13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 939

ESTIMATING ROTATIONAL DEMANDS IN HIGH-RISE CONCRETE WALL BUILDINGS

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SUMMARY

Results from numerous nonlinear dynamic analyses on high-rise concrete buildings, ranging in height from 120 to 480 ft, were used to develop simplified procedures for estimating maximum inelastic wall rotations and maximum coupling beam chord rotations. The results indicate that, due to higher mode effects and forces applied by coupling beams, maximum rotations in slender cantilever walls and in coupled walls usually do not occur at the same time as the maximum displacement. However, it is reasonable to estimate maximum inelastic rotation from maximum total displacement using a fictitious elastic displacement, which is proportional to actual wall strength to elastic demand ratio. Due to coupling beams "pulling back" on the coupled walls, the "elastic displacements" of coupled walls are smaller than cantilever walls. The maximum coupling beam rotation depends on the wall slope and floor slope at the critical level. A simplified procedure that gives reasonable results is to assume that the combination of wall and floor slope at the critical level is equal to the maximum global drift.

INTRODUCTION

Concrete walls in high-rise buildings are often located around the perimeter of elevator and stair shafts. Access into elevators and stairways requires large openings in the walls, and as a result, the lateral-force-resisting system consists of a number of large vertical wall segments interconnected by small horizontal wall segments (coupling beams) above and below the openings. Figure 1 shows an example of four walls arranged in a rectangular building core with door openings on two sides. In this case, the system consists of two cantilever walls and two coupled walls. Coupled walls are designed to act like a frame with very strong columns (walls) and weak coupling beams. Most of the inelastic deformation of a coupled wall system occurs within the coupling beams, and this reduces damage in the vertical wall segments, which are also part of the gravity-load-resisting system.

Rotational Demand in Walls

To ensure that a concrete wall has adequate displacement (drift) capacity, the inelastic rotational demand on the wall must not exceed the inelastic rotational capacity of the wall. The inelastic rotational capacity is estimated from the product of the plastic hinge length and the inelastic curvature capacity of the wall. The latter is determined from a plane sections (flexural) analysis of the wall for the given level of axial compression and with the maximum compression strain of concrete limited to between 0.003 and 0.004

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for unconfined concrete. The inelastic rotational capacity of a wall is increased by adding special boundary elements (confinement) at the edges of the wall.

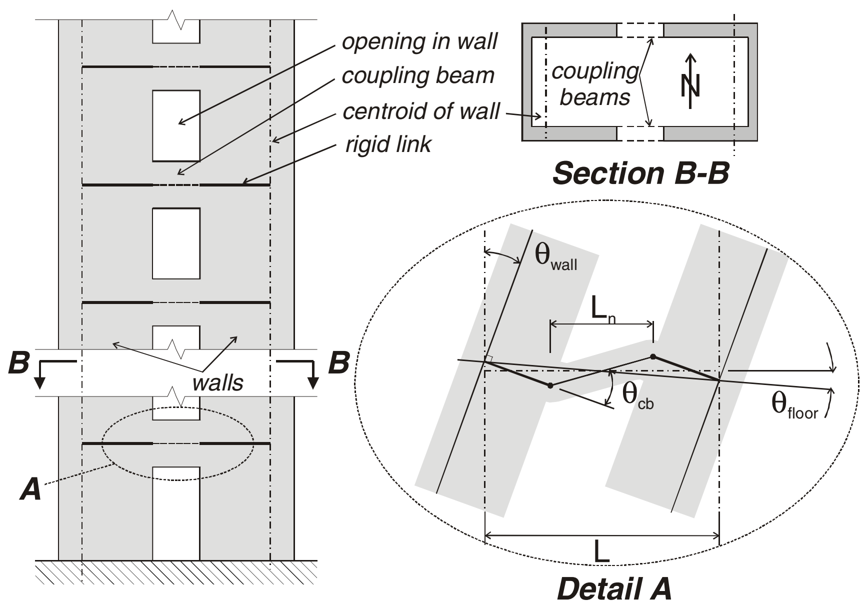

Figure 1 Illustration of an idealized rectangular building core and the geometry associated with coupling beam rotations

The inelastic rotational demand, on the other hand, is determined from the total displacement demand of the wall. The relationship between total displacement and inelastic rotation is summarized in Figure 2. The total displacement is made up of an elastic portion and an inelastic portion. The inelastic rotation θ_i is equal to the inelastic displacement Δ_i at the top of the wall divided by the effective height of the wall h_w' above the center of the plastic hinge as shown in Figure 2. The inelastic rotation of the hinge is equal to the inelastic drift of the wall.

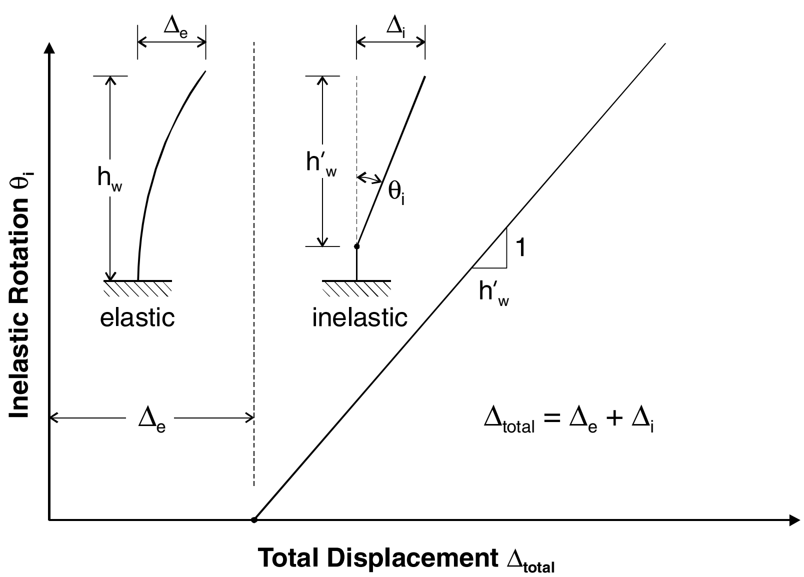

Figure 2 Relationship between inelastic rotation and total displacement of cantilever concrete walls showing the elastic and inelastic portions of the displacement

The challenge is determining what portion of the total wall displacement is inelastic or conversely, what portion is elastic. The general procedure in the 1997 Uniform Building Code (UBC [1]) for evaluating whether special boundary zones are needed, takes the elastic portion equal to the yield displacement of the wall. This approach is consistent with that suggested by Paulay [2], who further specified that the yield displacement be calculated assuming a first-mode curvature distribution (Paulay [3]). He presented the following simple expression for the elastic portion of the total displacement of a cantilever wall in terms of the wall height h_w, and the yield curvature ϕ_y, which is inversely proportional to the wall length.

$$\Delta_e = \Delta_y = 0.28\varphi_y h_w^2 \tag{1}$$

Equation (1) predicts that the elastic displacement increases dramatically as a wall becomes more slender. Prior to the current study, it was not known whether Equation (1) is appropriate for estimating the elastic portion of the displacement of slender cantilever walls in high-rise buildings.

The 2002 ACI 318 building code (ACI [4]) contains a simplified procedure for determining the inelastic rotational demand on a concrete wall when determining if special boundary zones are required. The procedure is based on the simplifying assumption that the inelastic drift (which is equal to the inelastic rotation) $\Delta_i / h_w' = \theta_i$ is equal to the total global drift Δ_{max} / h_w (Wallace [5]). Prior to the current study, it was not known if this simplified procedure, which was first introduced by Moehle [6] for bridge columns, is also suitable for slender cantilever walls in high-rise buildings.

Rotational Demand in Coupling Beams

Concern has been raised (e.g., Harries [7]) that the rotational demands on coupling beams may exceed the rotational capacities. Building codes, such as ACI 318 or the Canadian concrete code A23.3, provide guidelines on how to detail coupling beams, but do not specify any displacement or rotational limits. The NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273 [8]) contains suggested limits on acceptable plastic hinge rotations for all types of structural components including coupling beams of concrete shear walls. According to FEMA 273, the maximum chord rotation of diagonally reinforced concrete coupling beams should to be limited to 0.03 for the collapse prevention performance level.

Recently, two high-rise buildings located near Seattle area were designed with coupled walls as the lateral-force-resisting system (Mutrie et al. [9]). The steel braced frame alternative for one of the buildings was found to be $2 million more expensive to construct. The coupled walls were considered to be an "undefined system" according to UBC as the buildings were well over 400 ft high, and did not have a moment resisting frame capable of resisting 25% of the earthquake base shear. Non-linear dynamic analysis was used to determine the maximum chord rotation, and as it turned out, the geometry of the coupled wall system had to be modified to meet the FEMA 273 chord rotation limit.

This recent experience suggests that building codes should include a requirement to evaluate the rotational demand on coupling beams. As it is not practical for designers to undertake non-linear dynamic analysis as part of the design of every coupled wall building, a simplified procedure is required to estimate the chord rotation of coupling beams.

The deformations of a coupled wall system are summarized in Detail A of Figure 1. The total coupling beam rotation at a particular level in the structure can be calculated from:

$$\theta_{cb} = \left(\theta_{wall} - \theta_{floor}\right) \times \frac{L}{L_n} \tag{2}$$

where θ_{wall} is the slope of the walls at that level, θ_{floor} is equal to the difference in vertical displacement at the wall centroids divided by L, which is the distance between the wall centroids, and L_n is the clear span of the coupling beam. Equation (2) gives the total chord rotation, i.e., both the elastic and inelastic portions. While rotational limits are usually in terms of inelastic rotations, it is reasonable for coupling beams to consider rotational limits in terms of total rotation. Due to the short span-to-depth ratio of coupling beams, which are bent in double curvature, the elastic chord rotation is very small. Also, the assumption used for the plastic hinge length will have a very significant effect on the calculated plastic rotation. FEMA 273 specifies chord rotation limits in terms of total rotation.

L and L_n in Equation (2) are known for a given wall geometry. The issue that remains is what values of wall slope and floor slope should be used to estimate maximum coupling beam rotation. For first-mode dominated walls, the wall slope and floor slope can be estimated from a first-mode pushover analysis to a target displacement determined from a linear dynamic analysis. The procedure is somewhat more complex for taller walls.

The objectives of the current study were to develop simplified methods for estimating the maximum inelastic rotation (drift) of cantilever and coupled walls, and the maximum coupling beam rotation in coupled walls. The methods would cover a range of building heights, including tall high-rise buildings. The methodology that was used was to take the results from numerous nonlinear dynamic analyses on idealized wall models and develop simple rational equations based on a displacement demand from a linear dynamic analysis. The methods presented here were used as background for developing the procedures in the draft 2004 Canadian concrete code.

NON-LINEAR DYNAMIC ANALYSIS

A total of 442 dynamic analyses were conducted on a variety of 2-D cantilever and coupled wall models. The walls that were investigated ranged from 10 to 40 stories in height. The walls were all assumed to represent office towers with 12 ft story heights, resulting in overall heights of 120, 300, 360, 420 and 480 ft. In a high-rise building with a core layout similar to that shown in Figure 1, the fundamental periods in the two directions are usually not identical because the flexural stiffnesses in the two principal directions of the core are not usually identical. To simplify the comparison of results for cantilever and coupled walls, the masses were adjusted to give the same fundamental periods for the two cases. These turned out to be 0.5, 1.5, 2.1, 2.9 and 3.7 seconds for the different building heights.

Ten different earthquake records were used for each building. All records were from Phase 2 of the FEMA/SAC Steel Project (Somerville et al. [10]), and were scaled to a 2% in 50-year probability of occurrence in the intended locations. The records represent a variety of ground motions in western U.S., specifically Los Angeles and Seattle. Table 1 summarizes the earthquake properties.

The quantity of reinforcement in the concrete walls and coupling beams were selected so that the elastic-demand-to-strength ratios (i.e., the force-reduction-factors) for the system ranged from 1 to 6 for the 10 time histories. All nonlinear and linear analyses were done using computer program CANNY-99 (Li [11])

12 ft high “column” elements with bending, shear and axial degrees of freedom were used to model the vertical wall segments. Beam elements with rigid links extending from the centroid of the walls to the sides of the wall openings were used to model the horizontal wall segments (coupling beams). That is, the coupled walls were modeled as frames. The degree of coupling (DOC), which is the portion of the base overturning moment that is resisted by axial forces in the walls resulting from coupling beam shears, was 85% for all buildings except the 10 story buildings, which had a DOC of 71%. The coupling beam shear strengths were assumed to be uniform over the height of the building.

Table 1 Properties of earthquake records used in this study

Sac Name	Record	Earthquake Mechanism	Earthquake Magnitude	Distance (km)	Duration (sec)	PGA (g)	PGD (cm)
SE 21 SE 22	1992 Mendocino	thrust	7.1	8.5	23	0.76 0.49	31.9 11.6
SE 25 SE 26	1949 Olympia	subduction intraplate	6.5	56	36	0.89 0.82	15.6 15.0
SE 31 SE 32	1985 Valparaiso	subduction interplate	8.0	42	75	1.27 0.90	21.8 11.4
LA 21 LA 22	1995 Kobe	strike-slip	6.9	3.4	19	1.28 0.92	37.5 34.3
LA 23 LA 24	1989 Loma Prieta	oblique	7.0	3.5	10	0.42 0.47	14.8 31.7

Perimeter columns were included in the building models to take an appropriate portion of the gravity load. Due to the flexibility of the flat plates that connect the columns and core walls together, these elements did not contribute to the lateral resistance of the buildings. The axial compression, at the base of the walls, was about 10% of $f_c'A_g$. The horizontal displacement of all vertical wall segments and all gravity load columns were linked at each floor level to simulate the rigid diaphragm action of the concrete floor plates.

Selection of the hysteresis models for the walls and coupling beams were guided by the results from recent large scale tests on a concrete wall (Ibrahim [12]) and a diagonally reinforced coupling beam (Gonzalez [13]). Both types of elements were modeled using the Modified Clough (CL2) hysteresis model, which uses a bilinear skeleton curve with reloading directed towards the most exterior peak. The elastic range was modeled using an average (effective) stiffness less than the uncracked-section stiffness to account for the effect of cracking in a simple way. The material parameters (E and G) were selected assuming 55 MPa (8000 psi) concrete. The post-yielding stiffness of the walls was taken as 1% of the elastic stiffness. As the maximum wall curvatures were typically about 10 times the yield curvature, the ultimate strength was typically 10% greater than the yield strength. The post-yielding stiffness of the coupling beams was taken as .001% of the elastic stiffness so that there would be no significant strength increase after yield. Further details of how the nonlinear analyses were done can be found elsewhere (White [14]).

DISCUSSION OF RESULTS

Figure 3 shows how the top displacement, largest coupling beam rotation at any level of the building, and base rotation varied during earthquake LA24 for: (a) 120 ft high wall (T=0.5sec), and (b) 480 ft high wall (T=3.7sec) coupled walls. These two cases are indicative of the results for their respective heights. In first mode dominated (shorter) coupled walls, the top wall displacement, the largest coupling beam rotation, and base rotation had virtually identical time histories and the maximum values occurred at the same time [Figure 3(a)]. With taller walls, which have higher mode influence, the top wall displacement, largest coupling beam rotation, and base rotation had similar time histories; but the maximum values did not occur at the same instant [Figure 3(b)]. At the instant of maximum base rotation (also maximum inelastic drift) in the example shown in Figure 3(b), the top wall displacement is about 73% of the maximum top wall displacement, and at the instant of maximum top wall displacement, the base rotation is about 51% of the maximum base rotation during the earthquake. The maximum coupling beam rotation occurs at a different time than either the maximum top displacement or the maximum base rotation.

Rotational Demand of Walls

Figure 4 summaries the displacement profiles at the time of maximum base rotation for (a) the shortest (120 ft high) walls and (b) the tallest (480 ft high) walls, including both cantilever walls and coupled walls. The displacement profiles are all shown with positive inelastic rotations at the base, and are normalized by the maximum displacement for the particular time history. For all 120 ft high cantilever walls and a few coupled walls, the normalized displacements at the top of the walls equal 1.0, indicating that the displacement at maximum base rotation equals the maximum displacement during the earthquake. For a number of walls, the displacement at maximum base rotation is considerably less than the maximum displacement, and for a few of the very slender (480 ft high) coupled walls, the top displacement is in the opposite direction from the inelastic displacement resulting from the hinge rotations at the base.

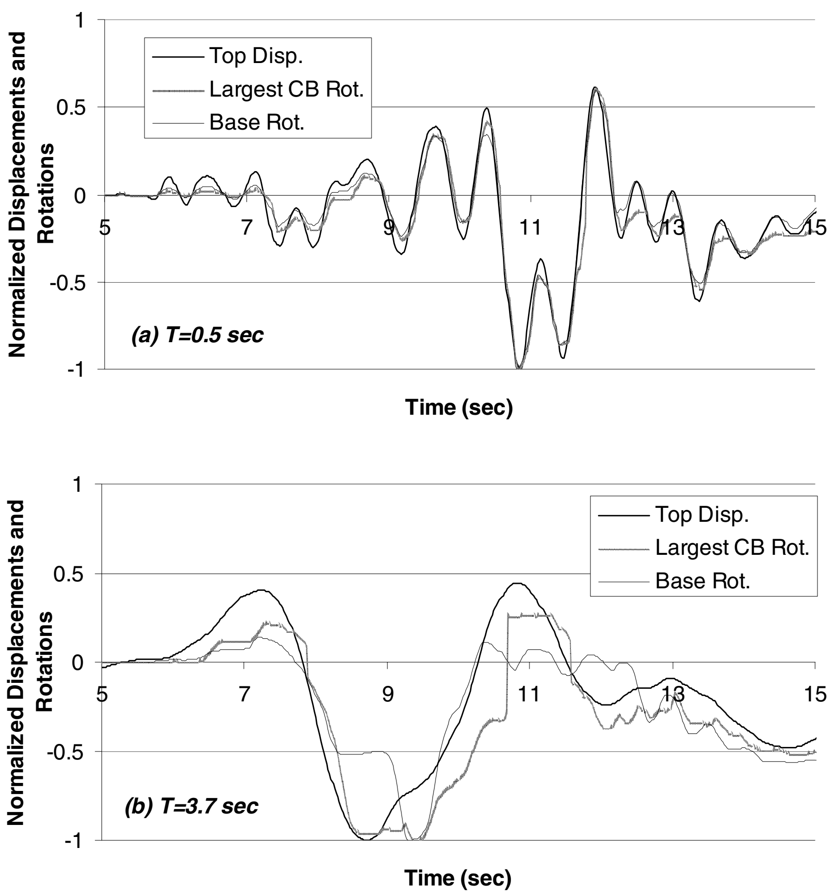

Figure 3 Time histories for LA24 of top displacement, largest coupling beam rotation, and base rotation for: (a) 120 ft high, and (b) 480 ft high, coupled walls.

There is significantly more scatter (in the displacement profiles at the time of maximum base rotation) in the tall (T=3.7 sec) walls than in the short (T=0.5 sec) walls, with coupled walls having consistently more scatter than cantilever walls. The increased scatter in the tall walls is a result of different amounts of higher mode influences because of the different frequency content of the earthquake records. The coupled walls have more variation in their response at the time of maximum base rotation because of subtle variations in their coupling beam shear profiles (White [14]).

From the total displacement profiles at the time of maximum hinge rotation and the inelastic rotation of the wall, the profile of elastic wall displacements can be determined. The inelastic displacement profile is assumed to be the mechanism shown in Figure 2 where the wall above the hinge has a constant drift equal to the inelastic rotation of the hinge.

Figure 5 summarizes the elastic displacement profiles at the time of maximum base rotation for the same two wall heights shown in Figure 4. The elastic displacements were normalized by the cantilever wall first mode yield displacements given by Equation (1). With the large inelastic displacements removed, the variation in elastic displacements becomes more visible. The elastic displacements of the shorter (120 ft high) cantilever walls are clearly first mode, and are reasonably well predicted by Equation (1). The influence of the coupling beams "pulling back" on the walls can be seen by comparing the deflections of the shorter coupled walls with the deflections of the shorter cantilever walls in Figure 5(a). The elastic displacements of all taller (480 ft high) walls (Figure 5b) are influenced by higher modes, and the influence of the coupling beams is not discernable. Equation (1) is clearly not appropriate for the taller walls. There is considerably more scatter in the elastic displacements compared to the total displacements at the top of the wall.

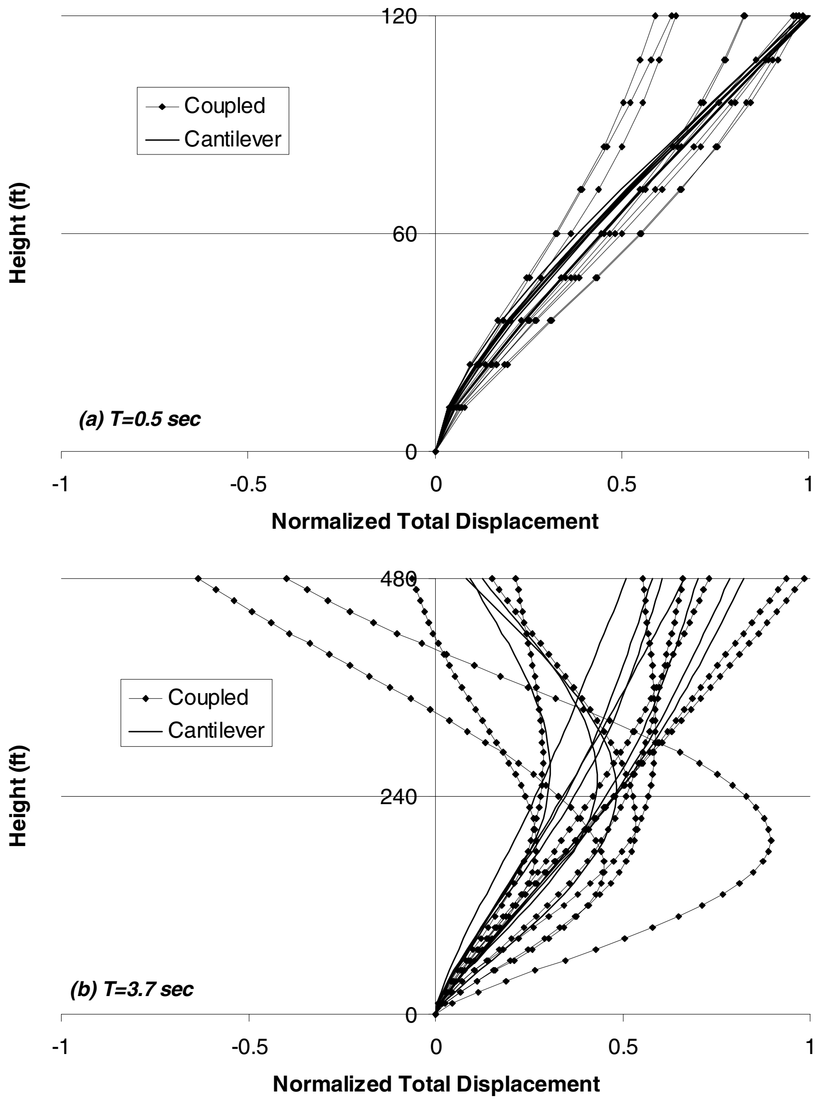

Figure 4 Total displacement profiles at time of maximum base rotation normalized by the maximum top displacement of each earthquake for: (a) 120 ft high, and (b) 480 ft high, cantilever and coupled walls

One objective of the current study was to identify a simple relationship between inelastic rotational demand and total displacement demand. Two separate issues have been identified which make it difficult to develop such a simple relationship: (1) the variation in the displacement at maximum hinge rotation (as a ratio of maximum displacement), and (2) the variation in the elastic portion of the displacements. Rather than deal with these two separately, it was decided to combine them together and develop the concept of an equivalent elastic displacement that is the difference between maximum total displacement and inelastic displacement corresponding to the maximum inelastic rotation. Except for the shortest walls, the equivalent elastic displacements are fictitious, as the maximum inelastic displacement and the maximum displacement demand do not occur at the same time.

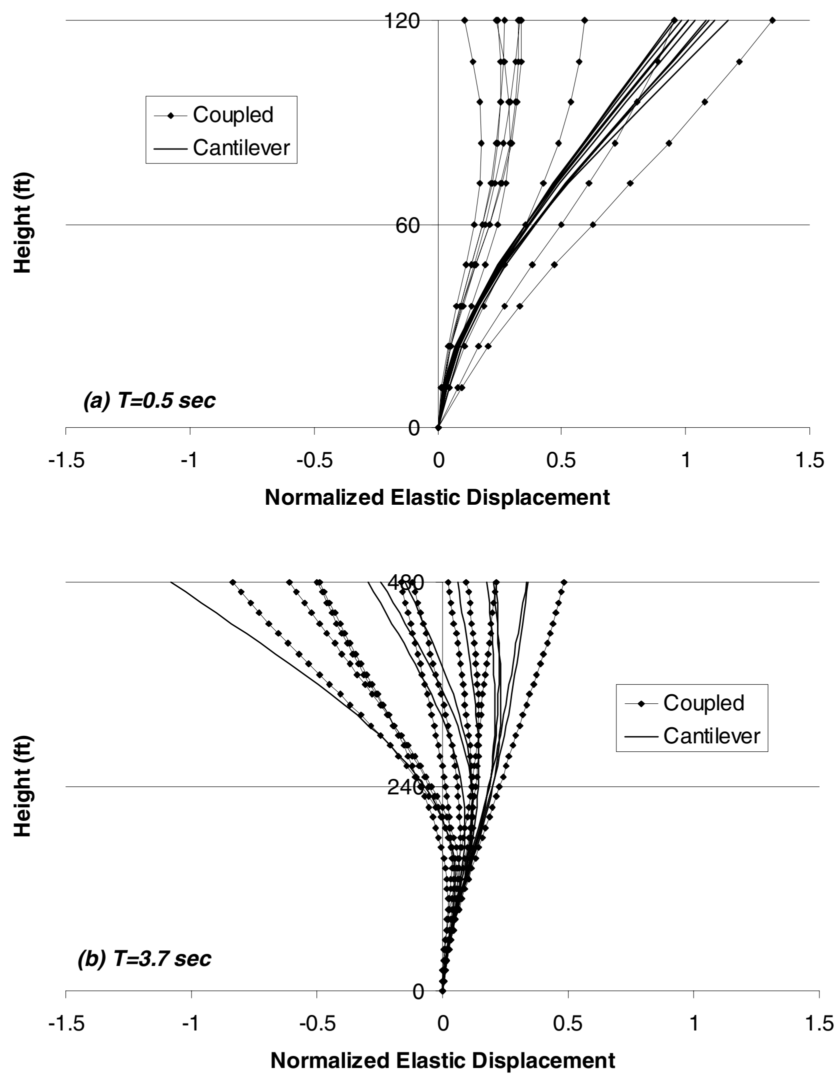

Figure 5 Elastic displacement profiles at the time of maximum base rotation normalized by the first mode yield displacement for: (a) 120 ft high, and (b) 480 ft high, cantilever and coupled walls

The elastic demand and strength refer to the over-turning bending moment at the base of the wall, and the elastic-demand-to-strength ratio is commonly referred to as the seismic force reduction factor R. In seismic design it is common to assume that the ratio of elastic displacement to total displacement is equal to the ratio of wall strength to elastic demand ($1/R$). Figure 6 summarizes the equivalent elastic displacements at the top of the wall normalized by the maximum total displacement for each wall, and plotted against R. Figure 6 indicates that this simplified approach gives very good estimates when using the equivalent elastic (fictitious) displacement for cantilever walls. However, this approach does not work for coupled walls. To account for the reduced elastic displacements of coupled walls, the ratio of equivalent elastic displacement to total displacement can be assumed to be proportional to R^{-2} for coupled walls. Using an R-based approach captures the response of walls with both low and high elastic to total displacement ratios.

The results of the current study led to the development of a simplified procedure in the 2004 Canadian concrete code (CSA [15]) in which the inelastic top wall displacement is estimated from the elastic-demand-to-strength ratio of the wall for cantilever walls (Adebar et al. [16]).

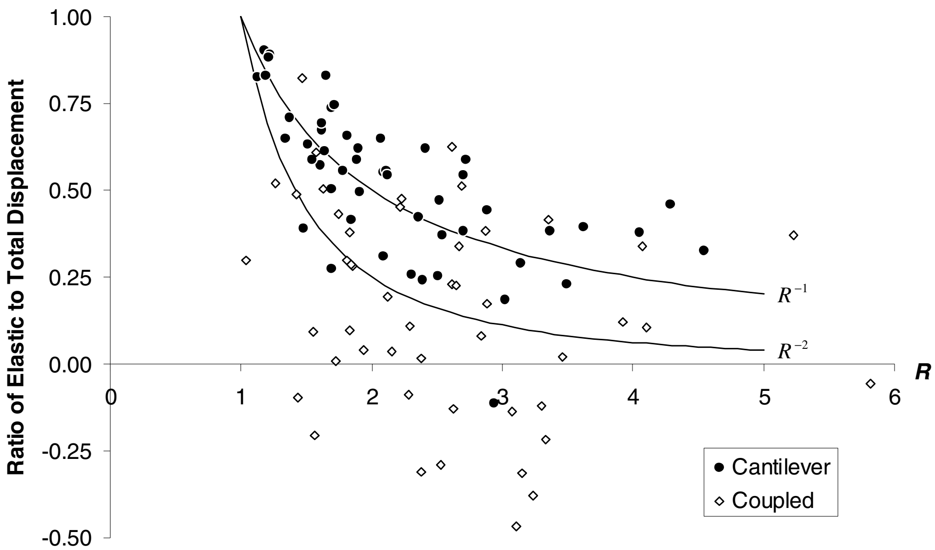

Figure 6 Ratio of equivalent elastic displacement to total displacement versus the elastic-demand-to-strength-ratio (R) at the base of the wall for cantilever and coupled walls

Method for Estimating Inelastic Rotations in Walls

The inelastic θ_i hinge rotation at the base of a concrete wall can be taken as:

$$\theta_i = \frac{\Delta_i}{h'_w} \tag{3}$$

where h'_w is the wall height above the center of the plastic hinge, and Δ_i is defined as:

$$\Delta_i = \Delta_{\max} - \Delta_e \tag{4}$$

$\Delta_{\max}$ is the displacement demand of the wall determined from a linear dynamic analysis, and Δ_e is estimated as for cantilever walls:

$$\Delta_e = \frac{\Delta_{\max}}{R} \tag{5}$$

and as for coupled walls:

$$\Delta_e = \frac{\Delta_{\max}}{R^2} \tag{6}$$

Three different approaches for estimating inelastic drift (rotational demand) of a concrete wall from the total displacement demand are compared in Figure 7, where the estimated value is plotted against the results from nonlinear dynamic analysis. The first approach, shown in Figure 7(a), is to assume that the difference between maximum top wall displacement and inelastic top wall displacement at time of maximum base rotation is equal to the first mode yield displacement given by Equation (1). As expected from the previous discussion, this approach gives good results for shorter cantilever walls (with shorter periods); but is inappropriate for coupled walls. However, it should be pointed out that this equation was never intended for coupled walls. The second approach, shown in Figure 7(b), is to assume that the inelastic drift Δ_i / h_w', which is equal to the inelastic rotation of the wall θ_i, is equal to the maximum global drift Δ_{max} / h_w. This approach gives conservative results for most cantilever walls; but gives very good results for coupled walls. This is the method in ACI [4] for all walls, and the method in the 2004 Canadian Concrete Code (CSA [15]) for coupled walls. Figure 7(c) compares the predictions using Equations (5) and (6), which generally give better results than either of the previously discussed approaches.

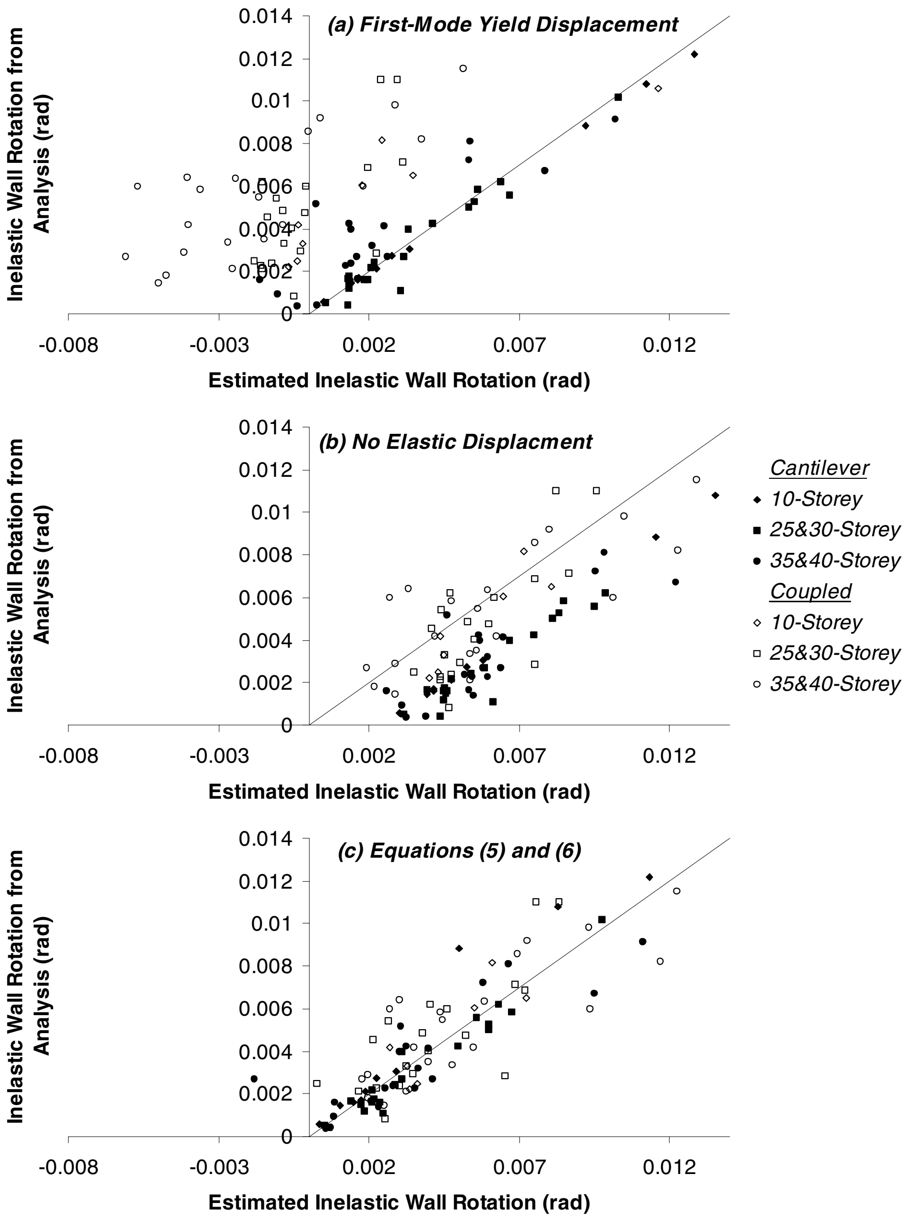

Figure 7 Comparison of estimated inelastic drifts (rotations) assuming: (a) the elastic displacement is equal to the first mode yield displacement, (b) no elastic displacement, and (c) the elastic displacement is a function of *R*

Another approach is to use the maximum mid-height displacement and an estimate of the equivalent elastic mid-height displacement (White [14]). This method is based on the premise that the mid-height elastic displacements typically have less scatter (see Figure 5), and the total mid-height displacements correlate better with the base rotations.

Rotational Demand of Coupling Beams

Figure 8 presents the displacement profiles at the time of maximum coupling beam rotation for the 120, and 480 ft high coupled walls. Each of the profiles has been normalized by the maximum top wall displacement for the particular earthquake. The orientation of the displacement profile was chosen to result in positive wall slopes at the level of the maximum coupling beam rotation (critical level).

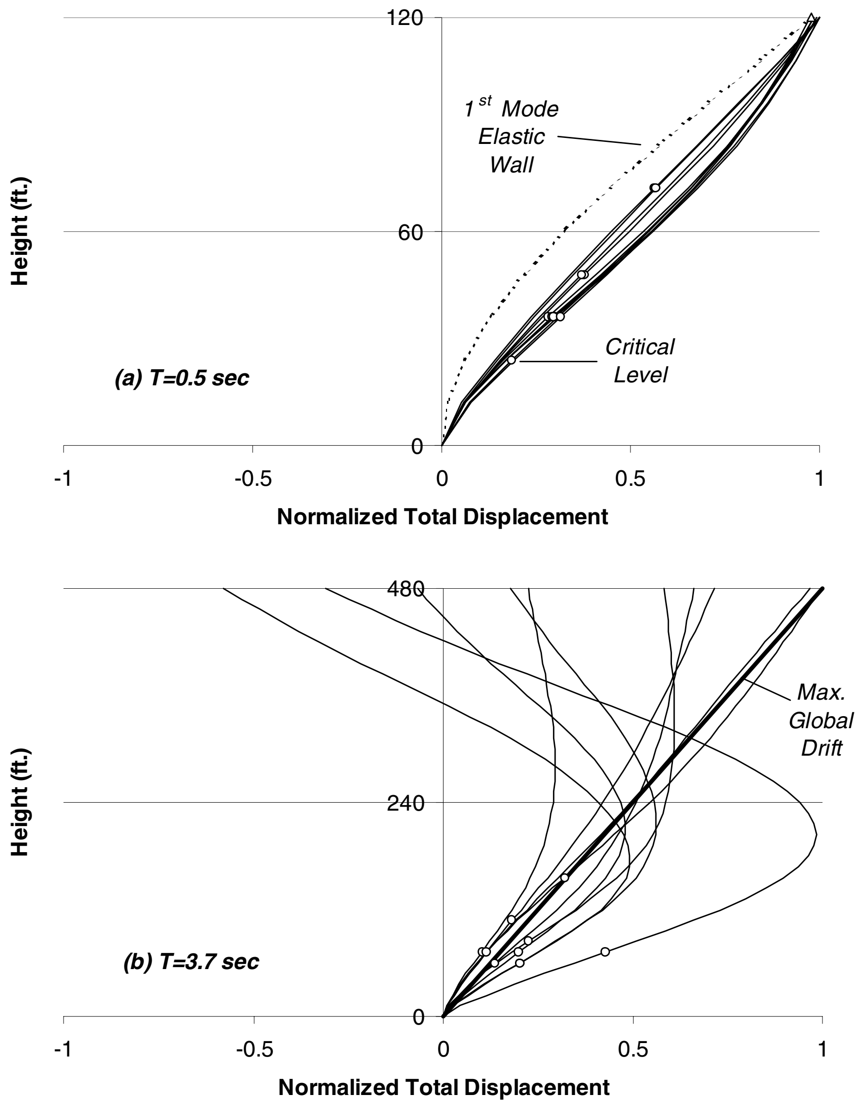

Figure 8 Total displacement profiles at the time of maximum coupling beam rotation normalized by the maximum top displacement of each earthquake for: (a) 120 ft high, and (b) 480 ft high coupled walls

Figure 8(a) confirms that for shorter wall heights, the top displacements at the time of maximum coupling beam rotation are equal to the maximum top displacements during all earthquakes as the normalized displacement profiles have a value of 1 at the top. The first mode (elastic) shape is also shown for comparison. Although the shortest walls are first mode dominated, their deflected shape is significantly affected by plastic hinging at the base and the "pull-back" from the coupling beams. Figure 8(b) confirm that for taller walls, the top wall displacement at the time of maximum coupling rotation is not necessarily equal to the maximum top wall displacement. The 480 ft high walls have normalized top wall displacements ranging between 1 and -0.5. The displacement profiles of the taller walls show evidence of higher mode influence.

The hollow circles on the displacement profiles in Figure 8 indicate the critical level (the level of maximum coupling beam rotation). For the shortest walls, the critical levels varied from 20% to 60% of the height; but most were at about 40% of the height. The critical level appears to shift down the walls, as the walls get taller. For the 480 ft high walls, the critical level was at about 20% of the height. The difference in critical height for the 120 ft high walls may be associated with the lower DOC of these walls (71% compared to 85% for all other walls). In spite of the variations in critical level and higher mode influence for the walls of different heights, the slopes of the normalized displacement profiles do not seem to vary that much at the critical levels. The maximum global drift is equal to the maximum top wall displacement divided by the height of the wall, i.e., it is equal to the slope of a line connecting a normalized top displacement of 1 to a bottom displacement of 0 as shown in Figure 8(b). The wall slopes at the critical level are reasonably similar to the maximum global drift.

The location of the critical level is a result of a combination of the wall slopes and the floor slopes. Figure 9 presents profiles of wall slope, floor slope and resulting coupling beam rotation all on one plot. Two typical analysis results are shown for two different wall heights. All slopes were normalized by the maximum coupling beam rotation so that the relative values are preserved. The normalized maximum coupling beam rotations are equal to 1, and a hollow circle again indicates the location. The wall slopes tend to be largest near the bottom of the wall, and decrease near the top due to the coupling beams pulling back on the walls and due to higher mode influence in the taller walls. The floor slopes are smallest near the bottom of the wall, and increase towards the top. The coupling beam rotation is greatest where the difference between wall slope and floor slope is maximum. This tends to be in the lower portion of the wall slightly below the location of maximum wall slope. In Figure 9(b) the floor slopes are about equal to the wall slopes near the top, and as a result, the coupling beam rotations are very small.

At the critical level, the wall slopes are much greater than the floor slopes. This would suggest that the floor slopes do not play a very important role. If the floor slope were negligible at all times, the maximum coupling beam rotation would result from maximum wall slope; however, this is not the case with many of the taller walls. The reason is that the floor slope is significantly larger at the time of maximum wall slope (White [14]). Thus, although floor slopes may not be significant in determining the maximum coupling beam rotation, they do play an important role in determining the location and time of maximum coupling beam rotation.

The second objective of the current study is to develop a simple relationship between the rotational demand of coupling beams and the total displacement demand. The two most significant results thus far have been (1) the critical wall slope (i.e. the wall slope at the critical level) is approximately equal to the maximum global drift, and (2) the critical floor slope is very small compared to the critical wall slope. This suggests that the maximum coupling beam rotation might be proportional the to maximum global drift, as it depends on the critical wall and critical floor slopes (see Equation (2) above).

The difference between the critical wall and critical floor slopes normalized by the maximum global drift are summarized in Figure 10 for all building heights. The height of the wall is represented by the initial fundamental period of the wall. The scatter in the results increases as the period increases: the values range from 0.94 to 1.05 for the shortest walls and range from about 0.67 to 2 for the tallest walls. A value of 1, indicated with a heavy line, represents the case when the difference between the critical wall and floor slope is equal to the maximum global drift. While there is much scatter, a value of 1 appears to be an acceptable approximation for the majority of the data presented.

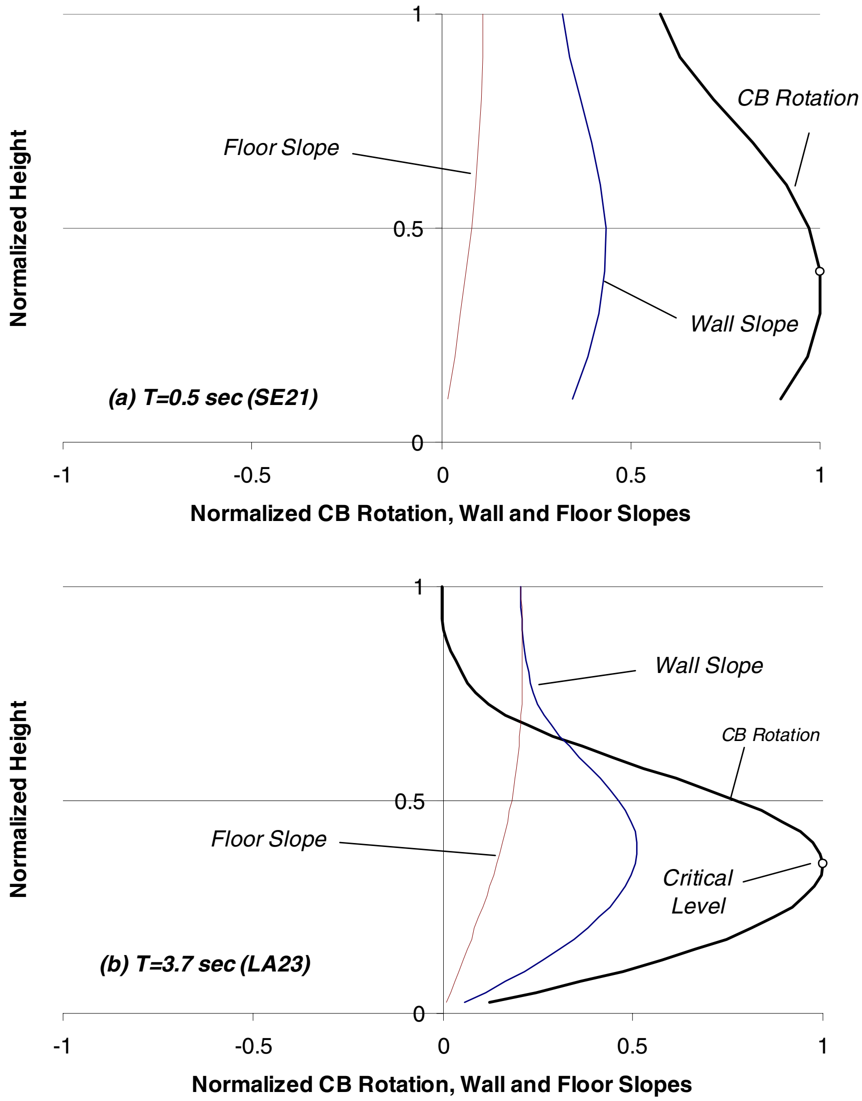

Figure 9 Profiles of coupling beam rotations, wall slopes, and floor slopes normalized by the maximum coupling beam rotation for: (a) 120 ft high, and (b) 480 ft high coupled walls

It should be noted that these results are all for coupled walls with fairly high degrees of coupling (greater than 70%). Walls with lower degrees of coupling generally have higher normalized critical wall slopes (White [14]).

Proposed Method for Estimating Coupling Beam Rotations in Coupled Walls

The maximum coupling beam rotation $\theta_{cb\text{-}max}$ can be estimated as:

$$\theta_{cb-\max} = \left(\frac{\Delta_{\max}}{h_w}\right)\frac{L}{L_n} \tag{7}$$

where $\Delta_{\max}$ is the displacement demand of the wall calculated by a linear dynamic analysis, h_w is the height of the wall, L is the distance between the wall centroids, and L_n is the clear span of the coupling beam.

This simplified procedure was adopted by the 2004 Canadian concrete code (CSA [15]).

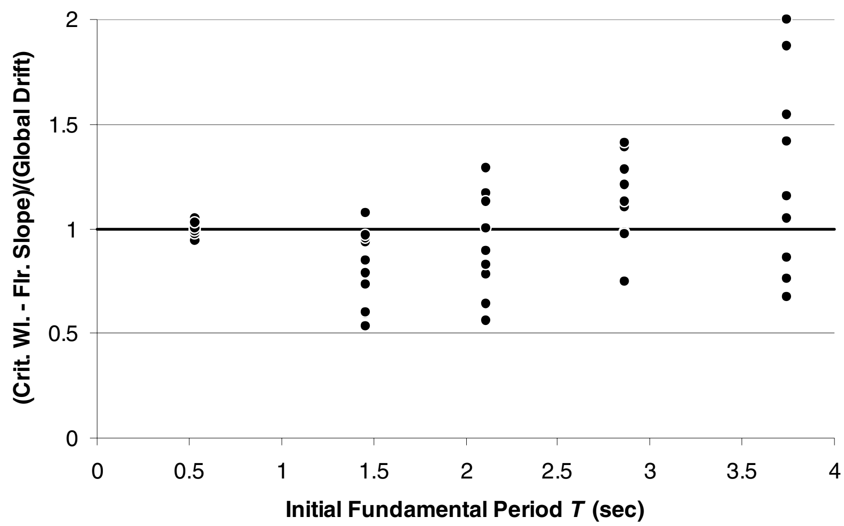

Figure 10 Critical wall minus critical floor slope normalized by the maximum global drift for coupled walls ranging in height from 120 to 480 ft

CONCLUSIONS

In order to develop a simplified relationship between total displacement of a concrete wall and inelastic rotation of the wall, nonlinear dynamic analyses were conducted on a variety of high-rise walls. The walls ranged in height from 120 to 480 ft, and each was subjected to 10 different earthquake records. Based on the results of these analyses, the following conclusions can be made.

For short walls the maximum inelastic wall rotations (inelastic drifts) and maximum coupling beam rotations usually occur at the time of maximum wall displacements, and the elastic portions of the wall displacements are first-mode dominated. For more slender cantilever walls, the maximum inelastic wall rotations and maximum coupling beam rotations often occur at wall displacements that are much less than the maximum displacements during the earthquake. Due to the effect of higher modes and coupling beams "pulling back" on the top of coupled walls, the elastic portion of the displacements may be very small, and in some cases can be in the opposite direction from the total displacements.

Rather than estimate the actual wall displacements at the point of maximum inelastic drift, a simpler and more accurate approach is to estimate the maximum inelastic displacement directly from the maximum total displacement using the concept of an equivalent (fictitious) elastic displacement, which is equal to the difference between the maximum total displacement and the maximum inelastic displacement even though the two of these may not occur at the same time. A reasonable estimate of the elastic displacement, for any height cantilever wall, is equal to the product of the maximum total displacement and R^{-1} (the ratio of actual wall strength to elastic demand). Due to coupling beams "pulling back" on the coupled walls, the "elastic displacements" of coupled walls are much smaller than cantilever walls. Assuming that the elastic displacements are equal to the product of the maximum total displacement and R^{-2}, or assuming that the inelastic drift is equal to the total global drift ($\Delta_i/h_w' = \Delta_{max}/h_w$) are both acceptable methods.

Coupling beam rotations depend on the wall slope and floor slope. The critical wall slope, which is the wall slope associated with the maximum coupling beam rotation, is proportional to the maximum global drift. The critical wall slope is much greater than the critical floor slope. Thus, the level of maximum coupling beam rotation occurs near to where the wall slope is largest. This is usually in the lower levels of the coupled walls due to the large inelastic drifts that are uniform over the height of the walls, and the coupling beams pulling back at the top of the walls. Due to the floor slopes, the maximum coupling beam rotations do not necessarily result from the maximum wall slopes during the earthquake. A simplified procedure that gives reasonable results is to assume that contribution of the wall and floor slope at the critical level is equal to the maximum global drift.

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