Evaluation of Displacement Amplification Factor for Seismic Design Codes

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ABSTRACT

To estimate the roof and story drifts occurring during severe earthquakes, the Uniform Building Code (UBC) uses $3R_w/8$ as a displacement amplification factor (*DAF*) to amplify elastic design drifts. A comparison of several seismic design codes shows that UBC's *DAF* is very low. A study conducted on four instrumented buildings indicates that the maximum story drifts developed in severe earthquakes are much higher than that predicted by the UBC. This study also shows that the *DAF* of multi-degree-of-freedom systems is similar to that of single-degree-of-freedom systems.

INTRODUCTION

Modern seismic design provisions assume that buildings will undergo inelastic deformations during severe earthquakes. Therefore, these provisions allow the designer to reduce the elastic seismic force demand by a force reduction factor (*FRF*). Since reduced seismic forces are used in design, the computed displacements from an elastic analysis have to be amplified in order to estimate the actual deformations that develop in severe earthquakes. Building codes usually use a displacement amplification factor (*DAF*) for this purpose.

Consider the typical lateral force versus deformation relationship of a structural system in Fig. 1. The elastic force demand, expressed in terms of a base shear ratio, for a severe earthquake is expressed as C_e. Idealizing the actual structural response curve by the linearly elastic-perfectly plastic curve in Fig. 1, the structural ductility factor can be defined as

$$\mu_s = \frac{\Delta_{\max}}{\Delta_y} \tag{1}$$

where the deformation is expressed in terms of story drift Δ. The ratio between the elastic force demand (C_e) and the structure's yield strength level (C_y) is defined as the ductility reduction factor, R_μ:

$$R_\mu = \frac{C_e}{C_y} \tag{2}$$

Based on these definitions, the force reduction factor (*FRF*) for working stress design can be derived from Fig. 1 as follows (8):

$$FRF = \frac{C_e}{C_w} = R_\mu \frac{C_y}{C_w} \tag{3}$$

The displacement amplification factor (*DAF*) can also be derived from Fig. 1 as follows:

$$DAF = \frac{\Delta_{max}}{\Delta_w} = \frac{\Delta_{max}}{\Delta_y}\frac{\Delta_y}{\Delta_w} = \mu_s \frac{\Delta_y}{\Delta_w} \tag{4}$$

Since Δ_y/Δ_w is equal to C_y/C_w (see Fig. 1), it follows that

$$DAF = \mu_s \frac{C_y}{C_w} \tag{5}$$

From Eqs. 3 and 5, the ratio between *DAF* and *FRF* is

$$\frac{DAF}{FRF} = \frac{\mu_s}{R_\mu} \tag{6}$$

For a single-degree-of-freedom system, the ratio between μ_s and R_μ can be expressed as follows (4):

(a) in the velocity and displacement amplification regions:

$$\frac{\mu_s}{R_\mu} = \frac{\mu_s}{\mu_s} = 1 \tag{7a}$$

(b) in the acceleration amplification region:

$$\frac{\mu_s}{R_\mu} = \frac{\mu_s}{\sqrt{2\mu_s - 1}} \geq 1 \tag{7b}$$

If a multistory building tends to deform as a single-degree-of-freedom system, which appears to be a valid assumption if the structure deforms into a global mechanism in severe earthquakes, it follows from Eqs. 6 and 7 that the *DAF* should not be less than *FRF*.

A review of the 1991 Uniform Building Code (10) of the U.S.A., the 1990 National Building Code (3) of Canada, the 1987 Mexico Code (5), and the 1988 Eurocode (7) indicates that the ratios between the *DAF* and *FRF* vary considerably from one code to another. Table 1 lists the ratio between the *DAF* and *FRF* used in these codes. On one extreme, both the Mexico Code and Eurocode use a *DAF* which is no smaller than the *FRF*. At the other extreme, UBC uses a *DAF* which is only 3/8 that of the *FRF*.

OBJECTIVE AND SCOPE

The main objective of this research is to evaluate the appropriate *DAF* for seismic design of multistory building frames. Dynamic analyses of four instrumented buildings in California (two steel and two reinforced concrete buildings) with fundamental periods ranging from 0.31 to 2.2 seconds were performed in order to investigate the relationship between the *DAF* and *FRF*.

METHOD OF ANALYSIS

The four buildings selected for this study have been instrumented by CDMG (6); they are designated as CSMIP 57357, 58496, 57355, and 58490. See Table 2 for a brief description of these buildings. For each building, the dynamic response was computed by the computer program DRAIN-2D (1). As input motions, eight historical earthquake records (see Table 3) were used. The average normalized response spectra of this set of earthquakes resembled the UBC elastic design spectra (see Fig. 2). By scaling the intensity of each earthquake record to different intensities, the roof drift and story drifts from the inelastic dynamic analyses were compared to those that would develop if the structure were to respond elastically. Referring to Fig. 1, the ratio between the inelastic drift (Δ_{max}) and elastic drift (Δ_e) is the same as the ratio between *DAF* and *FRF* because:

$$\frac{\Delta_{max}}{\Delta_e} = \frac{DAF\ (\Delta_w)}{FRF\ (\Delta_w)} = \frac{DAF}{FRF} \tag{8}$$

Both the ratios obtained from roof drift and story drifts were evaluated. The results obtained from the ratio of story drifts can be used to determine a suitable *DAF* for estimating the critical story drift, while the ratio of roof drifts can be used to determine an appropriate *DAF* for estimating the required building separations in order to avoid pounding. For each scaled earthquake record, the scale factor is defined as the ratio between the average pseudo-acceleration (*PSA*), expressed in terms of the gravitational acceleration, at the fundamental period of the structure and the UBC prescribed design base shear ratio, C_w. For the buildings and earthquake records selected in this study, the results of elastic dynamic analyses indicate that the base shear ratio, C_e, approximately equals the average *PSA*. Therefore, the earthquake scale factor can also be expressed as the ratio between C_e and C_w.

SUMMARY OF RESULTS

CSMIP Building No. 57357 — The lateral-force-resisting system of this building is a steel moment-resisting space frame (R_w = 12); the fundamental period is 2.18 seconds. A review based on the 1991 UBC indicates that story drift limit controlled the design (9). A 2-D interior frame in the E-W direction was modeled for dynamic analysis (see Fig. 3a). Elastic analysis of a 3-D mathematical model of the building using the ETABS program (2) indicated that the reactive mass associated with the selected frame is about 11% of the total building mass.

To calibrate the properties of the 2-D model, nonlinear dynamic analyses were performed using the 1989 Loma Prieta earthquake acceleration record as the input ground motion at the building base. In the analysis, the damping ratios were assumed to be 5% of critical damping for the first two modes. Fig. 4a shows that the displacements computed from the model correlate very well with the measured responses from the Loma Prieta earthquake. A nonlinear static analysis was also conducted to determine the lateral strength of the building; the UBC lateral load profile was used. The analysis indicated that the lateral load capacity of the building significantly exceeds the UBC design seismic forces (see Fig. 5a). The figure also shows that the failure mechanism is initiated at the base and continues to the seventh floor.

By scaling the earthquake record to different intensity levels, nonlinear dynamic analyses were performed for eight earthquake records to calculate the ratios between Δ_{max} and Δ_e (i.e., *DAF*/*FRF*). Fig. 6a shows that the displacement ratio at the roof level is slightly less than one; the ratio is about 0.8 at the UBC severe design earthquake level (i.e., C_e/C_w = 12.) Fig. 7a shows that the ratio of story drifts tends to be larger than one if the earthquake intensity is increased; the ratio is about 1.1 at the UBC design earthquake level. Therefore for this type of long period structures the UBC approach of using 3/8 of the *FRF* as *DAF* may significantly underestimate the roof and story drifts.

CSMIP Building No. 58496 — The lateral-force-resisting system of this hospital building (see Table 2) in the E-W direction consists of two eccentrically braced and one moment-resisting steel frames (see Fig. 3b); the fundamental period is 0.31 seconds. A low frequency noise was observed in the relative displacement records of this building. This noise was filtered for the dynamic correlation study. Fig. 4b shows that the displacements computed from a 2D mathematical model correlate reasonably well with the measured responses from the Loma Prieta earthquake. Fig. 5b shows that the lateral load capacity of the building is 5.8 times greater than the UBC prescribed design base shear. Fig. 6b shows that the ratio of roof displacements remains smaller than 1.0 as the earthquake intensity is increased; the ratio equal 0.8 at the level of the UBC severe design earthquake (i.e., $C_e/C_w = R_w$ = 12.) For a structure of such a short period, Fig. 6b indicates that the ratio may be greater than 1.0 if the intensity is further increased; this is consistent with the observations for single-degree-of-freedom systems (4). Fig. 7b shows the variations of the ratio for story drifts; the trend is similar to that observed in the previous long-period structure. Since the ratio from Fig. 7b is about 1.0 at the UBC severe design earthquake level, it appears that for such a short period structure the *DAF* should also be equal to *FRF*.

CSMIP Building No. 57355 — The lateral-force-resisting system of this building (see Table 2) in the N-S direction consists of four reinforced concrete ductile moment-resisting frames; the fundamental period is 0.96 seconds. An interior frame was selected for the dynamic analysis (see Fig. 3c). Elastic analysis of a 3-D mathematical model of the building indicated that the reactive mass associated with this frame is about 19% of the total building mass. Fig. 4c shows that displacements computed from the 2D mathematical model correlate reasonably well with measured responses from the Loma Prieta earthquake; the dynamic analysis indicated that some minor yielding might have occurred in a number of beams. Fig. 5c shows that the ultimate base shear of the building is about four times the UBC prescribed design base shear. The figure also shows that the building has a partial failure mechanism (from the base to the seventh floor). Since this is a reinforced concrete frame, the Takeda model was used to simulate the stiffness degradation of the members. Fig. 6c shows that the effect of stiffness degradation on the ratio of roof drifts is small; the ratio is about 0.8. Fig. 7c shows that the ratio of story drifts increases with the ground motion intensity. At the UBC severe design earthquake level, the ratio is about 1.2, much higher than the 3/8 recommended by the UBC.

CSMIP Building No. 58490 — The lateral-force-resisting system of this building (see Table 2) in the N-S direction consists of two perimeter moment-resisting frames; the fundamental period is about 0.84 seconds. The design of this building satisfies the UBC strength and stiffness requirements; the lateral strength controlled the design. Fig. 4d shows that displacements computed from the 2D mathematical model correlate well with the measured responses from the Loma Prieta earthquake; dynamic analysis indicated that some yielding might have occurred in a number of beams. Fig. 5d shows the overstrength of this building is relatively low, which is expected for perimeter frame systems. The ultimate base shear of the building is about 2.6 times the UBC prescribed design base shear. Since the column and beam strengths are constant along the height of building, the failure mechanism consists of a soft first story (see Fig. 5d). During severe earthquake excitation, it is expected that most of the damage will concentrate in the first story. Fig. 6d shows that the ratio of roof drifts is about 0.9 at an earthquake intensity similar to that of the UBC severe design earthquake. For the critical story drift, Fig. 7d shows that the ratios are 1.7 and 1.3 for models with and without stiffness degradation, respectively. These ratios are much greater than 3/8, as recommended by the UBC.

CONCLUSIONS AND RECOMMENDATIONS

Based on the observed dynamic responses of four regular planar frames subjected to eight historic earthquake records, the following conclusions and recommendations can be drawn.

(1) For steel frames, the ratio of *DAF/FRF* for estimating the critical story drift ranges from 1.0 to 1.3. For reinforced concrete frames with stiffness degradation, the ratio can be as high as 1.7. To estimate roof drift, the ratio of *DAF/FRF* ranges from 0.8 to 0.9.

(2) The ratio of *DAF/FRF* is insensitive to the fundamental period of the building. One flexible steel frame (T = 2.2 sec.) and one stiff eccentrically braced frame (T = 0.3 sec.) gave practically the same *DAF/FRF* ratio.

(3) The ratio of *DAF/FRF* for estimating the story drift is affected by the type of failure mechanism under severe ground shaking. For the two reinforced concrete frames studied, the 6-story perimeter frame having a soft first story exhibits a higher *DAF/FRF* ratio.

(4) Stiffness degradation has insignificant effect on roof displacement response. For frames with a soft story, the stiffness degradation effect is more pronounced on story drifts.

(5) This study shows a similarity of the *DAF/FRF* ratio for single- and multi-degree-of-freedom systems. The UBC recommended ratio of 3/8 is too low. For simplicity, it is recommended that a *DAF* equal to *FRF*, which is the approach used by the Eurocode and Mexico Code, be used for estimating both the story drift and roof drift.

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Building Code	*FRF*	*DAF*	$\frac{DAF}{FRF}$
UBC (1991)	R_w	$\frac{3R_w}{8}$	0.375
NBCC (1990)	$\frac{R}{0.6}$	R	0.6
Mexico (1987)	Q^*	Q	1.0
Eurocode (1988)	q	q	1.0

* less than Q in short period range

Table 1 *DAF/FRF* Ratios in Four Building Codes

CSMIP Bldg. No.	Construction Material	Reactive Weight (kips)	C_w	R_w	Direction of Frame	Period (sec)		
						UBC	Measured	Model
57357	13-story steel	25,200	0.043	12	E-W	1.77	2.18	2.20
58496	2-story steel	4,550	0.115	12	E-W	0.22	0.31	0.31
57355	10-story R.C.	24,500	0.058	12	N-S	1.11	0.96	1.19
58490	6-story R.C.	6,430	0.049	12	N-S	0.79	0.84	1.06

Table 2 Description of Buildings

Earthquake	Station	Comp.	PGA (g)	EPA (g)
Imperial Valley (1940)	EL Centro	S90E	0.35	0.28
Washington (1949)	Olympia	S86W	0.28	0.22
Kern County (1952)	Taft	S69E	0.18	0.15
Parkfield (1966)	Cholane	N85E	0.43	0.33
San Fernando (1971)	Pacoima Dam	S16E	1.17	0.80
Imperial Valley (1979)	I. V. C.	S40E	0.33	0.20
Loma Prieta (1989)	Corralitos	S00E	0.63	0.52
Loma Prieta (1989)	Santa Cruz	S90E	0.41	0.33

Table 3 Earthquake Records

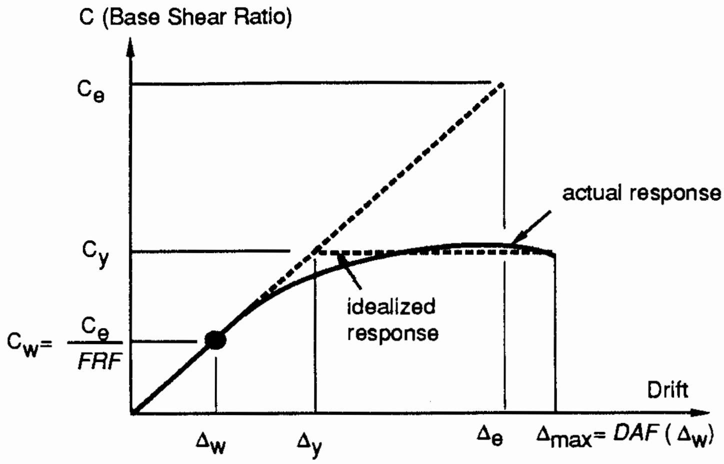

Fig. 1 General Structural Response

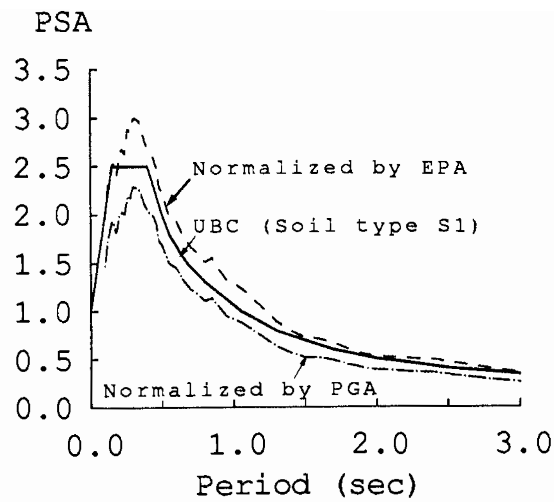

Fig. 2 Normalized Response Spectra (5% damping ratio)

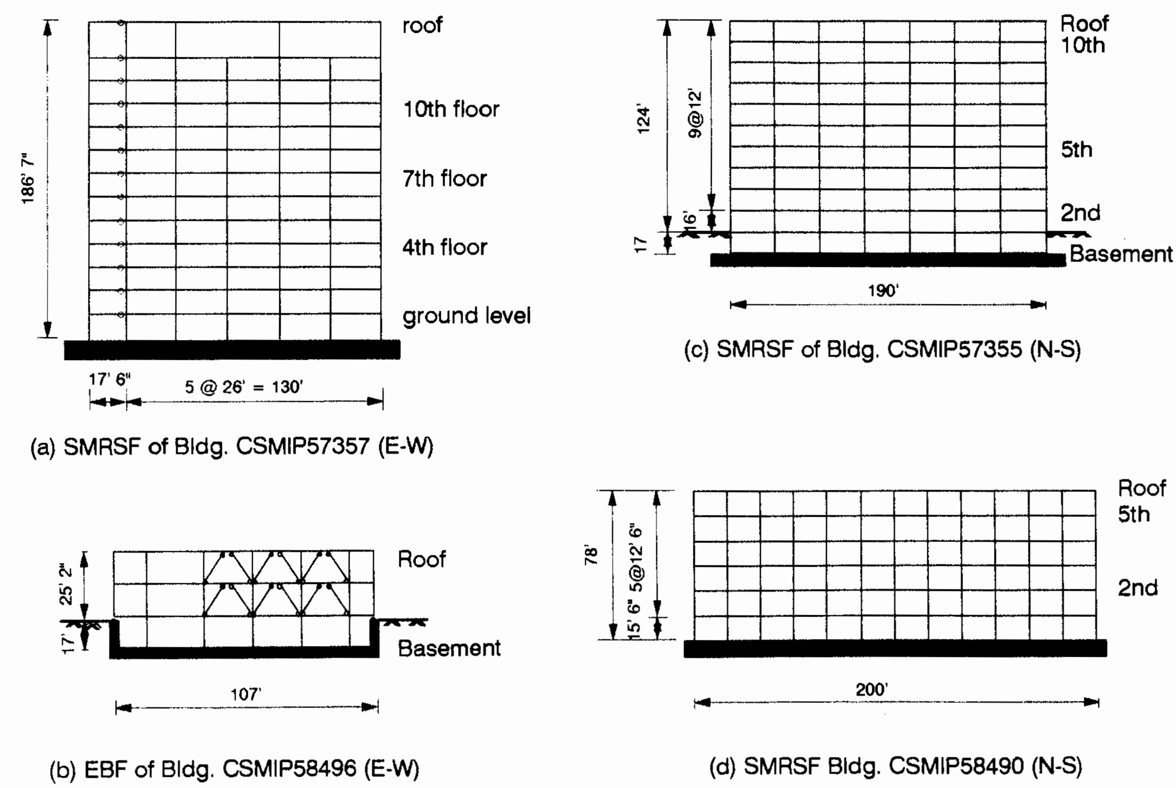

Fig. 3 Elevations of Building Frames

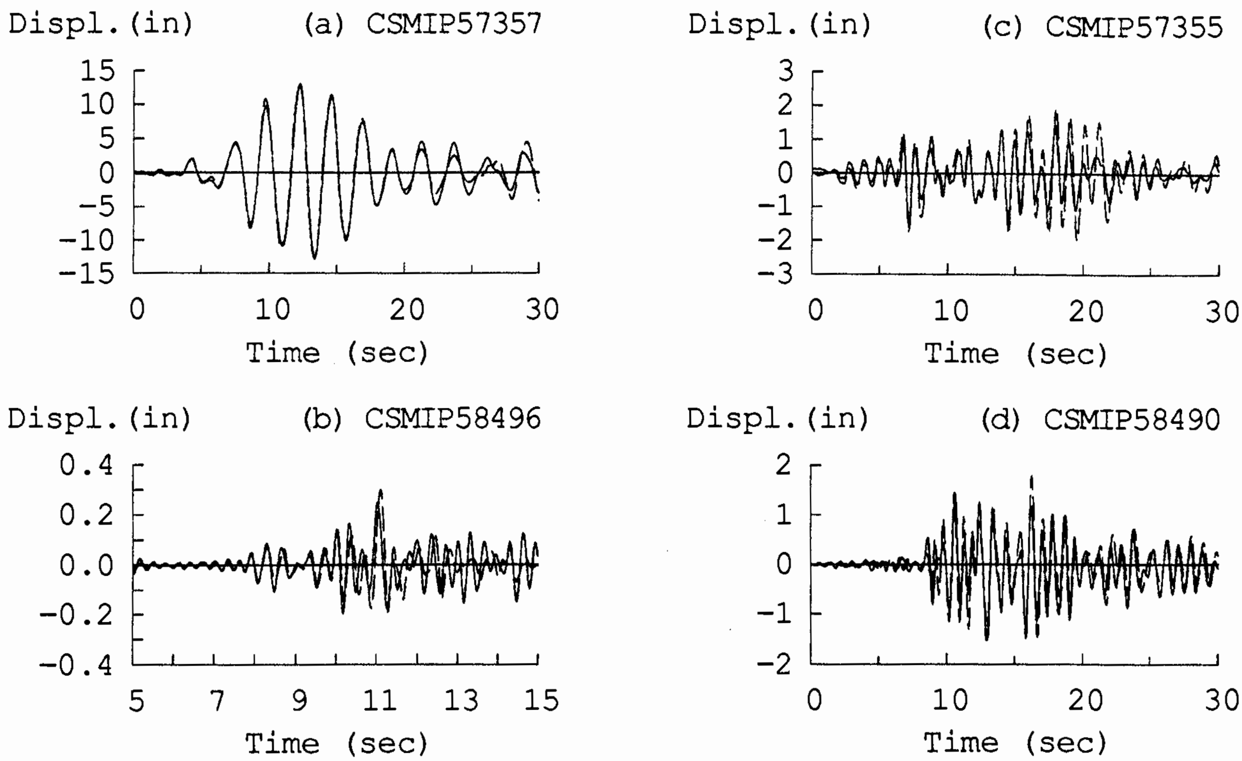

Fig. 4 Dynamic Correlation of Roof Response

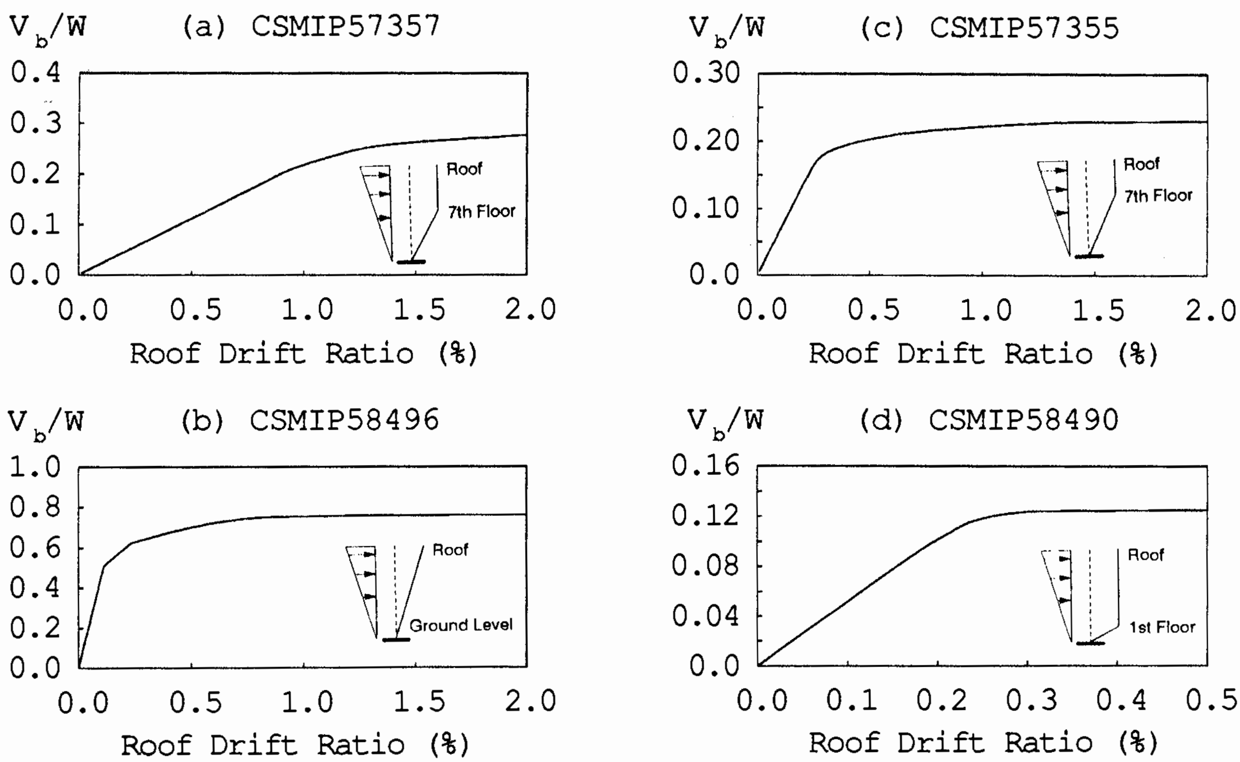

Fig. 5 Structural Lateral Strength and Collapse Mechanism

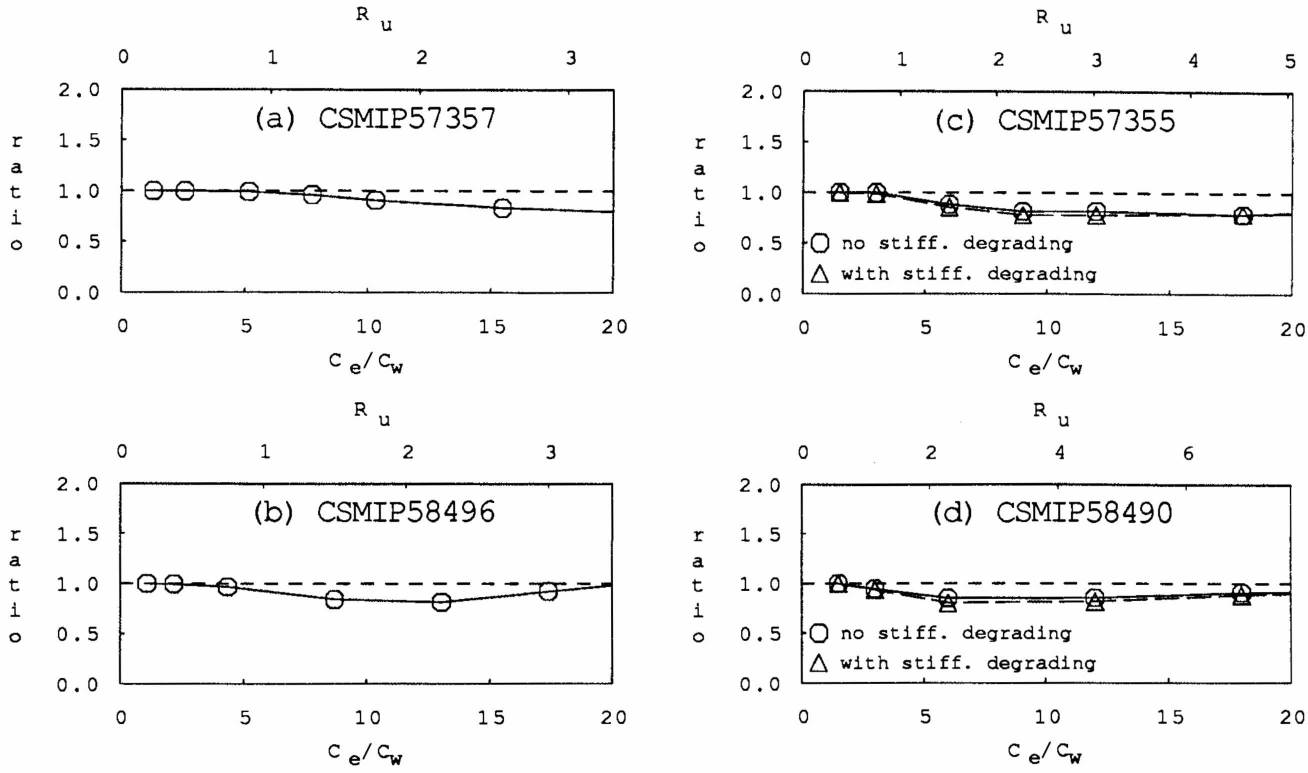

Fig. 6 Mean $\frac{\Delta_e}{\Delta_{max}}$ Ratios for Roof Drift

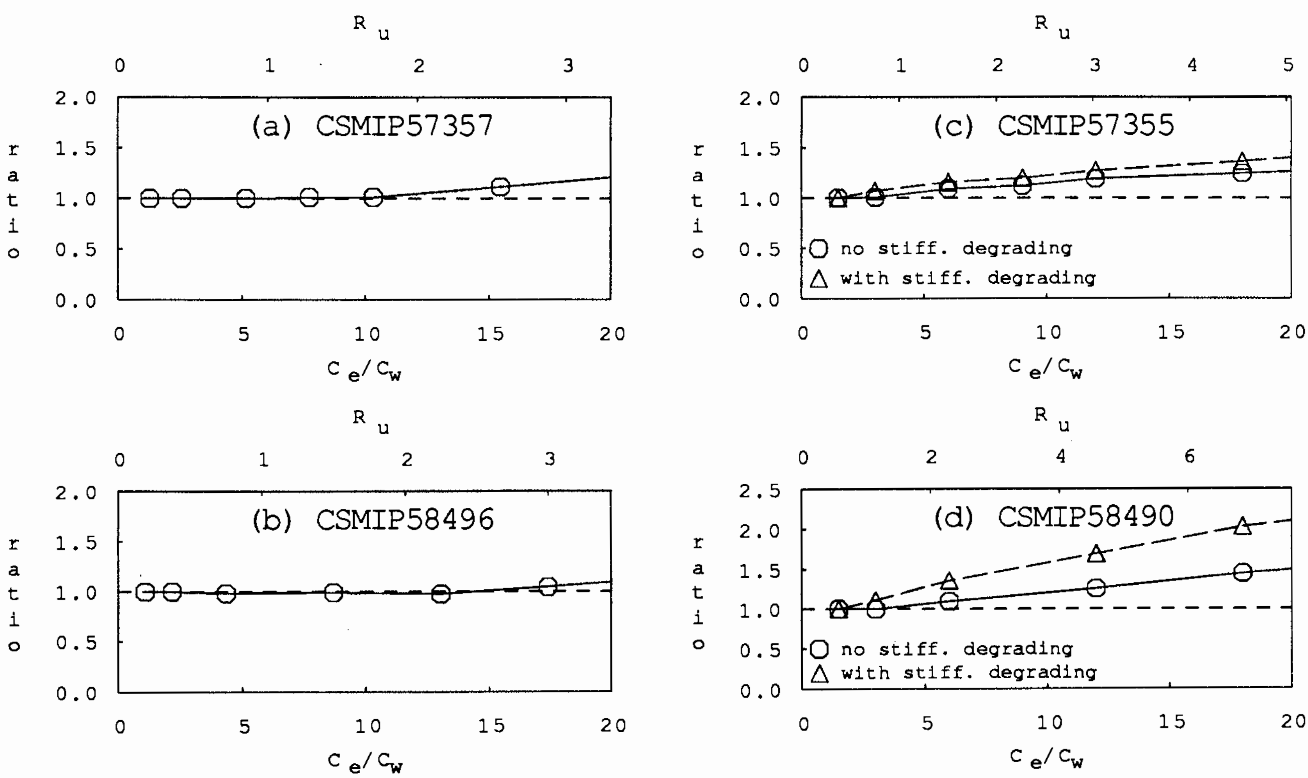

Fig. 7 Mean $\frac{\Delta_e}{\Delta_{max}}$ Ratios for Story Drift