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MODIFICATIONS OF SEISMIC DEMANDS FOR MDOF SYSTEMS

G.D.P.K. SENEVIRATNA and H. KRAWINKLER

Department of Civil Engineering, Stanford University, Stanford, CA 94305-4020, USA

ABSTRACT

SDOF spectra provide a convenient tool for the quantification of both elastic and inelastic seismic demands. However this basic information needs to be modified to become of use in the design of real structures, which are mostly MDOF systems. This paper presents quantitative information on MDOF modifications obtained from two research studies on MDOF effects in moment resisting frames and structural walls.

KEYWORDS

Seismic design, seismic demands, higher mode effects, frame structures, shear walls, inelastic response.

INTRODUCTION

Spectral representations of inelastic seismic demands (strength, displacement, energy) provide a convenient starting point for seismic design, especially in the conceptual design phase. Aided by the large increase in the number of ground motions recorded in the last decade, extensive studies have been conducted to quantify seismic demands for single degree of freedom (SDOF) systems (e.g., Nasser and Krawinkler, 1991; Miranda, 1993; Rahnama and Krawinkler, 1993; Vidic et al., 1994). The information contained in SDOF spectra, although relevant as baseline data, needs to be modified to become of direct use for the design of real structures, which are mostly multi degree of freedom (MDOF) systems with system dependent response characteristics. If a MDOF structure is expected to respond elastically to ground motions, elastic response spectra together with modal combination methods (SRSS, CQC) provide good estimates of the peak responses for the structure. However, if the performance level is life safety or collapse prevention, significant inelastic response is to be expected and modal analysis methods cannot be used with confidence.

Seismic demands for MDOF systems may differ from those of their SDOF counterparts due to higher mode effects and many other structural characteristics such as global mode of deformation, torsional effects, distribution of strength and stiffness over the height of the structure, structural system redundancy and the mode of failure at both element and global levels. The dynamic behavior of real structures is governed by a complex interaction of many of the above factors and can be determined accurately only through dynamic inelastic analyses on a case by case basis. The needs for a global understanding of MDOF inelastic response characteristics and for approximate tools that aid in the conceptual design process make it attractive to use parametric simulation techniques to assess MDOF effects and to develop modification factors that quantitatively describe the differences between MDOF behavior and the behavior of a counterpart SDOF system.

Such modification factors are discussed in this paper. The results reported here are extracted from two research studies (Seneviratna, 1995; Nassar and Krawinkler, 1991) that provide some of the answers needed to assess inelastic seismic demands for MDOF systems located on rock or stiff soils. The focus of these studies is to quantify the seismic demand modifications due to higher mode effects, and the effect of the type of "failure" mode on these modifications. Two types of lateral load resisting systems are investigated; moment resisting frames (MRFs) in which the inelastic behavior is governed by story shear, and structural walls in which it is desirable to control the inelastic behavior by flexural yielding. The motivation is to draw quantitative conclusions on the effectiveness of each yielding mechanism in limiting strength, displacement and energy demands imposed on the MDOF system.

A statistical evaluation of the inelastic response of the analytical models to an ensemble of 15 recorded ground motions is used in the parametric studies to develop relationships between MDOF seismic demands and SDOF spectra. Only mean values of response parameters are presented here unless stated differently. A statistical response study can be attempted only for ground motions with similar frequency characteristics. Accordingly, the constituent time histories are selected only from ground motions recorded on rock and stiff soil sites and whose elastic acceleration response spectrum resembles the ATC-3 S_1 spectrum (ATC, 1978). The mean and mean $\pm\ \sigma$ of the spectral shapes of the selected records together with the ATC-3 reference spectrum are shown in Fig. 1. The derived modification factors are applicable only for seismic input with similar characteristics; issues particular to ground motions recorded on soft soil sites are not addressed here.

MDOF MODELS USED IN THIS STUDY

As defined here, MDOF modification factors quantify the seismic demands for an MDOF system relative to the demands for the first mode SDOF system; i.e., an SDOF oscillator having a period equal to the first mode period of the MDOF system. The generic models for both frames and walls are "designed" individually for each ground motion so that the design base shear, V_y, is equal to the inelastic strength demand, $F_y(\mu)$, of the first mode SDOF system with a prescribed value of ductility, μ(SDOF). The UBC (UBC, 1994) static load pattern (hereafter referred to as design load pattern) is used to distribute the base shear over the height of the structure, and the member strengths are tuned so that a specific mechanism is formed.

Moment Resisting Frames (MRFs). Three types of single bay frame models are used, representing different yield mechanisms as illustrated in Fig. 2. The first type, designated as "beam hinge" (BH) model, represents MRFs designed according to the strong column - weak beam philosophy. The relative member strengths are tuned so that under the design load pattern a complete mechanism forms with simultaneous plastic hinging in all beams and supports. The second type, designated as "column hinge" (CH) model, represents MRFs designed to accommodate plastic hinging in the columns (weak column - strong beam). The relative member strengths are tuned so that under the design load pattern all columns form plastic hinges simultaneously, which permits individual story mechanisms. The third type is a "weak story" (WS) model in which the member strengths are tuned so that a story mechanism develops in the first story and all other stories remain elastic, representing a strength discontinuity in the first story. The relative member stiffnesses of all three types of models are tuned so that the interstory drifts are constant in each story.

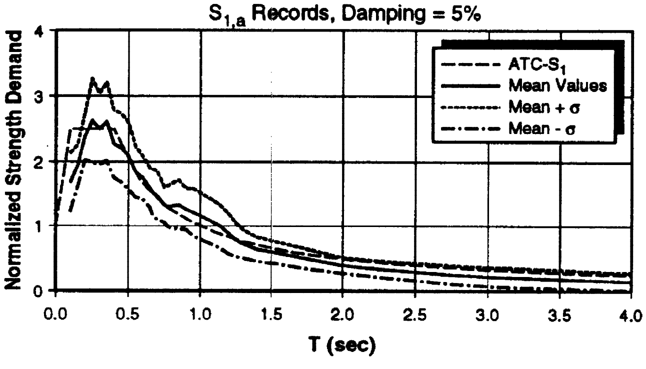


Fig. 1 Mean spectral shapes of selected records

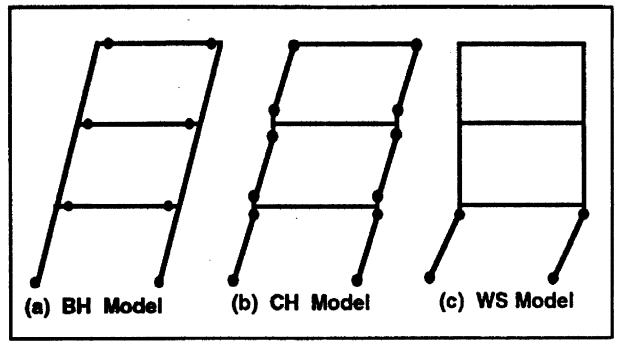


Fig. 2 Frame models used in this study

Structural Walls. The wall structures are modeled as vertical cantilevers with equal masses lumped at each floor level. Analyses are limited to walls with a constant flexural stiffness over the height. The effects of wall shear deformations are neglected. In the wall design, the SDOF inelastic strength demand, $F_y(\mu)$, is distributed over the height of the structure using the design load pattern, and the resulting base moment is assigned as the flexural strength at the base, M_y. The remainder of the structure is assigned a high flexural strength so that plastic hinging is limited to the base.

The stiffness values are selected so that the first mode period of each structure is equal to that given by the code equation $T = .02h_n^{2/3}$ sec., where h_n is the total height of the structure. This equation is used for both frames and walls to permit a direct comparison of the two structural systems. Structures with 2, 5, 10, 20, 30 and 40 stories are considered, with the first mode periods being 0.22, 0.43, 0.73, 1.22, 1.65, and 2.05 seconds, based on a constant story height of 12 ft (3.6m). For the time history analyses Rayleigh damping is used to obtain a damping ratio of .05 of critical in the first two modes. Inelastic time history analysis was performed for each structure and the 15 time history records using the computer program DRAIN-2DX.

MDOF MODIFICATIONS FOR FRAME STRUCTURES

Roof Displacement and Interstory Drift Demands

The maximum roof displacement of MDOF frames is mostly controlled by first mode response. The top (roof) displacement of elastic frames, $\delta_{t,el}$, can be predicted rather accurately by multiplying the spectral displacement of the first mode elastic SDOF system, $\delta_{SDOF,el}$, by the first mode participation factor. The accuracy of this approximation can be assessed from Fig. 3, which shows mean and mean + σ values of the ratio $\delta_{t,el}/\delta_{SDOF,el}$, denoted as α_1, together with period (number of lumped masses) dependent participation factors. Inelastic behavior de-amplifies the roof displacement for longer period structures and amplifies it for short period structures. This pattern is illustrated in Fig. 4, which presents mean values of the ratio $\delta_{t,in}/\delta_{t,el}$, denoted as α_2, for BH models. Almost identical results for this ratio are obtained for CH and WS models, demonstrating that the maximum roof displacement is not sensitive to the deformed shape of the structure, which differs greatly between BH and WS models. The ratio $\delta_{t,in}/\delta_{t,el}$ for the MDOF systems shows the same period and ductility dependent pattern as has been observed for SDOF systems in many studies reported in the literature. But the ratio is consistently smaller for MDOF systems than for SDOF systems.

Expected values of the roof displacement demands for inelastic frames can be obtained from the equation

$$\delta_{t,in} = \alpha_1 \alpha_2 \delta_{SDOF,el} \qquad (1)$$

utilizing SDOF elastic displacement spectra and the information presented in Figs. 3 and 4.

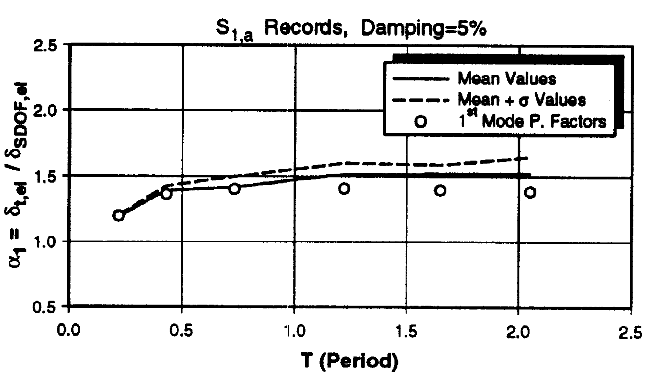


Fig. 3 Normalized roof displacement demands for elastic frame structures

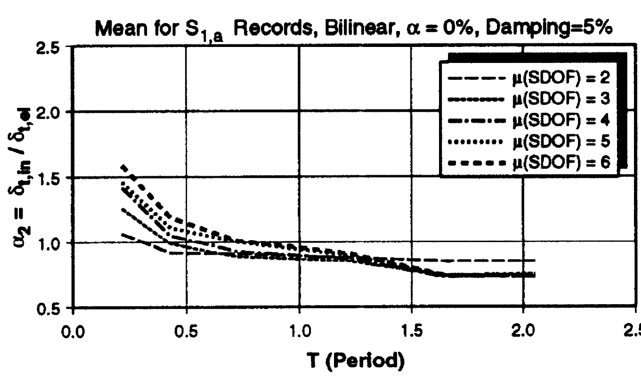
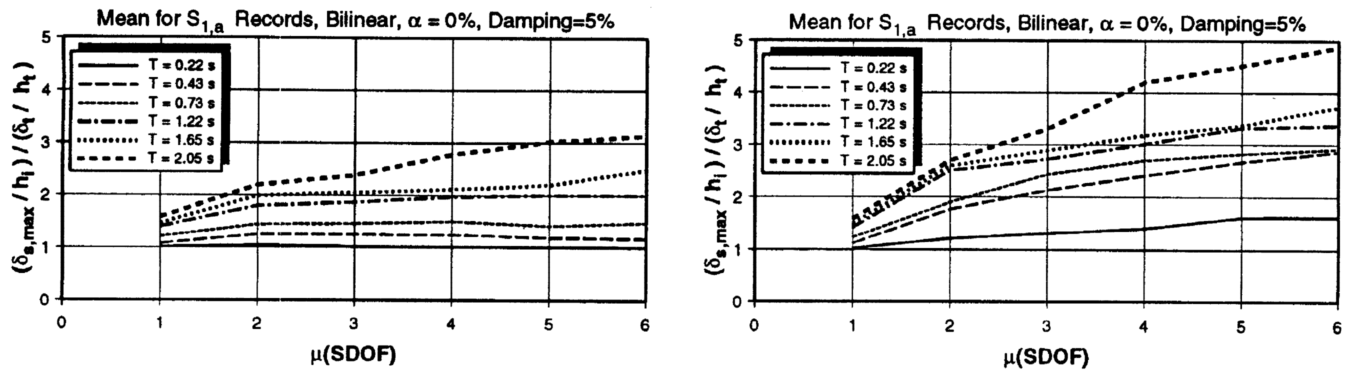


Fig. 4 Normalized roof displacement demands for inelastic frame structures

Interstory drift demands are of concern for damage control and P-delta control. When post earthquake functionality is a performance objective, interstory drift demands may have to be considered explicitly in the design process. These demands are greatly affected by higher mode effects and the type of mechanism, vary considerably over the height of the structure, and their maximum may be significantly larger than the global drift demand. Mean values of the maximum interstory drift, $\delta_{s,max}/h_i$ (h_i = height of story), normalized by the global drift δ_t/h_t (h_t = total height of structure) are shown in Fig. 5 for BH and CH models. The ratios clearly demonstrate the importance of the period dependent higher mode effects and the dependence on the yield strength (represented by μ(SDOF), and show that the maximum story drift increases significantly when undesirable story mechanisms are permitted (CH models) compared to frames designed according to the strong column - weak beam concept (BH models).



(a) BH Models (b) CH Models

Fig. 5 Mean values of normalized maximum interstory drift demands

Story Ductility Demands

Story ductility is defined here as the ratio of maximum interstory drift, $\delta_{s,i}$, to the yield interstory drift, δ_y. Figure 6 shows a typical example of the variation of story ductility demands over the height for a relatively tall frame structure. Because of higher mode effects, the story ductility demands are high near the bottom and top of the structure and relatively low around midheight. In most but not all cases the maximum story ductility demand occurs in the first story. In all but very short period structures the mean of the maximum story ductility demand is larger than the target ductility ratio for which the structure was designed. Thus, MDOF effects lead to an amplification of maximum story ductility demand, $\mu_{s,max}$, compared to the ductility demand for the first mode SDOF system, μ_{SDOF}. Mean values of the ratio $\mu_{s,max}/\mu_{SDOF}$ for CH models are presented in Fig. 7. This ratio increases significantly with period (number of stories) and somewhat with a decrease in yield level (represented by μ_{SDOF}). For BH models the ratio is consistently smaller (approximately 1.8 at the period of 2.05 sec.), and for WS models it is much larger (in the order of 8 at the period of 2.05 sec.) than for the CH models. The small amplification of the ductility ratio for BH models clearly demonstrates the superior performance of structures in which story mechanisms are avoided and inelastic deformations are distributed more uniformly over the height of the structure.

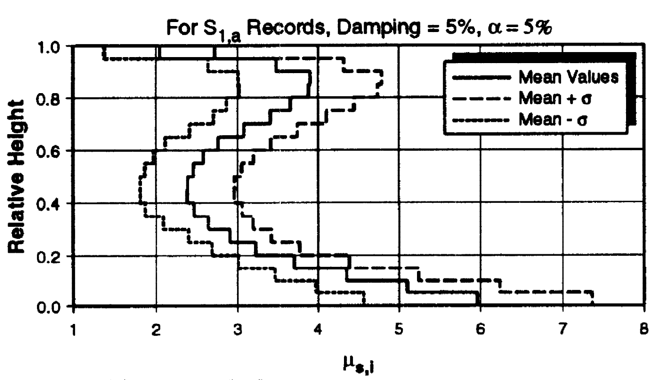


Fig. 6 Variation of story ductility ratio over height; 20 story frame, CH model, $\mu_{SDOF} = 4$

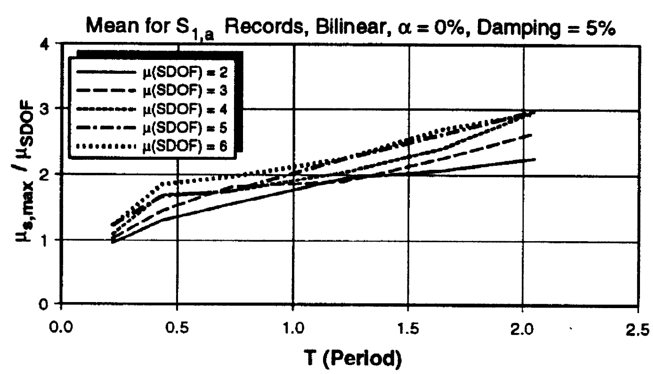
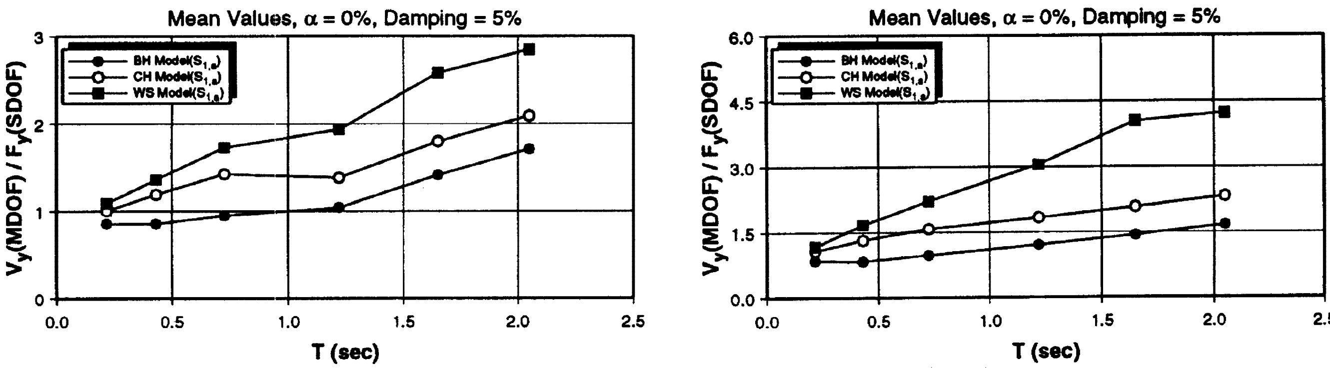


Fig. 7 Mean amplification of maximum story ductility ratio; CH models

Base Shear Demands

The results presented so far show that, in order to limit the maximum story ductility demand of the MDOF system to the target ductility ratio, the base shear strength of the MDOF system, V_y(MDOF), has to be greater than the strength demand of the SDOF system with the specified target ductility ratio, F_y(SDOF). Nasser and Krawinkler (1991) used an approach based on an interpolation procedure to compute the required modification factor for base shear strength, V_y(MDOF)/F_y(SDOF). Representative results obtained in their study are shown in Fig. 8 for target ductilities μ(SDOF) of 3 and 6. It can be seen that the modification factor increases significantly with period and target ductility ratio, and is largest for the very undesirable WS models and smallest for the desirable BH models.



(a) Target ductility ratio = 3 (a) Target ductility ratio = 6

Fig. 8 Modification factor for base shear strength demand

The design implications of the results presented in Fig. 8 can be illustrated on a simple example. If a MRF structure with a fundamental period of 1.22 sec. can tolerate a story ductility of 6, the base shear strength needs to be increased by a factor of approximately 1.2, 1.9, or 3.1, respectively, for BH, CH, or WS structures, compared to the inelastic strength demand of the corresponding SDOF system with the same period and target ductility. Clearly, these results apply only for sites whose design ground motion spectrum resembles the ATC-3 S_1 spectrum.

MDOF MODIFICATIONS FOR STRUCTURAL WALLS

Deformation Demands

In wall structures the definition of a ductility ratio becomes ambiguous and interpretations in terms of such a ratio may be misleading. Hence, the focus is on absolute deformation quantities that are most relevant for design and on relationships that permit the evaluation of these quantities from the corresponding SDOF spectral displacement. The design quantities of primary interest are judged to be the roof displacement and the plastic hinge rotation at the base of the wall. The following discussion presents a procedure that was found to be most effective to relate these quantities to each other and to the first mode SDOF displacement.

Roof Displacement Demands. Figures 9 and 10 present the same parameters ($\delta_{t,el}/\delta_{SDOF,el}$ and $\delta_{t,in}/\delta_{t,el}$) for wall structures as are shown in Figs. 3 and 4 for frame structures. The results for frame and wall structures are very similar and the conclusions drawn previously apply also for wall structures. Thus, Eq. (1) can be utilized also to predict the roof displacement for wall structures.

Plastic Hinge Rotation at Base. The maximum displacement at the top of wall structures results from two sources, the plastic hinge rotation at the base and the elastic flexural deformations over the height of the structure. The plastic rotation at the base results in a rigid body displacement at the top of the wall. The additional top displacement comes from the multi-mode response of the elastic portion of the wall above the base. If it is assumed that damage is caused only by inelastic deformations, then the plastic hinge rotation at

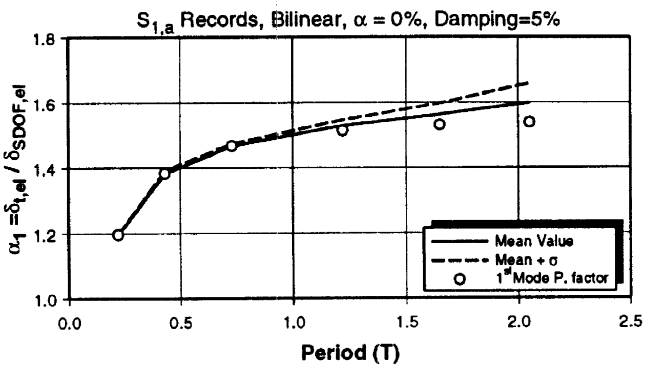


Fig. 9 Normalized roof displacement demands for elastic wall structures

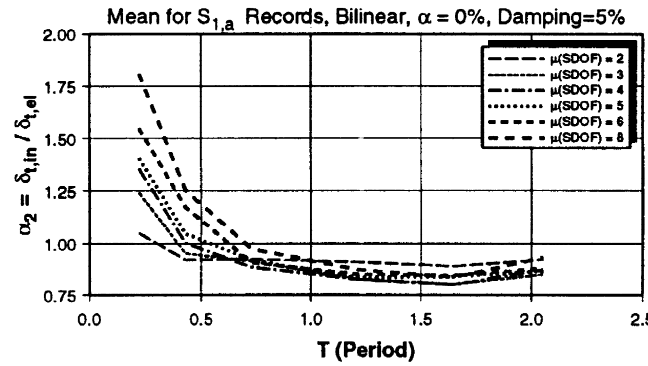


Fig. 10 Normalized roof displacement demands for inelastic wall structures

the base becomes the basic parameter for deformation based design. In the interpretation of the results discussed next it must be considered that the wall bending strength above the base is assumed to be large enough to prevent plastic hinging in all but the bottom story. This may not be the case in many real designs in which hinging may be distributed over several of the bottom stories.

Figure 11 present mean values of the plastic hinge rotation demands at the base of wall structures. The maximum plastic hinge rotation computed in the dynamic analysis, θ_p^{max}, is normalized by the global drift index, $\theta^{max}_{tot} = \delta_t/h_t$, where δ_t is the maximum displacement at the top of the structure, and h_t is the total height of the structure. This ratio is denoted as α_3. Since θ_p^{max} and δ_t do not necessarily occur at the same time, the difference between $\theta^{max}_{tot}h_t$ and $\theta_p^{max}h_t$ is not an accurate measure of the elastic displacement component. It is observed that the contribution of the plastic hinge rotation to the total displacement at the top, given by the ratio $\theta_p^{max}/\theta^{max}_{tot}$, is strongly dependent on the bending strength of the structure (defined by μ(SDOF) and the design load pattern), but is not very sensitive to the initial period of the structure.

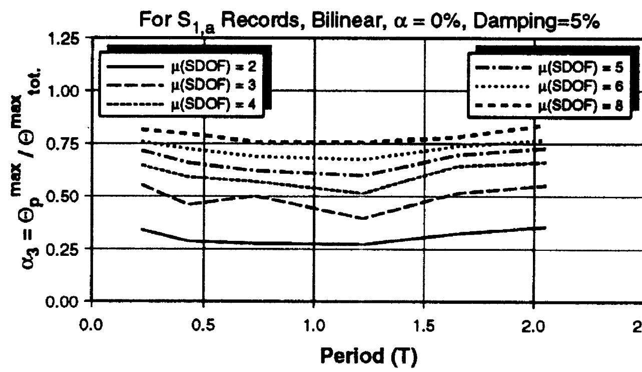


Fig. 11 Normalized plastic rotation demands at base

The results presented in Figs. 9 to 11 permit an assessment of the plastic hinge rotation demand for wall structures in terms of SDOF elastic displacement spectra, SDOF ductility ratios μ(SDOF), and wall height h_t. This demand can be computed as:

$$\theta_p^{max} = \alpha_1\alpha_2\alpha_3\frac{\delta_{SDOF,el}}{h_t} \qquad (2)$$

As a word of caution, it must be considered that the wall bending strength depends on the design load pattern, which is imbedded in the μ(SDOF) on which the graphs in Figs. 9 to 11 are based.

Strength Demands

Base Shear Demands. In the wall structures utilized in this study the bending strength at the base is a quantity that is related to the inelastic strength demand of the corresponding first mode SDOF system through the design load pattern. The load pattern determines the lever arm h', the height to the resultant of the load pattern from the base of the wall. As shown in the left sketch of Fig. 12, the lever arm of the code load pattern, $(h')_{code}$, relates the flexural strength at the base, M_y, to the design shear force, $F_y(\mu)$. Under dynamic excitations, the lever arm will change and may be significantly smaller than $(h')_{code}$ if higher mode effects become important. Since the flexural strength at the base is a given quantity, any decrease in lever arm will lead to an increase in base shear demand compared to the design base shear force $F_y(\mu)$. This increase can be very large if the wall bending strength is low (large μ(SDOF) values) and the second (and perhaps third) modal period is in the high acceleration range of the elastic response spectrum. Figure 13 shows mean values of the base shear amplification, expressed by the ratio V_b(MDOF)/F_y(μ(SDOF)), where V_b(MDOF) is the maximum base shear from the dynamic analysis and F_y(μ(SDOF)) is the design base shear defined as the inelastic strength demand for the first mode SDOF system and the selected target ductility ratio. The graph shows very large amplifications for longer period structures. This is of much relevance for shear design of walls since the design objective is to make walls strong enough to prevent shear failure. It should be noted that the amplifications for μ(SDOF) > 4 are of mostly academic value since present code designs for wall structures usually result in a bending strength at the base corresponding to μ(SDOF) < 4.

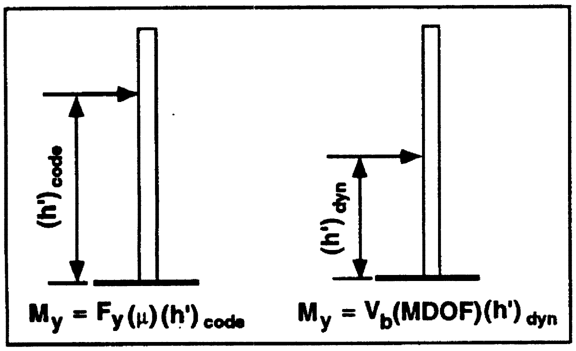


Fig. 12 Relationship between bending strength at base and base shear

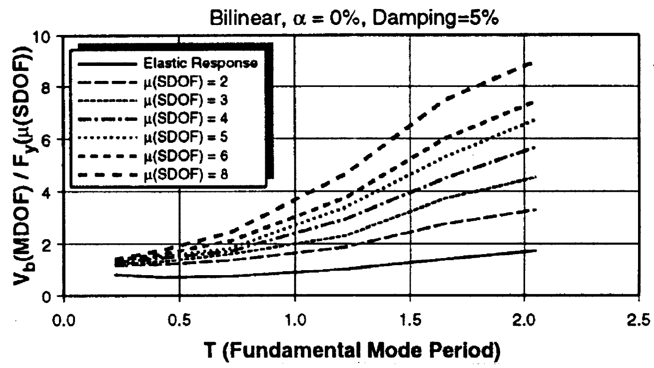


Fig. 13 Mean amplification of base shear demand for wall structures

Story Shear Demands. The significant amplification of the dynamic base shear demand, compared to the demand obtained from a code load pattern, necessitates a re-evaluation of shear design forces for the full height of wall structures. For a story by story evaluation, dynamic story shear envelopes normalized by the design story shears for a 20-story wall structure are shown in Fig. 14. The design story shears for this purpose are computed from the base shear on which the wall bending strength is based (F_y(μ(SDOF)), and the UBC 1994 seismic load pattern. Results are presented for elastic dynamic response and for values of μ(SDOF) = 4 and 8. For the inelastic wall structures the dynamic story shears are higher than the design story shears over the full height of the wall. The amplification is large at the base, decreases rapidly in the lower stories, reaches a minimum around midheight, and increases again rapidly in the upper stories (except for the top story). Higher mode effects are responsible for the amplification and its variation over the height. Thus, shear design based on the code load pattern will not provide adequate protection against shear failure, particularly in the bottom stories and in stories close to the top of a wall structure.

In order to permit a direct comparison between the earthquake induced story shear force patterns and the code design shear force pattern, the dynamic story shear demands are normalized by the corresponding dynamic base shear demand, and the design story shears are normalized by the design base shear. The resulting graphs for the 20-story wall structure are presented in Fig. 15 for the same cases as illustrated in Fig.

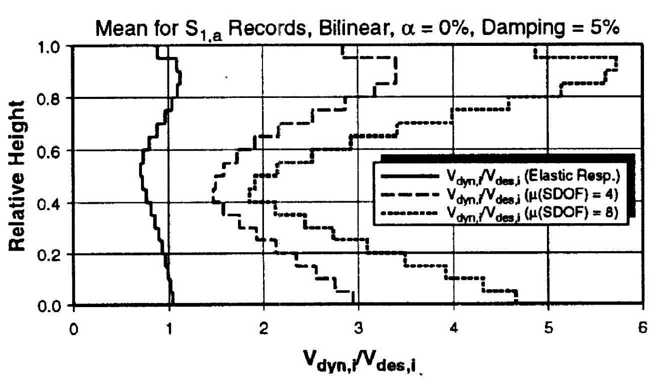


Fig. 14 Normalized story shear demands, 20-stories

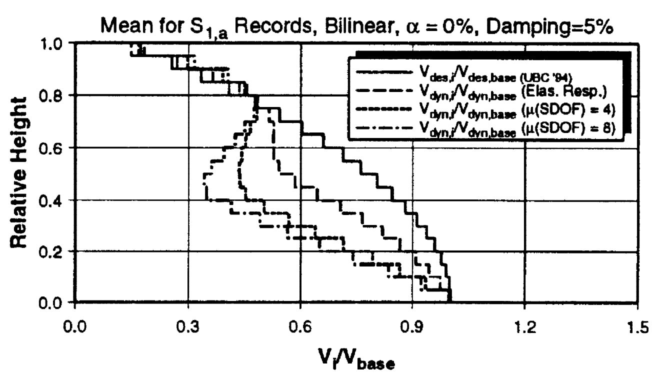


Fig. 15 Normalized story shear force patterns

14. The figure shows that the code load pattern overestimates the relative story shear force demands for large portions of the height of the structure. Thus, compared to presently employed procedures, the design base shear needs to be increased but the story shear force pattern can be modified to represent the more rapid decrease in shear force demands in the lower stories.

SUMMARY AND CONCLUSIONS

This paper presents some of the results obtained from statistical studies on MDOF effects for two-dimensional frame and wall structures. The effects are evaluated by subjecting generic structural systems from 2 to 40 stories to an ensemble of 15 ground motions recorded on rock and stiff soil sites. The objective is to assess the modifications needed to utilize seismic demand parameters obtained from inelastic SDOF spectra in the design of MDOF structures. Only strength and deformation demands are addressed here; information on energy demands can be obtained from Seneviratna (1995). The modifications are sensitive to the type of structure, the deformation mode, the mechanism that controls post-elastic response, the strength of the structure, and the relative spectral amplitudes at the low (primarily first and second) natural periods of the MDOF structure.

The results obtained in these studies are intended to contribute to the understanding of inelastic dynamic response characteristics of frame and wall structures. They are also intended to provide quantitative data needed to estimate strength, stiffness, and ductility requirements for conceptual design. It is important to point out that MDOF modifications are sensitive to strength and stiffness distributions over the height and to the frequency characteristics of the ground motions. Thus, the application of the presented modifications for cases not covered by the assumptions made here is subject to questions.

ACKNOWLEDGMENTS

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REFERENCES

ATC (1978). *Tentative Provisions for the Development of Seismic Regulations for Buildings*, Report No. ATC-3-06, Applied Technology Council, Redwood City, California, USA.

Miranda, E., (1993). Evaluation of site-dependent inelastic seismic design spectra. *J. of Struct. Division*, ASCE, 119,. 5.

Nassar, A.A. and H. Krawinkler (1991). Seismic demands for SDOF and MDOF systems. *John A. Blume Earthquake Engrg. Center Report No. 95*, Dept of Civil Engrg., Stanford University, USA.

Rahnama, M. and H. Krawinkler (1993). Effects of soft soils and hysteresis models on seismic design spectra. *John A. Blume Earthquake Engrg. Center Report No. 95*, Dept of Civil Engrg., Stanford U., USA.

Seneviratna, G.D.P.K., (1995). Evaluation of inelastic MDOF effects for seismic design. *Ph.D. Dissertation*, Dept. of Civil Engineering, Stanford University, USA.

UBC, (1994). *Uniform Building Code*. International Conference of Building Officials, Whittier, California.

Vidic, T., P. Fajfar, and M. Fischinger (1994.). Consistent inelastic design spectra: strength and displacement. *Earthquake Engineering and Structural Dynamics*, 23.