target COF for multi-story frames applying FORM and equation

		3story	4story	5story	6story	7story
$\beta_T=2$	FORM	1.22	1.31	1.45	1.58	1.70
	Equation	1.20	1.32	1.45	1.58	1.73
$\beta_T=3$	FORM	1.11	1.17	1.23	1.31	1.38
	Equation	1.08	1.15	1.23	1.31	1.40
$\beta_T=4$	FORM	1.07	1.09	1.13	1.16	1.19
	Equation	1.04	1.09	1.13	1.17	1.22

3.3 Effect of Height Irregularity on Target COF

Irregular buildings constitute a large portion of the modern infrastructure. The group of people involved in constructing the building facilities, including owner, architect, structural engineer, contractor and local authorities contribute to the overall planning, selection of the structural system and to its configuration. This may lead to the building structure with irregular configuration. Irregular configurations either in plan or in elevation were often regarded as one of the main causes of failure during past earthquakes (Athanassiadou 2007). Therefore, irregularity in any form in the building structures has been a major concern in the earthquake engineering society over the last several decades.

It is observed in the usual construction practices that the story height of the first story (first level) is higher than that of the remaining upper stories. This most common form of irregularity due to story height variation is mentioned hereinafter as height irregularity.

Three basic frames are utilized in this analytical investigation: four story, five story and six story two bay frames. For each frame bay width is selected 8m. Each frame is analyzed for reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$).

The failure probability and COF requirement of the frames with higher floor height in the first story has been evaluated probabilistically in this section. For regular case, all the story height is considered to be 4m ($h_1 = h$), where h_1 is the story height of the first story and h is the story height of the remaining each story. Five other cases are considered. These are $h_1 = 1.1h$, $h_1 = 1.2h$, $h_1 = 1.3h$, $h_1 = 1.4h$ and $h_1 = 1.5h$. So in all the cases, the typical floor height is same but the height of the first story is changing from 4m to 6m at an increment of 0.4m.

The performance function for the three story failure patterns can be established as follows:

$$G_U(\mathbf{X}) = 2 \sum_{i=1}^m M_{bni} + 2 \sum_{j=n-n_c+1}^{n-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^2 M_{cl} + \sum_{l=1}^{m-1} M_{cl} - \sum_{j=n-n_c+1}^n \left(\sum_{i=n-n_c+1}^j h_i P_j \right) \quad (3.8)$$

$$G_M(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^4 M_{csl} + \sum_{l=1}^{2m-2} M_{cl} - \sum_{j=1}^{n_c} \left(\sum_{i=n_b+1}^{n_b+j} h_i P_{j+n_b} \right) - \sum_{i=n_b+1}^{n_b+n_c} h_i \sum_{j=n_c+1}^{n-n_b} P_{j+n_b} \quad (3.9)$$

$$G_L(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} \sum_{i=1}^m M_{bij} + \sum_{l=1}^4 M_{csl} + \sum_{l=1}^{2m-2} M_{cl} - \sum_{j=1}^{n_c} \left(\sum_{i=1}^j h_i P_j \right) - \sum_{i=1}^{n_c} h_i \sum_{j=n_c+1}^n P_j \quad (3.10)$$

where G_L , G_M and G_U are the performance functions of the lower story failure pattern, the middle story failure pattern and the upper story failure pattern respectively. M_{bni} is the moment strength of the beam of the top story, M_{bij} is the moment strength of the beam of the i th span and j th story, M_{cl} is the moment strength of an interior column, M_{csl} is the moment strength of an exterior column, P_j is the load acting on the j th story of the structure, n is the number of stories, n_c is the number of failure stories, n_b is the number of unbroken stories at the bottom of the structure, m is the number of spans and h_i is the story height i th level of the structure.

Failure probabilities of the frames and the target COF requirements have been computed using FORM under different reliability level and different configuration of frames. It is observed that the failure probability of the frame changes with the change of the hl/h ratio. A case of lower story collapse mode (LM 2) of a six story frame under reliability level 2 ($\beta_T=2$) is shown in Fig. 3.9 which shows that failure probability increases with the increase of the hl/h ratio.

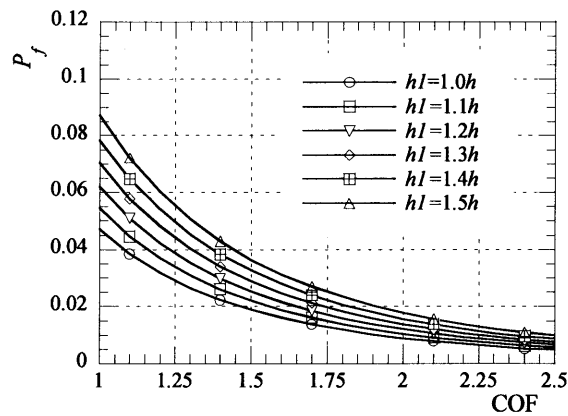


Fig. 3.9. Failure probability with height irregularity

It is also observed that the target COF requirement avoiding story mechanism is minimum in

case of regular frame having $hl = h$. When $hl = 1.5h$ the COF requirement is maximum. As the hl/h increases from 1.0 to 1.5 at a step of 0.1 the target COF requirement also increases gradually. That means the higher the height irregularity, the higher the target COF requirement and as the frame comes closer to regular frame the target COF requirement is also decreases gradually.

Fig.3.10 shows the γ -COF curve for six story frame under reliability level 2($\beta_T=2$) with height irregularity.

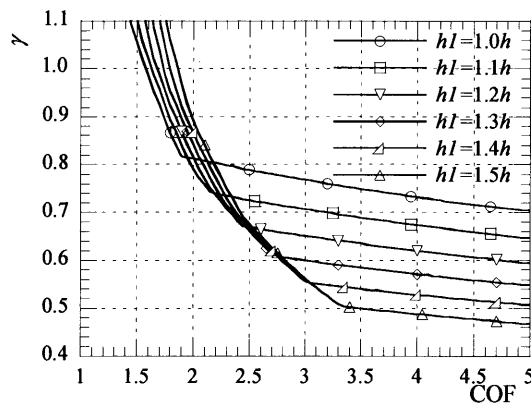


Fig. 3.10. γ -COF curves with height irregularity ($\beta_T=2$)

Table 3.3 shows the target COF for four story to six story frame with reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$) under different height irregularity of the frames.

Table 3.3 Target COF requirement with height irregularity

Configuration	$\beta_T=2$			$\beta_T=3$		
	Four story	Five story	Six story	Four story	Five story	Six story
$hl = 1.0h$	1.31	1.45	1.58	1.17	1.23	1.31
$hl = 1.1h$	1.38	1.51	1.63	1.23	1.30	1.37
$hl = 1.2h$	1.44	1.57	1.69	1.30	1.36	1.42
$hl = 1.3h$	1.50	1.62	1.74	1.38	1.42	1.49
$hl = 1.4h$	1.57	1.68	1.80	1.46	1.50	1.56
$hl = 1.5h$	1.63	1.74	1.87	1.55	1.58	1.62

Fig. 3.11 shows the rate of change of target COF with height irregularity for four to six story frames. It is observed that the increase of the COF with the increase of the height irregularity is almost linear.

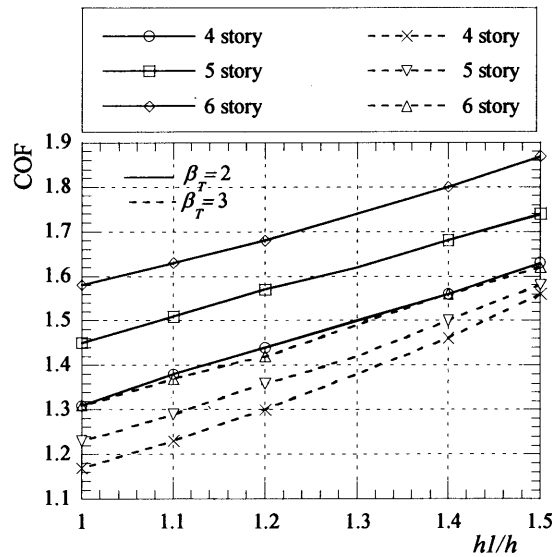


Fig. 3.11. Target COF with height irregularity

3.4 Effect of Mass Irregularity on Target COF

A building structure fulfills different functions at various levels over their height, e.g., buildings with floors used for commercial purposes, car parking floors or heavy mechanical equipment. The different use of a specific floor compared to the adjacent ones result in mass irregularity.

Current seismic codes specify rules for characterizing a structure as irregular in elevation due to mass discontinuities. According to UBC (1997) mass irregularity is considered to exist where the mass of any story is more than 150% of the mass of an adjacent story.

The mass irregularity is described here by the mass ratio, m_r which is the ratio of the heavier mass (w_1) applied to an arbitrarily selected floor over the mass of the adjacent floor (w). The floor for heavier mass is selected in the middle region of the frames.

The mass ratio is incrementally changed to observe its effect on failure probability and target COF requirement of the frames. The frames are considered with mass ratio, m_r equal to 1.0, 1.2, 1.4, 1.6, 1.8 and 2.0 i.e., both above and below the value mentioned in the design code (1.5) have been considered.

Three basic frames are utilized in this analytical investigation: four story, five story and six story two bay frames. For each frame bay width is selected 8m. Each frame is analyzed for reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$). The selected floor for heavier mass is shown in Fig. 3.12.

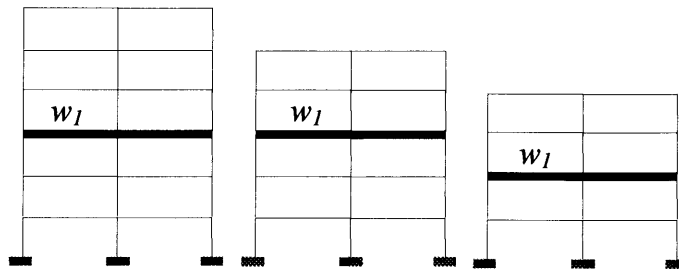


Fig. 3.12. Frame with selected floor for heavier mass

Failure probabilities of the frames and the target COF requirements have been computed using FORM under different reliability level and different mass ratio. It is observed that the failure probability of the frame changes with the change of the mass ratio. A case of lower story collapse mode (LM 2) of a six story frame under reliability level 2 ($\beta_T=2$) is shown in Fig. 3.13 which shows that failure probability increases with the increase of the mass ratio.

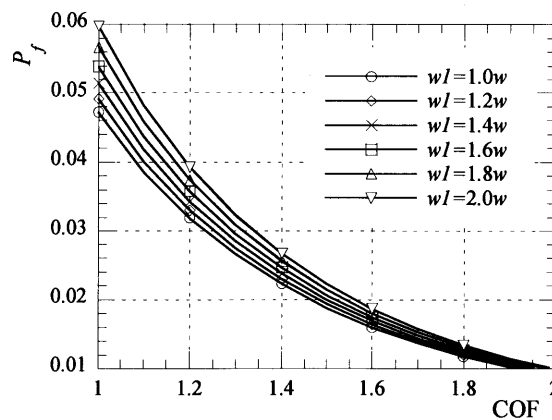


Fig. 3.13. Failure probability with mass irregularity

It is observed that the target COF requirement avoiding story mechanism is minimum in case of regular frame having $m_r = 1.0$ and when $m_r = 2.0$ the COF requirement is maximum. As the m_r increases from 1.0 to 2.0 at a step of 0.2 the target COF requirement also increases gradually. That means the higher the m_r , the higher the target COF requirement and as the frame comes closer to regular frame with $m_r = 1.0$ the target COF requirement is also gradually decreasing.

Fig.3.14 shows the γ -COF curve for six story frame under reliability level 2($\beta_T=2$) with mass irregularity.

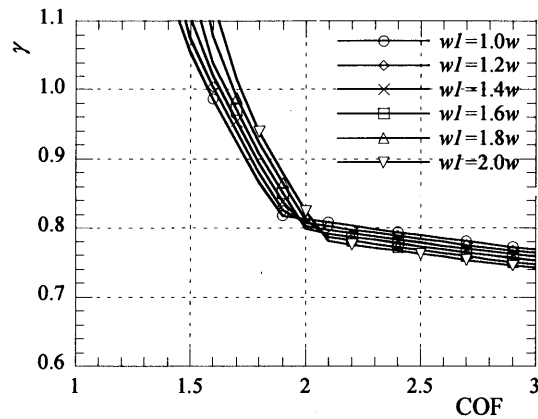


Fig. 3.14. γ -COF curves with mass irregularity ($\beta_T=2$)

Table 3.4 shows the target COF for four story to six story frame with reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$) under different mass irregularity of the frames.

Table 3.4 Target COF requirement with mass irregularity

Mass ratio	$\beta_T=2$			$\beta_T=3$		
	Four story	Five story	Six story	Four story	Five story	Six story
1.0	1.31	1.45	1.58	1.17	1.23	1.31
1.2	1.34	1.49	1.60	1.18	1.26	1.32
1.4	1.37	1.53	1.63	1.19	1.28	1.33
1.6	1.40	1.57	1.66	1.20	1.29	1.35
1.8	1.44	1.62	1.69	1.23	1.31	1.37
2.0	1.49	1.67	1.72	1.24	1.33	1.38

Fig. 3.15 shows the nature of the change of target COF with mass ratio for four to six story frames. It is observed that target COF requirement increases almost linearly with the increase of the mass ratio.

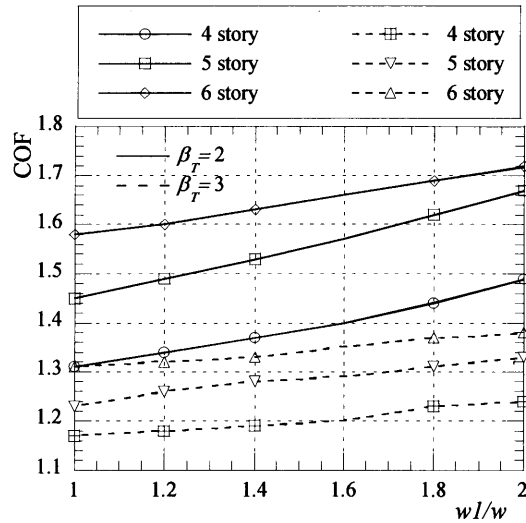


Fig. 3.15. Target COF with mass irregularity

3.5 Effect of Combined Height and Mass Irregularity on Target COF

In this section the frames with given mass ratio ($m_r=1.6$) is selected first. Then the height of the first level is changed from 4m to 6m at an increment of 0.4m. That means the cases considered are $h1 = 1.0h$, $h1 = 1.1h$, $h1 = 1.2h$, $h1 = 1.3h$, $h1 = 1.4h$ and $h1 = 1.5h$.

Three basic frames are utilized in this analytical investigation: four story, five story and six story two bay frames. For each frame bay width is selected 8m. Each frame is analyzed for reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$).

Failure probabilities of the frames and the target COF requirements have been computed using FORM under different reliability level and different configuration of frames. It is observed that the failure probability of the frame changes with the change of the $h1/h$ ratio. These failure probabilities are higher than the failure probability of the frames considered with only height irregularity or only mass irregularity. A case of lower story collapse mode (LM 2) of a six story frame under reliability level 2 ($\beta_T=2$) is shown in Fig. 3.16 which shows that failure probability increases with increasing height irregularity of the mass irregular frame. Comparing with Fig. 3.9 and Fig. 3.13 it is observed that failure probabilities of the frame with combined height and mass irregularity are higher than the failure probabilities of the frames considered with only height irregularity or only mass irregularity.

It is also observed that as the $h1/h$ increases from 1.0 to 1.5 the target COF requirement also increases gradually. These COF values are higher than the COF required for the frames considered with only height irregularity or only mass irregularity. That means combined effect of mass and height irregularity is the most critical and COF requirement is higher than that

with only height irregularity or only mass irregularity.

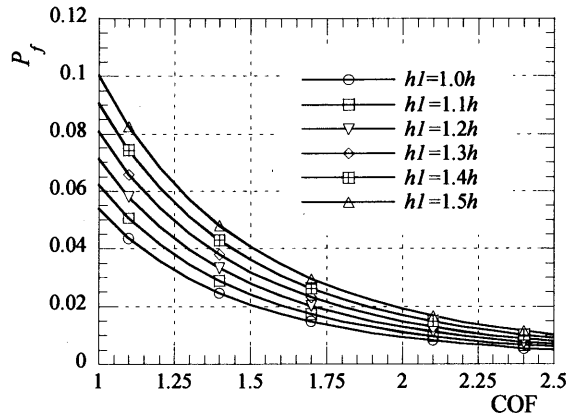


Fig. 3.16. Failure probability with height and mass irregularity

Fig. 3.17 shows the γ -COF curve for six story frame under reliability level 2 ($\beta_T=2$) with height and mass irregularity ($m_r=1.6$).

Table 3.5 shows the target COF requirement for four to six story frames with reliability levels 2 and 3 ($\beta_T=2$ and $\beta_T=3$) with height and mass irregularity ($m_r=1.6$).

Fig. 3.18 shows the nature of the change of target COF with height and mass irregularity for four to six story frames. It is observed that target COF requirement of the frames increases almost linearly.

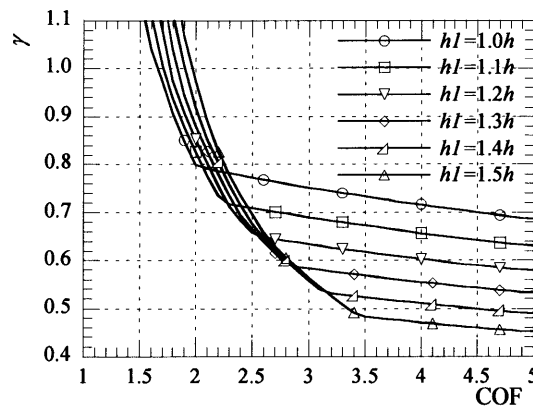


Fig. 3.17. γ -COF curves with height and mass irregularity ($\beta_T=2$)

Table 3.5 Target COF requirement with height and mass irregularity($m_r=1.6$)

Configuration	$\beta_T=2$			$\beta_T=3$		
	Four story	Five story	Six story	Four story	Five story	Six story
$hl=1.0h$	1.40	1.57	1.66	1.20	1.29	1.35
$hl=1.1h$	1.48	1.63	1.70	1.28	1.35	1.41
$hl=1.2h$	1.54	1.69	1.77	1.34	1.40	1.48
$hl=1.3h$	1.61	1.73	1.84	1.40	1.47	1.54
$hl=1.4h$	1.68	1.79	1.91	1.48	1.53	1.60
$hl=1.5h$	1.73	1.84	1.97	1.57	1.59	1.67

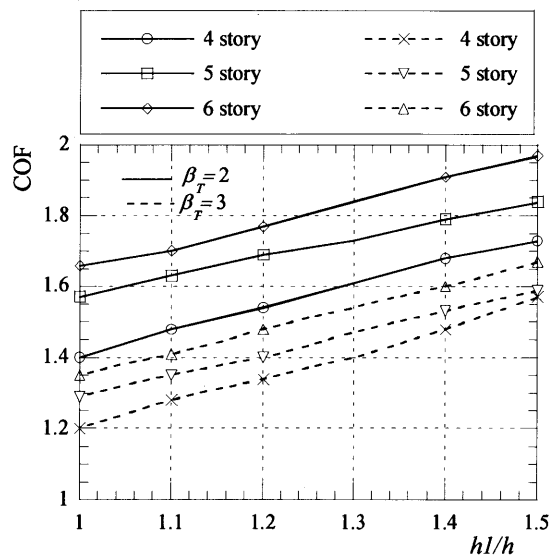


Fig. 3.18. Target COF with height and mass irregularity

3.6 Evaluation of Target COF for Multi-story Frames Considering UBC-1994

In this section the target values of COF for three story to seven story frame under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) have been evaluated based on aforementioned evaluation index of section 3.2.1 considering the base shear distribution of Uniform Building Code (UBC-94).

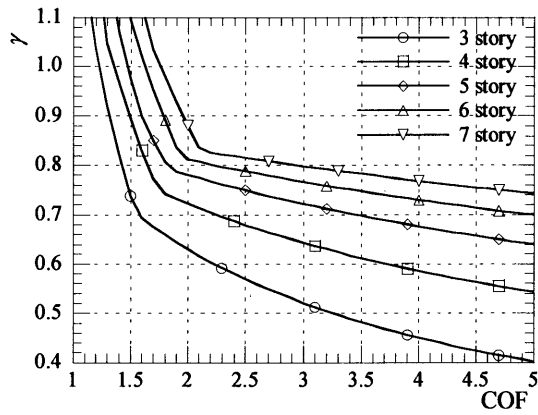


Fig. 3.19. γ -COF curves for three to seven story frame ($\beta_T=2$)

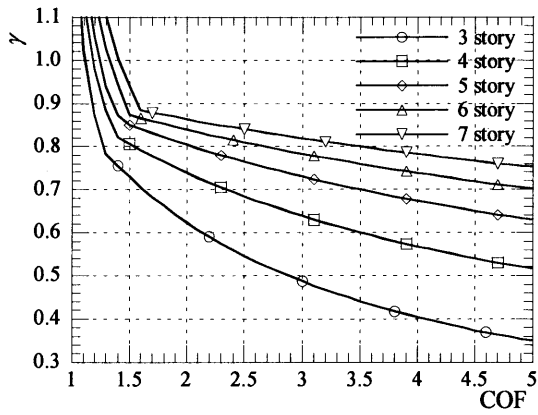


Fig. 3.20. γ -COF curves for three to seven story frame ($\beta_T=3$)

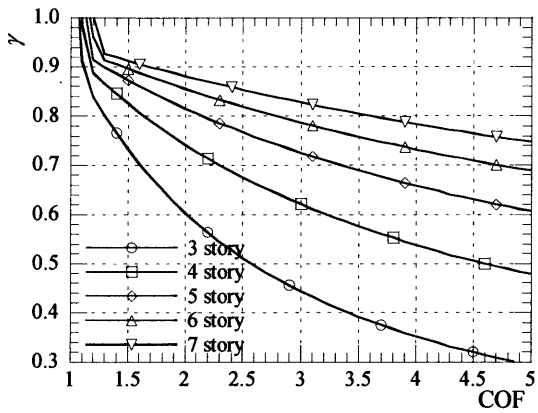


Fig. 3.21. γ -COF curves for three to seven story frame ($\beta_T=4$)

Fig. 3.19, Fig. 3.20, and Fig. 3.21 show the γ -COF for three story to seven story frames considering base shear distribution of UBC-94 under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$ and $\beta_T=4$) respectively. From these Figs. it is observed that target COF increases with the

number of story and decreases with the increase of the reliability levels.

The target COF for three to seven story building frames under reliability levels 2, 3 and 4 ($\beta_T=2, \beta_T=3,$ and $\beta_T=4$) have been presented in Table 3.6.

Table 3.6 Target COF for multi-story frames considering UBC-94

Reliability level	3story	4story	5story	6story	7story
$\beta_T=2$	1.23	1.36	1.48	1.63	1.76
$\beta_T=3$	1.11	1.19	1.24	1.33	1.41
$\beta_T=4$	1.07	1.09	1.13	1.17	1.20

Fig. 3.22 shows the change of target COF with number of story under reliability levels 2, 3, and 4 ($\beta_T=2, \beta_T=3,$ and $\beta_T=4$).

It is observed that under the same reliability level, the target COF requirement increases with the increase of the number of stories. It is also observed that under the same reliability level, the rate of change of target COF requirement with the number of stories is almost linear. A similar observation is also observed in case of Ai distribution of Japan as described in sec. 3.2.4.

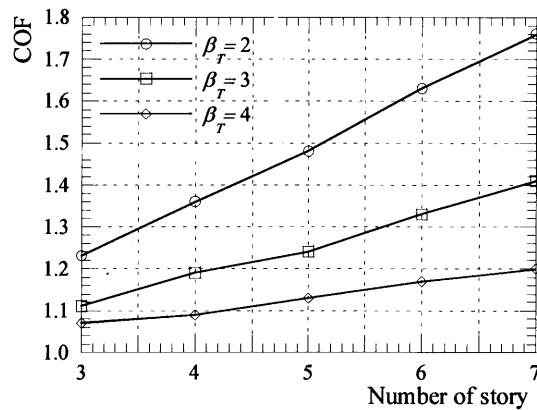


Fig. 3.22. Target COF with number of story

Fig. 3.23 shows the change of target COF with reliability levels ($\beta_T=2, \beta_T=3,$ and $\beta_T=4$) for three to seven story frames.

It is observed that for any frame, the target COF requirement decreases with the increase of the reliability level. It is also observed that the rate of change of target COF with the reliability levels is almost linear. A similar observation is also observed in case of Ai distribution of Japan as described in sec. 3.2.4.

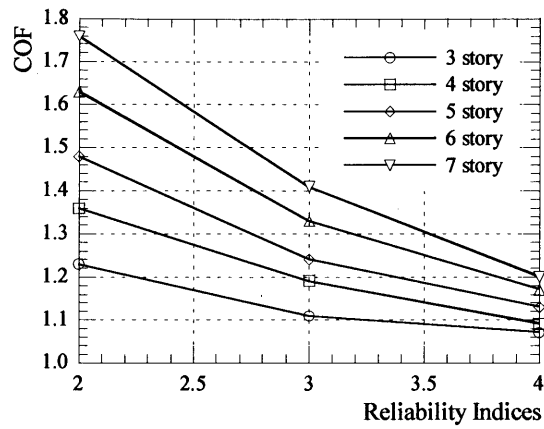


Fig. 3.23. Target COF with reliability indices

3.7 Evaluation of Target COF for Multi-story Frames Considering IBC-2006

In this section the target values of COF for three story to six story frame under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) have been evaluated based on aforementioned evaluation index of section 3.2.1 considering the base shear distribution of International Building Code (IBC-2006).

Fig. 3.24, Fig. 3.25, and Fig. 3.26 show the γ -COF for three story to seven story frames considering the base shear distribution of IBC-2006 under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$ and $\beta_T=4$) respectively. From these Figs. it is observed that target COF increases with the number of story and decreases with the increase of the reliability levels.

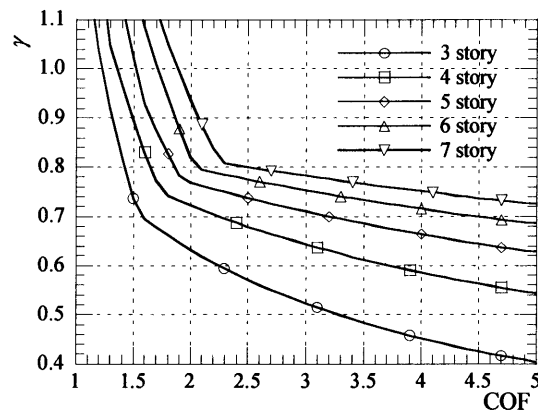


Fig. 3.24. γ -COF curves for three to seven story frame ($\beta_T=2$)

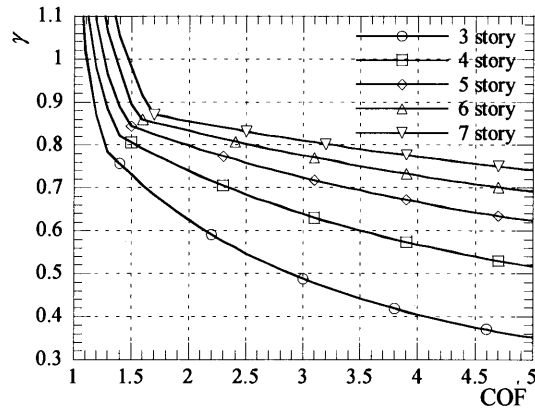


Fig. 3.25. γ -COF curves for three to seven story frame ($\beta_T=3$)

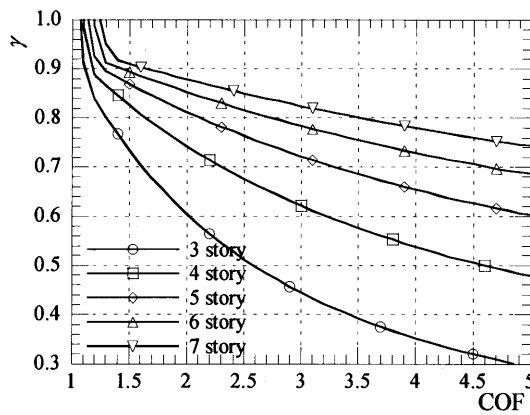


Fig. 3.26. γ -COF curves for three to seven story frame ($\beta_T=4$)

The target COF for three to seven story building frames under reliability levels 2, 3 and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) have been presented in Table 3.7.

Table 3.7 Target COF for multi-story frames considering IBC-2006

Reliability level	3story	4story	5story	6story	7story
$\beta_T=2$	1.23	1.36	1.52	1.71	1.89
$\beta_T=3$	1.11	1.19	1.27	1.37	1.48
$\beta_T=4$	1.07	1.09	1.14	1.19	1.24

Fig. 3.27 shows the change of target COF with number of story under reliability levels 2, 3, and 4 ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$).

It is observed that under the same reliability level, the target COF requirement increases with the increase of the number of stories. It is also observed that under the same reliability level,

the rate of change of target COF requirement with the number of stories is almost linear. A similar observation is also observed in case of Ai distribution of Japan as described in sec. 3.2.4. and in case of UBC-94 also as described in sec. 3.6.

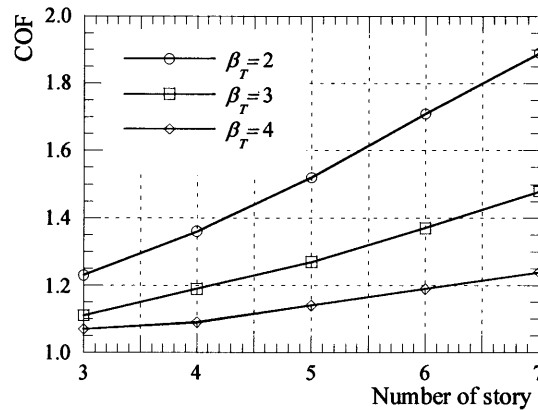


Fig. 3.27. Target COF with number of story

Fig. 3.28 shows the change of target COF with reliability levels ($\beta_T=2, \beta_T=3,$ and $\beta_T=4$) for three to seven story frames.

It is observed that for any frame, the target COF requirement decreases with the increase of the reliability level. It is also observed that under the same reliability level, the rate of change of target COF with the reliability levels is almost linear. A similar observation is also observed in case of Ai distribution of Japan as described in sec. 3.2.4. and in case of UBC-94 also as described in sec. 3.6.

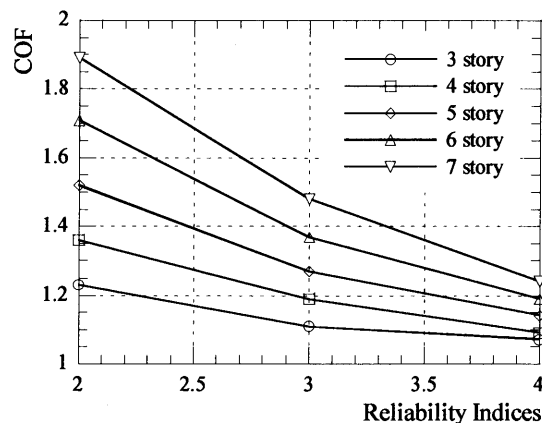


Fig. 3.28. Target COF with reliability indices

3.6 Conclusions

In this chapter, considering the uncertainties of earthquake load and strengths of structural members, the target COF requirement that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and avoid probabilistically the undesirable story collapse modes of the frame structures have been evaluated under reliability levels ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) considering three different load distributions. The load distributions are: the Ai distribution of the Building Standard Law of Japan, the load distributions of the Uniform Building Code (UBC-1994), and the International Building Code (IBC-2006). The effect of height and mass irregularity on target COF has been also investigated considering the Ai distribution of Japan. The investigation presented in this chapter can be summarized by the following conclusions.

- (1) The target COF is the minimum value of COF that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame. The target values of COF for three to seven story frames under reliability levels ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) based on Ai distribution of load and the distribution of UBC-94 and IBC-2006 are presented in this chapter.
- (2) Target COF increases with the increase in the number of stories and decrease with the increase in target reliability level of the frame. It is also observed that the rate of change of target COF with number of story and reliability level is almost linear. A similar observation is found in case of Ai distribution of load and the distribution of UBC-94 and IBC-2006.
- (3) Higher COF value has to be provided for frames with higher floor height in first story. The higher the height irregularity due to story height variation in the first story, the higher is the target COF requirement.
- (4) The target COF requirement increases with the increase of the mass irregularity, i.e. the higher the mass irregularity the higher the target COF requirement.
- (5) The target COF requirement further increases when both the height and mass irregularity is combined in the same frame, i.e., COF requirement of this frame is higher than the frame with only height irregularity or only mass irregularity.

Chapter 4

INVESTIGATION ON TARGET COF CONSIDERING SYSTEM RELIABILITY

4.1 Introduction

Most engineering system consists of many elements or component. Therefore, it is necessary to consider multiple failure modes. When considering system reliability, it is important to recognize that failure of a single component may or may not mean the failure of the system. Consequently, the reliability of an individual member may or may not be the representative of the reliability of the entire system. The calculation of the failure probability for a system is generally difficult even if the potential failure modes are known or can be identified, because of the large number of potential failure modes for most practical structures, the difficulty in obtaining the sensitivity of the performance function and the mutual correlations among the failure modes. The search for efficient computational procedures for estimating system reliability has resulted in several approaches such as bounding techniques, probabilistic network evaluation technique (PNET) and direct or smart Monte Carlo simulations. In the present research, a computationally more effective method using dimension reduction integration method for system reliability is adopted.

The column overdesign factor (COF) requirement that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame structure has been evaluated in this chapter considering system reliability based on A_i distribution of load along the height of the frames. In the previous chapter, the target COF was evaluated considering only the most likely failure modes with a single performance function at a time. Since most likely story mode is only one element of the system consists of many story failure modes, avoiding the most likely mode does not mean that all other story failure modes are avoided. Therefore, the consideration of system reliability is very important. The fish-bone model (Ogawa 1999) that condenses the columns and beams of each story into one column and one beam, respectively, employed in this study to simplify the computation. The target values of COF for three story to seven story frame under reliability levels 2, and 3 ($\beta_T=2$, and $\beta_T=3$) have been evaluated in this chapter.

4.2 System Reliability Analysis

4.2.1 Performance Function for System Reliability

A structural system will invariably have multiple modes of potential failure; e.g., E_1, E_2, \dots, E_m . In the case of a series system, occurrence of one or more of these failure modes will constitute failure of the system, i.e., the system failure is the union of all the modes or $E_1 \cup E_2 \cup \dots \cup E_m$.

Suppose each of the failure modes, E_i , can be defined by a smooth performance function $g_i = g_i(\mathbf{X})$ such that $E_i = (g_i < 0)$ and the failure probability of the system is then:

$$P_F = \text{Prob}[g_1 \leq 0 \cup g_2 \leq 0 \cup \dots \cup g_k \leq 0] \quad (4.1)$$

Conversely, the safety of a system is the event in which none of the k potential failure modes occur; again in the case of a series system, this means

$$\begin{aligned} P_S &= \text{Prob}[g_1 > 0 \cap g_2 > 0 \cap \dots \cap g_k > 0] \\ &= \text{Prob}[\min[g_1, g_2, \dots, g_k] > 0] \end{aligned} \quad (4.2)$$

Thus the performance function of a series system, G , can be expressed as the minimum of the performance functions corresponding to all the potential failure modes; that is,

$$G(\mathbf{X}) = \min[g_1, g_2, \dots, g_k] \quad (4.3)$$

where $g_i = g_i(\mathbf{X})$ is the performance function of the i th failure mode.

Similarly, for a parallel structural system, the failure probability of the system is:

$$\begin{aligned} P_F &= \text{Prob}[g_1 \leq 0 \cap g_2 \leq 0 \cap \dots \cap g_k \leq 0] \\ &= \text{Prob}[\max[g_1, g_2, \dots, g_k] \leq 0] \end{aligned} \quad (4.4)$$

Thus the performance function of a parallel system, G , can be expressed as the maximum of the performance functions corresponding to all the potential failure modes; that is,

$$G(\mathbf{X}) = \max[g_1, g_2, \dots, g_k] \quad (4.5)$$

Since the system performance function $G(\mathbf{X})$ will not be smooth although the performance function of a component is smooth, it is difficult to obtain the sensitivity of the performance function even for a series system as in Eq. 4.3, and derivative based FORM would not be applicable. The failure probability of a system may be determined using bounding techniques (e.g., Cornell, 1966) as a function of the failure probabilities of the individual modes; however, for a complex system the bounds would be wide; even though these bounds may be improved

by second-order bounds (Ditlevsen, 1979). The failure probability of a system may also be estimated approximately with the probabilistic network evaluation technique, PNET developed by Ang and Ma (1981), where the mutual correlations among the failure modes have to be computed. Other methods have been reviewed or discussed; in e.g., Moses (1982) and Bennett and Ang (1987).

In the present research, the first few moments of the system performance function are obtained by Dimension Reduction Integration (DRI), from which the moment-based reliability index based on the fourth moment standardization function and failure probability can be evaluated without Monte Carlo simulations.

4.2.2 Dimension Reduction Integration for Moments of Performance Function

For a performance function $Z = G(\mathbf{X})$, using inverse Rosenblatt transformation, the k th moments about zero, of Z can be defined as (Zhao and Ono 2000)

$$\begin{aligned}\mu_{kG} &= E\{[G(\mathbf{X})]^k\} = \int_{-\infty}^{\infty} \cdots \int_{-\infty}^{\infty} \{G[\mathbf{x}]\}^k f_{\mathbf{x}}(\mathbf{x}) d\mathbf{x} \\ &= \int_{-\infty}^{\infty} \cdots \int_{-\infty}^{\infty} \{G[T^{-1}(\mathbf{u})]\}^k \phi(\mathbf{u}) d\mathbf{u}\end{aligned}\quad (4.7)$$

where $f_{\mathbf{x}}(\mathbf{x})$ denotes the probability density function (PDF) of \mathbf{X} , $T^{-1}(\mathbf{u})$ denotes inverse Rosenblatt transformation, $\phi(\mathbf{u})$ denotes the PDF of standard normal variables.

Practically, the integral in Eq. (4.7) cannot be evaluated analytically because of the high dimensionality and the complicated integrand. An alternative approach is using the Gauss-Hermite quadrature, m^n times of function calls for computing $G(\mathbf{X})$ are required and the computations involved therefore, can be massive when n is large. In order to avoid this problem, Zhao and Ono (2000), Xu and S. Rahman (2004) proposed generalized multivariate dimension-reduction method, in which the n -dimensional performance function is approximated by the summation of a series of, at most, D -dimensional functions ($D < n$). In this research, the bivariate dimension reduction ($D = 2$) is used. Let $L(\mathbf{u}) = \{G[T^{-1}(\mathbf{u})]\}^k$ in Eq. (4.7). By the bivariate dimension reduction method

$$L(\mathbf{u}) \cong L_2 - (n-2)L_1 + \frac{(n-1)(n-2)}{2}L_0 \quad (4.8)$$

where

$$L_0 = L(0, \dots, 0) \quad (4.9a)$$

$$L_1 = \sum_{i=1}^n L(0, \dots, 0, u_i, 0, \dots, 0) \quad (4.9b)$$

$$L_2 = \sum_{i < j} L(0, \dots, 0, u_i, 0, \dots, u_j, 0, \dots, 0) \quad (4.9c)$$

in which $i, j = 1, 2, \dots, n$ and $i < j$. It is noted that L_1 is a summation of n one-dimensional functions and L_2 is a summation of $[n(n-1)]/2$ two-dimensional functions.

Substituting Eq. (4.8) in Eq. (4.7) reduces the n -dimensional integral of Eq. (4.7) into a summation of, at most, two-dimensional integrals

$$\begin{aligned} \mu_{kG} &= E\{G[T^{-1}(U)]^k\} \cong E\{L(U)\} \\ &= \sum_{i < j} \mu_{L2ij} + (n-2) \sum_i \mu_{L1i} + \frac{(n-1)(n-2)}{2} L_0 \end{aligned} \quad (4.10)$$

where

$$\mu_{L1i} = \int_{-\infty}^{\infty} L(0, \dots, 0, u_i, 0, \dots, 0) \phi(u_i) du_i \quad (4.11a)$$

$$\mu_{L2ij} = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} L(0, \dots, 0, u_i, 0, \dots, u_j, 0, \dots, 0) \phi(u_i) \phi(u_j) du_i du_j \quad (4.11b)$$

Using the Gauss-Hermite integration, the one-dimensional integral in Eq. (4.11a) can be approximated by the following equation.

$$\mu_{L1i} = \sum_{r=1}^m P_r L(0, \dots, 0, u_r, 0, \dots, 0) \quad (4.12)$$

The estimating points u_r and the corresponding weights P_r can be readily obtained as (Zhao and Ono 2000)

$$u_r = \sqrt{2}x_r, \quad P_r = \frac{w_r}{\sqrt{\pi}} \quad (4.13)$$

where x_r and w_r are the abscissas and weights for Hermite integration with weight function $\exp(-x^2)$.

Specially, for a five point estimate in standard normal space (Zhao and Ono 2000),

$$u_0 = 0 \quad P_0 = 8/15 \quad (4.14a)$$

$$u_{1+} = -u_{1-} = 1.3556262 \quad P_1 = 0.2220759 \quad (4.14b)$$

$$u_{2+} = -u_{2-} = 2.8569700 \quad P_2 = 1.12574 \times 10^{-2} \quad (4.14c)$$

whereas for a seven point estimate in standard normal space,

$$u_0 = 0 \quad P_0 = 16/35 \quad (4.15a)$$

$$u_{1+} = -u_{1-} = 1.1544054 \quad P_1 = 0.2401233 \quad (4.15b)$$

$$u_{2+} = -u_{2-} = 2.3667594 \quad P_2 = 3.07571 \times 10^{-2} \quad (4.15c)$$

$$u_{3+} = -u_{3-} = 3.7504397 \quad P_3 = 5.48269 \times 10^{-4} \quad (4.15d)$$

Similarly, the two-dimensional integral in in Eq. (4.11b) can be approximated by

$$\mu_{L2ij} = \sum_{r_1=1}^m \sum_{r_2=1}^m P_{r_1} P_{r_2} L(0, \dots, 0, u_{r_1}, 0, \dots, u_{r_2}, 0, \dots, 0) \quad (4.16)$$

Finally, the mean, standard deviation, the skewness, and the kurtosis of a performance function $G(\mathbf{X})$ with n random variables can be obtained as the follows

$$\mu_G = \mu_1 \quad (4.17a)$$

$$\sigma_G = \sqrt{\mu_2 - \mu_1^2} \quad (4.17b)$$

$$\alpha_{3G} = (\mu_3 - 3\mu_2\mu_1 + 2\mu_1^3) / \sigma_G^3 \quad (4.17c)$$

$$\alpha_{4G} = (\mu_4 - 4\mu_3\mu_1 - 3\mu_2^2 + 12\mu_2\mu_1^2 - 6\mu_1^4) / \sigma_G^4 \quad (4.17d)$$

4.2.3 Approximate Distribution of the Performance Function of the System

After the first three or four moments of $G(\mathbf{X})$ are obtained, the reliability analysis becomes a problem of approximating the distribution of a specific random variable with its known first three or four moments.

Approximating the distribution of a random variable using its moments of finite order is a well known problem in statistics, and various approximations such as the Pearson, Johnson and Burr systems, the Edgeworth and Cornish-Fisher expansions were developed (Stuart and Ord

1987). Their applications in structural reliability have been examined by Winterstein (1988), Grigoriu (1983), and Hong (1996). In the present study, the fourth moment standardization function will be used. For the standardized performance function,

$$Z_s = \frac{Z - \mu_G}{\sigma_G} \quad (4.18)$$

The fourth moment standardization function is expressed as (Zhao and Lu 2007)

$$Z_s = S(U, \mathbf{M}) = -l_1 + k_1 U + l_1 U^2 + k_2 U^3 \quad (4.19)$$

where

$$l_1 = \frac{\alpha_{3G}}{6(1 + 6l_2)} \quad (4.20a)$$

$$l_2 = \frac{1}{36} \left(\sqrt{6\alpha_{4G} - 8\alpha_{3G}^2 - 14} - 2 \right) \quad (4.20b)$$

$$k_1 = \frac{1 - 3l_2}{(1 + l_1^2 - l_2^2)} \quad (4.20c)$$

$$k_2 = \frac{l_2}{(1 + l_1^2 + 12l_2^2)} \quad (4.20d)$$

since

$$F(Z_s) = \Phi(U) = \Phi[S^{-1}(Z_s, \mathbf{M})] \quad (4.21)$$

The PDF of $Z = G(\mathbf{X})$, can be obtained as

$$f_z(z) = \frac{1}{\sigma} \phi(u) \left| \frac{dz_s}{du} \right| \quad (4.22)$$

where

$$u = S^{-1} \left(\frac{z - \mu_G}{\sigma_G} \right) \quad (4.23)$$

Therefore, the PDF of the performance function is expressed as

$$f_z(z) = \frac{\phi(u)}{\sigma(k_1 + 2l_1 u + 3k_2 u^2)} \quad (4.24)$$

where

$$u = S^{-1}\left(\frac{z - \mu_G}{\sigma_G}\right) = \frac{1}{D} - Dp - l \quad (4.25a)$$

$$D = \sqrt[3]{2\left(\sqrt{q^2 + 4p^3} - q\right)^{-1/3}} \quad (4.25b)$$

$$q = l(2l^2 - k_1/k_2 - 3) + \frac{z - \mu_G}{k_2\sigma_G} \quad (4.25c)$$

$$p = k_1/k_2/3 - l^2, \quad l = l_1/k_2/3 \quad (4.25d)$$

The probability of failure is expressed as

$$\begin{aligned} P_f &= \text{Prob}[G \leq 0] = \text{Prob}[Z_s\sigma_G + \mu_G \leq 0] \\ &= \text{Prob}\left[Z_s \leq -\frac{\mu_G}{\sigma_G}\right] = \text{Prob}[Z_s \leq -\beta_{2M}] \end{aligned}$$

That is

$$P_f = F(-\beta_{2M}) = \Phi[S^{-1}(-\beta_{2M}, \mathbf{M})] \quad (4.26)$$

where \mathbf{M} is the vector denoting the first several moments of $Z = G(\mathbf{X})$

Then the corresponding reliability index is expressed as

$$\beta = -\Phi^{-1}(P_f) = -S^{-1}(-\beta_{2M}, \mathbf{M}) \quad (4.27)$$

Therefore

$$\beta_{4M} = D_0 p - \frac{1}{D_0} + l \quad (4.28a)$$

$$D_0 = \sqrt[3]{2\left(\sqrt{q_0^2 + 4p^3} - q_0\right)^{-1/3}} \quad (4.28b)$$

$$q_0 = l(2l^2 - k_1/k_2 - 3) + \beta_{2M}/k_2 \quad (4.28c)$$

4.3 Evaluation Method

4.3.1 Fish bone Model and the Failure Modes

It has been shown that the second moment reliability index of the failure modes are not

affected by the mean value of member strength or the number of bays; the high-order moment of the performance function is also independent of number of bays and mean value of member strength, thus, the reliability indices of story failure patterns do not vary with the change of number of bays and mean value of member strength (Zhao *et al.* 2007). Therefore, COF is not affected by the number of bays and COF based on fish-bone model and real frame should be the same.

For simplification of the computation, the fish bone model is employed in the investigation of target COF. Fig. 4.1 shows the transformation of frame structure into the fish bone model.

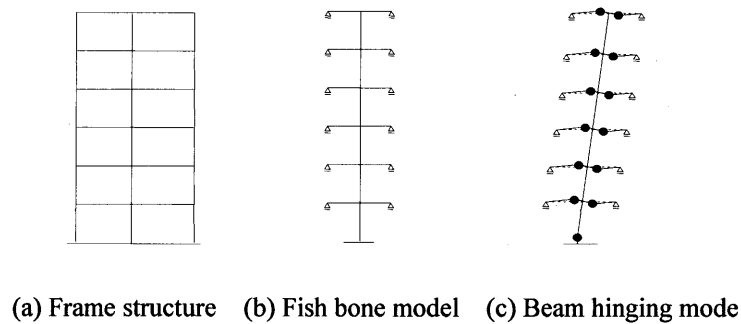


Fig. 4.1 Fish bone model and beam hinging failure mode

In fish bone model, each floor has two degrees of freedom, the lateral displacement and the rotation of node. In analysis, the frame was simplified into a stick model attached with a rotational spring at each floor. The rotational spring represents the resistance provided by all beams connected in a floor level. Multiple columns in one story are condensed into one column neglecting the elongation and contraction of the columns in a floor level. The detail discussion on fish bone model was described in Ogawa *et al.* (1999).

In the fish-bone model, the COF is generally expressed by the floor COF, which is defined for each floor level as the ratio of the sum of mean strengths of columns to the sum of mean strengths of beams, as follows:

$$COF = \frac{\sum_i \mu_{Mci}}{\sum_i \mu_{Mbi}} = \mu_{Mc} / \mu_{Mb} \quad (4.29)$$

where μ_{Mci} and μ_{Mbi} are the mean plastic moment strength of the column and beam, respectively, connected to a specific floor of the original frame, and μ_{Mc} and μ_{Mb} are the mean plastic moment strength of the column and beam, respectively, of the fish-bone model. One COF is

assigned for each floor.

The story failure modes are classified here into three patterns as before: upper story failure pattern, middle story failure pattern and lower story failure pattern. Fig. 4.2 shows the general forms of these three types of story mechanisms in fish-bone model.

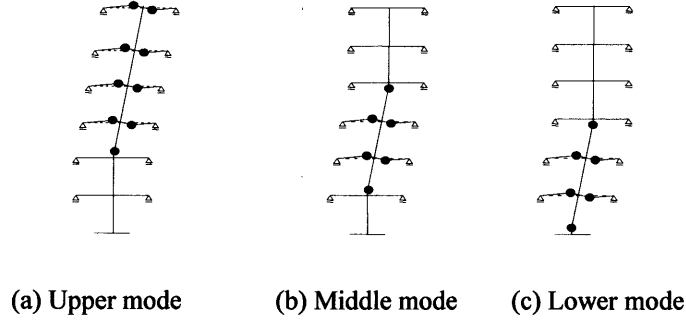


Fig. 4.2 Failure modes in fish bone model

Based on the principle of virtual work, performance function of the upper collapse mode, the middle collapse mode and the lower collapse modes in the fish bone model can be established as follows:

$$G_U(\mathbf{X}) = 2 \sum_{j=n-n_c+1}^n (M_{b1j} + M_{b2j}) + 2M_{csl} + M_{cl} - \sum_{j=n-n_c+1}^n (j + n_c - n)hP_j \quad (30)$$

$$G_M(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} (M_{b1j} + M_{b2j}) + 4M_{csl} + 2M_{cl} - \sum_{j=1}^{n_c} jhP_{j+n_b} - \sum_{j=n_c+1}^{n-n_b} n_c hP_{j+n_b} \quad (31)$$

$$G_L(\mathbf{X}) = 2 \sum_{j=1}^{n_c-1} (M_{b1j} + M_{b2j}) + 4M_{csl} + 2M_{cl} - \sum_{j=1}^{n_c} jhP_j - \sum_{j=n_c+1}^n n_c hP_j \quad (32)$$

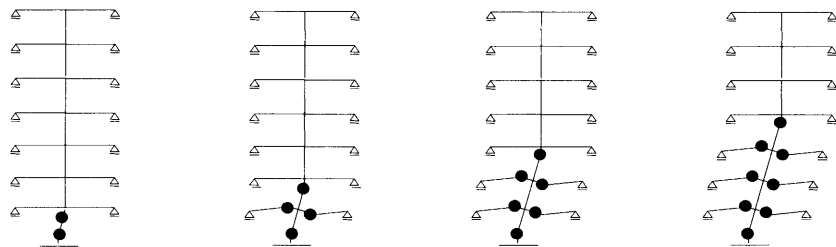
where G_U , G_M and G_L are the performance functions of the upper story failure pattern, the middle story failure pattern and the lower story failure pattern respectively. M_{b1j} and M_{b2j} are the moment strength of the beams of the j th story, M_{cl} is the moment strength of an interior column, M_{csl} is the moment strength of an exterior column, P_j is the load acting on the j th story of the structure, n is the number of stories, n_c is the number of failure stories and h is the story height of the structure.

The random variables, i.e., moment strength of the beams and columns and the load acting on the structure are assumed to follow the lognormal distribution. Moment strengths are statistically independent to one another and independent of the applied loads. The coefficient

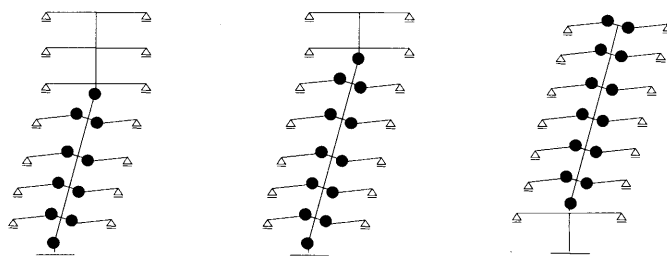
of variation of material strength is considered to be 0.1 and that of load to be 0.8.

The mean value of moment strength of the beam of the top story is assumed to be half of the mean value of moment strength of other beams of the lower stories. This is because the top beam has to sustain a lower load than the beams of the lower stories due to absence of walls and some other loads. The mean values of the moment strength of columns are obtained by multiplying respective COF value. For example when COF=1.1 and the mean strength of the top beam is 104.15 KN.m, the mean strength of other beams will be 208.30 KN.m and mean strength of column will be 229.13 KN.m. Since, this study has been conducted for low-rise structures of up to seven stories; therefore, columns of all the stories are considered to have the same strengths in order to simplify the calculation.

In system reliability assessment also the most likely failure modes are taken into consideration. For an example the most likely failure modes of a seven story frame are shown in Fig. 4.3.



(a) Lower mode (LM-1) (b) Lower mode (LM-2) (c) Lower mode (LM-3) (d) Lower mode (LM-4)



(e) Lower mode (LM-5) (b) Lower mode (LM-6) (c) Upper mode (UM-6)

Fig. 4.3 Likely failure modes of a seven story frame

Considering likely failure modes the performance of the system is as follows:

$$G = \min\{G_{L1}, G_{L2}, G_{L3}, \dots, G_{LK}, G_{UK}\} \quad (4.33)$$

where $K=n-1$ and n is the number of story.

4.3.2 Evaluation Index

To probabilistically avoid the story mechanisms, the probabilities of the story mechanisms should be controlled at least lower than that of the entire beam hinging failure mode. In the target COF evaluation, following evaluation index is used:

$$\gamma = P_{f2} / P_{f1} \leq \gamma_0 \leq 1 \quad (4.34)$$

where P_{f1} = the occurrence probability of the beam hinging failure mode and P_{f2} = the failure probability of the system.

If the above evaluation index is determined based on only likely failure mode and system reliability is not considered, the P_{f2} corresponds to one failure mode with one performance function, which does not necessarily mean that all other story failure modes are avoided. Therefore, the computation of P_{f2} considering system reliability is essential.

The method used in this paper is to assume a reliability index such as $\beta_T=2$ or $\beta_T=3$ for the entire beam hinging failure mode first to specify the safety level of the structure and then to compute the mean value of the earthquake load to ensure that the first order reliability index becomes equal to the target reliability index β_T for frame structures designed with various COFs. The obtained load is then applied to compute the failure probability of the system.

By conducting the failure analysis and the reliability analysis using a different COF for a frame structure, a γ -COF curve can be obtained and the target value of the COF for which Eq. 4.34 is satisfied can be determined.

When $\gamma_0=1$, the failure probabilities of the system and the preferable entire beam-hinging mode have the same likelihood of occurrence, i.e., both probabilities are equal. The threshold value of COF when $\gamma_0=1$ is defined here as the target COF. When the COF of the frame is higher than the target COF, the failure probability of the system is less than that of the expected entire beam hinging failure mode.

4.3.3 Monte Carlo Simulation and Investigation on Results of DRI

In many practical engineering situations, the problem may be complicated and not amenable to analytical solutions. In such situations, numerical methods are necessary and often provide the only practical and effective approach. When the problem involve random variables, or require consideration of probability, the numerical process may include repeated simulations through Monte Carlo sampling techniques. The numerical result from each repetition of the

numerical process may be considered a sample of the true solution, analogous to an observed sample from a physical experiment. When random variables are involved, the values of the different variables are sampled from the respective probability distributions in each repetition. An essential component of the process, therefore, is the generation of the values of the random variables, each with its prescribed probability distribution, known as random number generators.

However, in practice, Monte Carlo simulations (MCS) may be limited by constraints of economy and computer capability. Moreover, solutions obtained from MCS may not be amenable to generalization or extrapolation. Therefore, as a general rule, Monte Carlo methods should be used only as a last resort; that is, when analytical methods are not available or are ineffective. Monte Carlo solutions, however, are often the only means of verifying or validating approximate analytical solution methods. In the present study, some results of the DRI method are verified applying MCS.

It is observed that for any single mode as well as for the system the failure probability applying DRI and MCS are close to each other. Fig. 4.4 shows a comparison of the failure probability of LM-3 of a seven story frame as shown in Fig. 4.3 obtained from applying DRI and MCS under reliability level 2 ($\beta_T=2$).

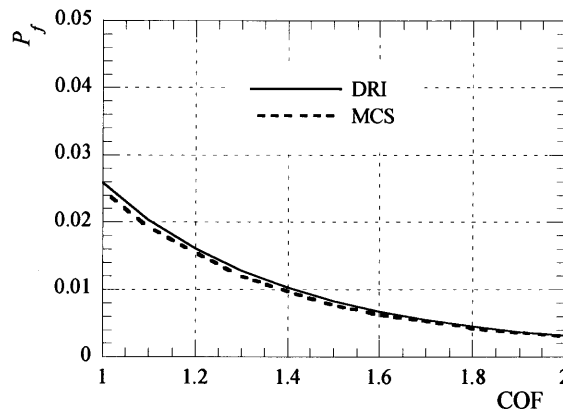


Fig. 4.4. Comparison of P_f of LM-3 of a 7- story frame

Fig. 4.5 shows a comparison of the failure probability of the system obtained from DRI and MCS for a seven story frame under reliability level 2 ($\beta_T=2$).

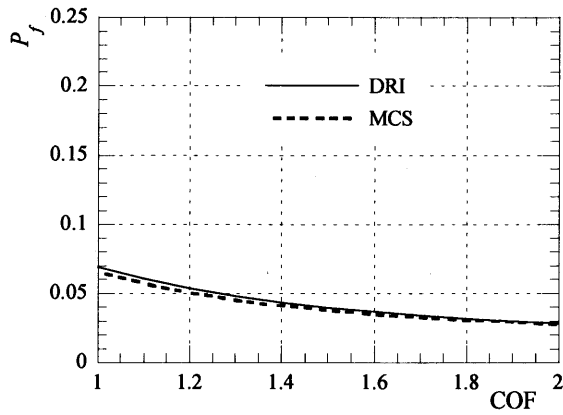


Fig. 4.5. Comparison of P_f of system of a 7- story frame

By comparing Fig. 4.4 and Fig.4.5 one can understand that the failure probability of any single mode is less than that of the system. It will be clearer from the following Fig. 4.6 which shows a comparison of failure probability of the system and that of a lower failure mode, LM-4 of a seven story frame under reliability level 2 ($\beta_T=2$). It is observed that failure probability of the system is always higher than that of any single mode.

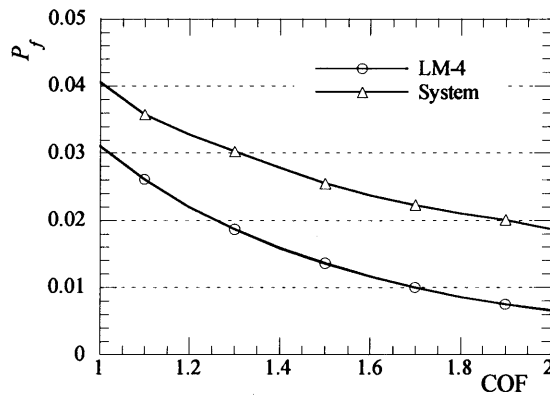


Fig. 4.6. Comparison of P_f of system and LM-4 of a 7- story frame

Fig. 4.7 shows a comparison of the reliability index of the system obtained from DRI and MCS for a seven story frame under reliability level 2 ($\beta_T=2$).

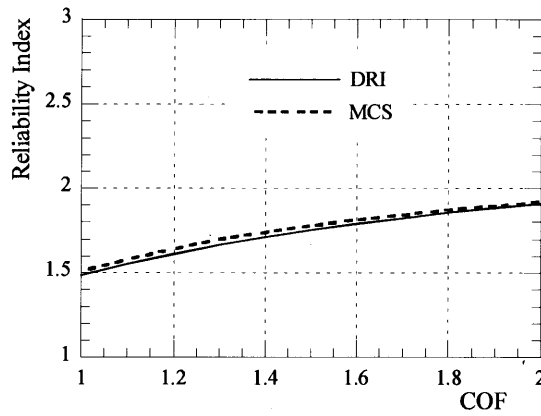


Fig. 4.7. Comparison of Reliability Index of system of a 7- story frame

It is also observed that the first four moments of the system performance function applying DRI and MCS are close to each other. For example for COF=1.5 the first four moments are obtained as $\mu_G = 16050.9$ and 16053.1 , $\sigma_G = 7939.49$ and 7759.76 , $\alpha_{3G} = -4.42$ and -4.37 , $\alpha_{4G} = 42.71$ and 42.22 respectively for MCS and DRI method.

Fig. 4.8 shows a comparison of mean of the system performance function obtained from DRI and MCS for a seven story frame under reliability level 2 ($\beta_T=2$).

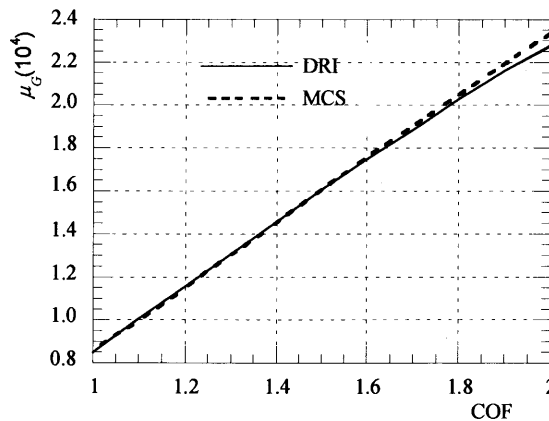


Fig. 4.8. Comparison of Mean of system of a 7- story frame

Fig. 4.9 shows a comparison of COV and skewness of the system performance function obtained from DRI and MCS for a seven story frame under reliability level 2 ($\beta_T=2$).

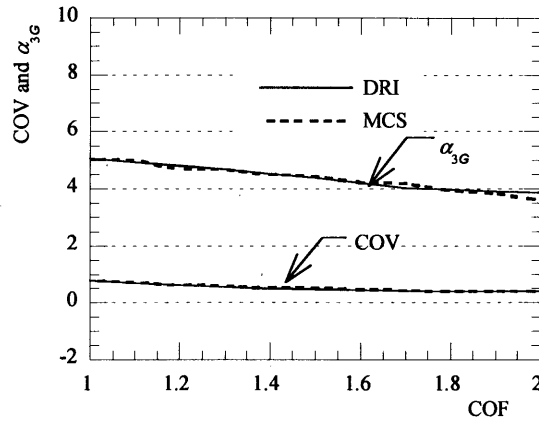


Fig. 4.9. Comparison of COV and skewness of system of a 7- story frame

Fig. 4.10 shows a comparison of kurtosis of the system performance function obtained from DRI and MCS for a seven story frame under reliability level 2 ($\beta_T=2$).

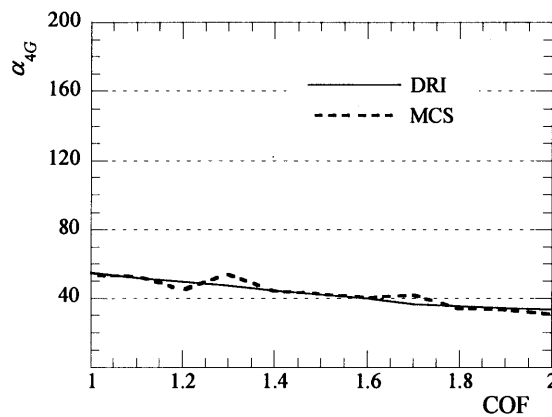


Fig. 4.10. Comparison of kurtosis of system of a 7- story frame

From the above discussion and the Figs. it is clear that results obtained from DRI show good agreements with that of MCS.

4.4 Target COF Applying System Reliability

In this section, the target COF requirement that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame structures have been evaluated under reliability levels 2 and 3 ($\beta_T=2$, and $\beta_T=3$) applying Dimension Reduction Integration (DRI) based on the Ai

distribution of Japan. Initially the failure probability of the system and that of the failure modes are investigated. After evaluation of the failure probability, the target values of COF are determined considering system reliability based on the evaluation index mentioned earlier in sec. 4.3.2.

Fig. 4.11 shows the γ -COF curve for four story to seven story frames under reliability level 2 ($\beta_T=2$) considering system reliability applying DRI.

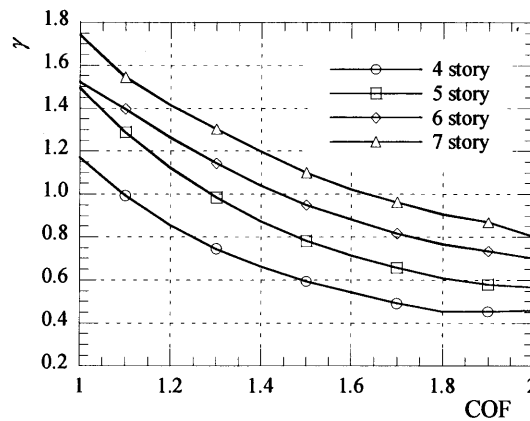


Fig. 4.11. γ -COF curves for four to seven story frame ($\beta_T=2$)

Fig. 4.12 shows the γ -COF curve for five story to seven story frames under reliability level 3 ($\beta_T=3$) considering system reliability applying DRI.

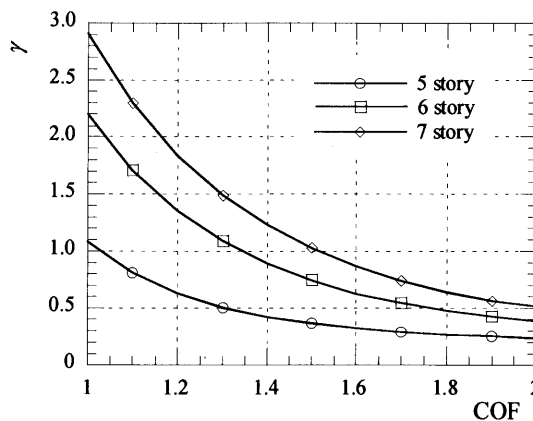


Fig. 4.12. γ -COF curves for three to seven story frame ($\beta_T=3$)

The target COF for three to seven story building frames under reliability levels 2, and 3 ($\beta_T=2$, and $\beta_T=3$) are presented in Table 3.7. The minimum value of target COF i. e., 1.0 is sufficient for frame structures below four story under reliability level 2 ($\beta_T=2$) and below five

story under reliability level 3 ($\beta_T=3$).

Table 4.1 Target COF for multi-story frames considering system reliability

Reliability level	3story	4story	5story	6story	7story
$\beta_T=2$	1.00	1.09	1.29	1.45	1.64
$\beta_T=3$	1.00	1.00	1.03	1.35	1.51

It is observed that under the same reliability level, the target COF requirement increases with the increase of the number of stories of the frame and decreases with the increase of the reliability level.

4.5 Conclusions

In this chapter, the target COF requirement that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame structures have been evaluated under reliability levels 2 and 3 ($\beta_T=2$, and $\beta_T=3$) considering system reliability applying Dimension Reduction Integration (DRI). The investigation presented in this chapter can be summarized by the following conclusions.

- (1) The system reliability of the frame structures are evaluated applying DRI method. The method directly calculates the reliability indices (and associated failure probabilities) based on the first few moments of the system performance function of a structure. It does not require the reliability analysis of the individual failure modes; also, it does not need the iterative computation of derivatives, or the computation of the mutual correlations among the failure modes, and does not require any design points. Thus, this method should be more effective for the system reliability evaluation of complex structures than currently available methods.
- (2) The accuracy of results obtained with the DRI method has been thoroughly examined by comparisons with large sample Monte Carlo simulations (MCS). It is observed that the results of DRI show good agreement with that of MCS.
- (3) The target values of COF for three to seven story frames under reliability levels 2 and 3 ($\beta_T=2$, and $\beta_T=3$) based on Ai distribution of Japan are presented in this chapter.

(4) It is observed that under the same reliability level the target COF requirement increases with the increase of the number of stories and decreases with the increase of the reliability level.

Chapter 5

CONCLUSIONS

In the present study, considering the uncertainties of earthquake load and strengths of structural members, the failure modes of the frame structures are investigated probabilistically. The A_i distribution of Japan, the distribution of the Uniform Building Code (UBC-1994) and International Building Code (IBC-2006) are taken into account. Based on the investigations, the target values of COF that probabilistically ensure the preferable beam hinging failure mode prior to story collapse are evaluated. An initiative has also taken to consider the system reliability in the COF evaluation. The major findings are summarized as follows:

1. Investigation on the story mechanisms of the frame structures

1. Under any specific reliability level, the mean value of the load is generally a linear function of COF. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006.
2. The failure probabilities of the middle story and upper story collapse modes follow some specific patterns but the failure probabilities of the lower story collapse modes do not follow any specific pattern. In case of upper story collapse modes, the failure probability steadily increases with the increase of the number of failure stories. In case of middle story collapse modes, the failure probability with higher n_b (number of unbroken story at the bottom of the frame) is less than that with lower n_b . The value of COF has a significant effect on the probabilistic order of the lower story modes. Each lower story mechanism has a special COF region in which the failure probability of that mode is the largest. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006.
3. With the increase of the COF the failure probabilities of all modes decrease. A similar observation is found in case of A_i distribution of load and the distribution of UBC-94 and IBC-2006.
4. Among all the failure modes of a multi-story ductile frame structure the lower failure modes and the upper failure mode with the maximum failure stories have the highest failure probability, i.e., these modes are the most likely failure modes of a multi-story

ductile frame structure. A similar observation is found in case of Ai distribution of load and the distribution of UBC-94 and IBC-2006. That means, most likely failure modes are independent of the type of the distribution applied in the evaluation.

2. Estimation of target column overdesign factor avoiding story mechanisms

1. The target COF is the minimum value of COF that probabilistically ensures the preferable entire beam hinging failure mode during earthquake and probabilistically avoids the undesirable story collapse modes of the frame. The target values of COF for three to seven story frames under reliability levels ($\beta_T=2$, $\beta_T=3$, and $\beta_T=4$) based on Ai distribution of load and the distribution of UBC-94 and IBC-2006 are presented in chapter three.

2. Under the same reliability level the target COF requirement increases with the increase of the number of stories and decreases with the increase of the reliability level. The rate of change of target COF with number of story and reliability level is almost linear. A similar observation is found in case of Ai distribution of load and the distribution of UBC-94 and IBC-2006.

3. Higher COF value has to be provided for frames with higher floor height in first story. The higher the height irregularity due to story height variation in the first story, the higher is the target COF requirement. The target COF requirement increases with the increase of the mass irregularity, i.e. the higher the mass irregularity the higher the target COF requirement. The target COF requirement further increases when both the height and mass irregularity is combined in the same frame, i.e. COF requirement of this frame is higher than the frame with only height irregularity or only mass irregularity.

3. Investigation on target COF considering system reliability

1. The system reliability of the frame structures are evaluated applying Dimension Reduction Integration (DRI) method. The method directly calculates the reliability indices (and associated failure probabilities) based on the first few moments of the system performance function of a structure. It does not require the reliability analysis of the individual failure modes; also, it does not need the iterative computation of derivatives, or the computation of the mutual correlations among the failure modes, and does not require any design points. Thus, this method should be more effective for the system reliability evaluation of complex structures than currently available methods.

2. The accuracy of results obtained with DRI method has been thoroughly examined by

comparisons with large sample Monte Carlo simulations (MCS). It is observed that the results of DRI show good agreement with that of MCS.

3. The target values of COF for three to seven story frames under reliability levels 2 and 3 ($\beta_T=2$, and $\beta_T=3$) based on A_i distribution of Japan are presented in chapter four.

4. Under the same reliability level the target COF requirement increases with the increase of the number of stories and decreases with the increase of the reliability level.

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LIST OF PUBLICATIONS

Reviewed Papers (Published/Accepted)

1. **Sharfuddin, M.** and Zhao, Y.G. (2010). Simple formulae for least and optimum COF for frame structure avoiding story mechanism. *Journal of Structural Engineering* 56B, 263-268.
2. **Sharfuddin, M.** and Idota, H. Comparison of probabilistically evaluated COF with base shear distribution of IBC and Japanese Code. *Journal of Asian Architecture and Building Engineering* (accepted for publication in May, 2011).
3. **Sharfuddin, M.**, Zhao, Y.G. and Duc, N.H. Target Column Overdesign Factor with Variation of Load and Strength Characteristics of Material. *International Workshop on Reliability Engineering and Risk Management (IWRERM 08)*, August 21-23, 2008, Shanghai, China.
4. **Sharfuddin, M.** and Zhao, Y.G. An investigation of COF with variation of load and strength characteristics of material. *10th International Conference on Structural Safety and Reliability (ICOSSAR 2009)*, September 13-17, 2008, Osaka, Japan.
5. Pu, W.C., **Sharfuddin, M.** and Lu, Z.H. Column over-design factor requirements for ensuring beam failure mechanism with specific reliability levels. *10th International Conference on Structural Safety and Reliability (ICOSSAR 2009)*, September 13-17, 2008, Osaka, Japan.
6. Zhao, Y.G. and **Sharfuddin, M.** Load Level and Target Column over-design Factors of Frames under a specific reliability level. *IFIP WG 7.5 Working Conference on Reliability and Optimization of Structural Systems*. April 7-10, 2010, Munich, Germany.
7. **Sharfuddin, M.**, Idota, H., Zhao, Y.G. and Ono, T. Effect of mass irregularity of frame on probabilistically evaluated Column Overdesign Factor. *International Symposium on Reliability Engineering and Risk Management (ISRERM 2010)*, September 23-26, 2010, Shanghai, China.
8. Zhao, Y.G., **Sharfuddin, M.** and Ang, A.H.S. A simple method for system reliability. Keynote lecture, *The 11th International Symposium on Structural Engineering (ISSE-11)*, Dec. 18-20, 2010, Guangzhou, China.

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9. **Sharfuddin, M.**, Zhao, Y.G., Idota, H., and Ansary, M.A. Probabilistic evaluation of column overdesign factor for frame structures considering seismic base shear distribution of BNBC. *Journal of Civil Engineering, IEB*, (submitted for review in May, 2010).
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12. Zhao, Y.G., **Sharfuddin, M.** Idota, H and Ang, A.H.S. System reliability assessment by dimension reduction integration. *Journal of Structural Engineering and Mechanics, ASCE* (submitted for review in October, 2010).

13. **Sharfuddin, M.**, Idota, H., and Zhao, Y.G. Probabilistic evaluation of column overdesign factor for frame structures with base shear distribution of UBC and IBC. *Electronic Journal of Structural Engineering* (submitted for review in October, 2010).

Papers without Review (Published)

14. **Sharfuddin, M.** and Zhao, Y.G. (2009).The Effect of the Configuration of the Frame on Column Overdesign Factor. *AIJ Tokai Chapter Architectural Research Meeting*, Vol 47, 125-128.

15. **Sharfuddin, M.** and Zhao, Y.G. (2009).A probabilistic approach to failure mode identification for frame structure. *Summaries of Technical Papers of Annual Meeting*, AIJ, B-1, 41-42.

16. **Sharfuddin, M.**, Zhao, Y.G. and Idota, H. (2010).Investigation of failure probability of frame structure with some common probability distributions. *Summaries of Technical Papers of Annual Meeting*, AIJ, B-1, 3-4.

17. Zhao, Y.G. and **Sharfuddin, M.** (2010). Relationship between earthquake load and COF under a specific reliability levels. *Summaries of Technical Papers of Annual Meeting*, AIJ, B-1, 1-2.

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