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Dependency of *COD* on ground motion intensity and stiffness distribution

Mark Aschheim† and Edwin Maurer‡

Civil Engineering Department, Santa Clara University, 500 El Camino Real, Santa Clara, CA 95053, USA

JoAnn Browning‡†

Civil, Enivironmental, and Architectural Engineering Department, University of Kansas, 2150 Learned Hall, 1530 W. 15th Street, Lawrence, Kansas 66045-7609, USA

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Abstract. Large changes in stiffness associated with cracking and yielding of reinforced concrete sections may be expected to occur during the dynamic response of reinforced concrete frames to earthquake ground shaking. These changes in stiffness in stories that experience cracking might be expected to cause relatively large peak interstory drift ratios. If so, accounting for such changes would add complexity to seismic design procedures. This study evaluates changes in an index parameter to establish whether this effect is significant. The index, known as the coefficient of distortion (COD), is defined as the ratio of peak interstory drift ratio and peak roof drift ratio. The sensitivity of the COD is evaluated statistically for five- and nine-story reinforced concrete frames having either uniform story heights or a tall first story. A suite of ten ground motion records was used; this suite was scaled to five intensity levels to cause varied degrees of damage to the concrete frame elements. Ground motion intensity was found to cause relatively small changes in mean CODs; the changes were most pronounced for changes in suite scale factor from 0.5 to 1 and from 1 to 4. While these changes were statistically significant in several cases, the magnitude of the change was sufficiently small that values of COD may be suggested for use in preliminary design that are independent of shaking intensity. Consequently, design limits on interstory drift ratio may be implemented by limiting the peak roof drift in preliminary design.

Keywords: reinforced concrete frames; interstory drift ratio; seismic response; seismic design.

1. Introduction

Reinforced concrete beams and columns exhibit a change in flexural stiffness as cracking and yielding take place. Relatively complex procedures can be used for the design of reinforced concrete structures that account for the influence of cracking on interstory drift (e.g., Chan and Wang 2005). However, it is not clear that high fidelity models, capable of tracking detailed component response including cracking, yielding, stiffness degradation, and other phenomena known to affect hysteretic

† Associate Professor, Corresponding author, E-mail: maschheim@scu.edu
‡ Assistant Professor, E-mail: emaurer@scu.edu
‡† Associate Professor, E-mail: jpbrown@ku.edu.

characteristics, are needed to assess the limits of interstory drifts, and interstory drift ratios (ratio of peak interstory drift to story height), in frame structures subjected to seismic actions. This study addresses whether relatively simple models, which do not explicitly represent the development of cracking within the structure, might be adequate for proportioning a reinforced concrete frame building to limit interstory drift ratio. The adequacy of relatively simple models for proportioning reinforced concrete frame buildings to limit interstory drift ratio is addressed in two parts. In this paper, the possible dependency of peak interstory drift ratio, relative to peak roof drift ratio (ratio of peak roof drift to total building height), on ground motion (or deformation) intensity is addressed using data obtained from numerical simulations. A second paper finds that relatively simple models are adequate for estimating roof drift when the estimated roof drift is greater than about twice the yield drift (as determined in a nonlinear static pushover analysis), while at lower drift levels, high fidelity models that represent the uncracked stiffness of reinforced concrete frame members are needed (Aschheim and Browning 2007). The term drift refers to the lateral displacement of the building relative to its base.

Peak interstory drift ratio is one of several important parameters used for assessing the performance and suitability of building frames. This parameter is useful because it provides a basis for assessing deformation-induced damage to both structural and nonstructural components of a building. Peak interstory drift ratio is emphasized in performance-based earthquake engineering, and is used for establishing performance levels in various guidelines and provisions for new design as well as the rehabilitation of existing structures (e.g., FEMA-351 (2000), FEMA-356 (2000), FEMA 450-1 (2004), and Vision 2000 (1995)). Peak interstory drift ratio also is used to characterize response in evaluative approaches such as Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002).

A normalized form of the peak interstory drift ratio is given by the coefficient of distortion, COD, which is defined as the ratio of the peak interstory drift ratio to the peak roof drift ratio, occurring in any one dynamic analysis. As noted by Moehle (1992), COD values can be expected to be a function of the number of stories, the distribution of stiffness (regularity), and the distribution of strength, which relates to the type of inelastic mechanism that develops and thus depends, to some extent, on the severity of inelastic response. The COD may depend on the frequency characteristics of the excitation, with impulsive (near-fault) or harmonic excitations being distinguished from the irregular excitation characteristic of most records. CODs have been investigated for non-deteriorating frames by Medina and Krawinkler (2005) and for deteriorating steel frames by Lee and Foutch (2002). Experimentally determined drifts for reduced scale concrete frame buildings and their estimation using simple methods are described by Matamoros *et al.* (2003). Methods to proportion elements of reinforced concrete frames to limit drift demand are described by Lepage (1997) and Browning (2001). An evaluation of drift demands for concrete frames in the Central and Eastern U.S. is presented in Browning (2002). The authors are unaware of research that has systematically evaluated the effect of the reduction in stiffness associated with cracking of RC members on the COD.

The cracking of sections is typically considered independently for excursions into the positive and negative directions of moment or deformation. While sectional analysis would indicate a sudden change of stiffness upon cracking, a gradual reduction in member stiffness is generally considered to be more realistic and is represented in many load-deformation models used for reinforced concrete members (e.g., Takeda *et al.* (1970), Ruauomoko (Carr 2003), LARZ (Saiidi 1979, Lopez 1988), and OpenSEES (McKenna and Fenves 2001)). The progression of cracking throughout a multistory reinforced concrete structure thus represents a continuous modification of structural properties throughout the dynamic response. In contrast to steel frames (typically characterized by a single and symmetric stiffness in the elastic range), reinforced concrete frames experience substantially greater changes in stiffness as cracking progresses. Because CODs are a function of regularity, the changes in stiffness associated with cracking may, in effect, introduce irregularities that cause the CODs in reinforced concrete frames to vary. In particular, larger CODs might be anticipated where cracking has a dominant influence on response; that is, for relatively small drifts in the vicinity of the drifts associated with cracking. The effect on peak drifts should diminish at larger drifts, because critical sections will generally already have cracked, and even may have yielded, in the cycles preceding the cycle in which the peak interstory drift ratio occurs.

2. Methods

To investigate the effect of cracking and yielding on the COD, an analytical study was conducted on four reinforced concrete moment resistant frames. A suite of ten recorded ground motions was established. The suite was scaled to five intensity levels in order to induce different intensities of structural deformation in the building frame models. The CODs were evaluated statistically to determine if differences in the CODs at each intensity were significant. Details are provided in the following.

2.1 Building frames and high-fidelity models

Four frames were considered in this study: a regular five-story frame, a regular nine-story frame, and tall (soft) story versions of these frames. The multistory planar frames were proportioned to be representative of construction common in the Central and Eastern United States. In much of this area, gravity loads control the design, and the frames have some nominal capacity to resist lateral loads. The expected peak drift demands vary widely, from low levels of drift (at or prior to yield), to very large demands for the extreme event. It is for this range of drift demand that the effects on cracking and yielding on computed values of COD are investigated in the current study.

The displacement response of the frames was based on flexural considerations only, assuming that detailing for shear and bond demands would be adequate at the calculated levels of drift. Although there is a possibility that the mechanisms that develop in these frames may not match those expected in designs satisfying current code requirements, the potential for cracking to affect COD values at intensities of ground motion of interest in structural engineering should be discernible and would be expected to affect both classes of frames.

Each frame had three bays, with columns at 30-ft (9.1 m) on center. The regular frames have 10-ft (3.0 m) story heights, while the height of the first story is 16 ft (4.9 m) in the case of the tall (soft) story variants. Typical nominal material properties were assumed: the concrete compressive strength (f_c') was 4000 psi (27.6 MPa) and the reinforcement yield strength was 60 ksi (414 MPa). The columns for the five-story frames were 22-in. (559 mm) square, while those for the nine-story frames were 28-in. (711 mm) square for the lowest four stories and 22-in. square for the remaining top five stories. The longitudinal reinforcement ratio in the columns was 1%. The beam depths were set equal to one-twelfth the span length; beam cross sections were reinforced with 1% top steel and 0.5% bottom steel at the face of the column. The initial stiffness of the rectangular girder sections

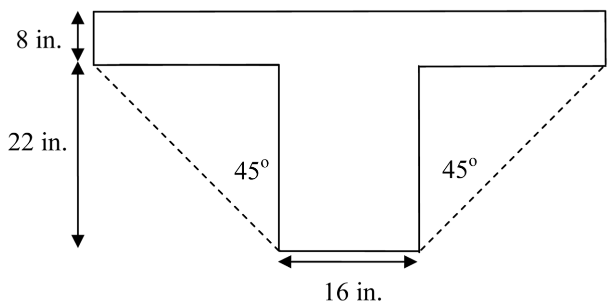

Fig. 1 Dimensions of representative T-beam elements

Table 1 Ground motion records

Event	Station	Component	Source	Duration (sec)	PGA* (g)	T_{gm} (sec)	Scaled PGA* (Moderate) (g)
San Fernando February 9, 1971	Castaic Old Ridge Route, Calif.	N21E	CALTECH (1973b)	30	0.32	0.35	0.39
Northridge January 17, 1994	Tarzana Cedar Hill Nursery, Calif.	NS	CSMIP (1994)	30	0.99	0.44	0.31
Chile March 3, 1985	Llolleo D.E.C., Chile	N10E	Saragoni *et al.* (1985)	75	0.71	0.5	0.28
Imperial Valley May 18, 1940	El Centro Irrigation District, Calif.	NS	CALTECH (1971)	45	0.35	0.55	0.25
Hyogo-Ken-Nanbu January 17, 1995	Kobe KMMO, Japan	NS	JMA (1995)	30**	0.83	0.7	0.20
Kern County July 21, 1952	Taft Lincoln School Tunnel, Calif.	N21E	CALTECH (1971)	45	0.16	0.72	0.19
Western Washington April 13, 1949	Seattle Army Base, Washington	S02W	CALTECH (1973a)	65	0.07	0.89	0.15
Miyagi-Ken-Oki June 12, 1978	Sendai Tohoku University, Japan	NS	Mori and Crouse (1981)	40	0.26	0.95	0.14
Kern County July 21, 1952	Santa Barbara Courthouse, Calif.	S48E	CALTECH (1971)	60	0.13	1.03	0.13
Tokachi-Oki May 16, 1968	Hachinohe Harbor, Japan	EW	Mori and Crouse (1981)	35	0.19	1.14	0.12

*Peak ground acceleration
**Cut from original record at 25 sec.

was doubled to account for the contribution of the slab (this is an approximation of the "effective" Tee beam, considered to be that portion contained within a 45 degree projection from the bottom corner of the girder to the lower slab face (Fig. 1)). Dead and live loads totaling 160 psf (159 N/mm^2) were assumed active on the frame during response to strong ground motion. The initial periods of the proportioned frames are listed in Table 2, and are calculated for the first mode based on uncracked gross section properties.

The beams and columns were modeled using a tri-linear moment-curvature relationship, which

Table 2 Initial periods T_i of frames based on gross section properties

Frame	Stories	Story height	T_i (sec)
A	5	10 ft. regular	0.58
B	5	10 ft. Tall First Story	0.77
C	9	10 ft. regular	0.94
D	9	10 ft. Tall First Story	1.06

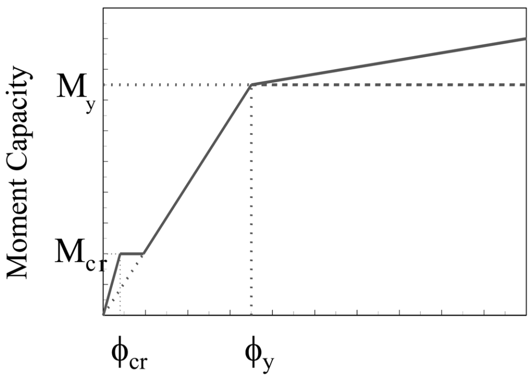

Fig. 2 Moment-curvature response represented in LARZ model

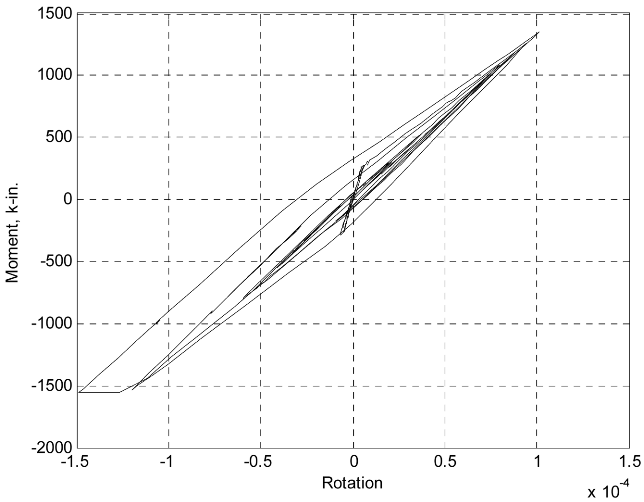

Fig. 3 Representative moment-rotation behavior determined in nonlinear dynamic analysis

represents the uncracked stiffness, cracking, and yielding portions of behavior (Fig. 2). This curve was established using the concrete stress-strain relationship defined by Hognestad (1951), with a limiting compressive strain of 0.004. Additional element rotation caused by the relative slip between the longitudinal reinforcement and concrete at the face of the joints also was included in the displacement calculations. The amount of slip rotation was calculated for developing the full yield stress in the steel with a uniform bond stress of $6\sqrt{f_c'}\,\psi$. Hysteretic response followed the rules defined by Takeda (1970) with an unloading slope coefficient of 0.4. While the moment-curvature response of column sections was established considering the dead and live loads present, the effects of overturning forces on member strength, stiffness, and deformation response was not considered in order to obtain results that are applicable to frames of varied geometries. Viscous damping equal to 2% of critical damping was used in the analyses. This relatively low level of damping was used in order to exaggerate the effects of changes in stiffness on the computed dynamic response, which are associated with hysteretic damping. Second-order (P-Δ) effects were represented. Fig. 3 illustrates a portion of the dynamic response of a member that has yielded.

Nonlinear static (pushover) analyses of the frames were conducted. The analyses were conducted using the program LARZ (Saiidi 1979, Lopez 1988). For simplicity, lateral loads were applied in an inverted triangular distribution (proportional to the product of mass and height from the base). A plot of base shear as a function of peak roof drift ratio, obtained in these analyses, is given in Fig. 5. Each of the frames has a relatively high stiffness at small drift ratios due to the uncracked stiffness. Although the frames were not designed for seismic loads, their lateral strength is not negligible, amounting to perhaps 8 to 15 percent of their weight. Noteworthy is that the yield displacements of the frames are all about 0.65 to 0.7% of their height. Somewhat smaller yield displacements (perhaps 0.5 to 0.6% of the height) would be expected for frames explicitly designed for seismic forces in regions of higher seismicity, because of the larger member depths required to limit response.

2.2 Ground motion records

A suite of ground motions was selected in relation to a representative smoothed design response spectrum. The dynamic response of the frames depends on the particular signature of each record,

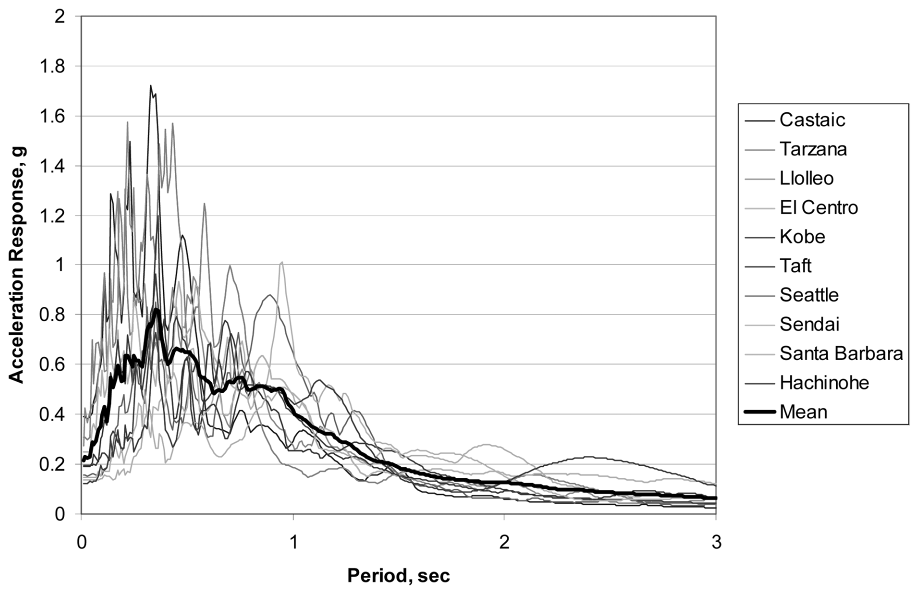

Fig. 4 Pseudo-acceleration response spectra of suite

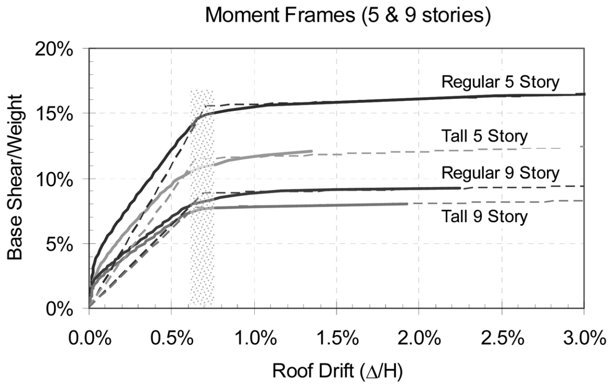

Fig. 5 Base shear versus roof drift ratios for the four frames (solid lines) and bilinear approximations (dashed lines). Yield drift ratios are in the range 0.6-0.75%

and hence is affected by the frequency content of the records as well as the dynamic properties of the frames. To limit variability in response that might be attributable to variability in ground motion, a single suite of records was established for the analyses, with different scale factors applied to obtain different intensities of response. While the dynamic properties of the frames vary as nonlinearity develops in the members, and hence vary for different scale factors, this approach was preferred over alternatives such as using suites containing unique (non-repeated) records at their naturally recorded intensities.

The ground motion suite comprises ten ground motion records that were selected and scaled so that their mean spectrum represents the design response spectrum for a stiff soil profile in Memphis, Tennessee (BSSC 2000). The motions and scale factors are listed in Table 1; their spectra are given in Fig. 4. This suite of scaled records was rescaled to five intensity levels in the analyses. Suite scale factors of 0.5, 1, 2, 3, and 4 were used. The records, scaled first by the scale factors of Table 1 and then by the suite scale factors, were applied to the same frame models. The same frames were used in order to identify a possible dependence of COD on ground motion or response intensities.

3. Results

Fig. 6 illustrates that the mean drift ratio increases monotonically with the scale factor applied to the suite of ground motions. The "mean drift ratio" represents the mean of the maximum absolute values of peak drift ratio (peak roof displacement divided by frame height) obtained for the ground motions within a suite at a given suite scale factor. The nearly linear increase in mean roof drift ratios with ground motion suite scale factor is expected.

Fig. 7 presents COD as a function of peak roof drift ratio, using one panel for each of the four frames. Symbols within each panel identify the suite scale factor. The variability in COD increases (a) for an increase from 5 to 9 stories (comparing Frames A and C, as well as B and D) and (b) due to the introduction of a tall first story in an otherwise regular frame (comparing Frames A and B, as well as C and D). Summary sample statistics are given in Table 3; mean CODs for each value of suite scale factor are given in Table 4. While the mean CODs are a function of the number of stories and regularity of the framing, a strong dependence of COD on ground motion suite scale

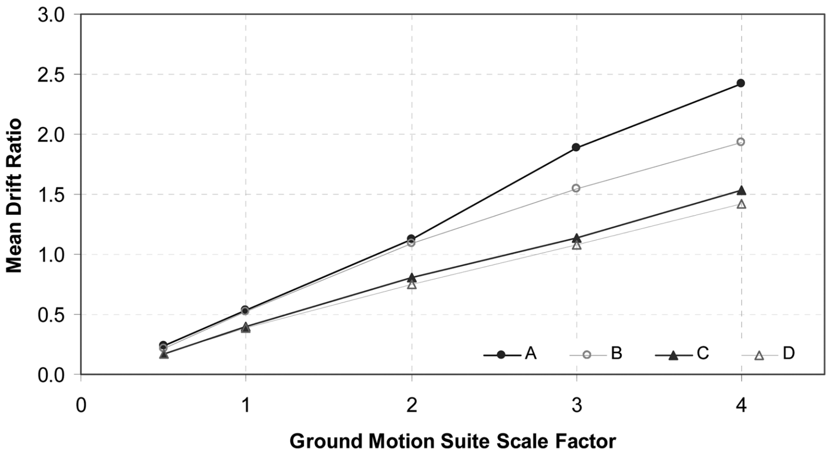

Fig. 6 Mean roof drift ratio as a function of suite scale factor

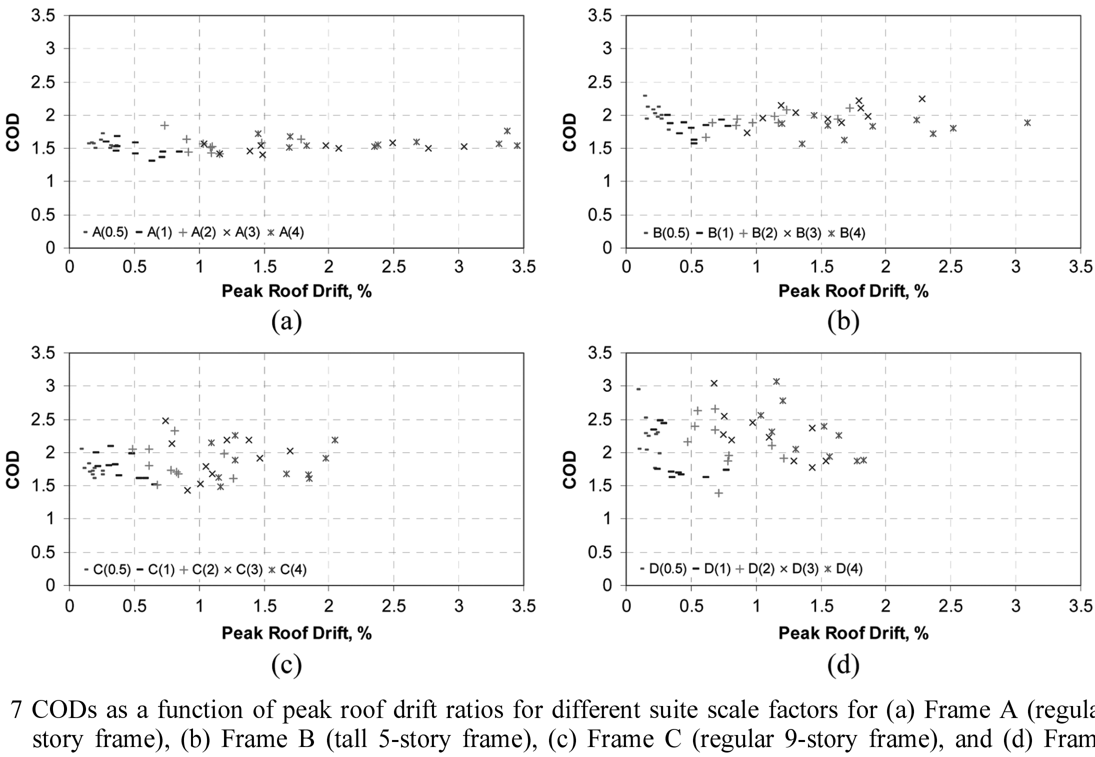

Fig. 7 CODs as a function of peak roof drift ratios for different suite scale factors for (a) Frame A (regular 5-story frame), (b) Frame B (tall 5-story frame), (c) Frame C (regular 9-story frame), and (d) Frame D (tall 9-story frame)

Table 3 Sample statistics for COD for the frames over all scale factors

Statistic	Frame A	Frame B	Frame C	Frame D
Mean	1.55	2.00	1.84	2.18
Standard deviation	0.10	0.21	0.24	0.38
Coefficient of variation	6.5%	10.3%	13.2%	17.7%

Table 4 Mean CODs for the frames as a function of suite scale factor

Suite scale factor	Frame A	Frame B	Frame C	Frame D
0.5	1.57	2.03	1.75	2.24
1	1.49	1.81	1.79	1.91
2	1.56	1.92	1.85	2.14
3	1.51	2.03	1.94	2.26
4	1.60	2.19	1.85	2.32

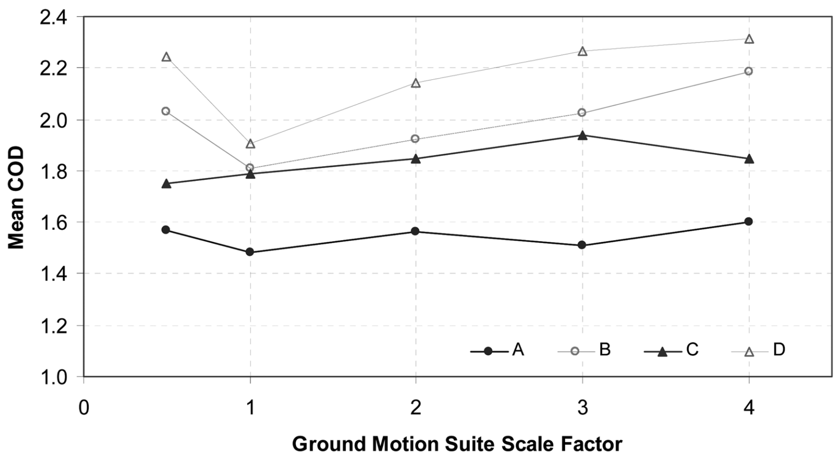

Fig. 8 Influence of ground motion suite scale factor on mean COD for each frame

factor is not evident in Fig. 7 and Table 4. Mean CODs for each frame as a function of the ground motion suite scale factor are plotted in Fig. 8. The mean CODs are observed generally to increase with an increase in suite scale factor above 1, and especially for frames with tall first stories. This increase in COD with suite scale factor may be attributable to localization of demands associated with mechanism formation. In contrast, mean CODs are observed to decrease for three of the four frames as the suite scale factor increases from 0.5 to 1. The frames subjected to the suite scaled by 0.5 had peak responses significantly below yield, with some elements remaining uncracked. The largest deformations occurred in the regular frames (five and nine story), which had maximum element ductilities close to 0.7. As the suite scale factor is increased to 1.0, the inelastic response of the frame increases, with some re-distribution of localized deformations before a mechanism forms. With larger suite scale factors, the mechanism forms and the CODs increase.

Three statistical tests were performed to determine if COD varies significantly with drift ratio or suite scale factor. Variation in the COD values as a function of peak roof drift ratio was examined in the first test. Variation in mean CODs with suite scale factor was examined using two additional tests.

The first test examines the confidence limits for the slope of a line regressed to fit the COD data as a function of peak roof drift ratio, for each of the building frames. The linear model

$$Y = \beta_0 + \beta_1 X + \varepsilon \tag{1}$$

Table 5 Regressed slope, b_1, for a line fitted to the data of Fig. 7 and 95% confidence limits for the slope

Statistic	Frame A	Frame B	Frame C	Frame D
b_0	1.531	1.856	1.781	2.175
b_1	0.0117	0.1310	0.0674	−0.0089
Lower limit for b_1	−0.0181	0.0605	−0.0559	−0.2232
Upper limit for b_1	0.0415	0.2015	0.1906	0.2054

Table 6 Regressed slope, b_1, for a line fitted to the data of Fig. 8 and 95% confidence limits for the slope

Statistic	Frame A	Frame B	Frame C	Frame D
B_0	1.520	1.855	1.758	2.037
B_1	0.0119	0.0667	0.0369	0.0657
Lower limit for b_1	−0.0324	−0.0353	−0.0109	−0.0652
Upper limit for b_1	0.0563	0.1688	0.0847	0.1966

was fitted to the COD data of Fig. 7, with X = peak roof drift ratio in percent, Y = COD, and ε = the departure of the COD from the best fit line. The estimate of the true y-intercept β_0 for each frame is given by the corresponding b_0 value in Table 5; the estimate of the true slope β_1 is given by b_1. While the frames have different y-intercepts, the estimated slopes, b_1, are near zero for the regular frames (A and C) and the tall 9-story frame (D). Confidence limits for b_1 were obtained using the t distribution for n-2 degrees of freedom, where n = 50 for each frame (e.g., see Draper and Smith, pp. 22-27). The confidence limits of Table 5 indicate that the null hypothesis (H_0: $b_1 = 0$) can be rejected at the 95% confidence level for only one of the four frames, Frame B. While there may be physically plausible grounds for COD to increase with roof drift ratio, due to localization of drift demands first as cracking forms and then as mechanisms form, variability in the computed data is such that the possibility that COD values are independent of roof drift ratio cannot be excluded at this confidence level for three of the frames.

The second test evaluates the confidence limits for the slope of a line regressed to fit the mean COD data for each frame as a function of the suite scale factor. The linear model (Eq. (1)) was fitted to the data of Fig. 8, with X = suite scale factor, Y = COD, and ε = the departure of the COD data from the best fit line. The estimate of the true slope β_1 is given by the b_1 values of Table 6. The estimated slopes are closest to zero for the regular frames. However, confidence limits for b_1, obtained using a t distribution with n-2 = 3 degrees of freedom indicates that the null hypothesis (H_0: $b_1 = 0$) cannot be rejected for all frames at this confidence level, indicating that the data does not support the notion that COD changes linearly with suite scale factor at this level of confidence.

Finally, the third test evaluates whether the mean COD for a frame subjected to the suite of ground motion records scaled by one factor differs significantly from the mean computed for the suite scaled by a different factor. In this case, a paired t-test was conducted, to consider the set of paired COD values obtained for a frame subjected to two scaled suites of ground motion records, where each pair constitutes the COD values associated with a given record scaled by different suite scale factors. Paired t tests were done for each frame and each possible pair-wise combination of suite scale factors. The p values resulting from these t tests are listed in Table 7, where p is the probability that the null hypothesis (H_0: mean CODs are equal) is rejected (because the sample data

Table 7 p values for a t-test on COD pairs associated with different suite scale factors

Frame	p_{01}	p_{02}	p_{03}	p_{04}	p_{12}	p_{13}	p_{14}	p_{23}	p_{24}	p_{34}
A	0.034	0.902	0.009	0.167	0.012	0.561	0.005	0.222	0.404	0.011
B	*0.015*	0.105	0.913	0.081	0.081	*0.017*	*0.006*	*0.027*	*0.002*	*0.017*
C	0.424	0.212	0.090	0.283	0.420	0.163	0.561	0.291	0.982	0.260
D	0.025	0.490	0.887	0.661	0.239	0.040	0.006	0.424	0.313	0.650

Values less than 0.05 are shown in italics.

are not representative) when in truth the mean CODs are equal. Evaluation of the p values at a significance level of 0.05 is equivalent to concluding that the mean CODs differ at a 95% confidence level; p values less than 0.05 provide strong evidence that the mean CODs vary with suite scale factor. The subscripts used in Table 7 refer to the suite scale factors; for example p_{13} is the p value comparing mean CODs for suite scale factors of 1 and 3; the subscript 0 is used to refer to a suite scale factor of 0.5.

Inspection of Table 7 shows that the null hypothesis cannot be rejected for Frame C at the 95% confidence level; that is, there is not strong evidence indicating the mean CODs for this frame change with suite scale factor. In contrast, for the short, regular frame (Frame A), the null hypothesis can be rejected at this confidence level for five of the ten combinations of scale factors represented in Table 7. The introduction of a tall first story increases the number of times that the null hypothesis can be rejected, from 5 to 6 for the 5-story frames (A and B) and from 0 to 3 for the 9-story frames (C and D). Mean CODs changed significantly with changes in suite scale factor from 0.5 to 1 and from 1 to 4 for three of the four frames, but more generally, increases in suite scale factor from 1 caused an increasing number of frames to have a significant change in mean COD, but this trend is absent for increases in suite scale factor from 0.5. (Specifically, the number of instances, n_{ij}, in which a p_{ij} value is less than 0.05 increases as the scale factor increases from 1, with $n_{1,2} = 1$, $n_{1,3} = 2$, $n_{1,4} = 3$, while for increases in suite scale factor from 0.5, $n_{0,1} = 3$, $n_{0,2} = 0$, $n_{0,3} = 1$, and $n_{0,4} = 0$.) The onset of significant cracking as the suite scale factor changes from 0.5 to 1 results in significant changes in mean COD values that are comparable to the development of inelasticity associated with the change in suite scale factor from 1 to 4, in all but the regular 9-story frame.

In the preceding paired t-tests, there is a probability of 5% that evidence against H_o as strong as or stronger than observed could have appeared by chance in any one hypothesis test. The problem of repeated trials, also known as the multiplicity problem, arises due to the joint evaluation of multiple, independent significance tests, each which either passes or fails a set threshold (e.g., Ang and Tang, p. 106 and Wilks, p. 151). Had the mean CODs within any one column of Table 7 been independent of suite scale factor for each frame, the probability of obtaining one or more rejections of the null hypothesis by chance in the four tests within any one column is 0.185 (given by the binomial distribution with $N = 4$, $X \geq 1$, and $p = 0.05$). Thus, it is not possible to reject the null hypothesis for changes in scale factor between 1 and 2, for which $n_{1,2} = 1$. The probability of obtaining two or more rejections of the null hypothesis by chance in four tests is 0.014 (given by the binomial distribution with $N = 4$, $X \geq 2$, and $p = 0.05$), while the probability of obtaining three or more rejections of the null hypothesis by chance is 0.0005. Thus, there is evidence to reject the null hypothesis for changes in scale factor from 0.5 to 1, from 1 to 3, from 1 to 4, and from 3 to 4 (for which $n_{0,1} = 3$, $n_{1,3} = 2$, $n_{1,4} = 3$, and $n_{3,4} = 2$).

The field significance tests show that the variation in mean CODs associated with paired responses is not likely to be due to chance, with the most significant changes in mean CODs occurring for increases in suite scale factors from 0.5 to 1 and from 1 to 3 or 4. When the frames are examined individually, this is particularly true for Frame B and absent for Frame C. This suggests that the more significant variation in mean CODs may be attributed to the changes in stiffness associated with cracking and/or yielding, particularly where soft or weak stories lead to a concentration of damage at higher intensities (e.g., Frames B and D, in contrast to Frame C). However, variation of the COD with suite scale factor could be established only for Frames B, C, and D.

4. Conclusions

During the dynamic response of a multistory reinforced concrete structure, large changes in stiffness, associated with cracking and yielding of reinforced concrete sections, may be expected to occur in a somewhat irregular pattern than varies temporally and spatially. For excitations that are large enough to induce cracking, these changes in stiffness generate irregularities that might be anticipated to result in relatively large COD values. Because cracking occurs at relatively small drift levels, the effect of cracking on peak response quantities might be diminished for very strong excitations that are associated with much larger drifts. If the COD is strongly dependent on excitation or deformation intensity, then design procedures that aim to limit peak interstory drift ratios would have to account for this dependency; alternatively, a negligible dependence of COD on shaking or deformation intensity would allow relatively simple displacement-based seismic design procedures to be used to limit interstory drift ratios of reinforced concrete moment-resistant frames.

Data obtained from the nonlinear dynamic analysis of four frames subjected to a suite of ten ground motion records scaled to five intensity levels was evaluated statistically. Mean CODs were found to change with suite scale factor by a relatively small amount. The changes in mean CODs with suite scale factor were not statistically significant for the regular 9-story frame, but were significant for the regular and irregular 5-story frames and the irregular 9-story frame. The changes in mean CODs were more pronounced for changes in suite scale factor from 0.5 to 1 and from 1 to 4; this is attributed to changes in stiffness associated with cracking and with development of inelasticity associated with yielding of reinforcement.

The increase in CODs with suite scale factor for the irregular frames is attributed to damage localization and softening associated with mechanism development. While the changes in mean CODs were statistically significant in several cases, the magnitude of the change was sufficiently small that approximate values of COD may be suggested for preliminary design that are independent of roof drift ratio or ground motion scale factor. The values to use clearly are a function of the number of stories and regularity of the structural system, and will also depend on the distribution of strength and stiffness throughout the structure.

The results were obtained for frames designed for the Central and Eastern United States, where gravity loads often control the dimensions and reinforcement of the lateral load-resisting frames. The frames were subjected to ground motions that lack the impulsive character of some near-field ground records and the harmonic character of some soft-soil records. The likelihood that critical sections may have already cracked due to gravity, wind, and prior shaking from lower intensity ground motions, which have shorter return periods, must be considered. Future studies might examine in more detail the dependence of CODs on stiffness distribution and mechanism type (by adjusting strength distributions) for a larger range of building heights, could include the effect of overturning forces on the response of column members, and could make comparisons with models in which the uncracked stiffness is not represented.

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