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RELATIONSHIP BETWEEN RESPONSE MODIFICATION COEFFICIENT AND DISPLACEMENT AMPLIFICATION FACTOR FOR DIFFERENT SEISMIC LEVELS AND SITE CLASSES

Bulent Erkmen[1]

[1] Ozyegin University, Department of Civil Engineering
Istanbul-Turkey
e-mail: Bulent.erkmen@ozyegin.edu.tr

Abstract

Modern seismic design provisions allow structural systems to be designed for reduced forces, which are typically much smaller than the corresponding elastic design loads. This reduction in seismic loads is done by using response modification coefficient. The maximum deflection or drift of structural systems is typically predicted by scaling deflections corresponding to this reduced force on elastic line by displacement amplification factor. The values of these two seismic design factors, which are independent of seismic level and site (soil) class for a given type of structure, are mostly based on accumulated experience from past earthquakes and engineering judgment.

In this study, a large number of five story parking garage structures were designed with a range of response modification factors for two seismic levels and for different site classes. The seismic performance of the structures was determined by performing nonlinear time history analysis with several recorded earthquake records on corresponding site classes. The computed maximum drift values were used to develop relationship between response modification coefficient and corresponding deflection amplification factor for each site class and seismic level. The results show that these two seismic design parameters are related but seismic level and site classes does not have significant effects on the relationship.

Keywords: Response Modification Coefficient, Displacement Amplification Factor, Ductility, Seismic Design.

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1 INTRODUCTION

Seismic design of a structural system to remain elastic or free of damage during a severe earthquake loading requires the structure to be designed to resists very large dynamic loads. Such a design approach results in significantly expensive and massive structures. Therefore, moderns seismic design provisions allow structures to be designed for much smaller loads then those required to maintain the structure elastic. This reduction is done by using reduction factor known as response modification coefficient (R), which solely depends on building seismic force-resisting system. The R coefficient was first introduced in 1978 by Applied Technology Council [1]. In the mid-1980s, researcher at the University of California at Berkeley performed a number of experiments to construct draft formulas for R as a function of reserve strength, ductility, and damping [2]. Relationships for R coefficient as a function of different parameters such as ductility, structure fundamental period, post-yielding stiffness, and soil characteristic were proposed by Newmark and Hall [3], Riddell et al. [4], Miranda [5], Nasar and Krawinkler [6], Vidic et al. [7], and Uang [8]. However, although value of R accounts for structural damping, ductility, over-strength, and redundancy, the code assigned values are mainly based on engineering judgment and seismic performance of structural systems in previous earthquakes [1, 8].

Because the reduced seismic forces are used in design, the computed lateral displacements from elastic analysis corresponding to design loads are amplified in order to estimate the actual nonlinear and inelastic lateral displacements that develop during design earthquake. This is done usually by using displacement amplification factor C_d. *Figure 1* show effects of these two seismic design factors on seismic design. The value of R as given in ASCE/SEI 7-16 [9] is between 1.5 an 8, where a value of 1.0 correspond to elastic structure, and those for C_d are between 1.0 and 6.5.

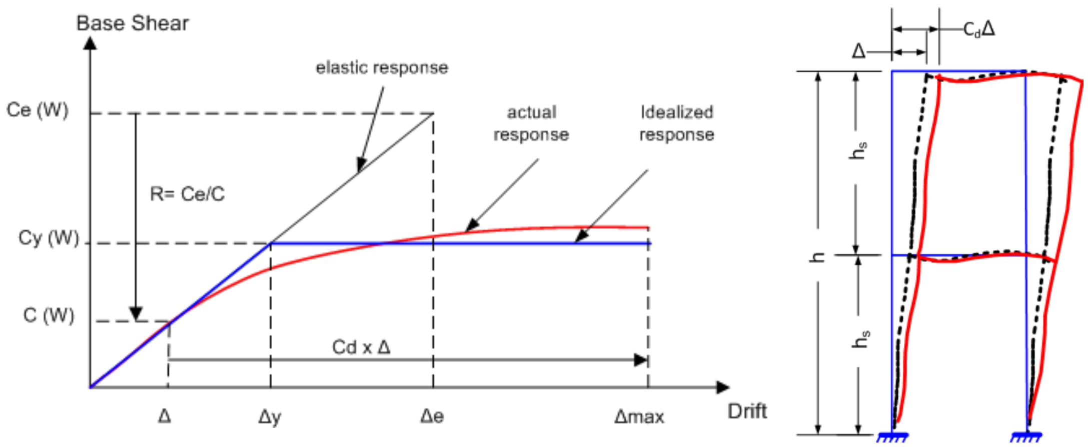


Figure 1: Design spectra and seismic design parameters.

Although it is well known that both R and C_d are interrelated, there is a limited number of studies investigating this relationship. The Uniform Building Code, which uses R_w as force reduction coefficient, assigns $3R_w/8$ for C_d factor [8]. The Commentary of the SEAOC Recommended Lateral Force Requirements, which is the basis for UBC seismic design provisions, states that C_d can be as high as R_w [8].

A review of national building codes indicates that the ratios of R and C_d vary considerably from one code to another. The objective of this study is to determine the relationship between these two seismic design parameters including effects of site classes and seismic levels. For this purpose, a large number of prototype parking garage structures were designed using *R* values from 4.0 to 8.0 with 0.5 increments. In total, two seismic levels (high and low) and four site classes were considered. For each site class, six actual earthquake time-acceleration data recorded on the same site class were used to preform nonlinear time history analysis of the prototype buildings. The computed deflections were used to establish relationship between seismic design parameters for each seismic level and site class.

2 PROTOTYPE STRUCTURE

A precast parking garage structure was selected as the prototype structure. The structure was designed as five story to include higher modes effects on results. The structure was designed for two seismic levels mainly 0.4*g* (low) and 0.8*g* (high) for spectral response acceleration per ASCE/SEI 7-16. For building location, five site classes (i.e., A, B, C, D, and E) defined in ASCE/SEI 7-16 were considered. However, site classes A and B were considered as a single site class (AB) since the corresponding design spectra and design requirements were not much different. The plan and elevation views of the building are given in *Figure 2*. Each story was 3.2 m high with a roof of 1-m high from top of the spandrels. The floor plan is typical for parking garage structures, and it features four unbonded post tensioned precast (PTT) shear walls as the only seismic force-resisting system in the N-S direction. Two lines of “light bearing walls” were used as seismic force resisting system in E-W direction. In this study, building seismic performance only in N-S direction was considered.

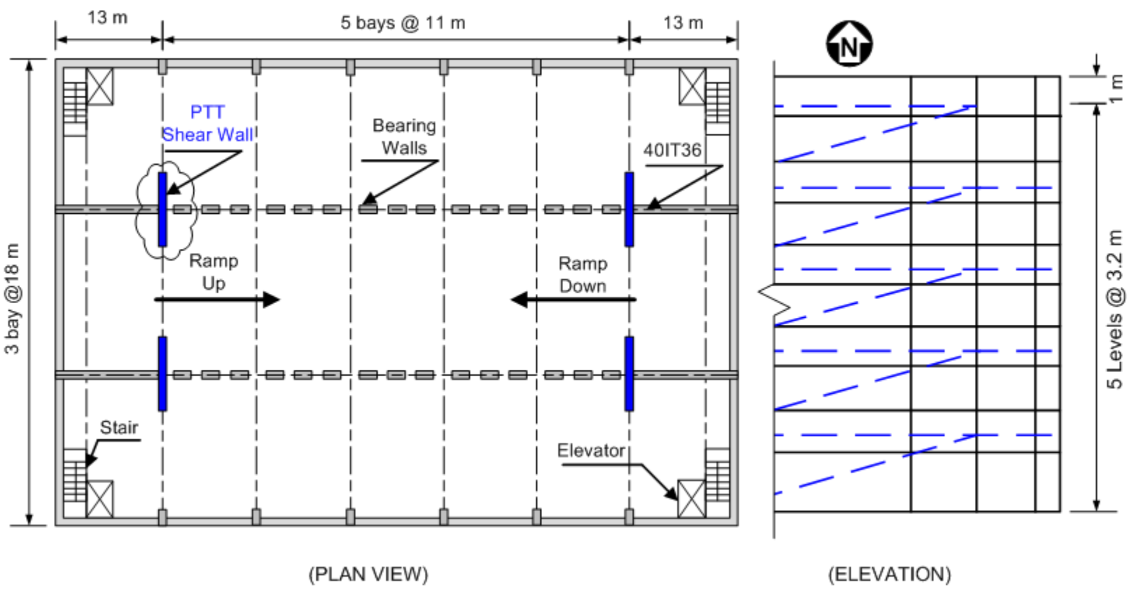


Figure 2: Plan and elevation views of prototype parking structure.

The unbonded post-tensioned precast shear walls are jointed construction, and they do not possess disadvantages of monolithic shear walls as shown by the PRESSS (PREcast Seismic Structural Systems) Research Program [10]. In such seismic force-resisting systems, concrete wall units remain almost undamaged due to lack of bond between post-tensioned tendons and concrete. The walls designed based on this design philosophy also known as jointed construction resist lateral loads by opening and closing of joints between precast members without causing significant damage to structural members. Unbonded precast concrete shear walls have been studied both experimentally and analytical by a number of researchers including Kurama et al. [11]., Shultz et al. [12], and Erkmen et al. [13].

The cross-section and elevation of prototype walls are given in *Figure 3*. Each wall has five precast wall panels with a horizontal joint at each floor level. The selected wall dimensions were based on site class and seismic level. The number and location of unbonded tendons were selected to ensure that each wall has the desired lateral strength level. Building overstrength was assumed to be unity. The amount and location of interlocking spirals (confining reinforcement) was determined based on design recommendations per ACI ITG-5.1-07 [14] and ACI ITG-5.2-09 [15].

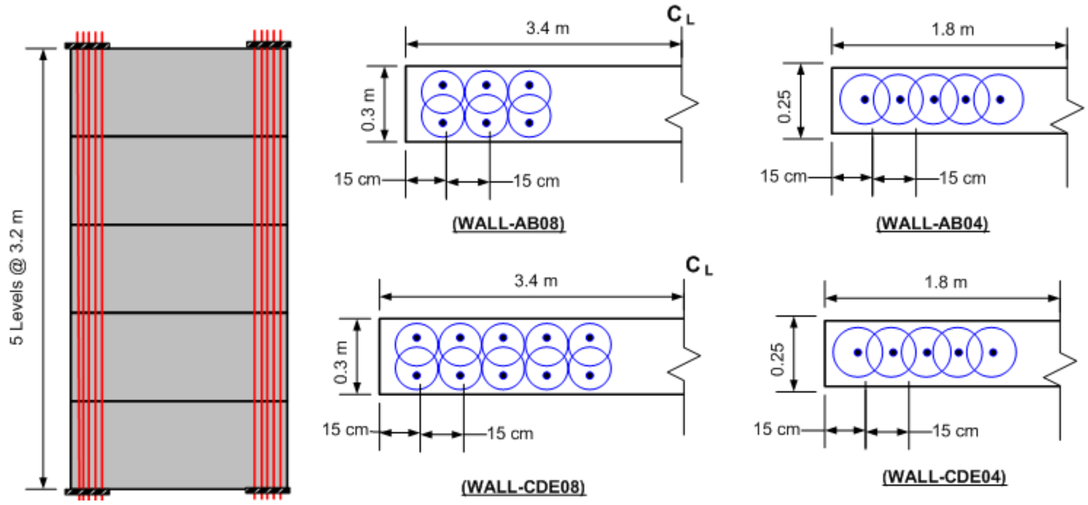


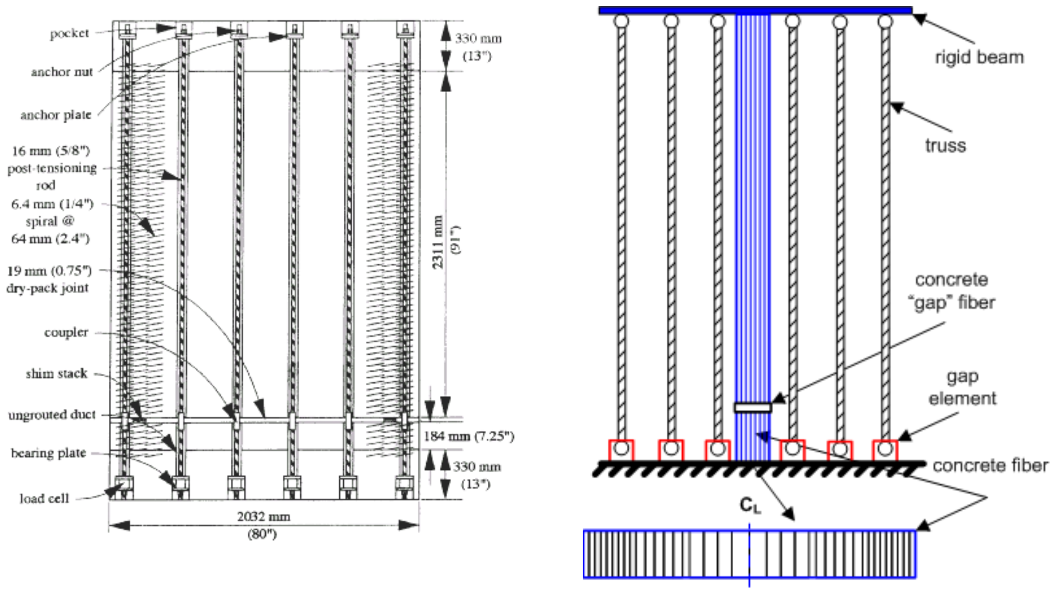
Figure 3: Plan and elevation views of prototype PTT walls.

2.1 Analytical Model and Verification

Nonlinear response-history analysis results of the porotype building were used to determine maximum lateral deflection of the building. Such nonlinear dynamic analyses shall utilize a structural mathematical model, which is capable of capturing nonlinear hysteretic behavior of structure [9]. The computational model was developed and verified using DRAIN-2DX program, which is a general-purpose computer program for both linear and nonlinear static and dynamic analysis of plane structures [16].

The computational model was verified using experimental test results given by Schultz et al [12]. The test PTT wall, which is given in *Figure 4*, was a 2/3-scale representation of the lowest two stories of a prototype precast concrete shear wall in a six-story office building. The wall featured six unbonded post-tensioned tendons and typical construction materials. The wall was tested under quasi-static cyclic loading and subjected to a combination of vertical and in-plane lateral loads history.

The main features of the developed analytical model are given in *Figure 4*, and a detailed description of the model and its verification are given by Erkmen et al [13]. The experimental and computed analytical responses given in *Figure 5* show no significant differences between the measured and computed stiffness, load capacity, and absorbed energy capacities of the wall.



(a) (b)

Figure 4: Experimentally tested PTT wall (a) dimensions and (b) analytical model.

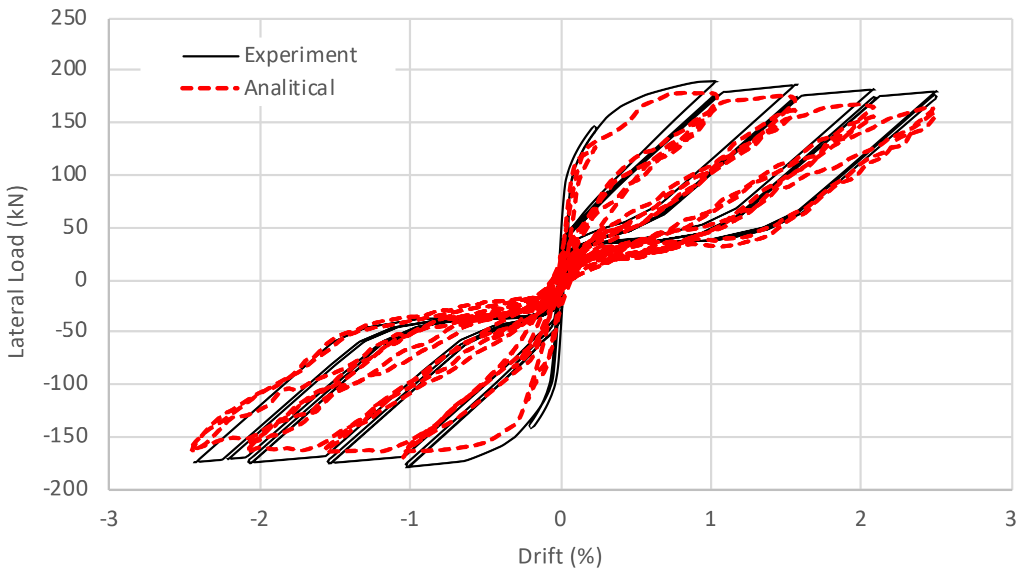


Figure 5: Experimental analytical force-displacement response of test PTT wall.

2.2 Selection and Scaling of Ground Motions

Six strong ground motion records were selected for each ASCE/SEI 7-16 Site Class A and B, C, D, and E. The ground motions records were obtained from the Pacific Earthquake Engineering Research (PEER) Center NGA-West2 database. A summary of all selected ground motions is given in *Table 1*. The ground motions records were selected giving preference to station site class, spectral shape over the period range of interest, free-field records, magnitude, and closest distance to the rupture plane.

No.	Motion, Year	Site Class	Station	Comp.	Mag.	Dist. km
EQAB1	Mendocino, 1992	AB	C. Mendocino	Cap.-Cpm000	7.1	8.5
EQAB2	Chi-Chi, 1999		HWA056	Coyotelk-G01230	7.6	48.8
EQAB3	Chi-Chi, 1999		ILA063	Chichi-Ila063N	7.6	71.6
EQAB4	Landers, 1992		12206 Silent Valley	Landers-Sil090	7.3	51.7
EQAB5	Northridge, 1994		24207 Pacoima Dam	Northr-Pul104	6.7	8.0
EQAB6	Northridge, 1994		90017 W. Ave.	Northr-Won095	6.7	22.7
EQC1	Northridge, 1994	C	90015 LA.Chalon Rd	Northr-Chl070	6.7	23.7
EQC2	Kern County, 1979		1095 T. L. School	Kern-Taf111	7.4	41.0
EQC3	Imp. Valley, 1979		6604 Cerro Prieto	I..H-CPE237	6.5	26.5
EQC4	Loma Prieta, 1989		57064 F. M. S. Jose	Lomap-Fre000	6.9	43.0
EQC5	Chi-Chi, 1999		CHY079	Chichi-Chy079N	7.6	55
EQC6	Kocaeli, 1999		Arcelik	Kocaeli-Arc090	7.4	17
EQD1	Erzincan, 1992	D	Erzincan	Erzincan-Erz-EW	6.7	4.4
EQD2	Chi-Chi, 1999		TCU072	Chichi-Tcu072-N	7.6	7.1
EQD3	I. Valley-02, 1940		El Centro A #9	Impvall.I-I-Elc270	7.0	6.1
EQD4	Northridge-0, 1994		Roscoe Blvd	North-Ro3090	6.7	10.1
EQD5	Big Bear-01, 1992		San B.E& Hosp.	Bigbear-Hos180	6.5	35.2
EQD6	S. Hills-02,1987		Imp. Co. Cent	Super.B-B-Icc000	6.5	18.2
EQE1	Kocaeli, 1999	E	Ambarli	Kocaeli-Ats090	7.5	69.6
EQE2	Chi-Chi, 1999		Chy002	Chichi-Chy002-N	7.6	25.0
EQE3	Chi-Chi, 1999		Chy025	Chichi-Chy025-N	7.6	19.1
EQE4	Chi-Chi, 1999		Tcu056	Chichi-Tcu056-N	7.6	19.5
EQE5	Loma Prietra, 1989		Treasure Island	Lomap-Tri000	6.9	77.4
EQE6	Taiwan, 1999		Tcu040	Chichi-Tcu040-N	7.6	22.1

Note: Mag. is moment magnitude, Dist. is distance to rupture plane.

Table 1: Summary of ground motion records used for dynamic analysis

The ground motions were scaled to match design spectrum based on ASCE/SEI 7-16 recommendations [9]. The ground motions were scaled using a constant scale factor to all acceleration ordinates of the motions such that the average value of the response spectra for the suite of all motions considered is not less than the design response spectrum for the site over the period range $0.2T$ to $1.5T$, where T is the natural period of the structure in its fundamental mode. The upper limit on the period is intended to account for period elongation due to inelastic actions and gap opening of the wall at its horizontal joints, and lower limit is intended to capture higher modes of response. The ASCE/SEI 7-16 design response spectrum for each site class at high seismic level (spectral response acceleration of $0.8g$) and scaled earthquake response acceleration spectra are given in *Figure 6*. The same approach was employed to scale the selected ground motions to low seismic level, which is $0.4g$.

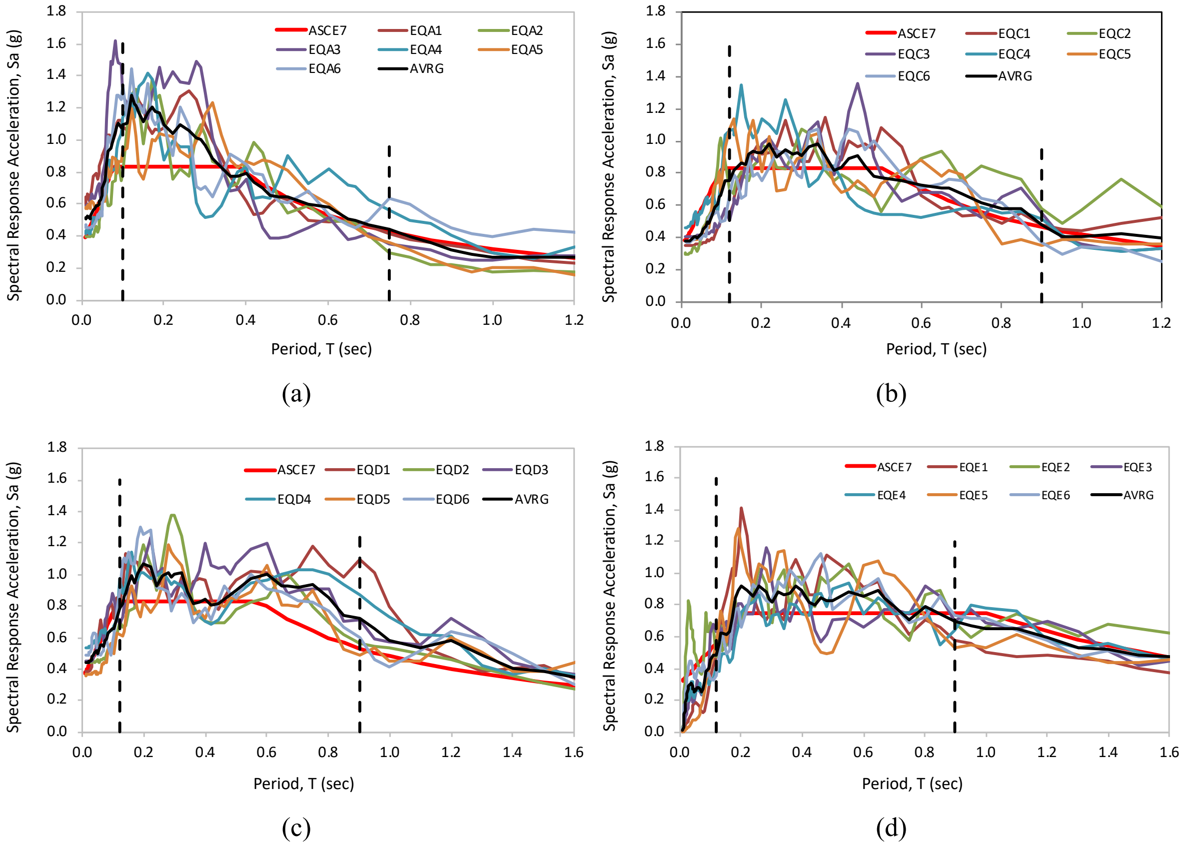


Figure 6: ASCE7 Design and spectral acceleration spectrum of selected ground motions at 0.8g

3 RESULTS AND DISCUSSIONS

The displacement amplification factor C_d was computed for the prototype buildings on the basis of computed building peak drift values using nonlinear time history analysis with the scaled ground motions. The C_d value was computed as the ratio of maximum drift to building drift corresponding to its design level lateral force. However, because the overstrength is neglected in the developed analytical models, the design level approximately corresponds to structure yielding. The computed C_d values for each site class and R value are given in *Figure 7* and *Figure 8* for seismic levels 0.4g and 0.8g, respectively. The results show that C_d value for prototype building increases with the increasing R value of the building. Large R values requires structural systems to be more ductile. Therefore, this observation is consistent with typical code values such as ASCE/SEI 7-16, where C_d value increases with increasing R value.

Figure 9 shows the computed average C_d value for each site class for both seismic levels. The average results show that structures located on sites with high seismic levels typically need to be designed with higher ductility capacities (larger C_d). However, the effects of seismic level and site class on C_d are not significant and probably can be ignored for design purposes. The only exception is Site Class E, which indicates that relatively larger C_d values are required as the site seismic level increases. However, because the structures designed with R values larger than 4.5 with high seismic level and 6.0 for low seismic levels failed (defined as maximum drift more than 2.5%), this observation is based on a limited number of structures. Finally, the average C_d values show that a C_d value that is equal to R value is conservative and reasonable for design purposes.

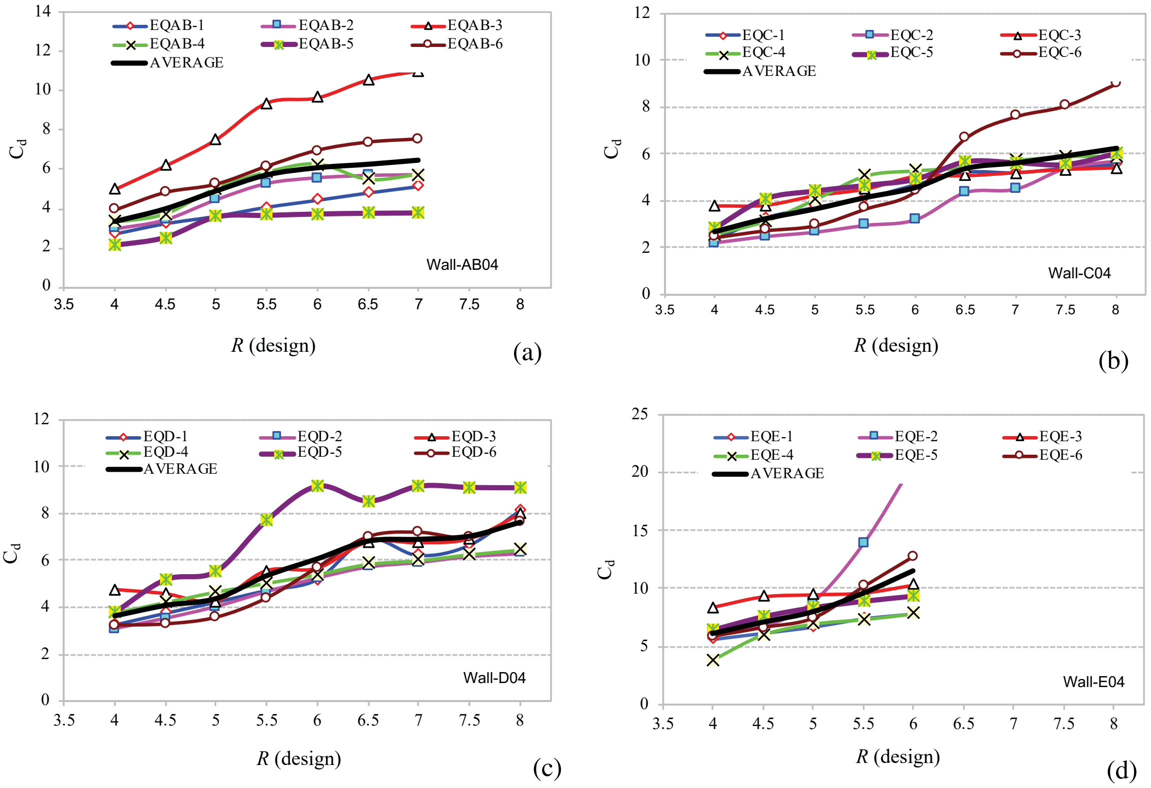


Figure 7: Computed C_d and R relationship at 0.4g for site class (a) AB, (b) C, (c) D, and (d) E

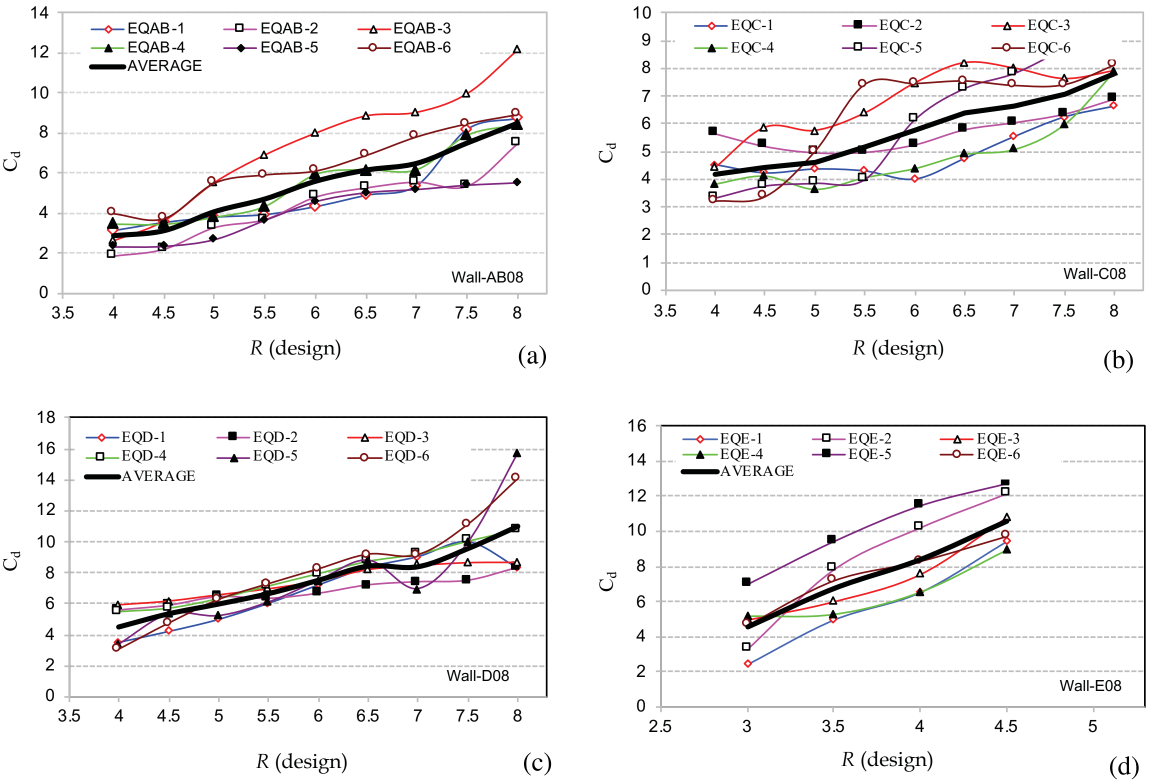


Figure 8: Computed C_d and R relationship at 0.8g for site class (a) AB, (b) C, (c) D, and (d) E

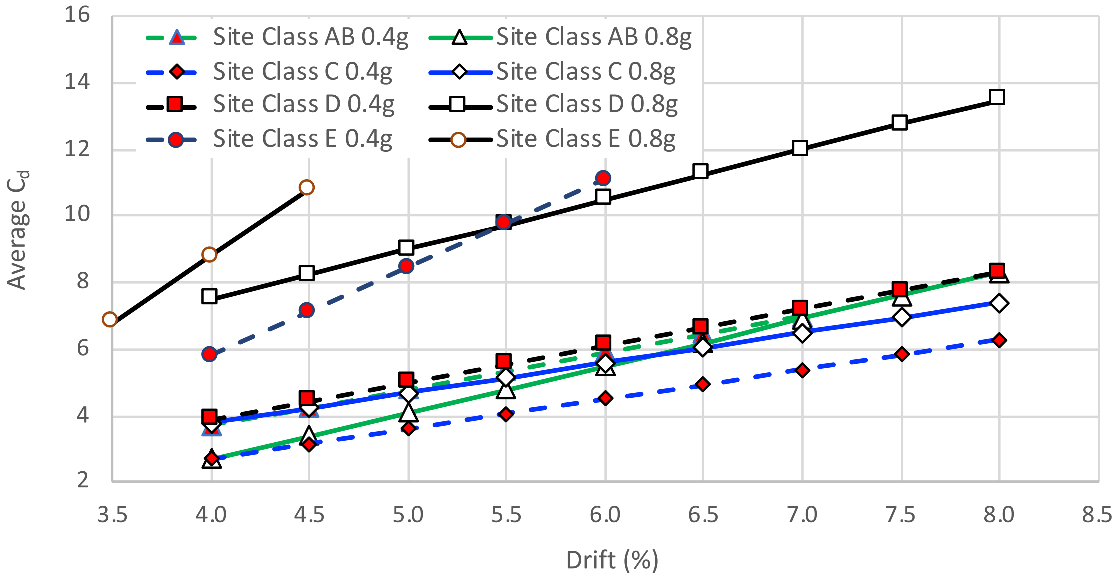


Figure 9: Average C_d and R relationship

4 CONCLUSIONS

A large number of prototype parking garage structures were designed for two seismic levels and several site classes. The structures were designed for values of response modification coefficient R ranging from 4.0 to 8.0. In total 24 recorded ground motions were used to perform nonlinear time history analysis of the designed structures. The computed maximum drifts were used to compute the required minimum displacement amplification factor C_d to ensure that structure does not collapse. Based on results presented the following main conclusions are made:

1. The seismic design parameters R and C_d are interrelated. For structures with larger R values the demand for C_d or ductility is also high.
2. Although demand for C_d increases as the site class become softer, the effects of site class are negligible for Site Class A, B, C, and D. However, structures located on Site Class E seems to have a requirement for larger values of C_d than those located on other site classes.
3. Increasing site seismic level increases the requirement for minimum value of C_d or building ductility, but the increase is typically not significant and can be ignored.
4. A value of C_d that is equal to building R value seems to be a reasonable design approach that envelope C_d requirements for structures presented in this study.

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