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DEFORMATION LIMITS FOR SIMPLE NON-DEGRADING SYSTEMS SUBJECTED TO NEAR-FAULT GROUND MOTIONS

Sinan AKKAR[1], Ufuk YAZGAN[1] and Polat GÜLKAN[1]

SUMMARY

In essence, the accuracy in maximum inelastic deformation estimates is the main concern for simplified nonlinear analysis procedures. The issue is more important for near-fault ground motions due to their high deformation demands on structures. The complex seismological features of these ground motions need to be mapped correctly on the proposed procedure to capture the most realistic demand variation on structural systems. This paper presents the results of a statistical study that describes the relation between the maximum inelastic and elastic deformation demands of near-fault ground motions on structural systems that do not experience strength deterioration. A total number of 145 near-fault ground motions recorded on firm-to-dense soil sites are used to investigate the variation of maximum inelastic to elastic deformation ratios for single-degree-of-freedom systems. The near-fault ground motion data set is evaluated in detail to distinguish records that contain pulse-like signal effects. The median variation of maximum inelastic to elastic deformation ratios is described separately for near-fault records that either do or do not contain pulse signals. The observations from these computations indicate that the existing pulse signal affects the inelastic deformation demand significantly and a categorization in near-fault ground motions in terms of signals with- and without-pulse is necessary for an accurate estimation of maximum inelastic deformation demand. The median error statistics for FEMA-356 nonlinear analysis procedure are presented that lacks such a categorization in near-fault deformation demand.

INTRODUCTION

The premise in simplified nonlinear analysis procedures is to describe the rigorous inelastic structural behavior via realistic approximations that in turn yield the most accurate deformation estimation for a given seismic hazard level. Of the various seismic design guidelines, the ATC-40 [1] and FEMA-356 [2] documents propose alternative simplified nonlinear analysis procedures for seismic rehabilitation of existing buildings in seismic zones. The ATC-40 document presents an equivalent linear method (Capacity Spectrum Method) that idealizes the nonlinear structural behavior by making use of a set of linear systems whose dynamic properties (i.e. lateral stiffness and structural damping) are different than those of the actual building system. Analytical expressions in the ATC-40 procedure are functions of displacement ductility ratio (maximum inelastic to yield displacement ratio) that constitutes the

[1] Department of Civil Engineering and Disaster Management Center, Middle East Technical University 06531 Ankara, Turkey.

fundamental variable in the theoretical development of equivalent linear methods. The implicit displacement ductility capacity in the existing buildings requires this procedure to be implemented in an iterative manner. The procedure proposed in FEMA-356 (Displacement Coefficient Method) approximates the nonlinear structural behavior by modifying the elastic dynamic response through some coefficients that account for the modifications in the actual structure when its response is beyond the yield strength capacity. FEMA-356 nonlinear procedure uses the normalized yield strength capacity of the actual system with respect to its elastic strength demand that is called as the strength reduction factor, R. This procedure does not require iteration for the prediction of lateral deformation capacity as the yield strength of existing buildings is known explicitly under a seismic hazard level.

The simplification *a priori* in these procedures is the representation of multi-degree-of-freedom (MDOF) building system by an equivalent single-degree-of-freedom (SDOF) system through pushover analysis. Given the pre-defined seismic hazard level, the dynamic properties of the equivalent SDOF system are implemented to these procedures for estimating the pertinent maximum inelastic SDOF displacement demand. Essentially, the approximate maximum inelastic displacement is related to the lateral deformation profile of the MDOF building system by using the proper modal participation factor. Thus, a major task for a well established simplified nonlinear analysis procedure is to set a reliable relationship between the elastic and inelastic maximum displacement demand on SDOF systems.

The performance of ATC-40 and FEMA-356 simplified nonlinear procedures has been evaluated by various researchers to explore their accuracy under certain cases. Chopra [3] showed the convergence problems in the ATC-40 equivalent linear method by using elastic-plastic (EPP) SDOF systems. Chopra and Goel also observed that even if the convergence problem is not experienced, the ATC-40 procedure may yield deceptive deformation estimates for near-fault (NF) ground motions by showing particular results from such records that exhibit either a pulse or not. Using different hysteretic models, Miranda and Akkar [4] compared the exact maximum SDOF inelastic displacements computed from the response history analyses with the estimations derived from the ATC-40 and FEMA-356 procedures. The mean error statistics presented by Miranda and Akkar were based on a total of 100 ground motions recorded on different sites and indicated that on average the approximate displacements of ATC-40 and FEMA-356 depart significantly from the exact response history results with a tendency either towards conservative or non-conservative side. Confined to a limited number of NF ground motions, they also pointed that the approximations for inelastic deformation demands of such records deserve special attention due to their complex seismological features. The studies conducted by MacRae [5, 6] urged the necessity of improving the ATC-40 and FEMA-356 procedures for estimating the SDOF inelastic response when buildings are subjected to NF ground motions. Using target displacement ductility ratios of 2, 4 and 6, and EPP hysteretic model they concluded that the inelastic displacement demand of SDOF systems subjected to NF ground motions with pulse are significantly large for medium and long period structures (i.e. T=1.0-3.0s) . Based on these studies, they also highlighted that the short period SDOF inelastic displacement was not affected significantly by the NF excitations with pulse signals. This observation is contradictory to those of Iwan [7] that pointed the importance of NF pulse signal period (T_p) in nonlinear structural response especially for short period structures with fundamental periods less than the pulse period.

This study highlights the effects of some essential structural and strong ground motion properties on the nonlinear displacement demands of SDOF systems subjected to NF ground motions. The nonlinear behavior is described by the bilinear and modified Takeda [8] hysteretic models that do not show strength deterioration on the maximum displacement response. These models are shown in Figure 1 and defined as non-degrading cyclic behavior as they display stable hysteresis loops and lack the strength deterioration during a severe ground shaking. The yield strength is used to define the structural capacity as it is the only capacity parameter known *priori* for existing structures when subjected to a given seismic hazard. A total of 145 NF ground motions recorded on firm-to-dense soil sites are used to display the general statistics of inelastic displacement demand variation as a function of structural period, pulse period, lateral strength capacity and hysteretic model. The statistics are derived from a total of 153990 response history analyses and they are presented separately for NF records with and without pulse signals for a more precise understanding of pulse effects on inelastic displacement demands. The results depicted from the nonlinear response history analyses are used to evaluate the accuracy of relevant modification factors in the FEMA-356 procedure for NF ground motions.

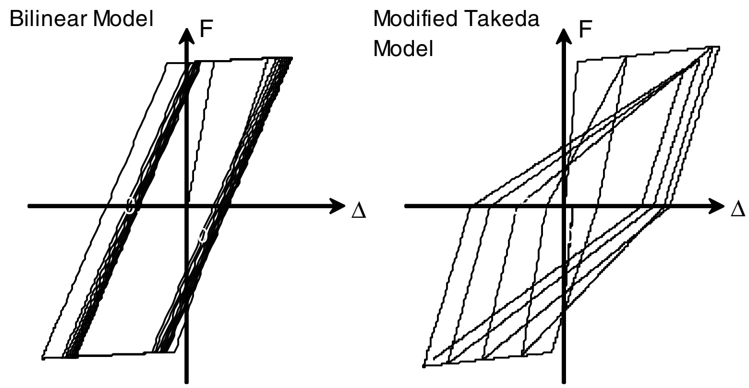

Figure 1. The non-degrading hysteretic models used in the study

NEAR-FAULT GROUND MOTION RECORDS

A collection of 145 NF ground motions that consists of 56 records with pulse signal and 89 records without pulse signal is used for the presented study. Some important seismological features of this comprehensive NF data set are listed in Tables 1 and 2 for records with and without pulse signals, respectively. The assembly comprises ground motion records of NEHRP C and D site classes that can be described as firm-to-dense soil sites with average shear wave velocities of 760m/s < v_s < 360m/s and 360m/s < v_s < 180m/s, respectively. The site class C records are relatively less than the records from site class D and they are shaded by gray color in the tables. The moment magnitude (M_w) range covered by the data set is mostly between 6.5 and 7.6 whereas the shortest distance between the fault rupture and recording station is less than 20 km for all records. The pulse periods (T_p) presented in Table 1 are the durations of maximum amplitude pulse cycles obtained from the velocity time history traces. A distinctive characteristic of the ground motion data set is the high peak ground velocity (PGV) values that are above 30cm/s for records either with or without pulse signals. The seismological properties revealed by these records suggest that the chosen assembly represents a ground motion data set of significantly high seismic hazard level; an eminent property of NF ground motions.

Table 1. Important features of NF records with pulse signals

Record	M_w	d (km)	PGA (g)	PGV (cm/s)	T_P (s)	Record	M_w	d (km)	PGA (g)	PGV (cm/s)	T_P (s)
Chi-Chi 1999,TCU087,NS	7.6	3.2	0.12	37	4.4	Northrdg.1994,Newh-W.Pico.,046	6.7	7.1	0.45	93	3.2
Chi-Chi 1999,TCU087,EW	7.6	3.2	0.13	41	9.5	Northrdg.1994,Newh-W. Pico.,316	6.7	7.1	0.33	67	1.9
Chi-Chi 1999,CHY080,NS	7.6	7.0	0.90	102	1.4	Northrdg.1994,Slymar Converter,052	6.7	6.2	0.61	117	2.5
Chi-Chi 1999,CHY080,EW	7.6	7.0	0.97	108	0.9	Northrdg.1994,Slymar Hospital,360	6.7	6.4	0.84	130	1.9
Chi-Chi 1999,NSY,EW	7.6	9.7	0.14	48	8.1	Northrdg.1994,Sepulveda VA,270	6.7	8.9	0.75	85	0.8
Chi-Chi 1999,NSY,NS	7.6	9.7	0.13	42	3.7	Northrdg.1994,EW. Lost Canyon,270	6.7	13.0	0.48	45	0.7
Chi-Chi 1999,TCU128,NS	7.6	9.7	0.17	69	4.0	Cape Mend.1992,Rio Del OVP,270	7.1	18.5	0.39	44	1.2
Chi-Chi 1999,TCU128,EW	7.6	9.7	0.14	73	7.8	Cape Mend.1992,Petrolia,090	7.1	9.5	0.66	90	0.7
Chi-Chi 1999,TCU057,NS	7.6	12.6	0.09	43	6.6	Loma Pr.1989,Corralitos,090	6.9	5.1	0.48	45	0.7

Chi-Chi 1999,TCU104,NS	7.6	13.6	0.09	47	6.4	Loma Pr.1989,Saratoga Aloha,090	6.9	13.0	0.32	43	3.1
Chi-Chi 1999,TCU068,NS	7.6	1.1	0.46	263	10.9	Loma Pr.1989,Gilroy #2,090	6.9	12.7	0.32	39	1.4
Chi-Chi 1999,TCU102,NS	7.6	1.8	0.17	77	2.8	Loma Pr.1989,Gilroy #3,090	6.9	14.4	0.37	45	2.1
Chi-Chi 1999,TCU101,EW	7.6	2.9	0.20	68	7.6	Superstition Hills 1987,PTS,225	6.6	0.7	0.45	112	2.2
Chi-Chi 1999,TCU103,EW	7.6	4.0	0.13	62	7.9	N.Palm Spr.1986,N.P.Spr.PO,210	6.0	8.2	0.59	73	1.4
Chi-Chi 1999,TCU053,NS	7.6	6.7	0.14	41	6.6	Imp.Val.1979,EC Overp FF,270	6.5	0.5	0.30	91	3.1
Chi-Chi 1999,CHY028,NS	7.6	7.3	0.82	67	0.8	Imp.Val.1979,El Centro #7,230	6.5	0.6	0.46	109	3.8
Chi-Chi 1999,TCU060,EW	7.6	9.5	0.20	36	8.4	Imp.Val.1979,El Centro #5,230	6.5	1.0	0.38	91	3.9
Chi-Chi 1999,TCU063,NS	7.6	10.4	0.13	73	5.2	Imp.Val.1979,El Centro #6,140	6.5	1.0	0.41	65	2.9
Chi-Chi 1999,CHY101,NS	7.6	11.1	0.44	115	4.7	Imp.Val.1979,El Centro #6,230	6.5	1.0	0.44	110	3.9
Chi-Chi 1999,CHY101,EW	7.6	11.1	0.35	71	3.4	Imp.Val.1979,El Centro #8,230	6.5	3.8	0.45	49	4.0
Chi-Chi 1999,WGK,EW	7.6	11.1	0.33	69	3.3	Imp.Val.1979,El Centro #4,230	6.5	4.2	0.36	77	4.3
Chi-Chi 1999,WGK,NS	7.6	11.1	0.48	74	4.5	Imp.Val.1979,El Centro DA,270	6.5	5.3	0.35	71	4.5
Chi-Chi 1999,TCU064,NS	7.6	15.1	0.12	54	7.2	Imp.Val.1979,Holtville PO,225	6.5	7.5	0.25	49	2.4
Chi-Chi 1999,TCU036,EW	7.6	16.7	0.14	60	5.8	Imp.Val.1979,Holtville PO,315	6.5	7.5	0.22	50	3.7
Chi-Chi 1999,TCU059,EW	7.6	17.8	0.17	59	6.0	Imp.Val.1979,EC Center FF,092	6.5	7.6	0.23	69	3.3
Chi-Chi 1999,CHY035,EW	7.6	18.1	0.25	46	1.3	Imp.Val.1979,Brawley Airport,225	6.5	8.5	0.16	36	3.4
Kobe 1995,KJM,090	6.9	0.6	0.60	74	1.4	Imp.Val.1979,El Centro #10,050	6.5	8.6	0.17	48	2.0
Kobe 1995,KJM,000	6.9	0.6	0.82	81	0.9	Imp.Val.1979,El Centro #10,320	6.5	8.6	0.22	41	3.9

Table 2. Important features of NF records without pulse signals

Record	M_w	d (km)	PGA (g)	PGV (cm/s)	Record	M_w	d (km)	PGA (g)	PGV (cm/s)
Chi-Chi 1999,TCU136,NS	7.6	8.97	0.18	48	Chi-Chi 1999,TCU106,EW	7.6	15.2	0.16	47
Chi-Chi 1999,TCU136,EW	7.6	8.97	0.17	56	Chi-Chi 1999,TCU036,NS	7.6	16.7	0.13	50
Chi-Chi 1999,TCU084,NS	7.6	10.4	0.42	46	Chi-Chi 1999,TCU061,EW	7.6	17.8	0.14	40
Chi-Chi 1999,TCU057,EW	7.6	12.6	0.12	35	Chi-Chi 1999,TCU061,NS	7.6	17.8	0.14	44
Chi-Chi 1999,TCU100,EW	7.6	12.7	0.12	35	Chi-Chi 1999,TCU059,NS	7.6	17.8	0.17	56
Chi-Chi 1999,TCU100,NS	7.6	12.7	0.12	47	Chi-Chi 1999,CHY035,NS	7.6	18.1	0.25	38
Chi-Chi 1999,TCU104,EW	7.6	13.6	0.11	37	Chi-Chi 1999,TCU107,EW	7.6	20.4	0.12	37
Chi-Chi 1999,TCU048,EW	7.6	14.4	0.12	33	Chi-Chi 1999,TCU107,NS	7.6	20.4	0.16	47
Chi-Chi 1999,TCU048,NS	7.6	14.4	0.18	48	Chi-Chi 1999,CHY036,EW	7.6	20.4	0.29	39
Chi-Chi 1999,TCU039,EW	7.6	16.7	0.21	50	Chi-Chi 1999,CHY036,NS	7.6	20.4	0.21	41
Chi-Chi 1999,TCU039,NS	7.6	16.7	0.14	54	Northrdg.1994,Pacomia Kagel C.,090	6.7	8.2	0.30	31
Chi-Chi 1999,TCU070,EW	7.6	19.1	0.25	52	Northrdg.1994,Pacomia Kagel C.,360	6.7	8.2	0.43	52
Chi-Chi 1999,TCU070,NS	7.6	19.1	0.17	62	Northrdg.1994,Tarzana-Cedar Hill,090	6.7	17.5	1.78	114
Chi-Chi 1999,TCU052,NS	7.6	0.24	0.42	118	Northrdg.1994,Tarzana-Cedar Hill,360	6.7	17.5	0.99	78
Chi-Chi 1999,TCU052,EW	7.6	0.24	0.35	159	Northrdg.1994,Slymar Converter,142	6.7	6.2	0.90	102
Chi-Chi 1999,TCU068,EW	7.6	1.09	0.57	177	Northrdg.1994,Slymar Hospital,090	6.7	6.4	0.60	78
Chi-Chi 1999,TCU102,EW	7.6	1.79	0.30	112	Northrdg.1994,Newhall,090	6.7	7.1	0.58	76
Chi-Chi 1999,EWNT,NS	7.6	2.21	0.63	42	Northrdg.1994,Newhall,360	6.7	7.1	0.59	97
Chi-Chi 1999,EWNT,EW	7.6	2.21	0.96	69	Northrdg.1994,Sepulveda VA,360	6.7	8.9	0.94	77
Chi-Chi 1999,TCU101,NS	7.6	2.94	0.25	49	Northrdg.1994,Lost Canyon,000	6.7	13.0	0.41	43
Chi-Chi 1999,TCU103,NS	7.6	4.01	0.16	27	Cape Mend.1992,Rio OVP,360	7.1	18.5	0.55	42
Chi-Chi 1999,TCU049,EW	7.6	4.48	0.29	48	Cape Mend.1992,Petrolia,000	7.1	9.5	0.59	48
Chi-Chi 1999,TCU049,NS	7.6	4.48	0.25	61	Loma Pr.1989,Corralitos,000	6.9	5.1	0.64	55
Chi-Chi 1999,TCU,NS	7.6	5.73	0.19	34	Loma Pr.1989,Saratoga Aloha,000	6.9	13.0	0.51	41
Chi-Chi 1999,TCU,EW	7.6	5.73	0.18	41	Loma Pr.1989,Saratoga Valley,000	6.9	13.7	0.25	42
Chi-Chi 1999,TCU082,NS	7.6	5.73	0.19	41	Loma Pr.1989,Saratoga Valley,270	6.9	13.7	0.33	62
Chi-Chi 1999,TCU082,EW	7.6	5.73	0.22	58	Loma Pr.1989,Gilroy #2,000	6.9	12.7	0.37	33
Chi-Chi 1999,TCU054,NS	7.6	5.92	0.19	39	Loma Pr.1989,Gilroy #3,000	6.9	14.4	0.55	36
Chi-Chi 1999,TCU054,EW	7.6	5.92	0.15	59	Loma Pr.1989,Gilroy #4,000	6.9	16.1	0.42	39

Chi-Chi 1999,TCU053,EW	7.6	6.69	0.22	41	Loma Pr.1989,Gilroy #4,090	6.9	16.1	0.21	38
Chi-Chi 1999,TCU055,EW	7.6	6.88	0.24	26	Superstition Hills 1987,PTS,315	6.6	0.7	0.38	44
Chi-Chi 1999,TCU055,NS	7.6	6.88	0.20	52	N.Palm Spr.1986,N.P.Spr.PO,300	6.0	8.2	0.69	34
Chi-Chi 1999,CHY028,EW	7.6	7.31	0.65	73	Imp.Val.1979,EC Overp FF,000	6.5	0.5	0.31	72
Chi-Chi 1999,TCU051,NS	7.6	8.27	0.23	38	Imp.Val.1979,El Centro #7,140	6.5	0.6	0.34	48
Chi-Chi 1999,TCU051,EW	7.6	8.27	0.19	49	Imp.Val.1979,El Centro #5,140	6.5	1.0	0.52	47
Chi-Chi 1999,TCU060,NS	7.6	9.46	0.11	45	Imp.Val.1979,Bonds Corner,140	6.5	2.5	0.59	45
Chi-Chi 1999,TCU050,EW	7.6	10.3	0.15	37	Imp.Val.1979,Bonds Corner,230	6.5	2.5	0.77	46
Chi-Chi 1999,TCU050,NS	7.6	10.3	0.13	42	Imp.Val.1979,El Centro #8,140	6.5	3.8	0.60	54
Chi-Chi 1999,TCU063,EW	7.6	10.4	0.17	59	Imp.Val.1979,El Centro #4,140	6.5	4.2	0.49	37
Chi-Chi 1999,TCU109,EW	7.6	13.1	0.16	51	Imp.Val.1979,El Centro DA,360	6.5	5.3	0.48	41
Chi-Chi 1999,TCU109,NS	7.6	13.1	0.15	53	Imp.Val.1979,EC Center FF,002	6.5	7.6	0.21	38
Chi-Chi 1999,CHY006,NS	7.6	14.9	0.35	43	Imp.Val.1979,Brawley Airport,315	6.5	8.5	0.22	39
Chi-Chi 1999,CHY006,EW	7.6	14.9	0.36	55	SanFernando 1971,Pacoima Dam,164	6.6	2.8	1.22	113
Chi-Chi 1999,TCU064,EW	7.6	15.1	0.11	39	San Fernando 1971,Pacoima Dam,254	6.6	2.8	1.16	54
Chi-Chi 1999,TCU106,NS	7.6	15.2	0.13	44					

INELASTIC DISPLACEMENT DEMANDS OF NEAR-FAULT GROUND MOTIONS

The median maximum inelastic to elastic SDOF displacement ratios for NF ground motions with and without pulse signals are shown in Figures 2 and 3, respectively. The maximum inelastic displacements are computed by using EPP model for different constant strength reduction factors, R. The initial viscous damping is taken as 5 percent and it is kept constant for all the computations presented in this study. The median ratios shown in these figures serve for the same purpose as the C_1 modification factor of the FEMA-356 and they are labeled with the same acronym. In Figure 2, the vibration periods are normalized with respect to the pulse period (i.e. T/T_p) in order not to single out the distinct pulse effect of NF records on inelastic displacement demands. The normalized T/T_p ranges between 0.1 and 3.0 with constant intervals of 0.05. Similarly, the period range for the non-pulse NF records is between 0.1s and 3.0s and period interval increments are chosen as 0.05s. The median statistics are presented separately for site class C, site class D, and site classes C and D to see the effects of local site conditions on the median statistics of inelastic to elastic maximum SDOF displacement ratio.

The median ratios of Figure 2 indicate that the structures with fundamental periods less than approximately 80 percent of the pulse period would be subjected to higher displacement demands than those of the corresponding maximum elastic displacements. In this critical region, the amplitude of inelastic displacement demand increases rapidly as the lateral strength capacity of the structure decreases. This observation is consistent with the conclusions presented by Iwan [7]. As the structural period goes beyond the pulse period, the displacement demands of NF records with pulse signals are in the level of elastic response and even less depending on the lateral strength capacity. For structures with fundamental periods greater than the pulse period, lower yield strength values with respect to the elastic strength demand implies the confident use of equal displacement rule in this particular spectral region. Similar plots presented in Figure 3 for NF records without pulse signals show that the maximum inelastic displacement demands on SDOF systems tend to be higher than the maximum elastic displacement demands except for very low R values (i.e. R=1.5 and 2.0) with periods of vibration greater than 0.5s. The inelastic displacement demands are of serious concern for periods of vibration less than 1.0s and $R \geq 4$ as they tend to increase asymptotically towards very large values regardless the local site conditions. The maximum inelastic displacement demand amplifications tend to stabilize for periods greater than 1.0s but they are still well above the maximum elastic displacements especially for $R \geq 6$. Even if the yield strength capacity of a structure is relatively high with respect to its elastic strength demand (i.e. R=1.5 and 2.0), the inelastic displacement demands show a very large amplification for periods of vibration less than 0.5s. Limited to the NF ground motion data set used in this study, the similarity in the median ratio statistics between site class C, site class D, and site classes C and D records suggest the negligible effect of local site conditions on inelastic displacement demand amplification.

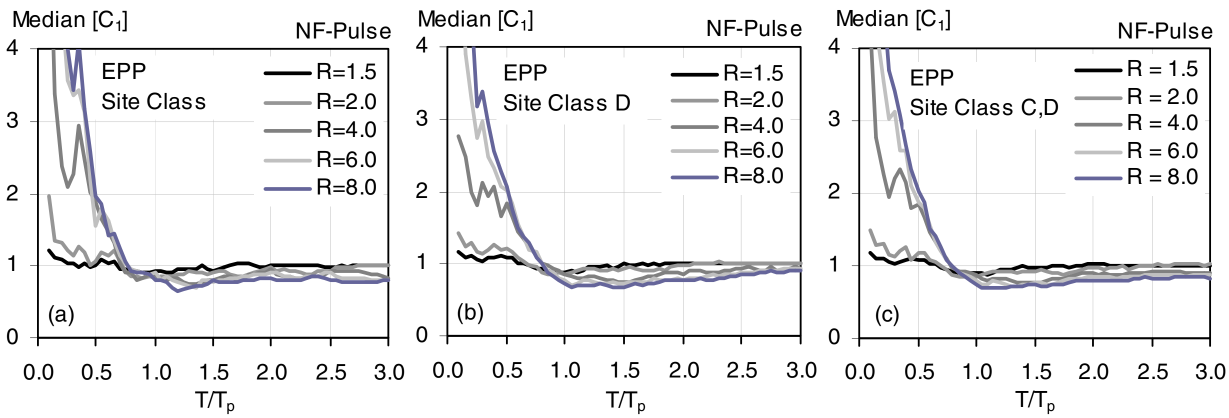

Figure 2. Inelastic to elastic maximum displacement ratios for NF records with pulse signals

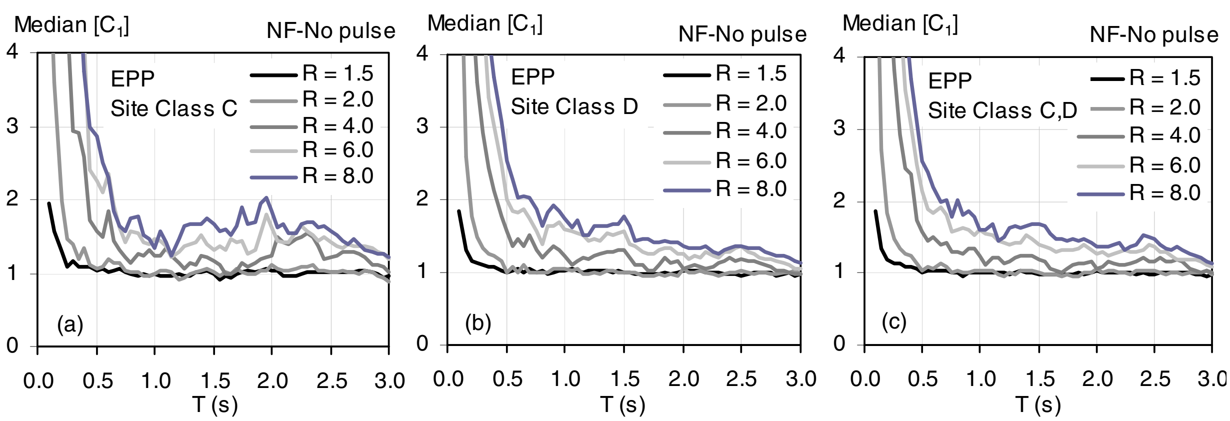

Figure 3. Inelastic to elastic maximum displacement ratios for NF records without pulse signals

The median inelastic to elastic displacement ratios displayed in Figure 3 are significantly higher than the results presented by Ruiz-García [9] that derived inelastic displacement prediction equations for similar site conditions by using EPP hysteretic behavior. The comprehensive statistical study conducted by Ruiz-García [9] used a typical subset of far-fault ground motions that, in general, are not expected to exhibit a dominant pulse signal that can be a common point with the non-pulse NF ground motion data used in the median plots of Figure 3. This common point between the two ground motion dataset would make the comparisons fair and allow a better understanding of high deformation demands in NF ground motions. The high PGV values and relatively short site-to-source distances of the chosen ground motions are the main reasons for the higher median statistics derived in this study. These seismological features are generally attributed to the NF ground motions and are not observed in the far-fault data set of Ruiz-García [9].

Complementary to the observations in the preceding paragraphs, Figures 4 and 5 display the effect of post yield stiffness ratio (α, post yield to initial stiffness ratio) on the inelastic displacement demands of NF records with and without pulse signals, respectively. These figures show the median curves for normalized maximum inelastic SDOF displacement ratios of α = 2 and 5 percents by the EPP (α = 0 percent) bilinear hysteretic behavior. The plots in Figure 4 are drawn for normalized period values with respect to the pulse period as in the case of Figure 2. The left columns in both figures show α = 2 to α = 0 percent median ratio whereas the right columns give the same statistics for α = 5 to α = 0 percent ratio. The plots in the first row are drawn for site class C records and the last row shows the results for site class D records.

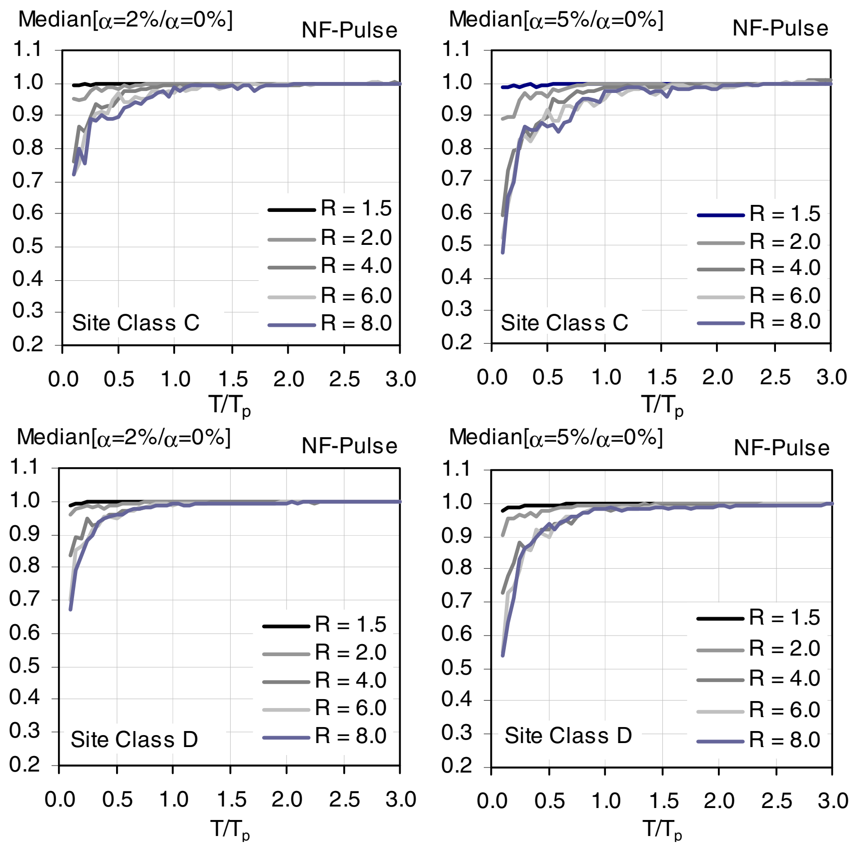

Figure 4. Effect of post yield stiffness on inelastic displacement demands of NF records with pulse

The median ratio curves presented in Figure 4 indicate that a non-zero post yield stiffness reduces the inelastic displacement demands for periods of vibration less than the pulse period. The reduction in the inelastic displacement demand is especially effective for $T/T_p \leq 0.5$ and $R \geq 4$. The reduced inelastic displacements for α = 2 and 5 percents can reach to a level of 30 percent and 50 percent of the EPP behavior, respectively when T/T_p ratio is approximately 0.1. The inelastic displacement demands of SDOF systems subjected to NF records with pulse are not affected significantly from the changes in post yield stiffness ratio when the periods of vibration are greater than the pulse period. The significant role of post yield stiffness ratio in decreasing the inelastic displacement demand of NF records without pulse signals becomes substantial for R values greater than 4 and periods of vibration less than 1.0s. The plots in Figure 5 show that the asymptotic decrease in the median inelastic displacement demand with respect to EPP inelastic displacements is more than 40 percent when α = 5 percent and T ≤ 0.5s for R ≥ 4. The observations highlighted in Figures 4 and 5 point that relatively short period structures associated with a moderate post yield strength would have lower deformation demands relative to the EPP behavior regardless the NF record exhibits a dominant pulse or not. The level of decrease in the deformation demands become more effective as the yield strength capacity attains lower values with respect to the elastic strength demand (i.e. larger R values).

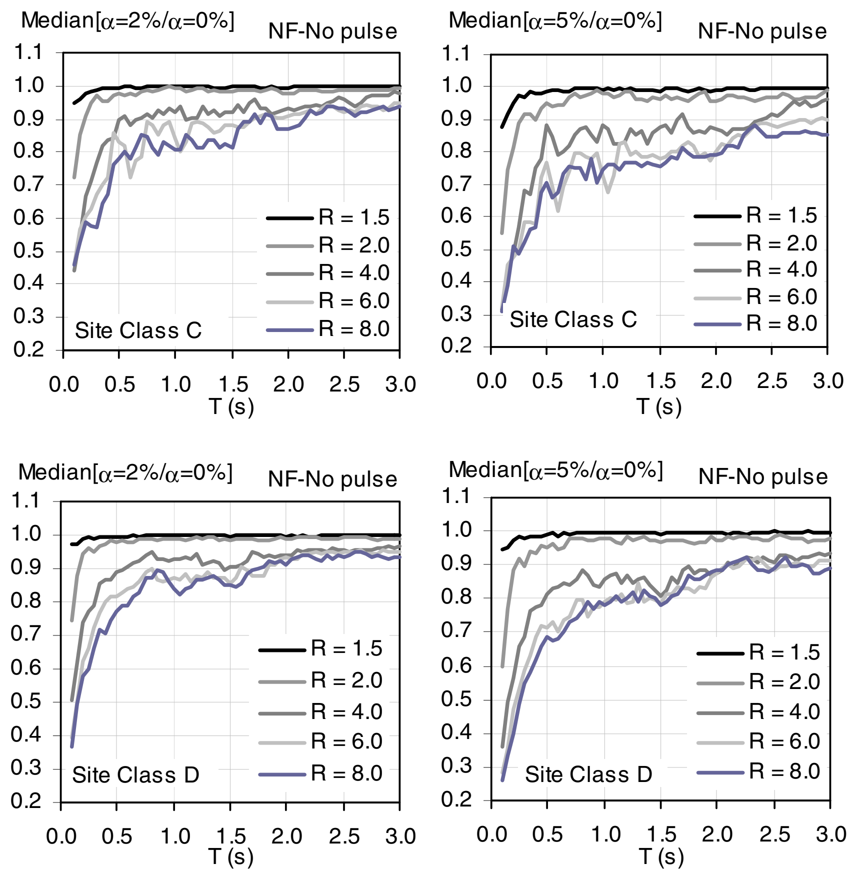

Figure 5. Effect of post yield stiffness on inelastic displacement demands of NF records without pulse

The effect of stiffness degradation on the maximum inelastic demands of NF ground motions are also investigated within the context of this study. Modified Takeda model as proposed by Otani [8] is used for this purpose. The stiffness degradation is accepted as the nonlinear behavior of a moderately damaged reinforced concrete (RC) building system subjected to a severe ground shaking. Such a system would not suffer a substantial damage due to the simultaneous effects of lateral stiffness and strength deterioration and it could be classified as a building that fulfills the life safety performance criterion in the FEMA-356 document. The maximum inelastic SDOF displacements of the modified Takeda model are normalized by the maximum inelastic SDOF displacements of the EPP behavior in order to display the effect of stiffness degradation on structural deformation demands. Null post yielding stiffness is assigned for the modified Takeda model as in the case of EPP model. The median ratio statistics for different strength reduction factors are computed separately and presented in Figures 6 and 7 for NF ground motions with and without pulse, respectively. Similar to the previous figures shown in the paper, the periods of vibration are normalized by the pulse period for NF ground motions that exhibit pulse signals in their waveforms. The figures show the median statistics for site class C, site class D, and site classes C and D to verify the significance of local site conditions on the inelastic deformation demands of stiffness degrading systems with respect to EPP behavior.

The median statistics of NF records with pulse presented in Figure 6 indicate that the stiffness degrading structural systems are subjected to higher inelastic displacement demands with respect to the EPP behavior when T/T_p is less than 1.5 and $R \geq 4$. The increase in inelastic displacement demand follows almost a linear trend for R ≥ 4 and $T/T_p \leq 1.5$ regardless the local site conditions and reaches approximately to an amplification factor of 1.2 with respect to the EPP behavior when $T/T_p = 0.1$. The systems with relatively large yield strength with respect to elastic strength demand (i.e. R = 1.5) are not sensitive to the stiffness degradation for the T/T_p range considered in this study. Similar median plots of NF records without pulse that are presented in Figure 7 do not show a clear trend for the influence of stiffness degradation hysteretic behavior over the EPP behavior. In general the median ratios are greater than 1.0 and fluctuate about 1.05 when T > 0.5s indicating a slight increase in inelastic displacements of stiffness degrading systems with respect to EPP behavior. The maximum inelastic displacements demands of stiffness degrading behavior tend to larger values for T ≤ 0.5s. In some particular cases (e.g. site class D, R = 1.5), the stiffness degradation requires a significantly large displacement demand at relatively short periods (i.e. T ≤ 0.35s). However, this particular pattern is arbitrary and does not follow the general trend described above. Miranda [4] derived similar mean statistics for far-fault ground motions of similar site classes. The mean ratios were derived for stiffness degrading modified Clough model [10]. Comparisons between the results of Miranda [4] and the curves presented in Figure 7 for non-pulse NF ground motions suggest larger median ratios for stiffness degrading hysteretic behavior of this study. Similar to the comments made in the preceding paragraphs, the difference can be attributed to the high PGV values and short site-to-source distances of the chosen NF ground motion data set in this study. This discussion once more brings forward the distinct behavior of near-fault ground motions either with or without pulse when compared to the general displacement demand trend in far-fault earthquake records.

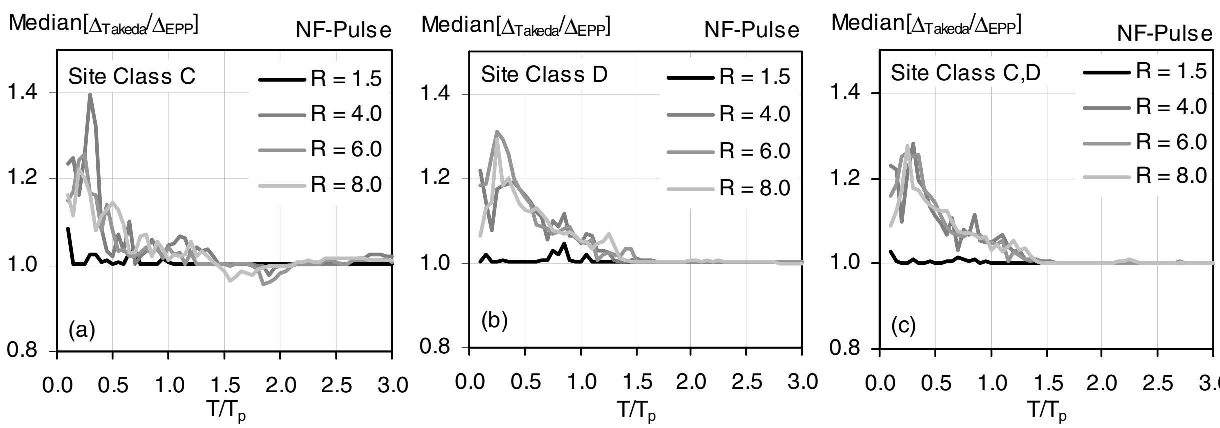

Figure 6. Median Takeda to EPP inelastic displacement ratios for NF records with pulse

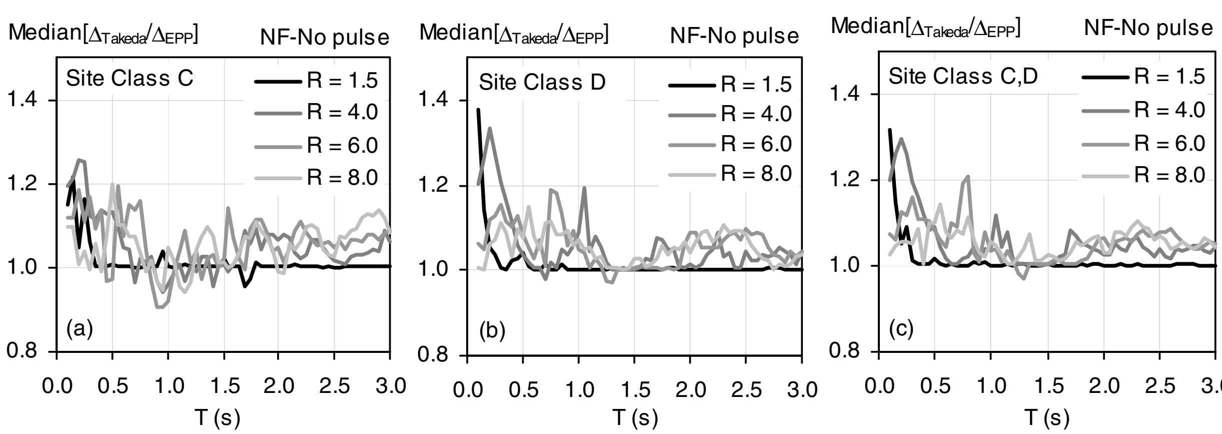

Figure 7. Median Takeda to EPP inelastic displacement ratios for NF records without pulse

The discussions in the above paragraph may give side information for the improvement of C_2 factor in the FEMA-356 nonlinear analysis procedure. This factor is proposed to modify the maximum inelastic SDOF displacements for hysteretic behaviors other than the bilinear behavior. For life safety performance level of ordinary RC frames (moderately damaged structural behavior in this study) C_2 value is 1.1 for periods of vibration greater than the characteristic period of design spectrum (designated as T_s in the FEMA-356) that is the corner period value between the so-called constant-acceleration and constant-velocity spectral regions. The value of C_2 is limited to 1.3 for $T \leq 0.1$s and varies linearly between $0.1s < T < T_s$. The discussions in this study suggest that the C_2 should be sensitive to T_p for NF records with pulse and the amplification due to stiffness degradation is only necessary for periods of vibration less than 1.5 times the pulse period. Similarly, for stiffness degrading systems, the C_2 value is calculated, on average, slightly above 1.0 for a broad range of periods when the subject ground motions are NF records without pulse. These results highlight that the C_2 coefficient of FEMA-356 can be either conservative or non-conservative depending on the NF ground motion type and the consistency of T_s computation with respect to T_p when the system is exposed to stiffness degradation. The significance of corner period and its accuracy for the prediction of maximum inelastic displacement demands of NF ground motions is discussed in detail while evaluating the FEMA-356 procedure in the next section.

EVALUATION OF FEMA-356 C_1 COEFFICIENT FOR NEAR-FAULT GROUND MOTIONS

The FEMA-356 nonlinear procedure proposes four coefficients to predict the deformation demand of building systems given the yield strength capacity for a pre-determined seismic hazard level. The coefficient C_1 modifies the maximum elastic spectral displacement for maximum inelastic displacement of a bilinear hysteretic behavior similar to the ones used in this study (i.e. either EPP or bilinear hysteresis associated with certain post yield stiffness). The stiffness degradation or simultaneous stiffness degradation with strength deterioration are accounted by the C_2 factor. The inelastic deformation modification due to secondary P-delta effects is determined by the C_3 coefficient. The fourth coefficient C_o converts the maximum SDOF inelastic displacement demands predicted by the product of C_1, C_2 and C_3 to the lateral deformation pattern of the MDOF system. The distinct behavior of C_1 and C_2 factors for NF ground motions with or without pulse signals are discussed in the previous section. This section evaluates the C_1 coefficient proposed by FEMA-356 to give complementary statistical information on the accuracy of maximum SDOF inelastic deformation estimates when the subject ground motions are NF records. Equation (1) shows the definition of error term used to measure the accuracy of FEMA-356 C_1 coefficient.

$$(E)^i_{T,R} = \left(\frac{C_{1,FEMA}}{C_{1,ex}} \right)^i_{T,R} \tag{1}$$

The variables $C_{1,FEMA}$ and $C_{1,ex}$ represent the C_1 coefficients computed from the FEMA-356 and response history results, respectively. Given the period of vibration T and strength reduction factor R, the ratios of $C_{1,FEMA}$ to $C_{1,ex}$ are computed for each ground motion listed in Tables 1 and 2. The characteristic period T_s is also determined for each NF ground motion as required by the FEMA-356 document to compute the $C_{1,FEMA}$ coefficient. For each particular T and R, the median statistics are computed for a total number of m earthquake records shown in Tables 1 and 2. The value of m is 56 for NF records with pulse whereas m is 89 for the median statistics of NF records without pulse. A median value greater than 1 indicates the 50 percent probability for $C_{1,FEMA}$ to give a conservative maximum inelastic SDOF estimate for that particular period of vibration and strength reduction factor. Conversely, a median value less than 1 implies a 50 percent probability of unsafe maximum inelastic SDOF displacement estimation for $C_{1,FEMA}$. The median error plots for NF records with or without pulse are shown in Figure 8. The left column in Figure 8 displays the median errors of NF records with pulse for post yield stiffness ratios of $\alpha = 0$, 2 and 5 percent bilinear behavior. The left column displays similar errors for NF records without pulse signals. Different from the previous figures presented in this study, the periods of vibration for NF records with pulse are not normalized with respect to pulse signals. The period range considered for error statistics is between 0.1s and 3.0 as in the case of all other figures.

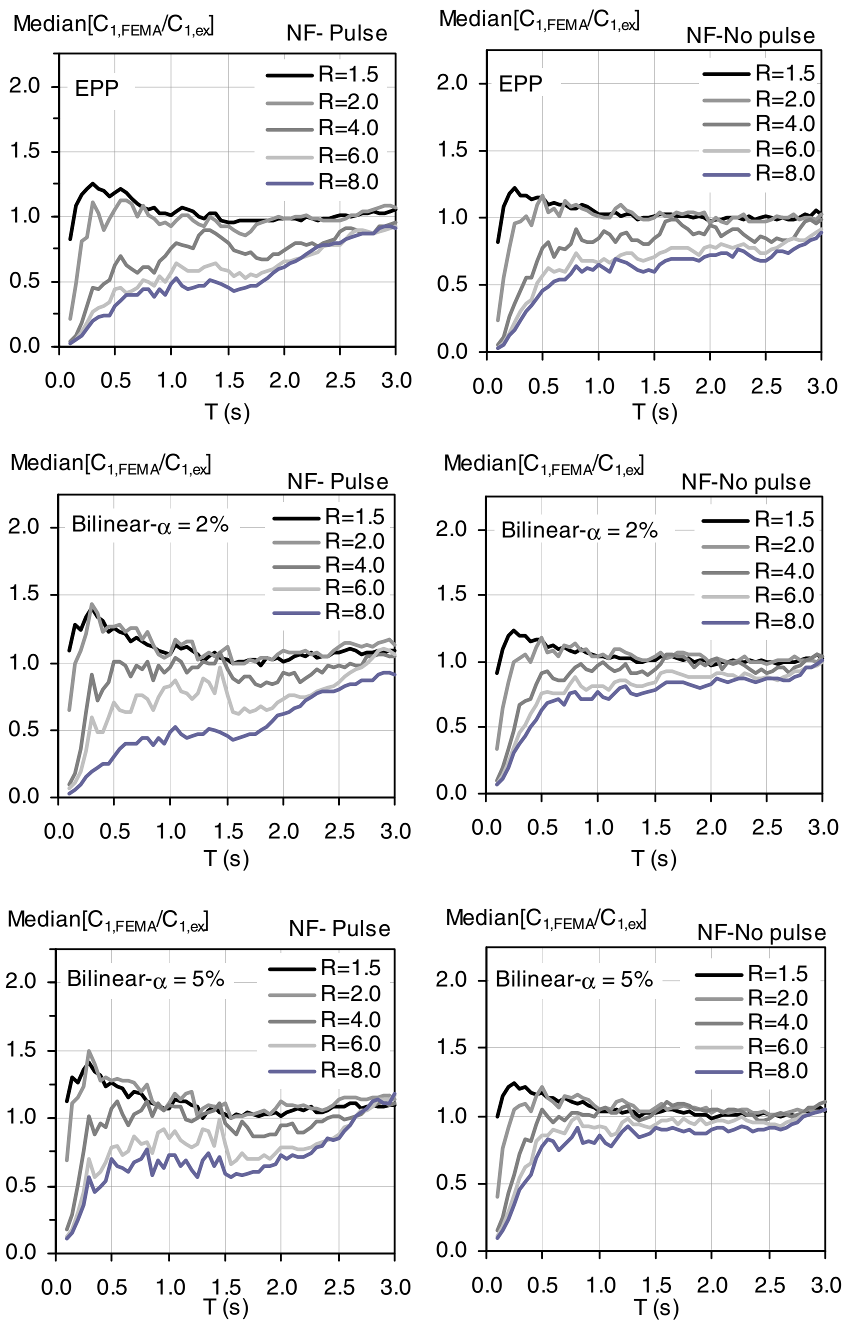

Figure 8. Median error statistics of FEMA-356 C_1 coefficient for NF records with and without pulse

The general trend of median errors presented in Figure 8 indicate a non-conservative tendency for the maximum inelastic displacement demand predictions of FEMA-356 when $R \geq 4$. The non-conservative estimations are larger for NF records with pulse and in general they tend to increase towards shorter periods. The median errors are largest for the EPP behavior (i.e. bilinear hysteretic model with $\alpha = 0$ percent) regardless the NF record type. When the yield strength capacity is relatively larger with respect to elastic strength demand (i.e. $R \leq 2$) the FEMA-356 C_1 coefficient estimations are compatible with the results of response history analyses for periods of vibration greater than 1.0s. Specific to such small strength reduction factors, the period shift towards smaller values makes the FEMA-356 maximum inelastic SDOF displacement predictions fluctuate from non-conservative to conservative side depending on the NF ground motion type, the period value and the post yielding stiffness ratio. The conservative median displacement estimations of FEMA-356 are significant with respect to the actual response of NF records with pulse when $R \leq 2$ and $0.35s \leq T \leq 0.7s$ for $\alpha = 2$ and 5 percent. As the lateral strength reduction factor increases, the unsafe maximum inelastic displacement estimations of FEMA-356 are substantial and should be a serious concern either for NF records with or without pulse. Regardless the NF ground motion type, the median unsafe estimations are greater than 30 percent for a moderate post yield stiffness ratio of 2 percent when the periods of vibration are approximately less than 0.7s and $R \geq 6$. The unsafe FEMA-356 estimations reach even much higher values for EPP hysteretic model that generally describes the nonlinear behavior of well detailed steel structures.

It is believed that the notable error margins described in the preceding paragraph are due to the limitations imposed on C_1 coefficient by FEMA-356 document. FEMA-356 favors equal displacement rule for periods of vibration greater than the characteristic period T_s and limits the C_1 value by 1.5 for periods less than 0.1s independent from the variation in yield strength capacity. The coefficient C_1 is also forced to vary linearly in the spectral region between T_s and 0.1s and attains almost the same values for the intermediate periods of this spectral region regardless the strength reduction factor R. A detailed description of these limitations is presented in Miranda [4]. The conclusions derived on the actual behavior of C_1 in Figures 2 and 3, and the effect of post yield stiffness ratio presented in Figures 4 and 5 displays very different rules for the inelastic displacement demands of SDOF systems subjected to NF records with or without pulse. The error statistics clearly show that the strong dominance of pulse period on the inelastic displacement demands of NF records with pulse cannot be captured by the characteristic period of the FEMA-356 procedure. The additive influence of imposed limitations on C_1 for periods of vibration less than T_s makes the displacement estimations of FEMA-356 even poorer for NF records with pulse. The median plots in Figure 2 show that the actual variation of C_1 attains values larger than 1.5 for periods less than $0.8T_p$ when $R \geq 4$. The sensitivity of maximum inelastic SDOF displacements on post yield stiffness ratio that are presented in Figure 4 for NF records with pulse reduces the error margins for $\alpha = 2$ and 5 percents. The decrease in error levels is insufficient as the inelastic displacement demands due to NF records with pulse are sensitive to post yield stiffness only for periods of vibration less than the pulse period and this property is not accounted for by the FEMA-356 nonlinear procedure. The large error levels of NF records without pulse signals can also be explained by comparing the pertinent error statistics with the relevant plots displayed in Figures 3 and 5. The actual trends in Figure 3 indicate that the equal displacement rule is not applicable in the case of NF records without pulse especially for $R \geq 4$ even if the periods of vibration take very large values. The median T_s value for the 89 NF records without pulse is computed as 0.74s. This value suggests that the C_1 coefficient of FEMA-356 is very unlikely to attain values above 1.0 to capture the actual inelastic displacement demands for very long periods and for large values of R as demonstrated in Figure 3. The unrealistic limitations imposed on C_1 for short periods also provoke the FEMA-356 unsafe errors for NF records without pulse even though the increased post yield stiffness decreases the inelastic deformations with respect to EPP behavior as presented in Figure 5.

CONCLUSIONS

The maximum inelastic displacement demands of SDOF systems subjected to NF ground motions are presented. A suite of 145 NF ground motions is used that consists of firm-to-dense soil records with $6.5 \leq$

$M_w \leq 7.6$ and $d \leq 20$km. The inelastic displacement demands of NF ground motions are evaluated separately for NF records with and without pulse signals. This way the effect of yield strength capacity, pulse period, post yield stiffness and hysteretic behavior with and without stiffness degradation are evaluated rationally for the general displacement demand characteristics of NF ground motions. A total of 153990 response history analyses are conducted during the course of this study to represent the observed trends in a statistical manner. Using these response history results, the accuracy of FEMA-356 C_1 coefficient is evaluated for NF ground motions. The following observations and conclusions are derived at the end of this study:

1. The pulse period plays an important role in the maximum inelastic displacement demands of NF records with pulse signals. The maximum inelastic displacements are amplified significantly for periods of vibration approximately less than 0.8 times the pulse period. The amplification increases with the increasing strength reduction factor. The equal displacement rule can be used for periods of vibration greater than the pulse period as the maximum inelastic displacements are below the maximum elastic displacements in this spectral region.
2. The maximum inelastic to elastic displacement demand ratios of NF records without pulse are above 1.0 for very long periods of vibration when $R \geq 4$. The amplification in the maximum inelastic displacements tends towards very large values asymptotically when periods are less than 1.0s. The computed median inelastic to elastic maximum displacement ratios for NF records without pulse are higher with respect to the previous observations made by using far-fault records of similar site conditions that are not expected to exhibit pulse in their wave forms. The higher PGV and shorter site-to-source distance that are of typical for NF records are the main reasons for the increase in the maximum inelastic to elastic SDOF displacement ratios.
3. The maximum inelastic to elastic SDOF displacement ratios exhibit similar trends for NEHRP site class C and D records either for NF records with or without pulse.
4. The post yield stiffness ratio reduces the maximum SDOF inelastic displacement demands with respect to elastoplastic hysteretic behavior. The decrease in the inelastic displacements due to non-zero post yield stiffness is related to the pulse period for NF records with pulse. For periods of vibration less than pulse period, the post yield stiffness effectively decreases the inelastic displacements with respect to elastoplastic behavior. The level of decrease in the inelastic displacements increases with increasing strength reduction factor R. The post yield stiffness ratio also reduces the inelastic displacement demands of NF records without pulse. The reduction in inelastic displacements due to non-zero post yield strength is prominent for $R \geq 4$.
5. The stiffness degrading hysteretic behavior increases the inelastic displacement demand of NF records with pulse for periods of vibration less than 1.5 times the pulse period. There is also a slight increase in the inelastic displacement demands of NF records without pulse due to stiffness degradation. This increase does not exhibit a clear trend as in the case of NF records with pulse.
6. The observation in item 5 is useful for the calibration of C_2 factor in the FEMA-356 procedure for NF ground motions. In its current state, this coefficient may yield conservative or non-conservative modification constants for stiffness degrading systems depending on the NF ground motion type and the consistency between the characteristic ground motion period and pulse period.
7. The C_1 coefficient proposed by FEMA-356 cannot capture the maximum inelastic displacement demands of SDOF systems subjected to NF ground motions. The predictions are on the unsafe side especially for large strength reduction, low post yield stiffness and short period of vibration. The limitations imposed on C_1 that are independent from the strength reduction factor and the erroneous guiding of characteristic ground motion period in the prediction of maximum inelastic displacement demands of NF ground motions are the main reasons for the low performance of C_1 coefficient in FEMA-356.

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