



DEVELOPMENT OF RESPONSE MODIFICATION COEFFICIENT AND DEFLECTION AMPLIFICATION FACTOR FOR DESIGN OF AAC STRUCTURAL SYSTEMS

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SUMMARY

This paper describes the development of a rational procedure to select the response modification coefficient (R) and the deflection amplification factor (C_d) for the seismic design of structures of autoclaved aerated concrete (AAC). In US seismic design codes, the coefficient R is intended to account for ductility, system over-strength and energy dissipation through the soil-foundation system. The factor C_d is used to convert elastic lateral displacements to total lateral displacements, including the effects of inelastic deformations.

For AAC shear-wall structures, values of the response modification coefficient (R) and the corresponding deflection amplification factor (C_d) have been proposed based on a combination of laboratory test results and numerical simulation. The test results are obtained from 14 AAC shear wall specimens and a two-story, full-scale AAC assemblage specimen tested under simulated gravity loads plus quasi-static reversed cyclic lateral loads representing the effects of strong ground motion. Using the results of those tests, conservative limits are proposed for the displacement ductility capacity and the drift ratio capacity of flexure-dominated AAC wall systems.

Those experimentally determined limits are then compared with the analytically predicted response of AAC structural systems subjected to suites of earthquake ground motions representative of design earthquakes in different regions of the United States. Analytical responses are predicted using nonlinear lumped-parameter models whose hysteretic characteristics are based on the experimentally observed responses. Using an iterative procedure

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similar to what would be used in design, four typical AAC shear-wall structures are designed using successively larger trial values of the response modification coefficient (R), until the response of the structure (either ductility or drift) exceeds the experimentally determined capacity. A lower fractile of those critical values, modified for probable structural overstrength, is taken as a reasonable value of 3 for R . Using an analogous procedure, a reasonable value of C_d is determined as 3.0. Those two selected values have been proposed for the seismic design of AAC shear-wall systems in the US.

INTRODUCTION

The seismic design philosophy of current United States building codes allows most structures to undergo inelastic deformations in the event of strong earthquake ground motions. As a result, the design lateral strength can be lower than that required to maintain the structure in the elastic range. In the International Building Code 2000 (IBC 2000 [1]), the response modification coefficient (R) is used to calculate the reduced design seismic forces of a structural system, and the deflection amplification factor (C_d) to convert elastic lateral displacements to total lateral displacements, including the effects of inelastic deformations. The values of R and C_d prescribed in the IBC 2000 [1] are based on observations of the performance of different structural systems in previous strong earthquakes, on technical justification, and on tradition (NEHRP 2000 [2]). The coefficient R is intended to account for ductility, system over-strength and energy dissipation through the soil-foundation system (NEHRP 2000 [2]).

Some research has been completed on the seismic behavior of autoclaved aerated concrete (AAC) walls, primarily focusing on the behavior of walls of AAC masonry-sized units. For example, one research project (de Vekey [3]) studied the performance of AAC wallettes and walls under lateral loads to study the effect of thickness, moisture content, and specimen size on the flexural strength. In another research project, the out-of-plane flexural behavior of non-load bearing AAC walls constructed with blocks in running bond was studied, to investigate the flexural strength of the walls parallel or perpendicular to the bed joints (Al-Shaleh [4]).

The literature review conducted on the behavior of AAC walls shows that there is insufficient prior research on the seismic performance of AAC structures to develop seismic design provisions or analytical models to predict the behavior of AAC shear-wall structures under earthquake ground motions. Sufficient information, however, has been acquired to permit the development of design provisions in areas with low seismic risk, such as Florida and Texas. Because there is insufficient prior research to verify the seismic performance of AAC structures, the selection of the seismic factors (R) and (C_d) for AAC structures needs to be based on laboratory test results and the numerical simulation of the behavior of AAC structures subjected to earthquake ground motions representative of different seismic zones of the United States.

Several research studies have been conducted on the selection of response modification factors (R) for the seismic design of structures. For example, Miranda [5] presents a summary of different investigations on the coefficient R , described in that study as a strength reduction factor (R_μ). Results from those different investigations were reviewed, and existing equations for R_μ were presented in a common format for a better comparison among them. Those

equations were in general based on the response of nonlinear single-degree-of-freedom systems subjected to real and synthetic earthquake ground motions. The extrapolation of these results to multi-degree-of-freedom systems required a relationship between local and global ductilities. The study of Miranda [5] suggests that the factor (R_{μ}) is mainly a function of the displacement ductility (μ), the natural period of the structure (T), and the soil conditions. One conclusion of that study is that the use of strength reduction factors based on ductility, period and soil conditions together with the evaluation of structural overstrength factors, and relationships between local and global ductility demands are needed to establish rational seismic design approaches.

Even though the equations presented by Miranda [5] seem reasonable and may be incorporated in future United States seismic codes, the reality is that today single values of the coefficient (R) are still proposed in those seismic codes to design different structural systems. Therefore, given this limitation, a rational procedure should be developed to select a single value of the coefficient R for the seismic design of AAC shear-wall structures in the United States. This procedure should address the behavior of AAC structures modeled as multi-degree-of-freedom systems, using a large number of real and synthetic suites of earthquake ground motions representative of different seismic regions of the United States.

The objective of this paper is to present the development and application of a rational procedure to select the response modification coefficient (R) and the deflection amplification factor (C_d) for the seismic design of autoclaved aerated concrete (AAC) shear-wall structures in the US.

PROCEDURE FOR SELECTING THE DUCTILITY FACTOR R_d

The coefficient (R) defined in the IBC 2000 is the product of the ductility reduction factor (R_d) and the structural overstrength factor (Ω_{system}) (NEHRP 2000[2]). An iterative procedure to select the ductility reduction factor (R_d) is presented in this section; the overstrength factor (Ω_{system}) is addressed later. The procedure to select R_d is explained for AAC shear-wall structures, and can be applied to other structural systems as well: (1) Select an AAC shear-wall structure. (2) Analyze the AAC structure using the modal analyses procedure specified in the IBC 2000 [1]. The elastic global drift ratio of the structure should be less or equal to 1%, and the flexural capacity of the walls and coupling beams if any, equal to the bending moments obtained from the elastic analyses ($R_d = 1$). (3) Select an earthquake from a suite of earthquakes representing the design spectrum. (4) Select a value of R_d greater than one, and redesign the structure for a reduced flexural capacity. For example, if R_d is selected as 2, then the required flexural capacity is reduced by a factor of 2. (5) Run a dynamic nonlinear analysis and calculate the drift ratio and displacement ductility demands. If the drift ratio demand is equal to 1%, the value of R_d assumed is the critical value of R_d based on drift ratio, similarly, if the displacement ductility demand is equal to 3.5, the assumed value of R_d is the critical value based on displacement ductility. (6) Repeat for other earthquakes of the same suite, for other suites of earthquakes, and AAC shear wall structures.

The general procedure for selecting values of the factor R_d based on drift ratio capacity and displacement ductility capacity is presented as a flow chart in Figure 1. Each step is explained in detail in the following sections.

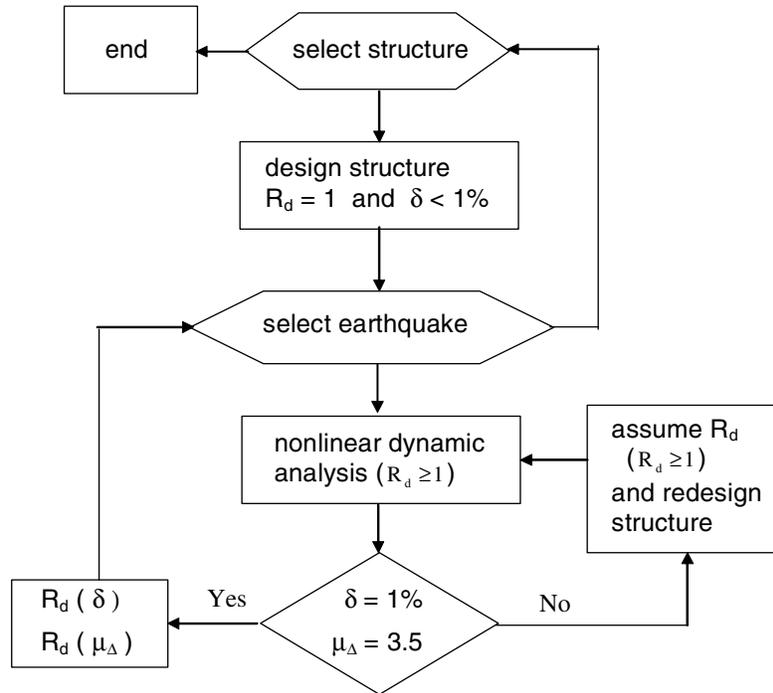


Figure 1 Procedure for selecting the factor R_d

SELECTION AND DESIGN OF AAC SHEAR-WALL STRUCTURES

Four AAC structures were selected for evaluation under earthquake ground motions from different seismically active regions of the United States. The four structures were selected as AAC shear-wall structures because shear walls are the major AAC structural elements resisting seismic forces. The AAC structures selected were a three- and a five-story cantilever-wall structure, and a three- and a five-story coupled-wall structure. Typical wall dimensions of 240 in. (6.1 m) long, 120 in. (3 m) high and 10 in. (0.25 m) thick were used in every story of each structure. The coupled-wall structures consisted of two cantilever walls connected by coupling beams at every story. All coupling beams were 48 in. (1.2 m) long, 40 in. (1 m) wide and 10 in. (0.2 m) thick. Slabs were made of AAC planks 10 in. (0.25 m) thick.

The structures were modeled as planar structures. A tributary width of 240 in. (6.1 m) was assumed to calculate the weights of each story. Design spectra for the seismic regions studied were calculated using site classes consistent with the suite of earthquakes selected. Elastic analyses were carried out using the program SAP2000 (SAP2000 [6]), with a reduced initial stiffness consistent with that used in the nonlinear analyses as presented later. Flexural capacities of walls and coupling beams are assumed equal to the bending moments obtained from the elastic modal analyses as described in the procedure to select the factor R_d . Actual bar

sizes for flexural reinforcement are not selected, to avoid introducing element overstrength. That issue is considered later in the selection of the system overstrength factor (Ω_{system}).

SELECTION OF EARTHQUAKE GROUND MOTIONS

Different suites of earthquake ground motions were selected based on areas with high potential for seismic activity. For the central and eastern US, three suites of earthquakes were selected: Charleston, SC; Carbondale, IL; and Memphis, TN. For the western US, two suites of earthquakes were selected: Los Angeles, CA; and Seattle WA. Each suite of earthquakes consists of ten earthquake ground motions and corresponds to 2% probability of exceedance in 50 years.

A model created by Frankel [7] was used to develop synthetic ground motions representative of the B-C soil class interface of Charleston, SC. The suite of earthquakes for Charleston, SC used in this project was taken from that work. Projects RR-1 and RR-2 of the Mid-America Earthquake Center (MAE) involved the development of uniform hazard spectra and synthetic ground motions for three major Mid-American cities: Carbondale, IL, Memphis, TN, and St. Louis, MO (Wen [8]). The ground motions used in this project for Carbondale, IL and Memphis, TN were taken from those projects RR-1 and RR-2. The selected suites are representative of the Soil Profile of Carbondale, IL and Memphis, TN. The SAC Phase 2 Steel Project provided suites of earthquake ground motions for three United States cities: Boston, MA, Los Angeles, CA and Seattle, WA (Somerville [9]). The suites of earthquakes for Los Angeles, CA and Seattle, WA used in this project were taken from that SAC project. The selected suites are representative of Soil Class D.

The selected five suites of earthquakes were scaled to represent the design seismic forces. Acceleration response spectra were calculated for each entire suite of earthquakes and compared with corresponding design spectra. For Charleston, Carbondale, and Memphis, acceleration response spectra were compared with corresponding IBC 2000 [1] Site Class C design spectra and for Los Angeles and Seattle, with IBC 2000 [1] Site Class D design spectra. Each entire suite was scaled using a single scaling factor calculated as follows: (1) Calculate the elastic response spectra for the suite of earthquakes. (2) Calculate the mean spectral accelerations of the response spectra for periods of 0.26 seconds and 0.62 seconds. In this step, periods of 0.26 seconds and 0.62 seconds are used because they represent the natural periods of the three-story and five-story AAC shear-wall structures studied. (3) Calculate a scaling factor for each period as the design spectral acceleration divided by the average spectral acceleration.

The final single scaling factor is the average of the two scaling factors calculated in Step 3. Two scaling factors, however, were used for the suite of Charleston because of the large difference between the two scaling factors calculated in Step 3. Table 1 shows the scaling factors selected for each of the suites of earthquakes studied.

Table 1 Scaling factors for each of the suite of earthquakes studied

Suite of Earthquakes	Site Class	Scaling Factor	Scaling Factor	Single Scaling Factor
		$T_n=0.26$ sec	$T_n=0.62$ sec	
Los Angeles	D	0.63	0.63	0.63
Seattle	D	0.55	0.47	0.51
Carbondale	C	0.60	0.58	0.59
Memphis	C	0.92	0.77	0.84
Charleston	C	0.81	1.16	--

MAXIMUM DRIFT RATIO AND DISPLACEMENT DUCTILITY CAPACITIES FOR AAC SHEAR-WALL STRUCTURES

The proposed procedure for selecting the ductility reduction factor (R_d) is based on a maximum drift ratio and displacement ductility capacity. The main objective on including drift and ductility capacities is to provide reasonable limits to avoid collapse of AAC shear-wall structures during severe earthquake ground motions. The drift ratio capacity is considered to limit damage and differential movement in AAC shear-wall structures, and the displacement ductility capacity to control the amount of inelastic deformation in those structures.

The maximum drift ratio and displacement ductility capacities are based on the drift ratios and displacement ductilities observed from six AAC flexure-dominated specimens (Table 2). These walls were tested under simulated gravity loads plus quasi-static reversed cyclic lateral loads representing the effects of strong ground motions (Varela [10] and Tanner [11]). The aspect ratio and normalized axial force variations of the specimens represent in general those expected in potential walls of AAC shear-wall structures up to five stories high (Varela [10]).

In Table 2, $\mu_{\Delta-on}$ and $\mu_{\Delta-os}$ represent the displacement ductilities observed in the south and north directions. Similarly δ_{os} and δ_{on} represent the maximum global drift ratios in the south and north directions. Although specimens were subjected to the same drifts in each direction during the displacement-controlled portion of each test, the selected maximum horizontal displacement capacity in each direction could be limited by strength degradation as noted above, and therefore could be different in each direction.

The displacement ductility was defined in this study as a maximum selected horizontal displacement divided by the corresponding displacement at yielding of the flexural reinforcement. The maximum global drift ratio was defined as the maximum selected horizontal displacement divided by the total height of the AAC structure. The selected horizontal displacement was based on one of the following criteria: (1) a degradation in the capacity of the AAC wall of more than 10%; or (2) a change in the shape of the hysteretic loop from the corresponding previous load cycle, for example, a large reduction in the energy dissipated.

A value of maximum drift ratio capacity of 1% was proposed to avoid collapse of AAC shear-wall structures. This value corresponds to the minimum observed selected drift ratio of the flexure-dominated specimens (Table 2). The maximum drift ratio of 0.4% for Shear Wall

Specimen 16 (not shown in Table 2) was not considered because this low value of drift ratio was associated with failure of the joint between the vertical panel and the U blocks which can be eliminated using Heli-fix ties[®], walls with flanges, or both (Varela [10]). This value of 1% was not based on a lower fractile because of the large dispersion observed in the selected drift ratios for each flexure-dominated specimen.

A value of maximum displacement ductility capacity of 3.5 was proposed to avoid collapse of AAC structures. This value corresponds to the 10% lower fractile of the selected displacement ductilities of the flexure-dominated specimens (Table 2). The maximum displacement ductility of 1.67 for Shear Wall Specimen 16 (not shown in Table 2) was not considered for the same reasons presented in the selection of the maximum drift ratio capacity.

Table 2 Maximum drift ratios and displacement ductilities for the flexure-dominated specimens not including those of Shear Wall Specimen 16 in the north direction

Specimen Number	$\mu_{\Delta-os}$	$\mu_{\Delta-on}$	δ_{os} (%)	δ_{on} (%)
13	4.8	4.0	1.4	1.1
14a	5.0	--	1.9	--
14b	5.4	2.6	1.9	1.0
15a	6.0	5.8	1.0	1.0
15b	4.8	4.8	1.0	1.0
16	5.0	--	1.0	--
	Average	4.8	Average	1.2
	COV	0.20	COV	0.32
	10% Lower Fractile	3.6	10% Lower Fractile	0.7

NONLINEAR ANALYSIS

In this study, the nonlinear analysis program CANNY 99 (CANNY 99 [12]) was selected to evaluate the performance of the four AAC shear-wall structures subjected to the different suites of earthquake ground motions. Structures in that program are idealized as rigid nodes connected by line elements and springs. All structural elements are treated as massless line elements represented by their centroidal axes, with mass concentrated at the nodes or at the center of gravity of floors.

The idealized wall element of that program considers the wall as a line element located at the wall centerline. The wall element is idealized using two nonlinear flexural springs, two rigid links, one nonlinear shear spring and one axial spring. The nonlinear flexural springs are located at the top and bottom of the wall centerline. Therefore, all nonlinearity is concentrated at the wall ends (lumped nonlinearity).

The five- and three-story cantilever-wall structures were modeled using five and three idealized wall elements respectively. Each wall of the five-story coupled wall structure was modeled

using five idealized wall elements, and each wall of the three-story coupled-wall structure was modeled using three idealized wall elements.

SELECTION OF PARAMETERS FOR THE NONLINEAR BEHAVIOR OF WALLS

The hysteretic model selected to represent the behavior of the nonlinear flexural and shear springs was the CANNY CA7 model which uses user-input hysteretic parameters to define the loading and unloading branches, degradation of strength and stiffness, and pinching of the hysteretic loops. The behavior of the nonlinear flexural spring is defined by a bilinear moment-rotation curve and the nonlinear shear spring by a bilinear force-displacement curve. The behavior of the axial spring was defined by the elastic model EL1 of CANNY 99 [12].

Based on the observed behavior of the six flexure-dominated walls, the hysteretic curve of the nonlinear flexural spring was defined as follows: (1) the initial stiffness is defined using the modulus of elasticity of AAC and a reduced moment of inertia equal to 40% of the gross moment of inertia of the wall; (2) the post-yielding stiffness is selected as 1% of the initial stiffness for the three-story structures and 0.5% for the five-story structures (Varela [10]); and (3) the degradation of the unloading stiffness is defined using a hysteretic parameter θ of 1 (CANNY 99 [12]). Strength degradation and pinching are not included because they were not observed up to a global drift ratio of 1% and a displacement ductility of 3.5%, and because the proposed unloading stiffnesses after yielding of the flexural reinforcement for each of the flexure-dominated specimens were calculated fitting a straight line on the observed unloading curve.

Based on the observed behavior of eight shear-dominated, the hysteretic curve of the nonlinear shear spring was defined as follows: (1) the initial stiffness is defined using the shear modulus of AAC and a reduced area equal to 40% of the gross area of the wall; (2) the stiffness after shear cracking is selected as 1% of the initial stiffness; (3) the degradation of the unloading stiffness is defined using a hysteretic parameter θ of 1; and (4) the degradation of the shear strength is defined using a hysteretic parameter λ_u of 0.3 and λ_e of 0 (CANNY 99[12]). Pinching of the hysteretic loops is not included because this phenomenon was not observed in all the shear-dominated walls.

PROPOSED VALUE OF R FOR AAC SHEAR-WALL STRUCTURES

The procedure described above to select the ductility reduction factor (R_d) was carried out for the four selected structures using the suites of earthquakes representative of Charleston, Carbondale, Memphis, Los Angeles, and Seattle. In most cases values of R_d of 1, 2, 3 and 4 were assumed in the proposed procedure. If the drift ratio or the displacement ductility demands changed significantly between two consecutive values of R_d , a new value of R_d equal to the average of those values was assumed, for example values of R_d of 2.5 and 3.5. Linear interpolation was used among those values to calculate critical values of R_d , (values of R_d that make the global drift ratio and displacement ductility demands equal to the maximum global drift ratio and displacement ductility capacities). A mean value of the factor R_d was selected for each different structure and suite of earthquakes, as the minimum value between the average critical values of R_d based on global drift ratio and displacement ductility capacities. In all cases

the critical value of R_d based on displacement ductility was smaller than that based on global drift ratio. In few cases during the nonlinear analyses, the global drift ratio demand for a value of R_d of 1 was greater than the global drift ratio capacity of 1%. Therefore, for those particular cases, values of R_d based on that global drift ratio were not selected. Table 3 presents the selected mean values of R_d based on displacement ductility for the different structures and suites of earthquakes.

The mean values of R_d presented in Table 3, for the three- and five-story cantilever-wall structures were smaller than those corresponding to the three- and five-story coupled wall structures. The reason is that the maximum inelastic displacement and displacement ductility demands for the cantilever-wall structures were greater than those corresponding to the coupled-wall structures. Mean values of R_d for the three-story structures were smaller than those corresponding to the five-story structures. This can be attributed to the following: (1) in the short-period range, the nonlinear response of the structure increases rapidly; and (2) the large dispersion among the spectral accelerations and the design spectral acceleration observed for a period of 0.26 seconds compared with that observed for a period of 0.62 seconds in the suites of earthquakes studied.

Based on the 10% lower fractile value of the mean values of R_d presented in Table 3, a value of R_d of 2 has been proposed for flexure-dominated AAC shear-wall structures. The approach adopted here was to select a value of R_d that would result in structural failure (exceedance of drift or ductility capacities) less than 10% of the time under suites of earthquakes representing in average the design spectra.

Table 3 Selected mean values of R_d based on drift ratio and displacement ductility capacities for different structures and suites of earthquakes

Suite of Earthquakes	Structure	Mean R_d	Structure	Mean R_d
Los Angeles	5-story cantilever	2.37	5-story coupled walls	2.48
Seattle	5-story cantilever	2.67	5-story coupled walls	2.92
Carbondale	5-story cantilever	2.80	5-story coupled walls	3.07
Memphis	5-story cantilever	2.46	5-story coupled walls	2.66
Charleston	5-story cantilever	2.93	5-story coupled walls	2.96
Los Angeles	3-story cantilever	1.95	3-story coupled walls	2.19
Seattle	3-story cantilever	2.15	3-story coupled walls	2.52
Carbondale	3-story cantilever	2.26	3-story coupled walls	2.4
Memphis	3-story cantilever	2.20	3-story coupled walls	2.43
Charleston	3-story cantilever	2.96	3-story coupled walls	3.19
Average				2.58
COV				0.14
10% lower fractile				2.13

The procedure presented in this study to select R_d is based on the design-level earthquake rather than the maximum considered earthquake because it was neither convenient nor safe to test large specimens to the point of collapse. Because of conservatism intentionally introduced in each

key step of the process, and justified below, the results are valid for the maximum considered earthquake as well.

For each flexure-dominated specimen, “failure” was determined as a reduction of about 10% in maximum capacity in a particular direction, compared with the maximum capacity in a previous cycle in the same direction. This criterion is obviously much more conservative (is triggered much earlier) than collapse of the system. Using that criterion, the average displacement ductility capacity (removing all effects of sliding) is 4.8. A lower 10% fractile of that capacity of 3.5 was used in establishing the trigger for the subsequent analytical work carried out in this study.

Using CANNY 99 [11], four typical AAC structural systems were subjected to 10 earthquake ground motions each. The systems represent realistic lower and upper bounds to the periods of AAC structural systems that can be used in practice. The suites of ground motions included recorded and synthetic motions, and included the suite used in the SAC study of steel moment frames. Each suite of ground motions was normalized to the design spectrum at two representative periods.

Displacement and ductility demand were calculated using the analytical models and the suites of ground motions. Ductility demand governs for this system. The value of R_d (no system overstrength) at which ductility demand reached the conservatively observed ductility capacity of 3.5, was 2.58 (Table 3). A lower fractile value of 2.13 was conservatively selected, and an even lower value of 2.0 was then recommended for convenience. In essence, neglecting possible system overstrength, the average ratio of actual ductility capacity of 4.8 (Table 2) to assumed ductility capacity (3.5) is 1.38; the average ratio of critical R_d (2.58) to proposed R_d (2.0) is 1.29. Both factors introduce conservatism; their combined effect is given by their product, or 1.78. This is larger than the difference between the design earthquake and the maximum considered earthquake in either the western or the eastern US.

The system overstrength factor (Ω_{system}) is the product of independent overstrength factors (NEHRP 2000 [2] and Uang [13]) defined as follows: (1) development of sequential plastic hinges in redundant structures; (2) material strengths higher than those specified in design; (3) strength reduction factors; (4) specified sections and reinforcement patterns greater than those required in design; (5) nonstructural elements; and (6) variation of lateral forces.

Independent overstrength factors are proposed for AAC shear-wall structures as follows: (1) Assume that plastic hinges at the base of the walls would form at the same time; that is, the redundancy factor would be equal to 1; (2) Assume actual yield strength of the flexural reinforcement 10% higher than that specified in design; (3) Assume a strength reduction factor for flexural design of walls equal to 0.9. This corresponds to an independent overstrength factor of 1.1; (4) Assume a selected amount of flexural reinforcement 10% greater than that required in design. (5) Ignore participation of nonstructural elements; (6) The minimum design seismic forces specified in the IBC 2000 [1] for the four selected structures were at least 20% greater than those obtained from the elastic modal spectral analysis. Two probable reasons are: (1) the

static analysis is a simplification of the modal spectral analysis; and (2) cracked properties of the walls were used in all modal spectral analyses.

The product of the above independent overstrength factors is equal to 1.6. A value of system overstrength factor (Ω_{system}) of 1.5 has been proposed for AAC shear-wall structures.

Using the proposed ductility reduction factor (R_d) of 2 and the system overstrength factor (Ω_{system}) of 1.5, a value of the response modification coefficient (R) of 3 has been proposed for the seismic design of flexure-dominated AAC shear-wall structures in the US. This value of R of 3 is equal to the value of R for detailed plain concrete shear walls, and is 20% greater than the value of R for ordinary reinforced and detailed plain masonry shear walls prescribed in the IBC 2000 [1].

PROPOSED VALUE OF THE FACTOR C_d FOR FLEXURE-DOMINATED AAC SHEAR-WALL STRUCTURES

The value of the deflection amplification factor C_d is defined as the maximum nonlinear displacement during an earthquake (D_{max}), divided by the elastic displacement (D_s) calculated using reduced seismic design forces (NEHRP 2000 [2]) as presented in Figure 2.

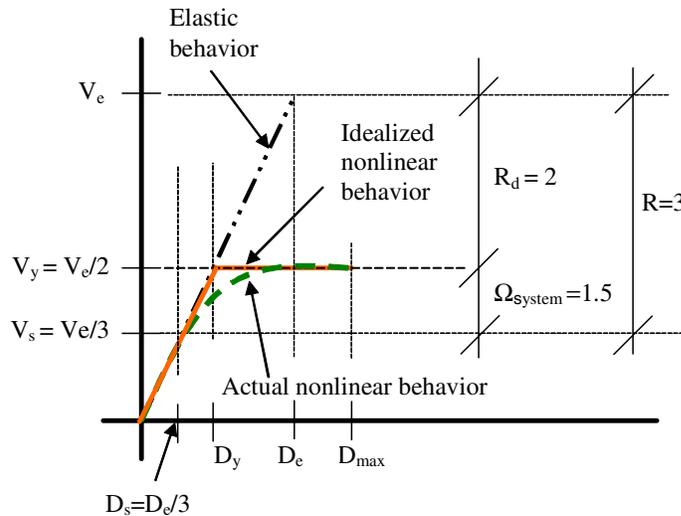


Figure 2 Maximum inelastic displacement and elastic displacements associated with a value of the factor R of 3

In Figure 2, V_e is the elastic design lateral force associated with a value of R of 1; V_y is the lateral force at which significant yield is observed in the structural system; and D_e and D_y are the elastic displacements calculated using V_e and V_y respectively. The factor C_d can be calculated as shown in Equation (1).

$$C_d = \frac{D_{\text{max}}}{D_y} \Omega_{\text{system}} \quad (1)$$

Setting the ratio D_{max}/D_y equal to the amplification parameter C_{du} , the displacement amplification factor (C_d) is given by Equation (2).

$$C_d = C_{du} \Omega_{system} \quad (2)$$

Equation (2) shows that the factor C_d depends on the selection of the amplification factor (C_{du}) and the system overstrength factor (Ω_{system}). Using the results of the dynamic nonlinear analyses carried out with a value of R_d of 2, mean critical values of C_{du} were calculated for each different AAC structure and suite of earthquakes studied. Each critical value of C_{du} was defined as the maximum nonlinear displacement divided by the elastic displacement calculated using reduced seismic forces ($R_d = 2$). The mean critical values of the factor C_{du} for the different structures and suites of earthquakes are presented in Table 4.

The 10% lower fractile, average, and 10% upper fractile values of the calculated mean critical values of C_{du} were equal to 1.86, 2.32, and 2.79 respectively. A value of C_{du} of 2 was selected as amplification factor for flexure-dominated AAC shear wall structures. Using this proposed value of C_{du} of 2, and the proposed value of Ω_{system} of 1.5, a value of C_d of 3 was proposed for the seismic design of flexure-dominated AAC shear wall structures in the US.

Table 4 Mean critical values of the factor C_{du} for different structures and suites of earthquakes

Suite of Earthquakes	Structure	Mean C_{du}	Structure	Mean C_{du}
Los Angeles	5-story cantilever	2.77	5-story coupled walls	2.49
Seattle	5-story cantilever	2.31	5-story coupled walls	2.14
Carbondale	5-story cantilever	2.08	5-story coupled walls	1.95
Memphis	5-story cantilever	2.27	5-story coupled walls	2.09
Charleston	5-story cantilever	2.08	5-story coupled walls	1.93
Los Angeles	3-story cantilever	3.01	3-story coupled walls	2.96
Seattle	3-story cantilever	2.11	3-story coupled walls	2.42
Carbondale	3-story cantilever	2.58	3-story coupled walls	2.89
Memphis	3-story cantilever	2.35	3-story coupled walls	2.34
Charleston	3-story cantilever	1.91	3-story coupled walls	1.85
Average				2.32
COV				0.16
10% lower fractile				1.86
10% Upper fractile				2.79

The value of R of 3 proposed for the seismic design of flexure-dominated AAC shear-wall structures was based on a 10% lower fractile value to be conservative in selecting the final design seismic forces. The value of C_d , however, should be based on an upper fractile value to be conservative in the estimation of the maximum inelastic displacements. If the factor C_{du} is based on the 10% upper fractile value of 2.79 and on the value of Ω_{system} of 1.5, then the value of C_d would be greater than the proposed value of R of 3. A value of C_d of 3 has been proposed for the seismic design of flexure-dominated AAC shear-wall structures to be consistent with the relationship between the values of R and C_d for other structural systems in the IBC 2000 [1] (for example, values of R are in most cases greater than or equal to those values of C_d).

CONCLUSIONS

Using a rational iterative procedure based on experimental and analytical results, values of the factors R and C_d have been proposed for the seismic design of AAC shear-wall structures in the US. The proposed value of the factor (R) is the product of the ductility factor (R_d) and an overstrength factor Ω_{system} , and is equal to 3. The proposed value of C_d is a function of the proposed value of (R) and the overstrength factor $\Omega_{\text{overstrength}}$, and is also equal to 3.

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