# Deflection Amplification Factor for Seismic Design Provisions

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**Abstract:** Seismic design provisions estimate the maximum roof and story drifts occurring in major earthquakes by amplifying the drifts computed from elastic analysis at the prescribed design seismic force level with a deflection amplification factor (*DAF*). A comparison of several seismic design provisions indicated that the deflection amplification factor in both the UBC and NEHRP Recommended Provisions, being equal to $3R_w/8$ and $C_d$, respectively, is very low and could lead to unconservative drift estimations. An analytical study of the seismic responses of four instrumented buildings confirmed that drifts developed in major earthquakes are much higher than those predicted by the UBC or NEHRP approach. It is recommended that the deflection amplification factor be increased to at least the seismic force reduction factor ($R_w$ in UBC and $R$ in NEHRP) for estimating maximum drifts. The effects of the ratio between building period and earthquake predominant period, types of yield mechanisms, and structural overstrength on the *DAF* are also presented.

## INTRODUCTION

### Current Seismic Design Procedure

It is well known that modern seismic design provisions reduce design seismic forces significantly to take advantage of the structure's capacity to dissipate earthquake input energy. This concept is explained by using Fig. 1, where the typical response envelope of base shear ratio ($C$) versus lateral drift ($\Delta$) of a ductile system is shown. The force demand at point "A" represents the required level of design seismic forces if the structure were to respond elastically in a major earthquake. For strength design, the NEHRP Recommended Seismic Provisions (*NEHRP* 1991) reduces this elastic force level to the level of point "B" by a force reduction factor (*FRF*), $R$. Since the Uniform Building Code (UBC) (*Uniform* 1991) specifies a design seismic force level for working stress design, the required elastic seismic force level can be further reduced to point "C" by a force reduction factor $R_w$.

To estimate the maximum inelastic deflections $\Delta_{max}$ (see point "D" in Fig. 1) that may develop in a major earthquake, the design deflections computed from an elastic structural analysis are amplified by a deflection amplification factor (*DAF*) as follows:

$$\text{NEHRP: } \Delta_{max} = \Delta_s \times C_d \tag{1}$$

$$\text{UBC: } \Delta_{max} = \Delta_w \times \frac{3R_w}{8} \tag{2}$$

[In addition to the $C_d$ factor, the NEHRP Seismic Provisions (Chapter 10) also uses $(2/5)FRF$ as the *DAF* for estimating maximum deformations of

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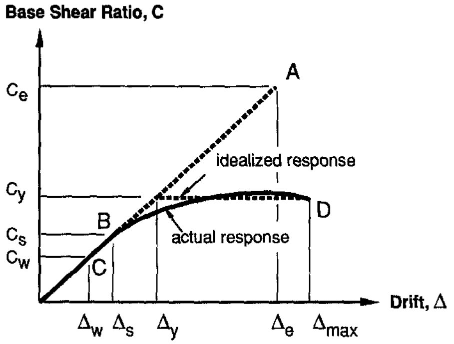


**FIG. 1. General Structural Response**

steel connections and shear links (Popov 1991). The (2/5)*FRF* in NEHRP is similar to, but slightly more conservative than, the (3/8)*FRF* in UBC.] For seismic design, it is important to estimate $\Delta_{max}$ for several reasons; these include: (1) estimating minimum building separation to avoid pounding; (2) estimating maximum story drifts; (3) checking deformation capacity of critical structural members (e.g., shear links in eccentrically braced frames); (4) checking *P*-delta effects; and (5) detailing connections for nonstructural components and so on.

## Seismic Force Reduction and Deflection Amplification Factors

If the actual response envelope in Fig. 1 can be idealized as an elasto–perfectly plastic response curve, and the following three quantities are defined as system ductility factor

$$\mu_s = \frac{\Delta_{max}}{\Delta_y} \tag{3}$$

system ductility reduction factor

$$R_\mu = \frac{C_e}{C_y} \tag{4}$$

structural overstrength factor

$$\Omega = \frac{C_y}{C_s} \tag{5}$$

then it has been shown (Uang 1991) that the force reduction and deflection amplification factors for strength design can be expressed by the following formulas:

$$FRF = R_\mu \times \Omega \tag{6}$$

$$DAF = \mu_s \times \Omega \tag{7}$$

(For working stress design, the foregoing two formulas are still valid, except that the structural overstrength factor, $\Omega$, has to be replaced by $\Omega_w$, where $\Omega_w = C_y/C_w$.)

## Ratio between *DAF* and *FRF*

From (6) and (7), the ratio between *DAF* and *FRF* is

$$\frac{DAF}{FRF} = \frac{\mu_s \times \Omega}{R_\mu \times \Omega} = \frac{\mu_s}{R_\mu} \tag{8}$$

For single-degree-freedom (SDOF) systems, the ratio between $\mu_s$ and $R_\mu$ can be expressed as (Newmark and Hall 1982).

- In the "velocity" and "displacement" amplification regions

$$\frac{\mu_s}{R_\mu} = \frac{\mu_s}{\mu_s} = 1 \tag{9}$$

- In the "acceleration" amplification region

$$\frac{\mu_s}{R_\mu} = \frac{\mu_s}{\sqrt{2\mu_s - 1}} \geq 1 \tag{10}$$

If a building frame were to respond similar to an SDOF system, substitution of (9) and (10) into (8) indicates that the *DAF* should not be less than *FRF*.

For multistory building design in accordance with the NEHRP Seismic Provisions, values of the *FRF* (i.e., $R$) and *DAF* (i.e., $C_d$) for several steel lateral-force-resisting systems are listed in Table 1; the *DAF*/*FRF* ratio is

**TABLE 1.** ***DAF*/*FRF* ($=C_d/R$) Ratios Used in NEHRP Recommended Provisions**

| Steel framing system (1) | *FRF* ($=R$) (2) | *DAF* ($=C_d$) (3) | $\frac{DAF}{FRF}\left(=\frac{C_d}{R}\right)$ (4) |
|---|---|---|---|
| Moment Resisting Frame System | | | |
| SMRF | 8 | 5.5 | 0.69 |
| OMRF | 4.5 | 4 | 0.89 |
| Dual System | | | |
| EBF + SMRF | 8 | 4 | 0.5 |
| CBF + SMRF | 6 | 5 | 0.83 |
| Building Frame System | | | |
| EBF | 8 | 4 | 0.5 |
| CBF | 5 | 4.5 | 0.9 |

Note: SMRF = special moment-resisting frame; OMRF = ordinary moment-resisting frame; EBF = eccentrically braced frame; CBF = concentrically braced frame.

**TABLE 2.** ***DAF*/*FRF* Ratios of Several Seismic Design Provisions**

| Building code (1) | *FRF* (2) | *DAF* (3) | $\frac{DAF}{FRF}$ (4) |
|---|---|---|---|
| Uniform Building Code (1991) | $R_w$ | $\frac{3R_w}{8}$ | 0.375 |
| NEHRP Seismic Provisions (1991) | $R$ | $C_d$ | 0.5–1.0 |
| National Building Code of Canada (1990) | $\frac{R}{0.6}$ | $R$ | 0.6 |
| Mexico Building Code (1987) | $Q$[a] | $Q$ | 1.0 |
| Eurocode No. 8 (1988) | $q$ | $q$ | 1.0 |

[a]Less than $Q$ in short period range.

also shown. Note that, in contrast to the SDOF system, the *DAF/FRF* ratio in NEHRP is never larger than one. Furthermore, a survey of *DAF* used in several building codes was also conducted; this survey included the UBC of the U.S.A., the 1990 National Building Code (*National* 1990) of Canada, the 1987 Mexico Code (*Seismic* 1988), and the 1988 Eurocode (*Structures* 1988). Since the magnitude of the *DAF* also depends on how much the elastic design seismic forces has been reduced, and as different seismic codes used different *FRF*, a comparison should be based on the ratio between *DAF* and *FRF*, not *DAF* alone. Table 2 indicates that the *DAF/FRF* ratios vary considerably. On one extreme, both the Mexico Code and Eurocode use a *DAF* which is not smaller than the *FRF*, that is, their approach is consistent with the SDOF systems. At the other extreme, UBC uses a *DAF* which is only three-eighths that of the *FRF*.

## OBJECTIVES AND SCOPE

Considering the significant differences of the *DAF/FRF* ratio among seismic codes, the first objective of this paper was to evaluate the appropriate *DAF* for seismic design of multistory building frames. The second objective

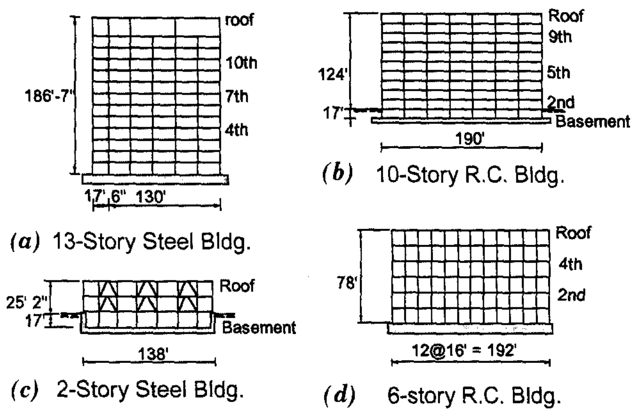


**FIG. 2. Elevations of Building Frames**

**TABLE 3. Description of Buildings**

| (1) | CSMIP Building Number 57357 (2) | 58496 (3) | 57355 (4) | 58490 (5) |
|---|---|---|---|---|
| Height | 13-story | two-story | 10-story | six-story |
| Lateral framing system | steel SMRF | steel EBF | RC SMRF | RC SMRF |
| Reactive weight (kN) | 112,100 | 20,200 | 109,000 | 28,600 |
| UBC base shear ratio, $C_w^a$ | 0.043 | 0.115 | 0.058 | 0.049 |
| UBC period (sec) | 1.8 | 0.3 | 1.1 | 0.8 |
| Measured period (sec) | 2.2 | 0.3 | 1.0 | 0.8 |
| Model period (sec) model | 2.1 | 0.3 | 1.0 | 0.8 |
| *PGA* ($g$)[b] | 8.7% | 11.7% | 8.6% | 13.7% |

[a]Based on $R_w = 12$.

[b]Measured at basement level (1989 Loma Prieta Earthquake).

was to investigate factors affecting the *DAF*. Since it is important that actual building response be investigated for this code issue, four buildings located in California and have been instrumented by the California Strong Motion Instrumentation Program (CSMIP) were selected in this study (Shakal et al. 1989).

## BUILDING DESCRIPTIONS AND MODELING

The elevations of these four buildings, which include two steel buildings (CSMIP Nos. 57357 and 58496) and two reinforced concrete buildings (CSMIP Nos. 57355 and 58490), are shown in Fig. 2. Several design data are also listed in Table 3. In addition to the UBC empirical periods, fundamental periods as identified from building responses recorded during the 1989 Loma Prieta earthquake are also listed in the table. Note that the fundamental periods of the two steel structures cover a wide range (0.3–2.2 sec). A review of the design indicated that all buildings satisfy the 1991 UBC (Uang and Maarouf 1992).

Based on the design drawings, two-dimensional mathematical models of the buildings were constructed for nonlinear dynamic analysis by DRAIN-2D computer program (Kannan and Powell 1973). To better represent the hysteresis behavior of reinforced concrete (RC) members under cyclic reversals, the Takeda model (Takeda et al. 1970) in DRAIN-2D was used for the two RC frames. The building responses recorded during the 1989 Loma Prieta earthquake by CSMIP were used to calibrate the mathematical models. (The peak ground accelerations measured at the basement level are listed in Table 3.) Good correlations between the analytically calculated dynamic responses and those obtained from the CSMIP records have been achieved (see Fig. 3). The analytically predicted results indicated that these buildings

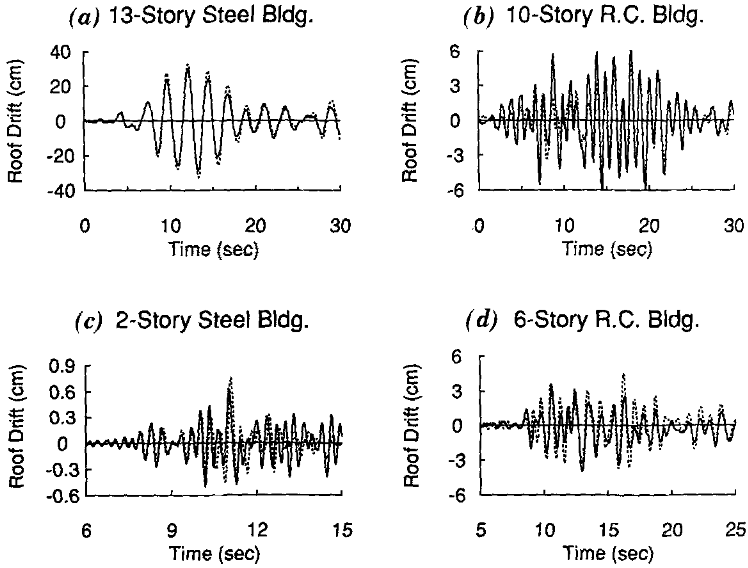


**FIG. 3. Time History Correlations of Roof Drift (—— = Analytical; ---- = CSMIP Data)**

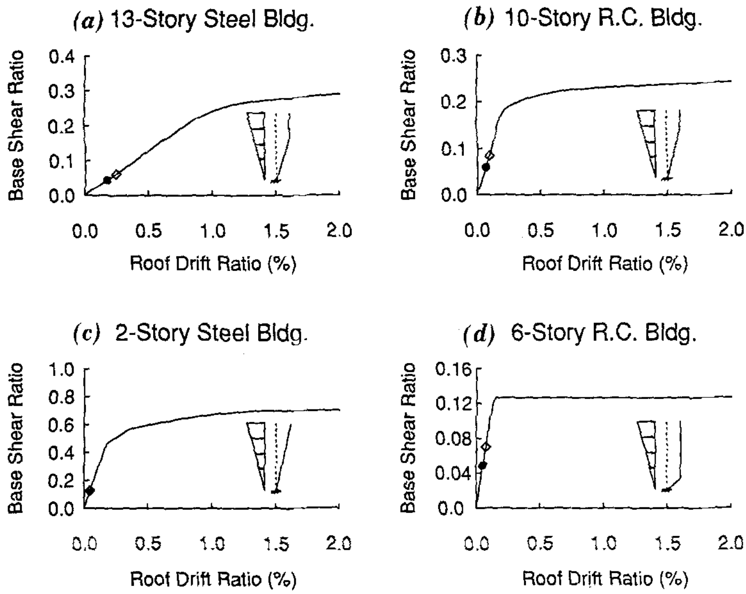


**FIG. 4. Lateral Strength of Building Frames (Levels of Design Base Shear Ratio: ● = UBC; and ◇ = NEHRP)**

responded practically in the elastic range during the 1989 Loma Prieta earthquake. The mathematical models of these frames were then used for subsequent dynamic analyses with a set of historical earthquake records as input motions.

To investigate the yield mechanisms that may develop in major earthquake and to calculate the yield base shear ratio ($C_y$) in Fig. 1, nonlinear static analyses were conducted on these frames; the UBC lateral load pattern was used for this purpose. In this paper, $C_y$ is defined as the strength level which corresponds to either the formation of a mechanism or a roof drift ratio equal to 1.5%, whichever is smaller. Fig. 4 shows the results together with the UBC and NEHRP prescribed design seismic force levels. Note that the structural overstrength varies considerably from one frame to the other. Because drift limitations govern the design of the 13-story steel moment-resisting frame, it has a significant structural overstrength. For the six-story RC building with a perimeter moment-resisting frame, since the same cross sections and similar amount of reinforcements are used for beams and columns, respectively, along the height of the frame, a weak first story is expected to develop.

## METHODOLOGY FOR EVALUATING THE *DAF/FRF* RATIO

The relationship between the *DAF* and *FRF* was investigated by conducting statistical analysis of the dynamic responses of these buildings. The procedure to compute the *DAF/FRF* ratio follows.

1. A set of eight historical earthquake records (see Table 4) was selected as input ground motions for dynamic analyses. Normalizing each record by its peak ground acceleration, *PGA*, or effective peak ground acceleration,

TABLE 4. Earthquake Ground Motion Records

| Earthquake (1) | Station (2) | Component (3) | $PGA$ ($g$) (4) | $T_g$ (s) (5) |
|---|---|---|---|---|
| Imperial Valley (1940) | El Centro | N90E | 0.35 | 0.55 |
| Washington (1949) | Olympia | S86W | 0.28 | 0.60 |
| Kern County (1952) | Taft | S69E | 0.18 | 0.44 |
| Parkfield (1966) | Cholame | N85E | 0.43 | 0.40 |
| San Fernando (1971) | Pacoima Dam | S16E | 1.17 | 0.40 |
| Imperial Valley (1979) | I.V.C. | S40E | 0.33 | 1.10 |
| Loma Prieta (1989) | Corralitos | S00E | 0.63 | 0.44 |
| Loma Prieta (1989) | Santa Cruz | N90E | 0.41 | 0.34 |

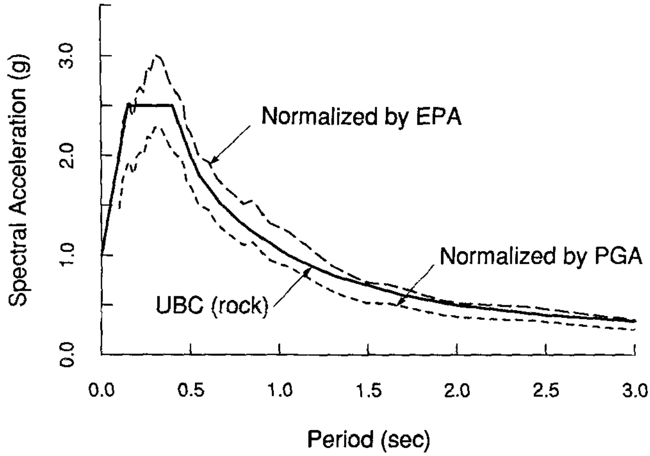


FIG. 5. Comparison of UBC and Mean Response Spectra

$EPA$, as defined in NEHRP (1991), Fig. 5 shows that the mean response spectra of the eight earthquake records are similar to the UBC design response spectrum for firm soils.

2. Each earthquake record was scaled to several intensity levels in order to produce varying degree of inelastic response in the structure. The scale factor can be defined as the ratio between the elastic base shear ratio, $C_e$, and the UBC design base shear ratio, $C_w$. (See Fig. 1.) But since Fig. 4 indicates that the ultimate strength of each frame varies considerably, another convenient definition of the scale factor is to take the ratio between $C_e$ and the yield strength, $C_y$, of the structure; this ratio can be interpreted from (4) as the ductility demand at the system level. Based on this latter definition, the $DAF/FRF$ ratio of each frame can be averaged over the whole set of earthquake records for a given value of $R_\mu$.

3. Referring to Fig. 1, it can be shown that the ratio between $DAF$ and $FRF$ is the same as the ratio between the inelastic drift ($\Delta_{max}$) and elastic drift ($\Delta_e$) because

$$\frac{DAF}{FRF} = \frac{\mu_s}{R_\mu} = \frac{\left(\frac{\Delta_{max}}{\Delta_y}\right)}{\left(\frac{C_e}{C_y}\right)} = \frac{\left(\frac{\Delta_{max}}{\Delta_y}\right)}{\left(\frac{\Delta_e}{\Delta_y}\right)} = \frac{\Delta_{max}}{\Delta_e} \tag{11}$$

Both drifts $\Delta_e$ and $\Delta_{max}$ were obtained from elastic and inelastic dynamic analyses. Rayleigh damping with 5% of critical for the first two modes were considered in the time-history analyses.

## SUMMARY OF RESULTS

The $\Delta_{max}/\Delta_e$ ratios [or *DAF/FRF* ratios from (11)] for both roof drift and story drift were computed. By averaging the results of eight earthquake records, the *DAF/FRF* ratios are summarized as follows.

### Roof Drift

Fig. 6 shows the variations of the mean *DAF/FRF* ratios for estimating maximum roof drift as a function of the earthquake scale factor, $R_\mu$; the corresponding coefficients of variation are shown in Fig. 7. Except for the two-story eccentrically braced frame (EBF), the trend of the other three buildings are very similar. Within the practical range of system ductility demand (say, $R_\mu$ = 2–5), the *DAF/FRF* ratio varies from 0.7 to 0.9. For

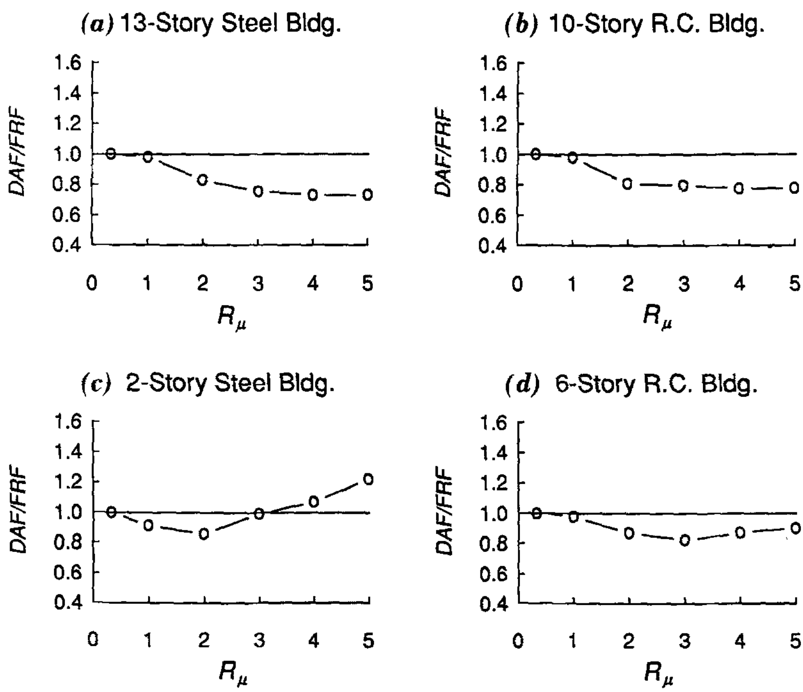


**FIG. 6. Mean *DAF/FRF* Ratios for Roof Drift**

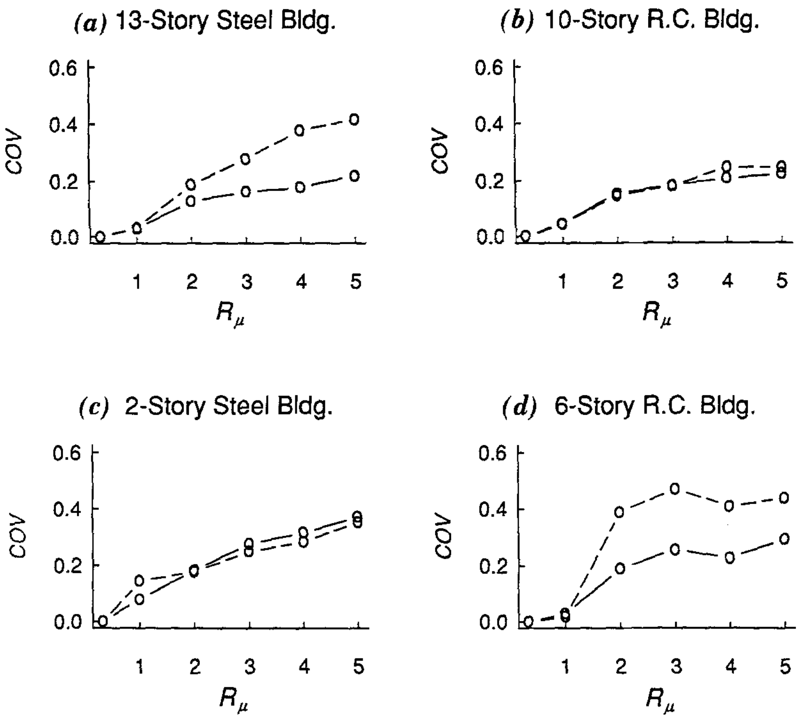


**FIG. 7. Coefficients of Variation (COV) for *DAF/FRF* Ratios (—— = Roof Drift; ---- = Story Drift)**

the two-story EBF, which has a very short period ($T$ = 0.3 sec.), Fig. 6 shows that the *DAF/FRF* ratio decreases and then increases above 1 as the intensity of the earthquake ground motion is increased.

**Story Drift**

Fig. 8 shows the variations of the mean *DAF/FRF* ratios for estimating maximum story drift as a function of the earthquake scale factor, $R_\mu$; the corresponding coefficients of variation are also shown in Fig. 7. Since structural damage in multistory frames tends to concentrate in a limited number of stories, it is expected that the *DAF/FRF* ratio will be larger than one. Fig. 8 shows that the *DAF/FRF* ratio increases with the system ductility reduction factor. Within the practical range of interest, this ratio varies from 1.0 to 1.5, significantly higher than the three-eighths as specified in the UBC. For the six-story RC frame with a weak first story, this ratio can be even higher. For this building, the maximum story drift is also sensitive to the types of earthquake ground motion; the larger coefficient of variation as observed in Fig. 7 results from the impulse type of ground motions (Cholame, Pacoima Dam and I.V.C. records in Table 4), which has been shown to be particularly damaging to certain types of structures (Anderson and Bertero 1987).

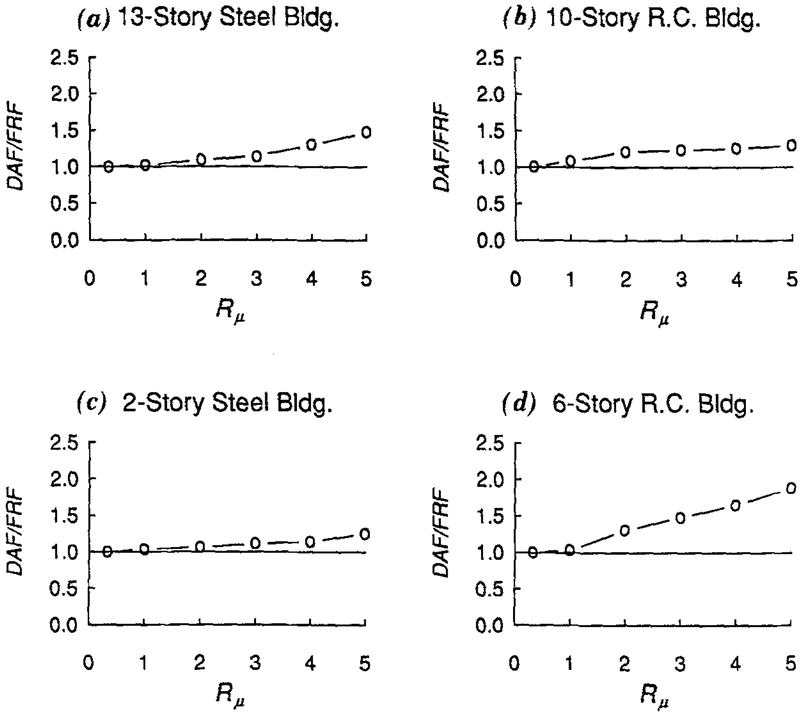


**FIG. 8. Mean *DAF/FRF* Ratios for Story Drift**

## FACTORS AFFECTING THE *DAF/FRF* RATIO

### Building Period and Earthquake Predominant Period

When a comparison of seismic responses is made between short- and long-period structures, it is important that the predominant period ($T_g$) of the ground motion also be considered. In this study, $T_g$ is calculated from the 5% damped pseudo-velocity response spectrum. By idealizing the spectrum as a bilinear curve, $T_g$ is defined as the period at which the two straight lines intersect. This period is approximately equal to the period at the intersection of constant-velocity and constant-acceleration regions of the earthquake response spectrum. Values of $T_g$ for the set of earthquake records are listed in Table 4.

To investigate how the $T/T_g$ ratio affects the *DAF* for multistory buildings, two more ground motion records with longer predominant periods were included in the analyses. These are the SCT (N00E) record of the 1985 Mexico City earthquake with a $T_g$ equal to 2.0 sec and the Point Bonita (N63W) record of the 1989 Loma Prieta earthquake with a $T_g$ equal to 1.5 sec. Among the four buildings studied, the two-story EBF has the shortest period ($T = 0.3$ sec). For estimating roof drift, it was shown in Fig. 6 that for this structure the trend of the *DAF/FRF* ratio tends to deviate from the other three frames when the ductility demand (i.e., earthquake intensity) is increased. For three levels of system ductility reduction factor ($R_\mu$), Fig. 9 shows the variations of the mean *DAF/FRF* ratio with $T/T_g$ for all the buildings. The figure indicates that the *DAF/FRF* ratio can significantly exceed one only when the $T/T_g$ ratio is smaller than a threshold value of

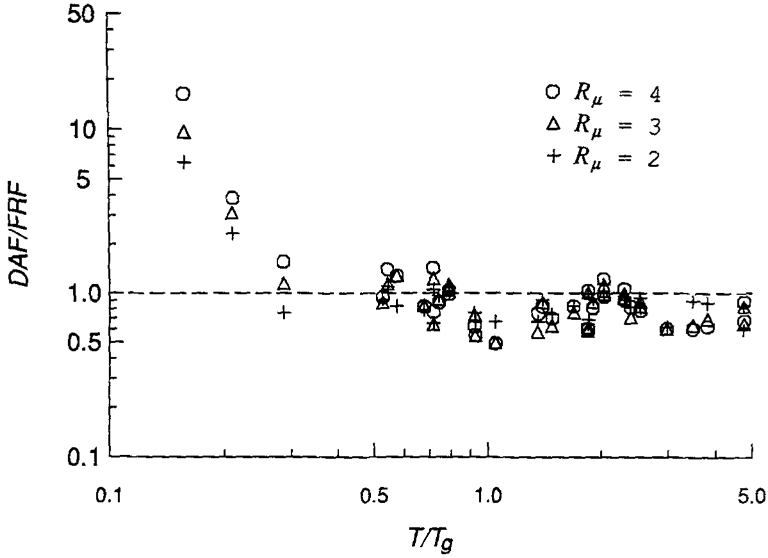


**FIG. 9.** $T/T_g$ **versus** $DAF/FRF$ **Relationship for Roof Drift**

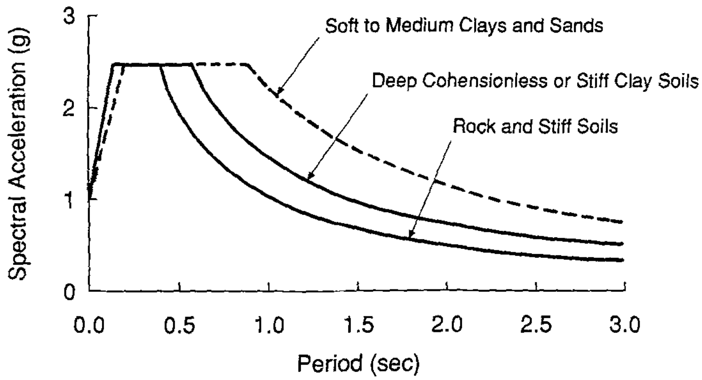


**FIG. 10. UBC Normalized Design Response Spectra (*Uniform* 1991)**

0.3. It implies that the $DAF$ can be taken as constant as long as the fundamental period of the structure is not less than $0.3T_g$. Taking the UBC design response spectra as an example, Fig. 10 shows that $T_g$ as implied by UBC is equal to 0.4 and 0.9 sec for stiff soils and soft clays, respectively. Therefore, the values of $DAF$ need to be significantly higher than those of $FRF$ only when the fundamental period of the structure is less than 0.12 sec for firm soils (soil type $S_1$) and 0.27 sec for soft clays (soil type $S_3$), respectively.

For 2% damped SDOF systems with stiffness degradation, Shimazaki and Sozen (1984) have shown that the $T/T_g$ ratio is also a key parameter to correlate the $DAF$ of SDOF systems. They observed that $\Delta_{max}$ is not less than $\Delta_e$, i.e., $\Delta_{max}/\Delta_e \geq 1$, if the following relationship is satisfied:

$$\frac{T}{T_g} + \frac{1}{R_\mu} \leq 1 \tag{12}$$

It is interesting to note that, according to (12), the $T/T_g$ ratio should be less than 0.50, 0.67, and 0.75 for $R_\mu$ equal to 2, 3, and 4, respectively. Nevertheless, this relationship was not observed in Fig. 9 for multistory frames.

**Yield Mechanism and Structural Overstrength**

Buildings CSMIP 57355 and 58490 are ductile reinforced concrete moment frames with approximately the same fundamental period. The main difference between these two buildings is the yield mechanism. The 10-story frame (CSMIP 57355) has a yield mechanism which extends from the base to the seventh floor, and the structural overstrength factor, $\Omega_w$ is equal to 4.0 (= 0.23/0.058, see Fig. 4). For the six-story perimeter frame (CSMIP 58490), the yield mechanism corresponds to the formation of a weak first story; the $\Omega_w$ is only equal to 2.7 (= 0.13/0.049). For estimating roof drift, Fig. 6 shows that the effect of the types of yield mechanism on the *DAF/FRF* ratio is insignificant. When a weak first story is combined with stiffness degradation, however, Fig. 8 indicates that the *DAF/FRF* ratio for story drift can be significantly larger than one. Therefore, yield mechanism and the associated structural overstrength play an important role for estimating maximum story drift. Unfortunately, the current seismic design procedure in the United States does not require designers to compute the ultimate strength of the structure; several drawbacks of this procedure have been pointed out recently (Uang 1993). Without knowing the yield mechanism and structural overstrength for frames with a weak story, it appears that it is difficult to develop a rational *DAF* for estimating maximum story drift.

## CONCLUSIONS AND RECOMMENDATIONS

Based on the analytically predicted dynamic responses of four actual building frames subjected to a set of eight historical earthquake records, the following conclusions on the ratio between seismic deflection amplification factor (*DAF*) and force reduction factor (*FRF*) can be made.

Within the practical range of system ductility demand ($R_\mu$ = 2–5), the *DAF/FRF* ratio for estimating roof drift ranges from 0.7 to 0.9. However, the *DAF/FRF* ratio for estimating maximum story drift can be much higher than 1.0; for a ductile frame system with stiffness degradation and a weak first story, the *DAF/FRF* ratio as high as 2.0 was observed.

Using the set of eight historical earthquake records and two additional long-period records as input motions, it was found that the *DAF/FRF* ratio is insensitive to the fundamental period of the building as long as the $T/T_g$ ratio is not less than 0.3. One flexible steel frame ($T$ = 2.2 sec) and one stiff eccentrically braced frame ($T$ = 0.3 sec) grave practically the same *DAF/FRF* ratios.

The *DAF/FRF* ratio for estimating the story drift is affected by the type of yield mechanism. For the two reinforced concrete frames studied, the six-story perimeter frame having a weak first story exhibits a higher *DAF/FRF* ratio. A much higher *DAF/FRF* ratio is needed when this type of structure is subject to impulse-type of earthquake and ground motions.

The deflection amplification factors used in UBC and NEHRP Recommended Provisions are low and, therefore, nonconservative. This study has shown that the *DAF* can be slightly less than and significantly larger than

*FRF* for estimating roof drift and story drift, respectively. Nevertheless, it is recommended that, for simplicity, a *DAF* which is equal to *FRF* be used for estimating both the maximum story drift and roof drift that may develop in major earthquakes. A rational *DAF* for building frames involving a weak story remains to be developed.

## ACKNOWLEDGMENTS

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## APPENDIX II. NOTATION

*The following symbols are used in this paper:*

$C$ = base shear ratio (= base shear/total weight of reactive masses);
$C_s$ = NEHRP design base shear ratio;
$C_w$ = UBC design base shear ratio;

$C_y$ = yield base shear ratio;
$DAF$ = deflection amplification factor;
$EPA$ = effective peak acceleration;
$FRF$ = force reduction factor;
$PGA$ = peak ground acceleration;
$R$ = response modification factor (NEHRP Recommended Provisions);
$R_w$ = system modification factor (UBC);
$R_\mu$ = structural (or system) ductility reduction factor;
$T$ = fundamental period of structure;
$T_g$ = earthquake predominant period;
$\Delta$ = lateral drift;
$\Delta_s$ = NEHRP elastic design story drift produced by seismic forces $C_sW$;
$\Delta_w$ = UBC elastic design story drift produced by seismic forces $C_wW$;
$\Delta_{max}$ = maximum (inelastic) story drift produced by earthquake ground motion;
$\mu_s$ = structural (or system) ductility factor;
$\Omega$ = structural overstrength factor ($= C_y/C_s$) for NEHRP strength design; and
$\Omega_w$ = structural overstrength factor ($= C_y/C_w$) for UBC working stress design.