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MODIFICATIONS ON EQUIVALENT LATERAL FORCE METHOD

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SUMMARY

It is important that deformation demands be predicted when evaluating the seismic performance of structures. In this paper, a statistical process is outlined to estimate the seismic demands of roof and story for MDOF structures. Roof drift demand is related to the maximum story drift demand through a simplified process that provides quick and reasonable estimates of seismic demands. An attempt has been made to adjust and extend the concept of response modification factor by introducing a modifying factor R_T. This factor accounts for the inelastic response in the dynamic analysis of MDOF structures. The analysis involves MDOF shear and frame buildings with various dynamic characteristics. Nonlinear and corresponding linear time history analyses were performed for severe earthquake ground motions. The data were used to derive empirical formulae for the maximum story and roof displacements. The results of the study can provide a simple and practical way to determine the nonlinear dynamic response of MDOF structures. An approximate approach is presented to calculate the maximum inelastic deformations in a structure with a given strength distribution. The relationships proposed in this paper could be useful in the conceptual design phase, estimating deformation demands for performance assessment, and improving the basic understanding of seismic behavior.

INTRODUCTION

There is no agreement regarding the main criterion for the preliminary earthquake resistant design of structures. Present practice emphasizes the use of strength in a preliminary design, perhaps because of past and present code requirements. More specifically, in many of the present codes the preliminary design is based only on base shear strength, with a requirement to check the drift using elastic analysis. There are good reasons for this design approach: it is well understood by the engineering profession; it is relatively easy to implement; and in most cases it has served the profession well. It is generally recognized that damage to structures during earthquakes is due to deformation and that to control damage it is necessary to control deformations. The question then becomes how to achieve such control at different levels of earthquake shaking that can occur during the life of a structure.

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The static lateral force method incorporated in most of the seismic codes accounts for the nonlinear response in a seismic framing system by using a response modification factor, R. The commentary to the NEHRP Provisions [1] describes the R factor as "an empirical response modification (reduction) factor intended to account for both damping and ductility inherent in a structural system at displacements great enough to approach the maximum displacement of the system." This definition provides some insight into the developers' qualitative understanding of the seismic response of buildings and the expected behavior of a code-compliant building in the event of an earthquake. In addition, it is important to note the use of the adjective "empirical" in the above description, because there is no technical basis for the values assigned to R in seismic codes.

New procedures for the seismic analysis, evaluation, and design of new and existing structures have been proposed. These procedures are performance-based and displacement-oriented. Their use in new and retrofit construction in the future will represent a shift in the paradigm for seismic design practice. Nonetheless, the seismic design of new construction in the near future will likely make use of force-based procedures and response modification factors.

For a performance-based displacement-oriented seismic design, it is important to estimate maximum inter-story drift as well as maximum roof displacement. Reasons include: (1) support to estimate maximum damage; (2) estimating minimum building separation to avoid pounding; (3) checking deformation capacity of critical structural members; (4) checking P-delta effects; (5) detailing connections for nonstructural components. While there is no recommendation for estimating the maximum roof displacement in the current seismic design codes, some of them estimate the maximum story drift occurring in major earthquakes by amplifying the drifts computed from elastic analysis at the prescribed design seismic force level with a deflection amplification factor. A survey of several seismic design codes indicated that the deflection amplification factor in most of the current codes is independent of important factors such as ductility ratio and fundamental period.

This paper discusses the results of an investigation into the determination of response modification factor and displacement amplification factor for multi-degree of freedom (MDOF) shear building structures. It then provides a statistical evaluation of a roof displacement amplification factor to indicate the factors affecting the ratio of the maximum roof displacement to the maximum inter-story drift. Nonlinear and corresponding linear time history analyses are performed to produce structural response data. These data are used to derive an empirical formula for the evaluation of the response (displacement) modification factor.

MODELS AND BASIC ASSUMPTIONS

There are several different structural models used to estimate the nonlinear seismic response of building frames. The shear beam is the one most frequently adopted. In spite of some drawbacks, shear-beam models are widely used to study the seismic response of multistory buildings because of their simplicity and their low computer time consumption, thus allowing for many parametric studies [2]. Lai et al. [3] have investigated the reliability and accuracy of such shear-beam models. The shear-beam models of five, ten and fifteen story structures with identical story heights as well as single-degree of freedom (SDOF) models have been used in the present study. In these models, each floor is considered as a lumped mass connected by perfect elastoplastic shear springs.

The total mass of the structure is considered to be uniformly distributed over its height. The damping matrix is defined as a linear combination of the mass and initial stiffness matrices, in order to obtain 5% damping for the first few effective modes. In all MDOF models, lateral stiffness obtained in accordance with the selected lateral-loading pattern is assumed to be proportional to shear strength at each story. Twenty-one accelerograms recorded for ten different earthquake events are used as the input excitation. Emphasis is placed on those recorded at a low to moderate distance from the epicenter (less than 45 km), with rather high local magnitudes (i.e., $M_L>6.5$). Since all these records demonstrated high intensities, they should represent severe earthquakes and are used directly without being normalized or amplified. The structural models are subjected to seismic excitations and time-history nonlinear dynamic analyses are conducted utilizing the computer program DRAIN-2DX [4]. For each accelerogram, the dynamic response of models with various periods is calculated.

In most seismic building codes [1, 5-8] the height-wise distribution of lateral forces is determined from the following typical relationship:

(1) $$F_i = \frac{w_i h_i^k}{\sum_{j=1}^{N} w_j h_j^k} V$$

where w_i and h_i are the weight and height of the ith floor above the base, respectively; N is the number of stories; V is the seismic shear force at the base of the structure; and k is the power that differs from one seismic code to another. In some provisions such as NEHRP-94 and ANSI/ASCE 7-95, k increases from 1 to 2 as the structure's period varies from 0.5 to 2.5 seconds. In some codes, such as UBC-97, the force at the top floor (or roof) computed from Equation [1] is increased by adding an additional force Ft = 0.07TV for a fundamental period T of greater than 0.7 second. In this case, the base shear V in Equation [1] is replaced by (V-Ft).

In the present study, two extreme loading patterns are considered by introducing two different values for k in Equation [1], i.e.,

Rectangular Loading Pattern, with k=0.

Concentrated Loading Pattern, with $k \rightarrow \infty$. In this pattern, the total base shear is applied concentrically on the top floor.

These loading patterns beside of some other patterns has been used for a parametric study on the effects of different values of k on the nonlinear dynamic responses of structures [9] and they may not necessarily reflect the realistic loading patterns. In addition to the above patterns, the loading patterns of UBC-97 and NEHRP-94 are also considered in the present study.

RESPONSE MODIFICATION FACTOR

One of the most controversial issues in seismic design provisions for buildings is the development of response modification (or force reduction) factor, R. The force reduction factor is used to reduce the linear elastic design response spectra:

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

where Z is seismic zone factor, I is importance factor, C is dynamic response spectrum value and W is the total weight of the structure. The numerator in Equation [2] defines a modified elastic strength demand and the denominator defines a system-dependent strength reduction factor that reduces elastic strength demand on the structure to a design shear force at the member strength level. Equation [2] recognizes that the inelastic behavior of the structural system must be tolerated in the design for economic reasons. The R factor, which shows up in various forms in seismic codes of most countries, has been blamed for most of the shortcomings of the present design approaches [10].

The quantitative value of the reduction factor strongly depends on the energy dissipation capacity of the structural system, which is closely related to the ductility of the structure. Since R corresponds to the seismic forces at the design level (Fig. 1), it can be idealized as a product of the conventional ductility reduction factors R_μ and R_f, which account for all safety factors incorporated in the design procedure as well as overstrength [11], i.e.

(3) $$R = R_\mu \, R_f$$

Several investigations [12-18] have been conducted in the last three decades to evaluate the ductility reduction factotr R_μ. These studies are based mainly on the dynamic response analysis of single-degree of freedom (SDOF) systems. However, most structures have multi-degree of freedom (MDOF) and display more complicated behavior than SDOF systems, particularly in the nonlinear range. Thus, R_μ factors for SDOF systems need to be modified accordingly. It has been proposed to multiply R_μ by a modifying factor, R_T, which takes into account the possible concentration of displacement ductility demands on a specific floor [19]. This reduction factor can then be used for seismic analysis of multistory structures.

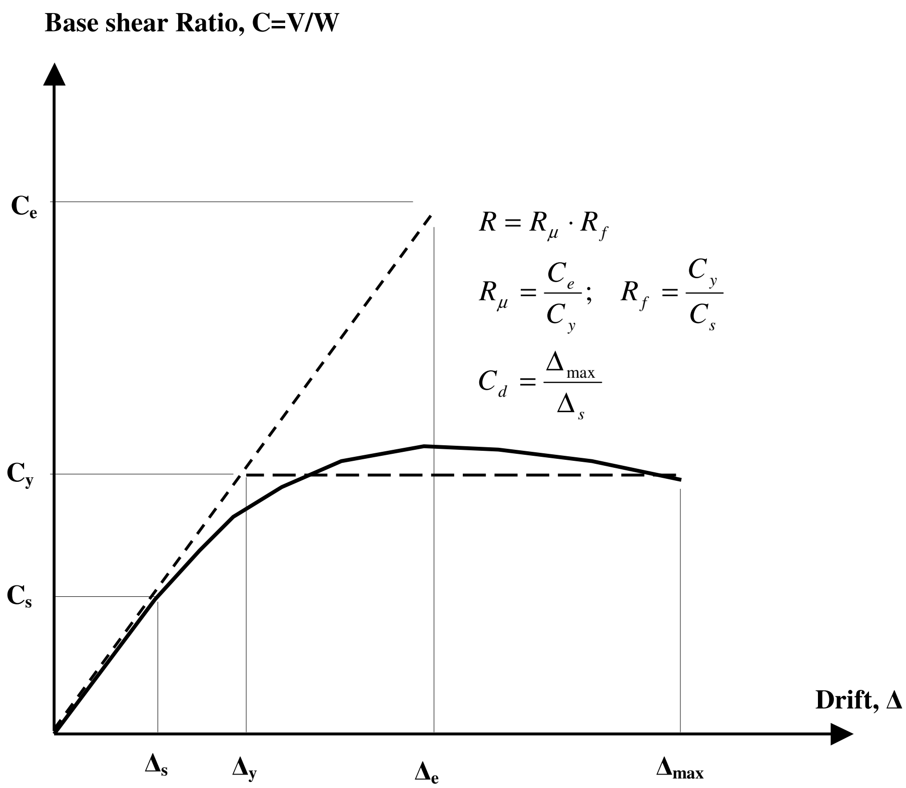

Figure 1. Effect of Inelastic Behavior on Seismic Response

Overstrength is a major factor that contributes to the seismic resistance of structures, and may stem from a variety of sources including: internal force redistribution (redundancy); conservatism of the design procedures; material overstrength; member oversize; the effect of nonstructural elements and strain rate effect. However, due to the uncertainty regarding many of the overstrength sources, the structural overstrength factor, R_o, may be employed only when it is quantified in the design process. In principle, the solution is to evaluate R_o using inelastic analyses such as a simplified limit analysis or a static pushover analysis. Alternatively, a comprehensive database of existing designs could be used to establish the necessary parameters for developing R_o factors. This can be carried out for particular structural systems designed according to definite code requirements. However, it is difficult to quantify the structural overstrength factor, especially when it depends on local characteristics of buildings and technology [10, 20-22].

According to the foregoing discussion, Equation [3] can be replaced by the following expression to define a more rational force reduction factor:

(4) $$R = R_\mu \; R_T \; R_o$$

MDOF MODIFYING FACTOR, R_T

The inelastic response of an MDOF structure designed for a base shear coefficient equal to that estimated from SDOF systems with periods equal to the fundamental period of the structure for a given ductility ratio μ_0, subjected to the same strong ground motion is considered. The results of nonlinear dynamic analysis of such a system may be used to estimate the base shear capacity that is required for a MDOF system to limit the maximum inter-story ductility demand on the target value, μ_0. However, the results of the present study (and previous studies by Nassar et al. [23] and Miranda [19]) indicate that, in general, the base shear demands of MDOF systems are greater than those of the corresponding SDOF systems, in order to limit the ductility demand in both systems to the same value of μ_0. The modifying factor R_T is then defined as:

(5) $$R_T = \frac{V_{ys}}{V_{ym}}$$

where V_{ys} and V_{ym} are the base shear demands of SDOF and MDOF systems, respectively. Therefore, the inelastic spectra for MDOF systems can be determined from the elastic spectra for SDOF systems as follows.

(6) $$V_{ym} = \frac{V_{ys}}{R_T} = \frac{V_{es}}{R_{\mu s} \cdot R_T}$$

where $R_{\mu s}$ and V_{es} are the reduction factor and elastic base shear for SDOF systems, respectively.
Research on multistory frames and walls [19, 23] indicates that the modifying factor, R_T, depends on the type of structural systems, the fundamental period, and the failure mode. In the present study, the dependence of R_T on the displacement ductility ratio, the fundamental period of structure, and the number of stories has been investigated. This study demonstrates that the modifying factor R_T is rather insensitive to the ductility ratio. Based on the outcomes of this study, the following expression has been suggested and discussed in Moghaddam and Karami Mohammadi [24]:

(7) $$R_T = N^{-0.26}$$

where N is the number of stories. A comparison of the exact results for 10-story building models with the approximate values from Equation [10] is given in Fig. 2. The distribution pattern of the design forces for proportioning the strength and stiffness of the structural models is chosen in accordance with the NEHRP provisions. The suggested R_T may only apply to regular shear buildings with a pattern of stiffness and strength distribution similar to the NEHRP recommendations. Therefore, more general conditions remains to be investigated. It was found that the distribution pattern of the design forces affects R_T [24].

DISPLACEMENT AMPLIFICATION FACTOR

In some seismic codes, such as the NEHRP Recommended Provisions, the deflections computed from an elastic structural analysis are amplified by a deflection amplification factor to estimate the maximum inelastic deflection that may occur during a major earthquake, i.e.

(8) $$\Delta_{\max} = \Delta_s \times C_d$$

where Δ_{max} is the maximum story inelastic deflection, Δ_s is the corresponding deflection computed from the static elastic analysis of the structure subjected to the equivalent seismic forces introduced by the code, and C_d is the deflection amplification factor (Fig. 1).

A survey of several current seismic design codes indicated that the deflection amplification factor in most cases is independent of some important factors such as ductility ratio, fundamental period and number of stories. Investigations [21] on multistory structures have shown that the deflection amplification factor, C_d, is sensitive to the type of yield mechanism and the fundamental period of structure T as long as the T/T_g ratio is less than 0.3, where T_g is the predominant period of the ground motion. Other investigations [25- 27] have shown that C_d is sensitive to the ductility factor, μ. Miranda (2000) and Baez and Miranda (2000) have indicated the effects of soil condition and some earthquake characteristics on C_d. These investigations have also indicated that the deflection amplification factor proposed in the NEHRP Provisions is very low and could underestimate drift values.

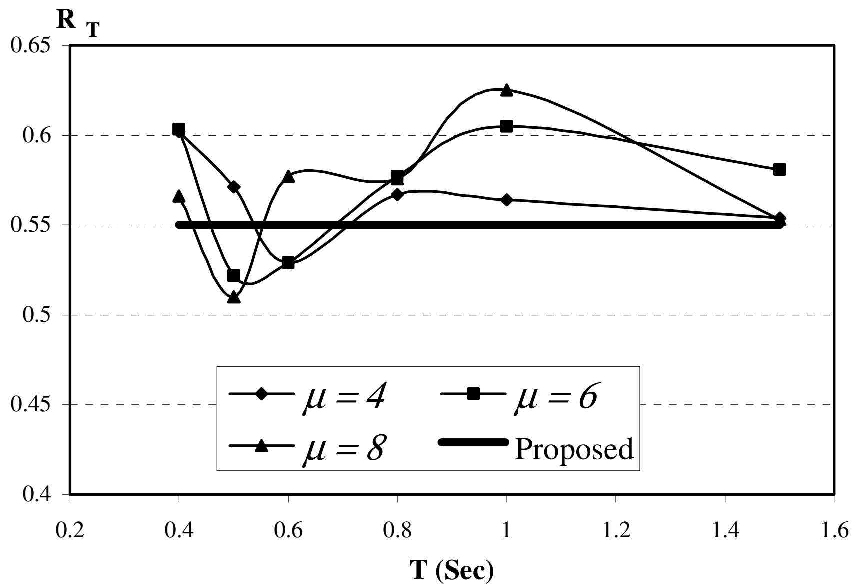

Figure 2. A comparison of the results of statistical studies with R_T as proposed by Eq.7 for 10-story models

The deflection factor C_d is computed in the present study by averaging the results of dynamic analyses of structural models subjected to twenty-one earthquake motions. The dependence of C_d on the displacement ductility ratio, the fundamental period of the structure, and the number of stories were explored. Based on the outcome of this study, the following expression has been suggested and discussed in Karami Mohammadi (2002) to determine C_d for MDOF systems:

$$C_d = R * \begin{cases} (0.78+0.17\mu)\cdot N^{(0.27-0.004N)} + (0.03-0.24\mu)\cdot T & T \le 0.8 \quad \text{sec} \\ (0.78+0.17\mu)\cdot N^{(0.27-0.004N)} + 0.8(0.03-0.24\mu) & T > 0.8 \quad \text{sec} \end{cases} \tag{9}$$

where R is the force reduction factor (Equation [3] or [4]) and μ is the allowable ductility ratio. A comparison of C_d values obtained from analyses of 10-story building models with the values established from Equation [9] is given in Fig. 3. It should be noted that the suggested formula for C_d may only apply to regular buildings with a pattern of stiffness and strength distribution similar to the NEHRP provisions. Therefore, more general conditions remain to be investigated. A complete discussion of this can be found in Karami Mohammadi [28].

ROOF DISPLACEMENT FACTOR

The value of maximum roof displacement is a direct and efficient measure used to quantify the overall displacement response of a building. However, the value of roof displacement provides no direct information about localized deformation within a structure. If the value of the inter-story displacement for each story is the same as the value of the roof displacement divided by the number of stories, the structure is said to deform uniformly. On the other hand, if in some stories the value of the inter-story displacement is much larger than the value of the roof displacement divided by the number of stories, concentrated local damages may occur in these stories. A roof displacement factor, C_m can be defined as the ratio of the maximum value of roof displacement, Δ^R_{max}, to the multiplication of the number of stories, N, by the maximum inter-story displacement, Δ_{max}:

(10) $$C_m = \frac{\Delta^R_{\max}}{N \cdot \Delta_{\max}}$$

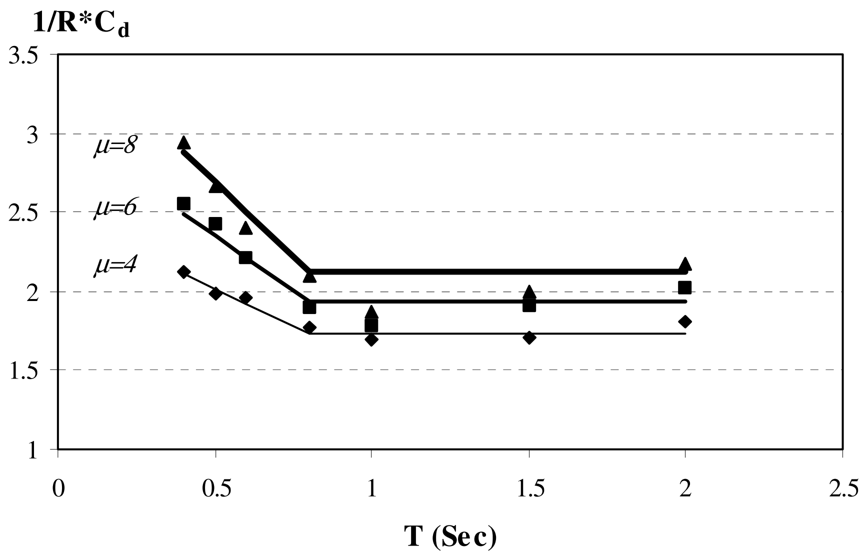

Figure 3. A comparison between the results of statistical studies and the C_d Proposed by Eq. 9 for 10-story building models

The value of C_m has to be less than unity, even if the dynamic response is limited to the elastic range (because inter-story drift of different stories is not reached the maximum simultaneously). In the case of inelastic response, a smaller value of C_m is usually expected.

Investigations [29, 30] on multistory structures have shown that the roof displacement factor, C_m, is sensitive to a multitude of factors including, among others, the relative strength and stiffness of the stories, the higher mode and P-delta effects, and the characteristics of the ground motions. In the present study, C_m is computed by averaging the results of response analyses of structural models subjected to twenty-one earthquake ground motions. The dependence of C_m on the displacement ductility ratio, fundamental period of structure, number of stories, and the distribution pattern of strength (and/or stiffness) are explored.

Distribution pattern of seismic load

The distribution of lateral strength over the height of structures is a function of the distribution of the seismic load considered in the analysis and design of the structure. Figure 4 shows the effect of different seismic load distribution patterns, including those postulated in the UBC and NEHRP provisions, on C_m for five-story structures with an allowable ductility ratio of six. It can be noted from Fig. 4 that C_m is generally sensitive to the load patterns. However, the difference between C_m values for the UBC and NEHRP load patterns is small and the results for the NEHRP load pattern could be also extended to buildings designed in accordance with the UBC load pattern. The remaining part of this study will examine the effect of different factors on C_m considering buildings that are designed in accordance with the NEHRP load pattern.

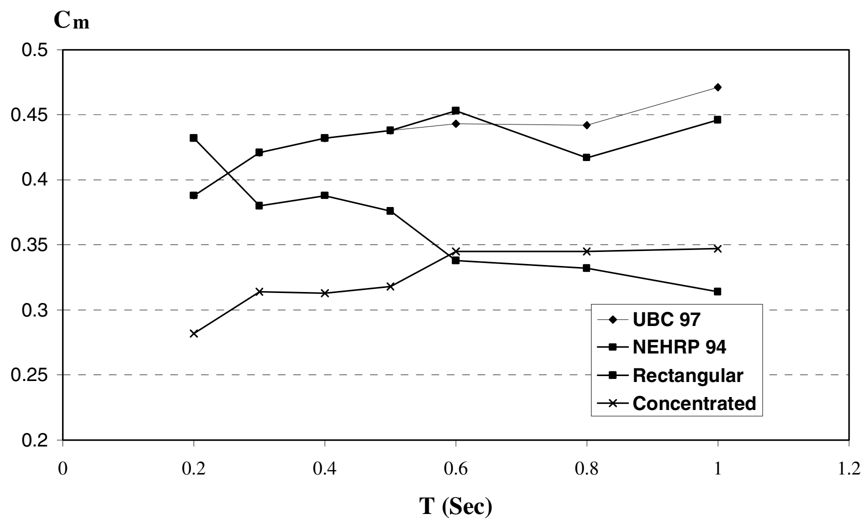

Figure 4. Effect of different distribution patterns of seismic load on C_m for 5-story models with μ=6

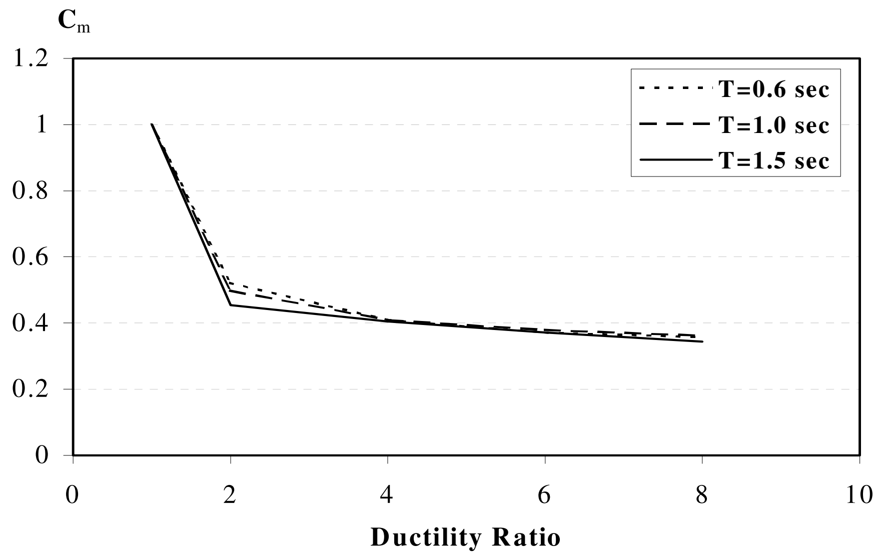

Figure 5. Effects of ductility ratio and period on the roof displacement factor, C_m, for 10-story models

Ductility ratio

The effects of the ductility ratio μ on C_m are shown in Fig 5. This figure shows the values of C_m for 10 story structures with periods varying from 0.6 to 1.5 seconds. Figure 5 shows the way that C_m decreases with an increase in μ. This figure has been also shown the low effect of fundamental period of the structure on C_m.

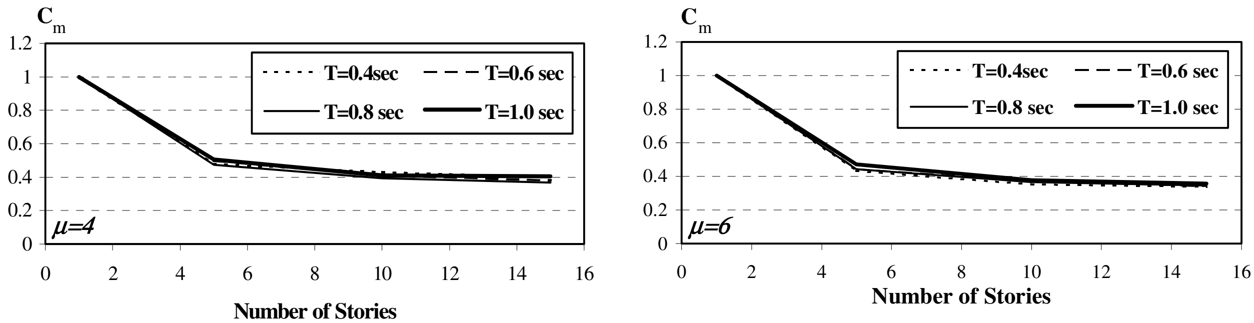

Figure 6. Effect of the number of stories on the roof displacement factor, C_m, for μ=4 and 6

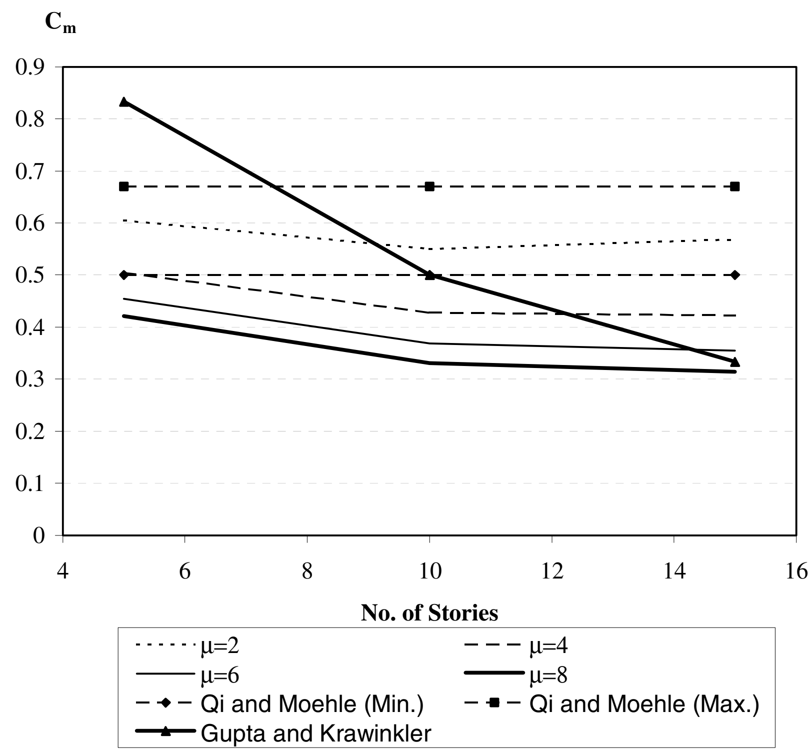

Figure 7. A comparison between the different suggested values of C_m for multi-story models

Number of Stories

The effect of number of stories on C_m is illustrated in Fig. 6 for ductility ratios of four and six. The results indicate that C_m decreases as the number of stories increases. Figure 6 also confirms that C_m does not highly depend on the fundamental period of the structure.

Determination of Empirical Formula for C_m

Based on the mean values of C_m for various ductility ratios and fundamental periods, the following expression can be suggested for the determination of C_m for MDOF systems:

(11) $$C_m = N^{(0.01N-0.25)} \cdot \mu^{-(0.15Ln(N)+0.02)}$$

where μ is the allowable ductility ratio based on the maximum displacement of stories and N is the number of stories. The values obtained from Equation [11] are in good agreement with the analytical results and represent a conservative approximation of C_m. A comparison of C_m values suggested by other researchers (Gupta and Krawinkler 2000, Qi and Moehle 1991) and those calculated using Equation [11] for multi-story buildings is depicted in Fig. 7. It is seen in this Figure that Equation [11] produces C_m values that lie within the range of values predicted by other researchers.

DETERMINATION OF MAXIMUM ROOF AND STORY DISPLACEMENTS

The results obtained in this study are based on shear-building models. However, a shear-building model rarely occurs in reality and further efforts should be made to examine the behavior of a wide range of real structural forms. Therefore, it is useful to verify the application of the method for some structures with moment-resisting frames. For this purpose, a five and a ten story steel building are selected. The structures are assumed to be located in a zone designated in the relevant code as seismically active with a design peak ground acceleration of 0.4g, and are founded on a soil profile type C specified in the NEHRP Provisions. The buildings consist of composite floors and ordinary moment resisting steel frames arranged along the east-west direction. The lateral load resisting system in the north-south direction is the concentric bracing. The moment resisting steel frames resist the seismic lateral forces in the east-west direction.

Design of frames

The steel frames are designed in accordance with the seismic provisions of NEHRP. The design is conducted using an iterative process assuming infinitely rigid joints and is based on centerline-to-centerline dimensions, an approach that will likely be used in practice. The fundamental period of a building is calculated from an empirical formula specified in the code that follows a proposed loading pattern. At the end of the iterative design process, the actual fundamental period set to the same period determined from the empirical formula of the code. The effect of accidental torsion is neglected. The ground motion is applied in the east-west direction only, and the combination effects of ground motions in two perpendicular directions are ignored.

The beams and columns are designed such that they can support both gravity and lateral loads in accordance with the allowable stress design procedure of AISC specifications (1994). Sets of expressions have been extracted to relate the geometrical characteristics of the actual sections to the cross-sectional area of the members. These expressions have been examined to choose the most suitable section for each member. Once the members are selected, the entire design is checked for code drift limitations. The NEHRP Provisions recommends that the inter-story drift under equivalent-static-force should not exceed 0.025h, where h is the story (or overall) height.

Analytical modeling and dynamic analyses

The nonlinear dynamic analysis computer program DRAIN-2DX is used to predict the dynamic responses of the two frames. A two-dimensional beam-column element that has both flexural and axial stiffness is used to model the beams and columns. This element allows the formation of plastic hinges at concentrated points near its ends. The yield strength of the beam is limited to $Z\sigma_y$, where Z is the plastic modulus of the beam section and σ_y is the yield stress. A yield interaction relationship involving both axial force and bending moment is prescribed for the columns, i.e.,

(12) $P_f/P_y+0.85M_{fx}/M_{pc}<1.0$

where P_f and M_{fx} are the axial load and the bending moment about the major axis due to gravity and lateral loads, while P_y and M_{pc} are the axial yield resistance and the moment yield resistance of the column section. No strain-hardening is considered.

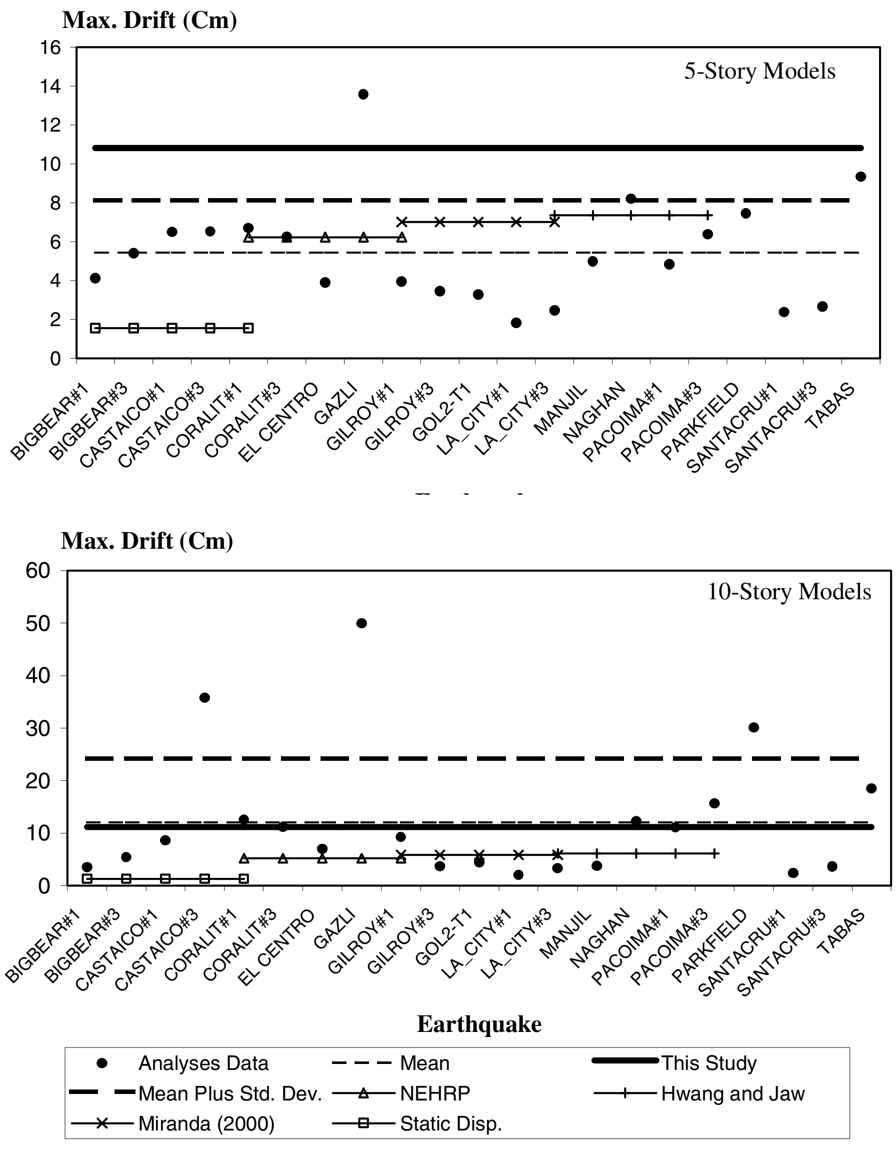

Figure 8. A comparison between the dynamic analyses results and the maximum inter-story drift as proposed by NEHRP, Hwang and Jaw, Miranda and this study for the 5 and 10-story moment-resisting frames

In the dynamic analysis, damping is considered proportional to both mass and initial stiffness with a damping ratio of five percent. Member stability considerations are included while lateral sway effects (P-delta) are excluded. The two structures are subjected to twenty-one earthquake records to assess their seismic performance. The maximum inter-story drifts of the two frames due to all ground motions are shown in Fig. 8.

Estimation of maximum drift

To apply the proposed method to determine maximum story displacement, it is necessary to estimate the over-strength factor, R_o of the frames. Based on the results of pushover analyses, an over-strength factor R_o of 1.16 and 1.31 is considered for five and ten story frames, respectively. Having determined factors R (NEHRP-94), R_T (Equation [7]) and R_o, the conventional force reduction factor, R_μ can be calculated from the relationship $R_\mu = R/(R_T . R_o)$ (Equation [4]). Equation [13] (Nassar et al. (1992)) is then used to estimate the ductility ratio, μ:

(13) $$R_\mu = \left[c(\mu - 1) + 1\right]^{1/c}$$

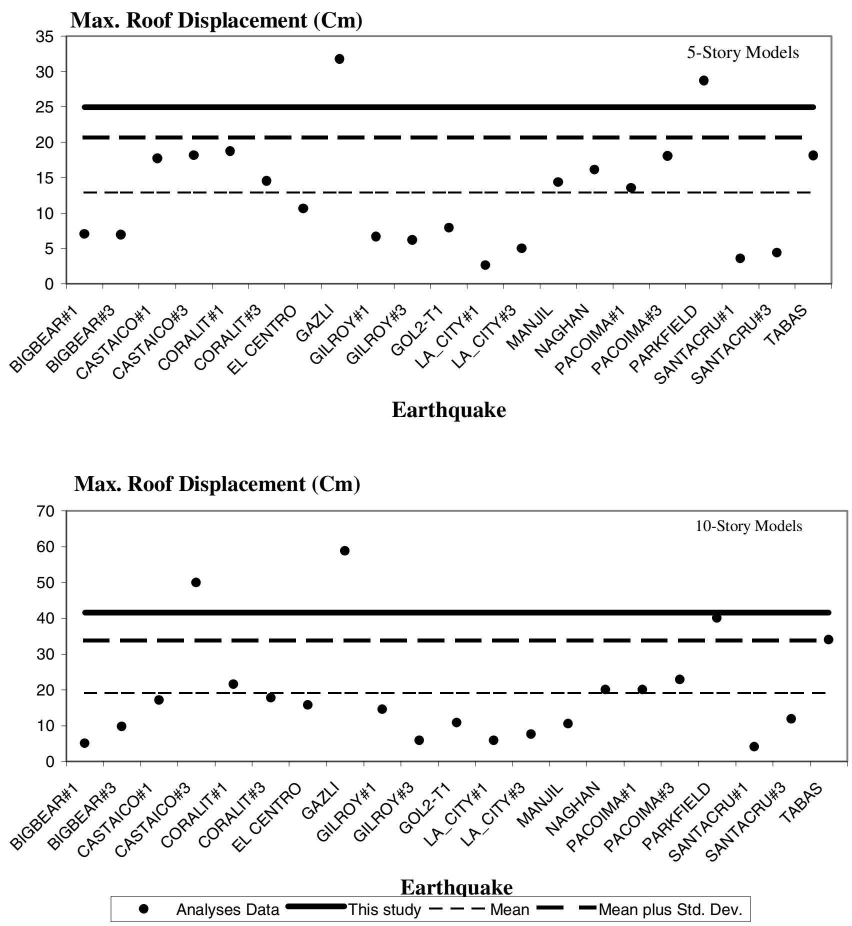

Figure 9. A comparison between the dynamic analyses results and the maximum roof displacement as proposed by this study for the 5 and 10-story moment-resisting frames

where the parameter c for elastic-perfectly plastic systems is given by:

$$c = \frac{T}{T+1} + \frac{0.42}{T} \qquad (14)$$

and T is the period of structure. Subsequently, the maximum inter-story drift and roof displacement are calculated using Equations [9], [10] and [11], respectively.

The maximum inter-story drifts suggested by the NEHRP, Miranda (2000), Hwang and Jaw (1990) and the present study are shown in Fig. 8. The mean and mean plus one standard deviation of the results for all earthquakes are also plotted in the same figure. This figure shows that the proposed method overestimated the maximum drift for five-story frame buildings, while the NEHRP Provisions and others suggested a maximum story displacement between mean and mean plus one standard deviation of maximum story displacement of the frame subjected to all earthquakes. For the ten-story building, all methods underestimated the maximum drift but the proposed method yielded the best estimate. This shows that the proposed method could be a more conservative approach to determine the maximum story displacement of the real structures.

Figure 9 shows the maximum roof displacement suggested by the present study compared with the mean and mean plus one standard deviation of the results for all earthquake records. As shown in this figure, the maximum roof displacements calculated using the proposed method is conservative and could be considered suitable for use in the preliminary design stage.

CONCLUSIONS

Nonlinear seismic analyses of shear building and steel frame models were performed. The results were examined and a statistical process was applied to estimate the seismic demands of roof and story for MDOF structures. The roof drift demand is related to the maximum story drift demand using a simplified process that provides quick and reasonable estimates of seismic demands. A modifying factor R_T was introduced to adjust and extend the concept of response modification factor. The following conclusions were drawn:

The inelastic base shear of an MDOF shear-building system can be calculated from an elastic SDOF response spectrum using the modifying factor R_T. The modifying factor R_T decreases with an increase in the number of stories approaching a constant value. The proposed expression provides a conservative estimate for R_T. An empirical formula is proposed to evaluate the code based deflection amplification factors relating the nonlinear responses of SDOF and MDOF systems. This formula evaluates the amplification factor as a function of the allowable ductility ratio, fundamental period, number of stories, and reduction factor. The deflections calculated using the proposed formula are in good agreement with those obtained from the nonlinear analyses, and seem to be more conservative than the values obtained from the NEHRP Recommended Provisions.

An empirical formula is introduced to relate the maximum roof and story displacements. It has been demonstrated that this relationship is a function of the strength (and/or stiffness) distribution within the structure, number of stories (fundamental period) and ductility ratio. The results of the study can provide a simple and practical way to determine the nonlinear dynamic response of multi-degree of freedom structures. An approximate approach is presented to calculate the maximum inelastic deformations in a structure with a given strength distribution. The relationships proposed in this paper could be useful in the conceptual design phase, estimating deformation demands for performance assessment, and improving basic understanding of seismic behavior.

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