Effects of Pulse-Like NFGM Spectral Contents on Deflection Amplification

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LISBOA 2012

ABSTRACT:
The behavior of cantilevered column systems modeled as a single degree-of-freedom (SDF) structure having *initial* period of vibration T_0 in the range $0.6 \leq T_0 \leq 4.0$ sec subjected to a series of Near-Fault Ground Motions (NFGM) is investigated. The results have shown that when the strength of the system is gradually reduced, its collapse potential generally increases along with an elongation in the system period. In addition, because of the anomalous spectral contents of pulse-like NFGM, the mean inelastic spectral displacements can be quite large at short *initial* period, T_0. Since the mean Displacement Amplification Factor C_d in most cases is observed to be larger than the Response Modification Factor R, in order to limit the displacement or ductility requirement as well as to ensure that the damage index falls within an acceptable level, it is proposed that R = 1.25, 1.7, and 2.5 be used for systems that exhibit elastic-perfectly plastic (EPP) hysteretic behavior under "Critical", "Essential" and "Other" Importance Categories, respectively, provided that the corresponding displacement amplification is not excessive. To account for the near-fault effect, the concept of an equivalent system period T_{Eqv} in conjunction with a new period amplification factor C_T is introduced, and a procedure to determine the base shear and spectral displacement of cantilevered column systems in Seismic Design Category E (SDC-E) is proposed.

Keywords: Seismic Performance Factors; Near Fault Ground Motions; Cantilevered Column Systems

1. INTRODUCTION AND OVERVIEW

Structures assigned to Seismic Design Category (SDC) E that are located in near-fault zone require special attention because of the potential for these structures to experience high seismic forces and/or the need for them to be designed for high ductility requirement.

The present study involves investigating the behavior of a special type of seismic-force-resisting systems (SFRS) – the cantilevered column systems in which stability of mass at the top is provided by a single column with base fixity acting as a single degree-of-freedom (SDF) structure. The *initial* natural period of vibration T_0 used in the study spans the range $0.6 \leq T_0 \leq 4.0$ sec. For performance evaluation, the system is assumed to have a 5% damping, exhibit elastic-perfectly plastic (EPP) hysteretic behavior with a zero hardening stiffness ratio, and an instability (P-Δ) effect of 5%. A series of pulse-like near fault ground motions (NFGM) are used as the excitation force. In the dynamic analysis, the spectral displacements and pseudo-accelerations are evaluated for different Strength Reduction Factors, R_d = 1, 1.5, 2, 2.5, 3, 3.5, 4 over a natural period range of 0.6 sec $\leq T_n \leq$ *HUP*, where the Highest Usable Period (*HUP*) is the reciprocal of the Lowest Usable Frequency (*LUF*) as given in PEER database. Acceleration Displacement Response Spectra (ADRS) from each record are examined for various R_d at different T_0 for normalized base shear A/g, ductility μ, μ/R_d (= C_d/R) ratio, and equivalent period T_{Eqv}.

The spectral analysis follows the methodology outlined in FEMA P-695 Quantification of Building Seismic Performance Factors (2009), FEMA P-750 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (2009) and ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures (2010), except that the system performance is examined against a Damage Index DI_{BB}, proposed by Park and Ang (1985) and modified by Bozorgnia and Bertero (2003).

By varying the Strength Reduction Factor R_d, it is observed that (1) as R_d increases, the Damage Index DI_{BB} of the system also increases as a function of T_0, (2) because of the spectral characteristics of pulse-like NFGM, the inelastic spectral displacements can be quite large, especially at shorter *initial* period, T_0, and (3) in most cases, the Displacement Amplification Factor C_d is larger than the Response Modification Factor R, i.e., $C_d/R > 1$.

Since the *mean* C_d is often larger than the *mean* R , to ensure that the displacement is not excessive and the damage index falls within an acceptable value, it is proposed that EPP systems under "Critical" "Essential" and "Other" Importance Categories be designed for R = 1.25, 1.7 and 2.5, respectively, provided that the corresponding displacement amplification is acceptable. For design purpose, the concept of an equivalent system period T_{Eqv} to account for the near-fault effect is introduced, and a new ADRS is provided to facilitate the seismic design of cantilevered column systems in SDC-*E*.

2. INTRODUCTION TO SEISMIC PERFORMANCE FACTORS, *R*, C_d AND Ω_0

Seismic Performance Factors are often used to facilitate the design of structures for earthquake loading. Three such factors as defined in FEMA P-696 (2009) are illustrated in Fig. 1. The definitions of these factors are given in the figure.

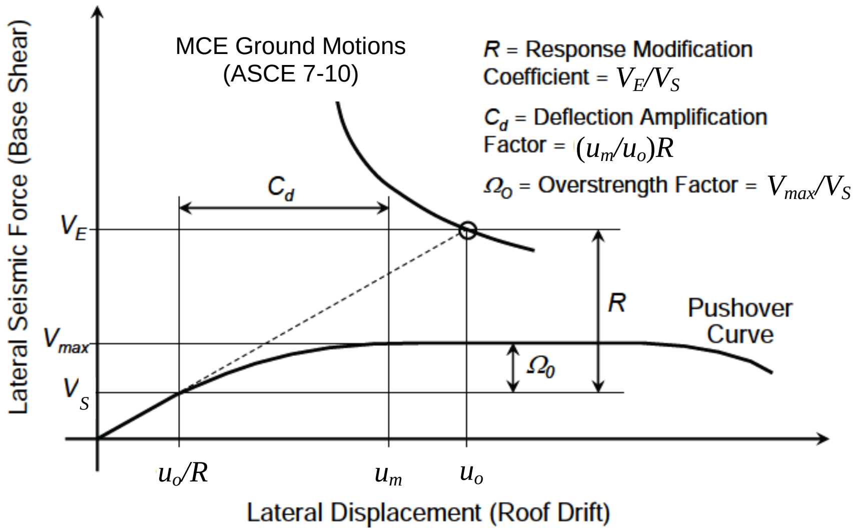

Figure 1. Seismic Performance Factors: R, C_d, and Ω_0. (Fig. 1.1 from FEMA P-695, 2009)

In the figure, the term u_o/R represents roof drift of the seismic-force-resisting system corresponding to design base shear V_S, assuming the system remains essentially elastic at this level of force. The term u_m represents the assumed roof drift of the yielded system corresponding to Maximum Considered Earthquake (MCE) ground motions. As illustrated in the figure, $C_d = (u_m/u_o)R$ (shown for $C_d < R$). However, it is not uncommon to have $C_d > R$. Another factor that is commonly used in seismic design is the Strength Reduction Factor R_d, it is related to R and Ω_0 by the equation $R_d = R/\Omega_0$.

3. CURRENT U.S. DESIGN CODE PROVISIONS

TABLE 12.2-1 of ASCE 7-10 gives Design Coefficients and Factors for cantilevered column seismic-force-resisting systems (Type *G*) as $R = C_d = 2.5$ and $\Omega_0 = 1.25$. For SDC-*E*, the limit on building height for *special* steel or reinforced concrete moment frames is 10.7 m (35 ft). The use of seismic design factors is not permitted for *intermediate* or *ordinary* (steel or RCC) moment frames.

As per AASHTO LRFD Bridge Design 2012 and Seismic Bridge Design 2011, for single column bridge-pier, the value of R is given as 1.5, 2.0 and 3.0 for "Critical", "Essential" and "Other" Importance Categories, respectively. No limitation on drift or height of column-pier is mentioned.

Appendix B of CALTRANS Seismic Design Criteria, Version 1.6 (2010) requires application of a near-fault adjustment factor [for a site-to-rupture plane distance (R_{Rup}) less than 15 km or 9.4 miles] consists of a 20% increase in spectral values for systems with period longer than one second. This increase is linearly reduced to zero at a period of 0.5 second.

4. SELECTION OF PULSE-LIKE NEAR-FAULT GROUND MOTION RECORDS

Based on Table A-6 of FEMA P-695 and the PEER database, 22 near-field pulse records were selected for the present study. They are summarized in Table 1. Since pulse-like NFGM are strong enough to cause collapse of modern structures, none of these Maximum Considered Earthquake (MCE) ground motion records are scaled (A.8 of FEMA P-695). It should be noted that only the FN and FP components of the records from Table A-6, which *necessarily* have pulse-liked contents as per PEER, are used in the present study. Record # 17 and 18 from Chi-Chi are the components in the direction of maximum velocity. From Table 1, the mean value of the velocity pulse period, $T_p \approx 3.6$ sec.

Table 1. Summary of the twenty-two NFGM records used in the present study

No	**Table-A6 ID#**	**NGA No**	**Soil Type**	**Vs30** m/s	**RECORD**	**LUF** Hz	**Mech**	***R*rup** km	***Tp*** (s) PEER
1	1	181	D	203	IMPV-HE06-233FN	0.125	SS	1.4	3.8
2	1	181	D	203	IMPV-HE06-323FP	0.125	SS	1.4	2.6
3	2	182	D	211	IMPV-HE07-233FN	0.125	SS	1.4	4.2
4	2	182	D	211	IMPV-HE07-323FP	0.125	SS	1.4	4.5
5	3	292	B	1000	IRPINIA-STU-223FN	0.16	N	10.8	3.1
6	3	292	B	1000	IRPINIA-STU-313FP	0.16	N	10.8	3.5
7	4	723	D	349	SPRST-BPTS-037FN	0.15	SS	1.0	2.3
8	5	802	C	371	LOMAP-STG-038FN	0.125	RO	8.5	4.5
9	6	821	D	275	ERZ-ERZ-032FN	0.125	SS	4.4	2.7
10	6	821	D	275	ERZ-ERZ-122FP	0.125	SS	4.4	2.2
11	7	828	C	713	CPMEND-PET-260FN	0.07	R	8.2	3.0
12	7	828	C	713	CPMEND-PET-350FP	0.07	R	8.2	1.0
13	8	879	C	685	LNDRS-LCN-239FN	0.1	SS	2.2	5.1
14	9	1063	D	282	NORTHR-RRS-032FN	0.113	R	6.5	1.2
15	9	1063	D	282	NORTHR-RRS-122FP	0.113	R	6.5	3.0
16	10	1086	C	441	NORTHR-SYL-032FN	0.12	R	5.3	3.1
17	12	1503	D	306	ChiChi-TCU65-N123E	0.075	RO	0.6	-
18	13	1529	C	714	ChiChi-TCU102-N232E	0.063	RO	1.5	-
19	14	1605	D	276	DZC-DZC-262FP	0.063	SS	6.6	5.6
20	22	825	C	514	CPMEND-CPM-350FP	0.07	R	7.0	4.9
21	25	1176	D	297	KOCAELI-YPT-180FN	0.09	SS	4.8	4.5
22	25	1176	D	297	KOCAELI-YPT-270FP	0.09	SS	4.8	4.6

5. CRITERIA FOR TARGETING SUITABLE DESIGN BASE SHEAR, *V*

Sec. 12.12.1.1 of ASCE 7-10 requires that the design story drift (Δ) shall not exceed Δ_a/ρ for any story, where in reference to ASCE 7-10 the allowable story drift Δ_a is obtained from Table 12.12-1 and ρ is the redundancy factor discussed in Sec. 12.3.4.2. For a maximum drift ratio limit of 0.02, the permissible lateral displacement limits are 12 cm and 24 cm, for columns that are 6-m and 12-m tall, respectively. Because of the very high spectral displacements, this drift ratio limitation (≈ 0.02) may not be realistically achieved for EPP systems subjected to most pulse-like NFGM records.

For optimal performance, it is recommended that the system be *targeted for minimum displacement* when determining a suitable Response Modification Factor, *R*. However, it should be noted that high

R values do not necessarily translate into low spectral displacements. Further, the ensuing cumulative effects of more repeated cycles of inelastic structural deformation over the duration of ground motion may result in a higher damage index or higher residual displacements, neither of which is desirable.

Selection of a suitable Response Modification Factor R would thus depend upon the ensuing spectral displacements, the damage indices and the residual displacements. This will be discussed in a later section in which performance is evaluated against different design parameters, and Design Base Shear is plotted against *equivalent* spectral displacements for different strength reduction factors (Fig. 7).

6. DAMAGE INDEX, DI_{BB}

Seismic design can be based on a number of criteria such as avoidance of dynamic instability, large lateral displacement, or excessive residual displacements after cessation of earthquake (Khanse and Lui 2009) or structural damage due to cumulative effects of repeated cycles of inelastic structural deformation. In the present study, the Damage Index proposed by Park and Ang (1985) DI_{PA}, as modified by Bozorgnia and Bertero (2003) DI_{BB} is adopted to predict "Irreparable Damage". DI_{BB} is calculated using the equation

$$DI_{\mathbf{BB}} = \frac{(1-\alpha)(\mu-1)}{\mu_{mon}-1} + \alpha\frac{E_H}{E_{Hmon}} \tag{6.1}$$

where α is a constant, $\mu = u_{max}/u_y$, E_H = dissipated hysteretic energy, $E_{Hmon} = F_y\, u_{mon}$, where F_y is the yield force and u_{mon} is the ultimate monotonic displacement capacity, and μ_{mon} = monotonic displacement ductility.

DI_{PA} (*DI* from Park and Ang) was calibrated against numerous experimental results and field observations in actual earthquakes. DI_{PA} less than 0.4 – 0.5 has been reported as the limit of damage that can be repaired. Experimental studies have demonstrated that failure of structural members and systems is influenced by the number of inelastic cycles of response. In the present study, DI_{BB} calculated using Eqn. 6.1 with μ_{mon}=8 and α=0.3 is used to determine if the structure has experienced "irreparable damage". Structures with $DI_{BB} \geq 0.5$ are considered unfit for repairs, which may therefore require dismantling. In Sec. 8.3, DI_{BB} are examined against *initial* period T_0 and ductility μ, and appropriate values of R factors are recommended.

7. ACCELERATION DISPLACEMENT RESPONSE SPECTRA (ADRS)

In this section, the DZC-DZC-262FP ground motion is used as an illustration on how a typical analysis is performed for each of the 22 selected NFGM records. In addition, this example is used to show that the SDF cantilevered column structure subjected to this ground motion can have a relatively high μ/R_d (= C_d/R) ratio. All analyses were carried out using the software BiSpec (www.eqsols.com).

Assuming the system under investigation has a 5% initial damping with *initial* natural period of vibration T_0 and Elastic-Perfectly Plastic (EPP) hysteretic behavior with a zero hardening stiffness and P-Δ = 5%, Acceleration Displacement Response Spectra (ADRS) are computed from a spectral analysis and plotted in Fig. 2 for Strength Reduction Factors, R_d = 1, 2, 3 and 4. In the plot, the pseudo-acceleration expressed non-dimensionally as A/g, where A=$4\pi^2 D/(T_n^2)$, g=acceleration due to gravity, is plotted against the spectral displacement D (= u_m), in cm for the range 0.6 sec $\leq T_n \leq HUP$. Note that for a SDF system $A/g = V/W$, where V is the base shear and W is the seismic weight of the system under consideration. The slope of the radial line as shown in the figure is equal to k/W, where k is the *initial* stiffness of the SDF system. This is because slope = $(A/g)/D = (V/W)/D = (V/D)/W = k/W = 4\pi^2/gT_n^2$. Therefore, when pseudo-acceleration is plotted against spectral displacement (i.e., an ADRS plot), the system period can be evaluated from the slope of a radial line as $2\pi/\sqrt{g \times slope}$.

In Table 2, the evaluation of system performance at a typical initial natural period T_0 = 1.2 sec is summarized. The first column in the table gives the R_d values under consideration. For R_d = 1 (i.e., elastic behavior), by performing a spectral analysis at T_0 = 1.2 sec, the elastic spectral displacement u_o (= u_m for an elastic system) = 26 cm is determined, from which *elastic* $V/W = A/g$ =0.7269 is obtained.

When the strength of the system having the same initial stiffness is reduced by 2 (i.e., R_d = 2), the system yields at $V/W = A/g$ = 0.7269/2 = 0.3634. A horizontal line drawn at this value of A/g will intersect the R_d = 2 curve at various points. By using the nearest intersection point, the inelastic spectral displacement is determined to be u_m = 27.6 cm, and by drawing a radial line from the origin to this intersection point, the equivalent period T_{Eqv} is calculated to be 1.78.

Using a similar approach, u_m and T_{Eqv} are calculated for other R_d values and are shown in Columns 3 and 2, respectively. For various R_d, the yield displacement u_y, ductility μ and the ratio $\mu/R_d = C_d/R$ are also calculated and tabulated in Columns 5, 6 and 7, respectively, while Columns 8 and 9 give the residual displacement *Dres* and Damage Index DI_{BB} evaluated for μ_{mon}=8 and α=0.3. The last column gives the status of the structure. It is designated as having "Irreparable Damage" when $DI_{\mathrm{BB}} \geq 0.5$.

This performance evaluation procedure is repeated for a range of initial periods $0.6 \leq T_0 \leq 4.0$ sec in increments of 0.2 sec. The results for C_d/R and the computed value for DI_{BB} for different T_0 and R_d are given in Table 3. Cells with the light yellow highlight signify $DI_{\mathrm{BB}} \geq 0.5$, and the structure is considered to have suffered "Irreparable Damages". It can be seen that "Irreparable Damages" occur at around 44% of the cases. Furthermore, it should be noted that $C_d/R > 1$ for almost 92% of the cases.

By repeating the analysis for each of the twenty-two NFGM given in Table 1, the mean and standard deviations for several design parameters are evaluated. Pertinent results will be discussed in Sec. 8. This ground motion has two *minor* pulses at T_p = 1.25 & 2.16 sec, and a *major* one at T_p = 5.6 sec. These pulses are visible as inclined peaks (from obtuse to acute angled, respectively), as seen from Fig. 2. When an elastic system yields due to a reduction in strength, the period of the resulting inelastic system elongates (Sec. 8.1). As typically observed from Fig. 2, the ensuing spectral displacements, u_m are generally higher, when $T_0 < T_{\mathrm{Eqv}} \leq T_p$. For $T_{\mathrm{Eqv}} \geq T_p$, the ensuing spectral displacements may or may not be higher than elastic spectral displacements, u_0. The pulse-effect results in *mean* value of C_d/R to be > 1, for T_0 < 3.5, approximately, as seen from Fig. 4a. It may be recalled from Sec. 4 that the *mean* value of the velocity pulse period, T_p is 3.6 sec. Notwithstanding, if the pulses are weak (i.e., lower values of Fourier amplitudes), the value of C_d/R is likely to be < 1.0.

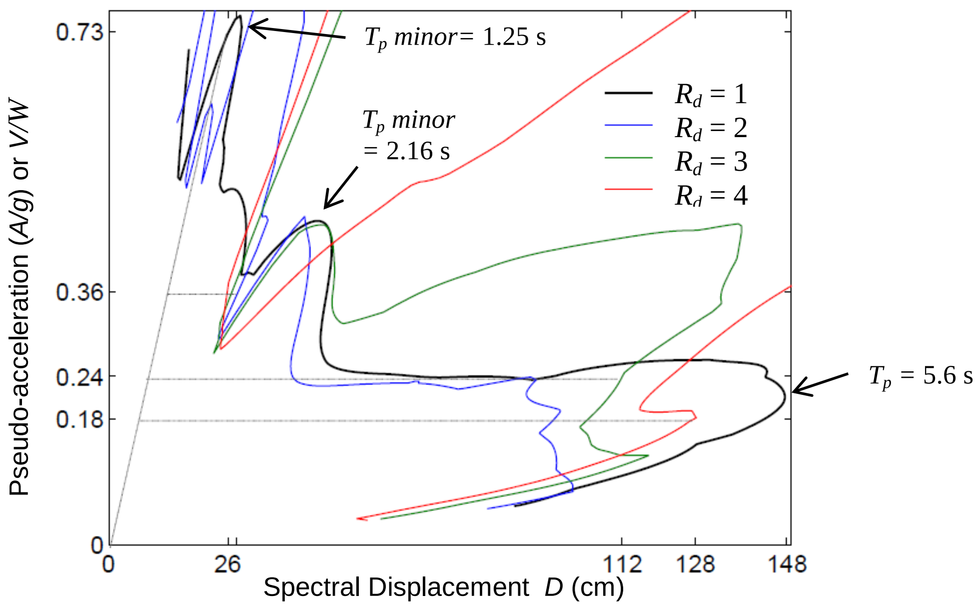

Figure 2. Acceleration Displacement Response Spectra (ADRS), EPP model, ζ = 5%, Duzce-Duzce-262FP

Table 2. Performance evaluation of a SDF system with T_0 = 1.2 sec subjected to DZC-DZC-262FP

R_d	T_{Eqv} (sec)	u_m (cm)	A/g or V/W	$u_y = u_o/R_d$ (cm)	$\mu = u_m/u_y$	$\mu/R_d = C_d/R$	*Dres* (cm)	DI_{BB} $\mu_{mon} = 8$	**Status**
1.0	1.20	26.00	0.7269	26.00	1.000	1.000	-	-	-
1.5	1.62	31.52	0.4846	17.33	1.818	1.212	4.54	0.068	ok
2.0	1.78	27.60	0.3634	13.00	2.123	1.061	10.58	0.185	ok
2.5	2.46	43.87	0.2907	10.40	4.218	1.687	9.82	0.339	ok
3.0	4.32	112.3	0.2423	8.67	12.956	4.319	37.86	0.715	Irr. Dmg.
3.5	4.67	112.5	0.2077	7.43	15.147	4.328	74.47	1.366	Irr. Dmg.
4.0	5.33	128.1	0.1817	6.50	19.708	4.927	165.84	4.462	Irr. Dmg.

Table 3. Period-dependent values of C_d/R and Damage Indices of a SDF system subjected to DZC-DZC-262FP

T_0 (sec)	C_d/R						Damage Index, DI_{BB}					
	R_d = 1.5	R_d = 2.0	R_d = 2.5	R_d = 3.0	R_d = 3.5	R_d = 4.0	R_d = 1.5	R_d = 2.0	R_d = 2.5	R_d = 3.0	R_d = 3.5	R_d = 4.0
0.6	1.711	1.809	3.230	2.801	2.543	2.487	0.098	0.271	0.571	0.986	1.651	19.282
0.8	0.927	0.777	1.906	1.522	1.333	1.295	0.073	0.142	0.182	0.276	0.373	0.467
1.0	1.770	1.577	1.237	6.377	6.380	7.244	0.125	0.217	0.416	0.789	1.371	7.246
1.2	1.212	1.061	1.687	4.319	4.328	4.927	0.068	0.185	0.339	0.715	1.366	4.462
1.4	1.021	1.614	4.226	4.180	4.920	4.519	0.144	0.390	0.799	1.325	1.869	2.180
1.6	0.806	1.364	3.418	3.504	3.926	3.585	0.103	0.264	0.500	0.809	1.018	1.086
1.8	1.188	3.156	3.213	3.580	3.438	3.118	0.102	0.210	0.315	0.479	0.640	0.800
2.0	0.518	0.912	2.220	2.288	2.536	2.320	0.060	0.177	0.290	0.380	0.458	0.498
2.2	0.766	1.926	2.031	2.224	2.251	2.036	0.052	0.143	0.252	0.357	0.467	0.639
2.4	0.795	2.026	2.170	2.353	2.110	1.886	0.052	0.135	0.236	0.361	0.550	0.884
2.6	1.760	2.091	2.307	2.179	1.916	1.706	0.053	0.136	0.248	0.420	0.772	1.496
2.8	2.600	1.984	2.259	1.960	1.704	1.520	0.059	0.146	0.271	0.570	1.112	1.994
3.0	2.315	1.781	1.995	1.729	1.514	1.329	0.062	0.153	0.350	0.666	1.231	1.958
3.2	2.048	1.582	1.762	1.523	1.330	1.180	0.064	0.167	0.373	0.708	1.210	1.677
3.4	1.830	1.423	1.575	1.357	1.183	1.048	0.071	0.183	0.381	0.738	1.111	1.489
3.6	1.631	1.269	1.405	1.210	1.055	0.934	0.072	0.191	0.427	0.707	0.996	1.275
3.8	1.465	1.140	1.262	1.087	0.948	0.839	0.070	0.196	0.406	0.615	0.843	1.063
4.0	1.328	1.037	1.142	0.989	0.854	0.754	0.078	0.209	0.373	0.556	0.725	0.915

8. PERFORMANCE EVALUATION

8.1 Period Amplification Factor, C_T

Figures 3a and 3b show the mean and mean plus one standard deviation (m+σ) values of T_{Eqv} as a function of the *initial* period T_0 for various R_d. As can be seen, T_{Eqv} increases with T_0 and R_d. This means when an elastic system yields due to a reduction in strength, the period of the resulting inelastic system elongates. Because of the near linear relationship between T_{Eqv} and T_0 observed in these figures, an equation in the form $T_{Eqv} = mT_0 + C_T$ is proposed. In the equation, the constant C_T is referred to as the "Period Amplification Factor". It is a factor to account for the period-elongation effect for inelastic systems subjected to pulse-like near-fault ground motions. Values for C_T and their correlation coefficients *CORREL* are tabulated in Table 4.

An approximate equation for C_T can be written as $C_T = 2.142\ ln(R_d) + 0.125$ and $C_T = 2.253\ ln(R_d) + 1.354$, for *mean* T_{Eqv} and (m+σ) T_{Eqv}, respectively. They are valid for $1.5 \leq R_d \leq 4$ over the range of period $0.6 \leq T_0 \leq 4.0$ sec.

From the study of more than seventy pulse-like ground motions (Khanse 2009b), it has been observed that the velocity or acceleration pulses are primarily dominant much higher Fourier amplitudes in low-frequency contents, which cause very high spectral displacements.

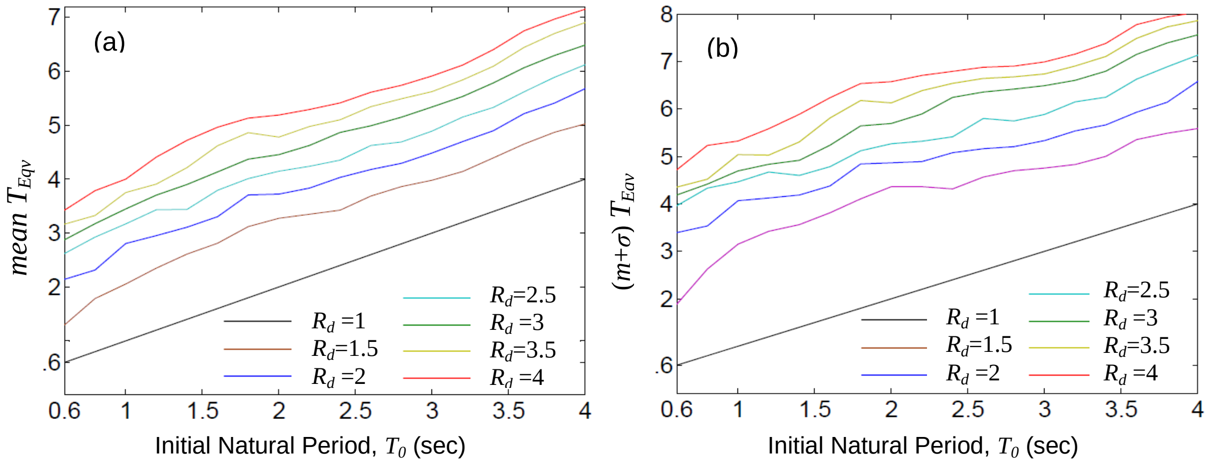

Figure 3. T_{Eqv} values of a SDF system subjected to 22 NFGM, EPP model, (a) mean values, (b) $(m+\sigma)$ values

Table 4. m and C_T values

	mean T_{Eqv}			$(m+\sigma)$ T_{Eqv}		
R_d	***m = slope***	C_T	*CORREL*	***m = slope***	C_T	*CORREL*
1.0	1.000	0.000	1.000	1.000	0.000	1.000
1.5	0.996	1.082	0.983	0.895	2.157	0.923
2.0	0.961	1.723	0.988	0.806	3.085	0.969
2.5	0.952	2.170	0.991	0.848	3.517	0.981
3.0	1.001	2.431	0.995	0.958	3.715	0.987
3.5	1.032	2.691	0.984	0.968	4.019	0.965
4.0	0.993	3.103	0.979	0.868	4.595	0.958

8.2 Deflection Amplification Factor C_d and C_d/R

Figures 4a and 4b show the *mean* and $(m+\sigma)$ values of C_d/R for the SDF system subjected to 22 NFGM, respectively. It can be seen that (1) C_d/R is period dependent, (2) in most cases $C_d > R$, and (3) for $1.5 \leq T_0 \leq 4.0$ sec, the effect for R_d on C_d/R is not significant.

At low values of T_0, it can be seen that C_d/R is quite high. Displacement magnification at short-period is acknowledged in AASHTO Seismic Bridge Design 2011 vide Sec. 4.3.3. Furthermore, in the case of systems subjected to far-fault ground motions, it is recognized in Sec. 7.7 of FEMA P-695 (2009) that "for short-period systems inelastic displacement generally exceeds elastic displacement, but it is not considered appropriate to base the deflection amplification factor on response of short-period systems, *unless* the systems are displacement sensitive. Short-period, displacement sensitive systems should incorporate the consequences of these larger inelastic displacements." SDF systems subjected to pulse-like NFGM have been shown to be very much displacement sensitive (Khanse and Lui 2010).

8.3 Damage Index DI_{BB}, Ductility μ and Residual Displacement *Dres*

Figures 5a and 5b show plots of the $(m+\sigma)$ period-dependent damage index DI_{BB} as a function of the system *initial* period T_0 and ductility μ, respectively. From Fig. 5a, it can be seen that the $(m+\sigma)$ DI_{BB} value exceeds 0.5 over the entire period range under investigation for systems with $R_d \geq 3$, and over a relatively large period range for systems with R_d = 2.5. As a result, the use of $R_d \geq 2.5$ is not recommended. From Fig. 5b, it can be observed that even when the condition of $DI_{BB} < 0.5$ is satisfied, the use of R_d = 1.5 and 2 could lead to large ductility μ requirement. These observations will be used as a guide to determine proper values of the Response Modification Factor R for use in design.

In Figure 6a, $(m+\sigma)$ values of DI_{BB} are plotted against *initial* period T_0. The thick heavier lines shown for R_d = 1.5, 2 and 2.5 represent μ values for which $DI_{BB} < 0.5$. The thin lighter lines represent conditions that are deemed *unacceptable* because $DI_{BB} \geq 0.5$.

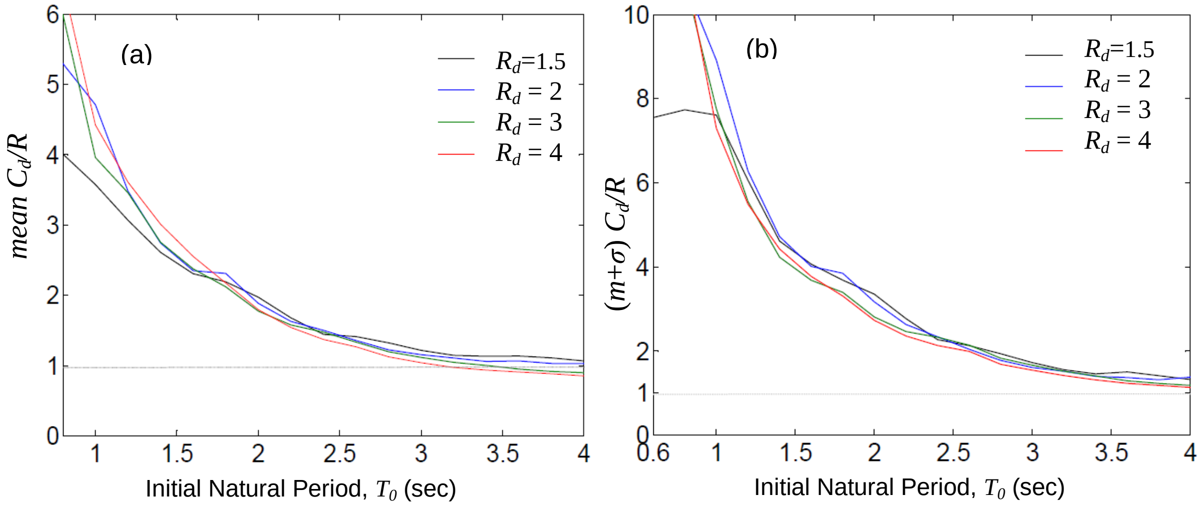

Figure 4. C_d/R values of a SDF system subjected to 22 NFGM, EPP model, (a) mean values, (b) $(m+\sigma)$ values

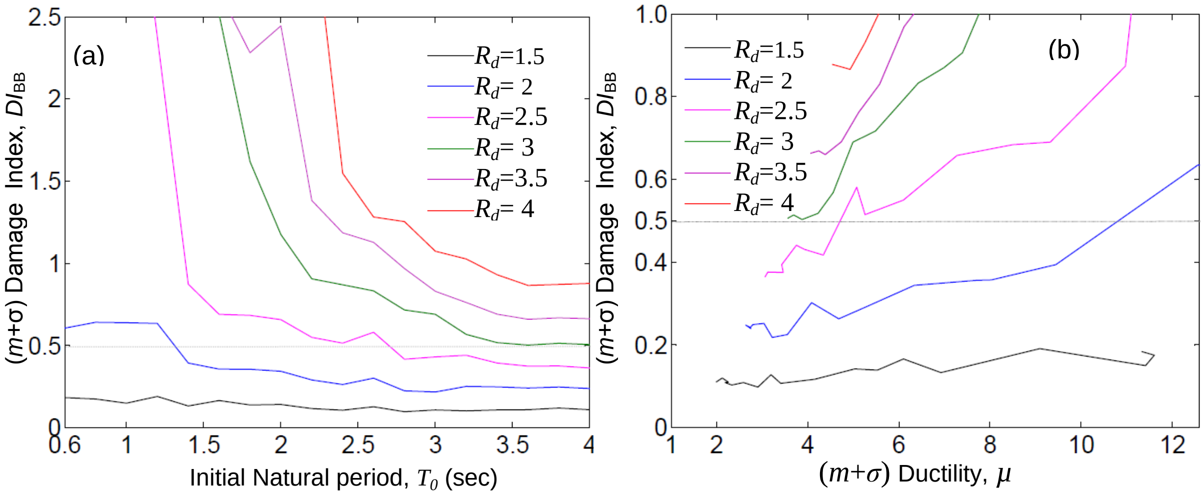

Figure 5. $(m+\sigma)$ DI_{BB} values of a SDF system subjected to 22 NFGM, EPP model,
(a) plotted against T_0, (b) plotted against μ

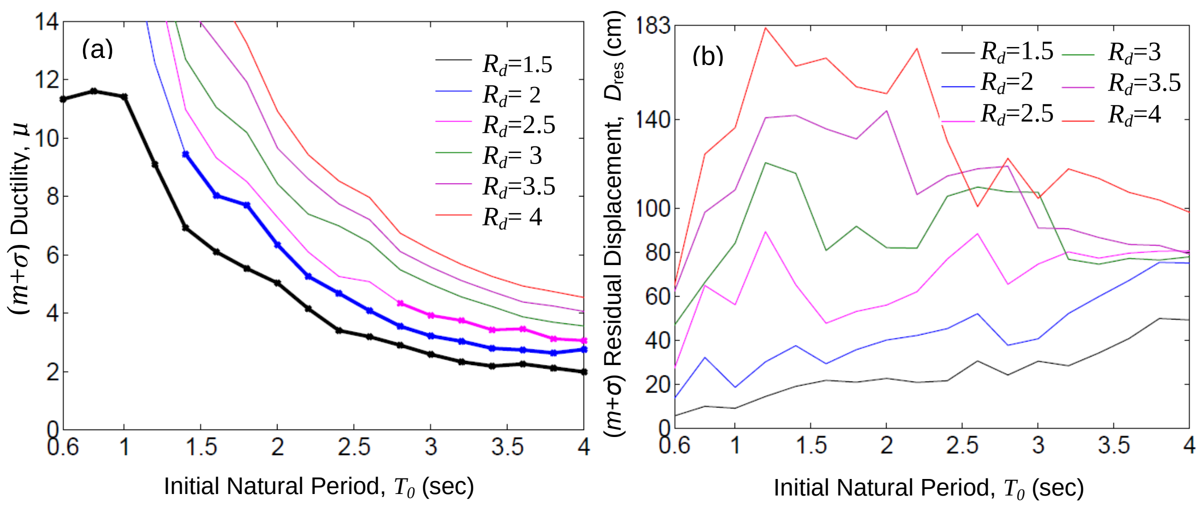

Figure 6. $(m+\sigma)$ values of a SDF system subjected to 22 NFGM, EPP model,
(a) μ plotted against T_0, (b) *Dres* plotted against T_0

Note the high ductility requirement for structures with short initial periods. In Figure 6b, (m+σ) values of Residual Displacement *Dres* are plotted against *initial* period T_0. Systems having *Dres* ≥ 60 cm (24 in.) may not be repairable. This is because *Dres* ≥ 60 cm roughly corresponds to DI_{BB} ≥ 0.5. The information presented in these figures further reinforces that the use of R_d ≥ 2.5 for design should be avoided.

8.4 Design ADRS

Based on the above analyses and observations, and using a system total overstrength factor Ω_0 = 1.25 for cantilevered column systems (ASCE/SEI 7-10), it is recommended that R = 1.25, 1.7 and 2.5 be used for the design of such systems under the "Critical", "Essential" and "Other" Importance Categories, respectively. In Figures 7a and 7b, the mean and mean plus one standard deviation (m+σ) acceleration displacement response spectra (ADRS) for three R_d values are plotted using data summarized in Table 5.

Table 5. *mean* and (m+σ) values of Design ADRS for R_d = 1, 1.5 and 2

mean values									**(m+σ) values**								
R_d = 1.0			**R_d = 1.5**			**R_d = 2.0**			**R_d = 1.0**			**R_d = 1.5**			**R_d = 2.0**		
T_0	**u_o**	**V/W**	**T_{Eqv}**	**u_m**	**V/W**	**T_{Eqv}**	**u_m**	**V/W**	**T_0**	**u_0**	**V/W**	**T_{Eqv}**	**u_m**	**V/W**	**T_{Eqv}**	**u_m**	**V/W**
0.6	8.3	0.933	1.32	26.9	0.622	2.14	53.2	0.466	0.6	12.4	1.386	1.46	49.2	0.924	2.43	101.8	0.693
0.8	13.5	0.849	1.71	41.0	0.566	2.21	51.7	0.425	0.8	20.7	1.300	1.81	70.6	0.867	2.42	94.8	0.650
1.0	17.9	0.719	2.01	48.1	0.479	2.67	63.5	0.359	1.0	28.1	1.132	2.18	89.2	0.754	2.74	105.3	0.566
1.2	22.3	0.624	2.27	53.4	0.416	2.86	63.2	0.312	1.2	33.6	0.940	2.4	89.7	0.626	2.97	103.2	0.470
1.4	27.0	0.555	2.59	61.6	0.370	3.04	63.7	0.277	1.4	39.7	0.815	2.73	100.3	0.543	3.19	103.2	0.407
1.6	31.6	0.497	2.81	64.9	0.332	3.23	64.3	0.249	1.6	46.6	0.732	2.96	106.2	0.488	3.36	102.4	0.366
1.8	36.6	0.455	3.03	69.3	0.303	3.58	72.5	0.227	1.8	54.3	0.675	3.1	107.4	0.450	3.61	109.3	0.338
2.0	42.7	0.430	3.17	71.6	0.287	3.6	69.2	0.215	2.0	62.0	0.624	3.26	109.6	0.416	3.7	106.0	0.312
2.2	48.8	0.406	3.27	71.9	0.271	3.75	70.9	0.203	2.2	69.4	0.577	3.38	109.1	0.385	3.87	107.3	0.289
2.4	53.9	0.376	3.39	71.7	0.251	4.05	76.6	0.188	2.4	74.8	0.523	3.54	108.3	0.348	4.34	122.3	0.261
2.6	57.9	0.345	3.73	79.4	0.230	4.21	75.8	0.173	2.6	79.0	0.471	3.99	124.1	0.314	4.5	118.3	0.235
2.8	61.6	0.316	3.94	81.4	0.211	4.37	75.1	0.158	2.8	83.4	0.428	4.25	128.0	0.286	4.74	119.4	0.214
3.0	65.0	0.291	4.08	80.3	0.194	4.58	75.7	0.145	3.0	88.3	0.395	4.39	126.0	0.263	4.93	119.4	0.197
3.2	67.7	0.266	4.24	79.4	0.177	4.83	77.2	0.133	3.2	94.2	0.370	4.52	125.2	0.247	5.23	125.8	0.185
3.4	69.8	0.243	4.48	80.8	0.162	5.02	76.2	0.122	3.4	99.7	0.347	4.69	126.7	0.231	5.34	122.7	0.174
3.6	71.8	0.223	4.75	83.4	0.149	5.27	77.0	0.112	3.6	106.6	0.331	4.96	134.8	0.221	5.45	122.0	0.166
3.8	73.5	0.205	4.93	82.5	0.137	5.45	75.6	0.102	3.8	112.8	0.315	5.06	133.4	0.210	5.59	122.2	0.157
4.0	75.7	0.191	5.07	81.1	0.127	5.66	75.8	0.095	4.0	119.7	0.301	5.13	131.3	0.201	5.73	122.8	0.151

A procedure for estimating the design base shear and spectral displacement for cantilevered column systems subjected to NFGM can be given as follows: (1) For a given initial period T_0 and R_d, calculate T_{Eqv} using the procedure outlined in Section 8.1, (2) use the equation T_{Eqv} = $2\pi/\sqrt{g \times slope}$ to determine the slope of a radial line to be drawn on the ADRS plot, (3) the radial line will intersect the one of the R_d=constant curves, (4) obtain the value of the base shear from the ordinate and the value of the spectral displacement from the abscissa. As an example, if the initial period T_0 of a system is 1.0 second, the *mean* Design Base Shear *A/g* (= *V/W*) is determined to be 0.72, 0.48 and 0.36, corresponding to a *mean* equivalent spectral displacement D = 18 cm, 48 cm and 63.5 cm, for R_d = 1, 1.5 and 2, respectively. Whereas the mean plus one standard deviation (m+σ) *A/g* (= *V/W*) would be 1.13, 0.75 and 0.57, corresponding to a (m+σ) D = 28 cm, 89 cm and 105 cm, for R_d = 1, 1.5 and 2, respectively.

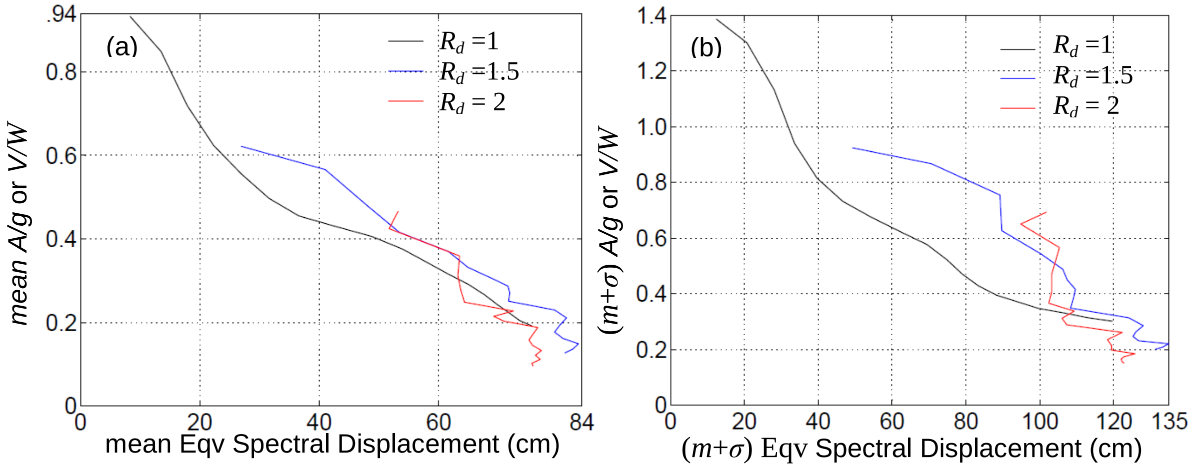

Figure 7. Design ADRS for R_d = 1, 1.5 and 2.
Radial lines define T_0 and T_{Eqv} for elastic and inelastic systems, respectively. (a) mean values, (b) (m+σ) values.

9. CONCLUSIONS

Pending further studies that involve the use of other hysteretic models and additional NFGM earthquake records, it is proposed that cantilevered column systems that exhibit elastic-perfectly plastic hysteretic behavior under "Critical", "Essential", and "Other" Importance Categories be designed using R = 1.25, 1.7 and 2.5, respectively, provided that the *resulting displacement amplification is acceptable*. Thus, to account for near-fault effect the values of R are proposed to be reduced by approximately 17% from those recommended by AASHTO LRFD Bridge Design 2012.

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