Parametric Study of Coupled Wall Behavior—Implications for the Design of Coupling Beams

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Abstract: Coupled walls are known to be efficient lateral load resisting systems; however, the relationship between their global and local behavior is not well understood and has been shown to result in structural systems having excessive internal deformation or strength demands on their component substructures. In order to investigate appropriate parameters for identifying efficient coupled wall geometries, an initial parametric study of over 2000 coupled wall geometries is reported. These analyses permitted the evaluation of the sensitivity of the structural response to various geometric parameters. The objective of this study is to investigate the elastic response parameters of coupled wall structures and to identify parameters that will permit an accurate initial estimate of the global behavior of a coupled system, the local behavior of the coupling beams and the interaction between the global and local behaviors. Using elastic analysis and gross section properties, the role of representative geometric parameters in the response of coupled structures is illustrated. The effect of using various code-prescribed reduced section properties is also discussed. The critical role of the coupling beam design is also illustrated.

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Introduction

There has been a considerable body of work investigating the response of coupled wall structures. The emphasis of the majority of studies of coupled wall behavior has been the global response of the walls. Coupled walls are known to be efficient lateral load resisting systems and therefore the majority of studies of their behavior concentrate on optimizing the design process. Recent investigations have included the classification of "efficient" coupled wall systems (Chaallal et al. 1996) and displacement-based approaches to ensuring efficient wall-pier response (Munshi and Ghosh 2000).

A question remains, however, based on the expected response of a coupled wall system. Can the coupling beams be detailed to provide the ductility and deformability necessary for the walls to achieve the proposed efficient response? There is a significant disparity between the flexural stiffness of the individual wall piers and the stiffness of the "frame," composed of the wall piers and coupling beams. The frame stiffness is largely a function of the axial stiffness of the piers.

The relationship between the wall and frame action is the degree of coupling. The degree of coupling (doc) of a coupled wall system is defined as the ratio of the total overturning moment resisted by the coupling action to the total overturning moment

$$\text{doc} = \frac{NL_w}{\Sigma M_w + NL_w} \tag{1}$$

where N = axial load in walls due to shears in coupling beams; L_w = lever arm between centroids of wall piers; and M_w = overturning moments in individual wall piers.

The axial force couple [NL_w in Eq. (1)] in the wall piers is developed through the accumulation of shear in the coupling beams. The hysteretic characteristics of coupling beams, therefore, may substantially affect the overall response of a coupled wall system particularly for structures having a high degree of coupling. As coupling beams become stiffer, the wall system behavior approaches that of a single pierced wall exhibiting little frame action. Similarly, flexible coupling beams result in the system behaving as two isolated walls.

An effective coupling beam is generally quite short, having a large shear-to-moment ratio. It is accepted that the ductility of such concrete members having steep moment gradients may be limited and that the moment capacity decays rapidly in the presence of the high shear. The expected coupling beam behavior strongly suggests the use of hybrid coupling beams (Shahrooz et al. 1993; Harries et al. 1997; Gong et al. 1998; Harries et al. 2000). For these reasons, it is necessary to investigate the behavior of coupling beams in light of the predicted demands placed on them.

In a recent review study of analytic coupled wall behavior and experimental coupling beam behavior (Harries 2001) it is concluded that the predicted displacement ductility demand of coupling beams is often greater than the experimentally demonstrated available ductility of these beams. It was also noted that coupled wall systems having a high degree of coupling are not necessarily practical in the form in which they are often presented (Harries 2001). A high degree of coupling is more practical for cases where the wall piers are flexible. This is illustrated by the wall

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structures presented by Guizani and Chaallal (1995). These walls are an excellent example of obtaining a high degree of coupling with a practical structure. The individual walls in these cases are quite slender, having height-to-width ratios between 10.5 and 23.3. In this case a high degree of coupling is relatively easily achieved with practical coupling beams having span-to-depth ratios of 5.5 and 4.4. Drift limits associated with the more flexible structure, rather than beam deformation capacity, serve to limit excessive beam ductility demands. For less flexible wall systems, ductility capacities are often exhausted before typical drift limits are achieved (Harries 2001).

It was concluded that the degree of coupling, alone, is not always a suitable parameter for predicting or defining expected coupled wall behavior (Harries 2001). An additional parameter capturing the wall slenderness and/or the relative stiffness of the walls and beams is necessary to accurately qualify coupled wall response and link this response to limits imposed by architectural geometry.

In order to investigate appropriate parameters for identifying efficient coupled wall geometries, a parametric study of over 2000 coupled wall geometries was conducted. These analyses permitted the evaluation of the sensitivity of the structural response to various geometric parameters. The intent of this study is to investigate the elastic response parameters of coupled wall structures and their impact on the local behavior and thus design parameters of the coupling beams. The results of this parametric evaluation will be used to:

1. Evaluate the role of critical geometric parameters in determining the response of coupled walls, focusing on the demands placed on the coupling beams;
2. Identify a number of representative prototype structures for further nonlinear evaluation; and
3. Identify additional parameters affecting the response of coupled structures.

The objective of this study is to identify parameters that will permit an accurate initial estimate of the global behavior of a coupled system, the local behavior of the coupling beams and the interaction between the global and local behaviors. The long-term objective is the development of a series of "selection algorithms" that will permit a designer to enter certain desired performance criteria and some predetermined geometric properties. The algorithms are used to determine reasonable values for some of the other unknown geometric properties and to estimate the behavior of the coupled system early in the design process. Such algorithms should permit the initial selection of coupled systems that will work within a performance-based design context.

Parametric Study

For this parametric study, only the coupled core wall of the structure is considered to contribute to the lateral resistance of the structure. The initial prototype structure considered is based on an 18 story prototype presented previously by the author (Harries et al. 1998). The general prototype geometry for the parametric study is shown in Fig. 1. The parameters investigated are provided in Table 1. For the initial parametric study, gross section properties were used for the wall piers and the coupling beam stiffness was only reduced to account for shear deformations. The effect of different assumptions of effective properties will be discussed later in this paper.

The prototype is a reinforced concrete double channel core wall with coupling beams spanning the flange wall toes. Both

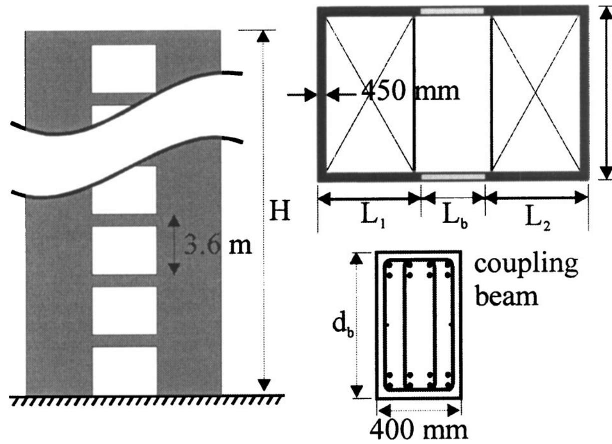

Fig. 1. Prototype geometry

wall piers are identical. The walls have a uniform thickness of 450 mm (18 in.) over their entire height. Story heights, h, are also constant at 3600 mm (142 in.). The coupling beams are all 400 mm (16 in.) wide. For evaluation purposes, the coupling beams are assumed to have a longitudinal reinforcing steel ratio, ρ, equal to 0.02. For the initial study it is sufficient to assume that the beams are detailed to satisfy the seismic requirements, Chap. 21, of ACI-318 (2002). As the parametric study is refined, the actual beam details will be discussed. The structure surrounding the core is assumed to be symmetric—torsion will not be considered in this initial investigation—and weigh 10,000 kN (2250 kips) per floor. It is assumed that concrete having a compressive strength, $f_c' = 30$ MPa (4350 psi), and a modulus, $E = 28.5$ GPa (4133 ksi), will be used throughout the structure.

All combinations of the parameters were investigated in the initial elastic analysis. While it is recognized that many of the resulting structures are architecturally or structurally impractical, including all combinations permitted a large range of responses to be investigated.

Finally, only the coupled direction (left to right, in Fig. 1) lateral resistance was investigated. It is acknowledged that some of the prototype structures—particularly those with a small value of b_w—may not be adequate to resist lateral loads in the perpendicular direction.

Elastic Analysis of Coupled Shear Walls

Continuous Medium Method

The initial elastic analyses of the 2016 prototype geometries were carried out using the continuous medium method (Chitty 1947) of modeling the coupled system. The continuous medium method

Table 1. Geometric Parameters Considered

Parameter	Values
Number of stories, n	6, 9, 12, 18, 24, 30 stories
Building height, H	21.6, 32.4, 43.2, 64.8, 86.4 and 108 m
Length of wall pier, L_1	2, 3, 4, 5, 6, 7 and 8 m
Breadth of wall pier, b_w	3, 6, 9 and 12 m
Length of coupling beam, L_b	1.2, 1.5, 2, 2.5, 3 and 3.5 m
Depth of coupling beam, d_b	700 and 1000 mm

Note: 1 m = 39.4 in.

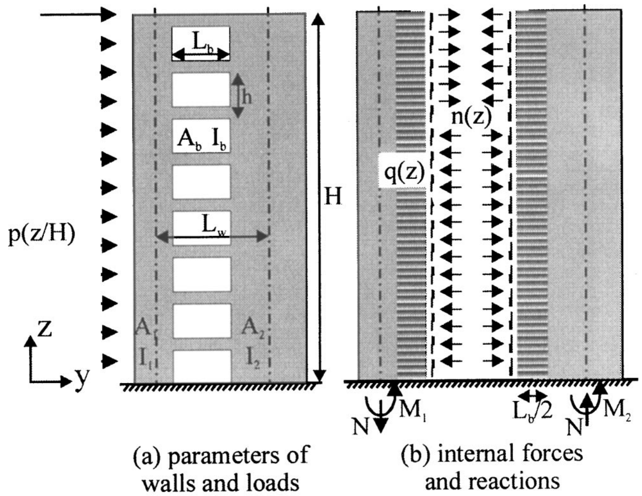

Fig. 2. Coupled shear wall geometry, loading and modeling

results in closed form solutions for the internal forces and deformations of the system. The complete derivation and resulting closed-form solutions for internal forces and displacements of coupled wall structures having two piers and one row of coupling beams is presented in Chap. 10 and Appendix A of Stafford-Smith and Coull (1991). The derivation is based on the plane coupled wall structure shown in Fig. 2(a), where the coupling beams are modeled as a continuous medium. The resulting internal forces and reactions are shown in Fig. 2(b). Having determined the internal forces in the continuum, it is a straightforward matter to collect these at each of the discrete coupling beams.

Structural Behavior by the Continuous Medium Method

The assumed lateral loading on the prototype structures is a triangularly distributed load varying uniformly over the height of the structure, $p(z/H)$, as shown in Fig. 2(a). All internal forces, reactions and lateral displacements of the structure are found using the continuous medium method. Having found the shear flow, $q(z)$, in the coupling medium and assuming fixed base conditions, the relative deflection of the ends of the coupling beams may be found to be

$$\delta=\frac{1}{(k\alpha H)^2}\frac{pH^3L_w}{EI}F_2[z/H,k\alpha H] \tag{2}$$

where p, H, L_w, and z are defined in Fig. 2; k, α and I are defined below; and E=Young's modulus.

Function F_2 for the triangular loading case is (Stafford-Smith and Coull 1991):

$$F_2=\left[\frac{\sinh(k\alpha H)-k\alpha H/2+1/k\alpha H}{(k\alpha H)\cosh(k\alpha H)}\cosh[k\alpha(H-z)]\right.$$
$$\left.-\frac{\sinh[k\alpha(H-z)]}{(k\alpha H)}+\left(1-\frac{z}{H}\right)-\frac{1}{2}\left(1-\frac{z}{H}\right)^2+\frac{1}{(k\alpha H)^2}\right] \tag{3}$$

As has been previously stated, the degree of coupling (doc) is typically used as an indicator of coupled wall behavior. The degree of coupling for the triangularly distributed loading case can also be found, in closed form, from the continuous medium method (Chaallal and Nollet 1997)

$$\mathrm{doc}=\frac{3}{k^2(k\alpha H)^2}\left[\frac{(k\alpha H)^2}{3}-\cosh(k\alpha H)\right.$$
$$\left.+\frac{\sinh(k\alpha H)-k\alpha H/2+1/k\alpha H}{\cosh(k\alpha H)}\sinh(k\alpha H)\right] \tag{4}$$

Significance of Geometric Parameter $k\alpha H$

In the previous equations, the parameters α and k are defined as follows:

$$\alpha=\sqrt{\frac{12I_cL_w^2}{L_b^3hI}} \tag{5}$$

$$k=\sqrt{1+\frac{AI}{A_1A_2L_w^2}} \tag{6}$$

where I=sum of the moments of inertia of the individual wall piers ($I=I_1+I_2$); A=sum of the areas of the individual wall piers ($A=A_1+A_2$); L_w, L_b, and h are defined in Fig. 2; and I_c=effective moment of inertia of the coupling beam accounting for shear deformations

$$I_c=\frac{I_b}{1+\left(\frac{12EI_b}{L_b^2GA_b}\lambda\right)} \tag{7}$$

where I_b and A_b=gross moment of inertia and area of the coupling beam, respectively; E and G=Young's modulus and the shear modulus of the coupling beam; and λ=shape factor, taken as 1.2 for rectangular sections.

The parameter α is a measure of the relative flexibility of the coupling beams and the walls. A low value of α indicates a relatively flexible coupling beam system. In such a case, the overall behavior of the system will be governed by the flexural response of the individual wall piers. A higher value of α leads to greater coupling (frame) action between the walls. The parameter k is a measure of the relative flexural to axial stiffness of the wall piers. This parameter has a lower limit of $k=1$ representing axially rigid wall piers and varies up to values of about $k=1.2$. It should be noted that a structurally and architecturally practical coupled structure will typically have a k value less than 1.1. The average k value of all structures in the present parametric study is 1.058.

The parameter

$$k\alpha H=\sqrt{\left(1+\frac{AI}{A_1A_2L_w^2}\right)\frac{12I_cL_w^2}{L_b^3hI}H^2} \tag{8}$$

may be interpreted as a measure of the stiffness of the coupling beams and is most sensitive to changes in either the stiffness or length of the coupling beam—that is, the α term. If the connecting beams have negligible stiffness ($k\alpha H=0$) then the applied moment is resisted entirely by bending of the wall piers. That is, the structure behaves as a pair of linked walls. If the coupling beams are rigid ($k\alpha H=\infty$) the structure behaves as a single cantilever wall.

Typically, if $k\alpha H$ is less than 1, the structure is considered to have negligible coupling action (doc<20%) and behaves as an arrangement of linked walls. For values of $k\alpha H$ greater than about 8, the coupling beams are considered to be stiff and the structural response is dominated by that of the wall piers as described by the factor k. In this case, a flexible wall pier system (higher values of k) results in greater coupling action as the flexibility of the walls engages the frame action of the coupling

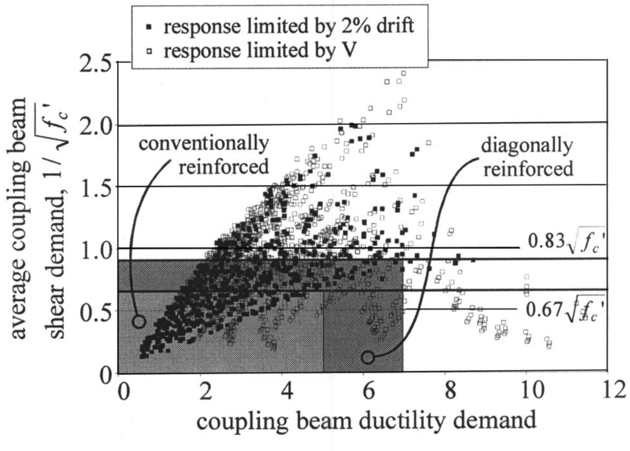

Fig. 3. Degree of coupling determined from Eq. (4)

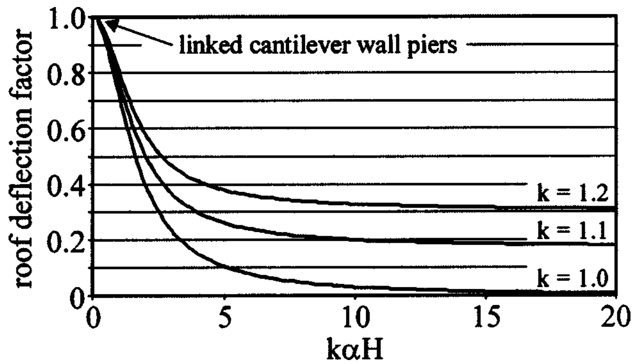

Fig. 5. Effect of coupling action on shear forces in coupling beams (adapted from Stafford-Smith and Coull 1991)

beams. The relationship between kαH and the degree of coupling (doc) is shown in Fig. 3.

For values of kαH greater than about 8, the incremental response of the structure is exceptionally stable. The doc shows little variance with a further increase of $k\alpha H$ (see Fig. 3). Global structural deformations, represented by the roof deflection, shown in Fig. 4, normalized by the roof deflection for a pair of linked cantilever walls, are also relatively unaffected beyond $k\alpha H=8$. The roof deflection of a coupled wall having a triangularly distributed lateral force is (Stafford-Smith and Coull 1991)

$$y_H=\frac{11pH^4}{120EI}F_3[k\alpha H] \tag{9}$$

The factor F_3 is the reduction in roof deflection provided by the coupling action compared to the roof deflection of a pair of linked walls $y_H=11pH^4/120EI$; and is given as

$$F_3=1-\frac{1}{k^2}+\frac{120}{11}\frac{1}{k^2(k\alpha H)^2}\times\left[\frac{1}{3}-\frac{1+(k\alpha H/2-1/k\alpha H)\sinh(k\alpha H)}{(k\alpha H)^2\cosh(k\alpha H)}\right] \tag{10}$$

Fig. 4 clearly demonstrates the advantages of coupling walls in order to controll lateral displacements. At higher degrees of coupling, the roof deflection falls below 33% of that expected if the same walls were simply linked, acting as a collection of individual cantilevers.

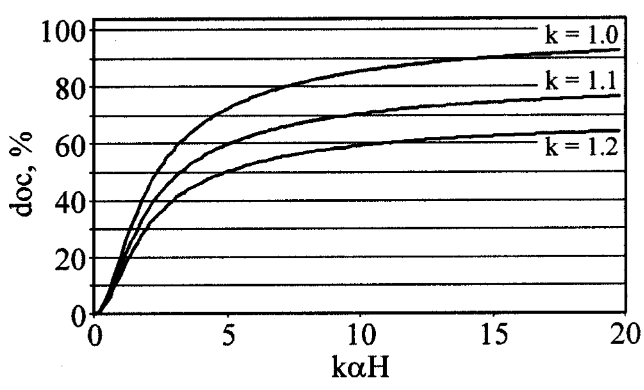

Fig. 4. Effect of coupling action on roof deflection (adapted from Stafford-Smith and Coull 1991)

Although the global response of the structure remains relatively consistent at values of kαH greater than 5 (see Figs. 3 and 4), once high levels of coupling are achieved, many of the local response parameters continue to be significantly affected by changes in $k\alpha H$. Fig. 5 shows the distribution of shear in the coupling beams as represented by the shear flow in the theoretical coupling continuum (Stafford-Smith and Coull 1991)

$$q=\frac{pH}{k^2L_w}F_2[z/H,k\alpha H] \tag{11}$$

Function F_2 is defined in Eq. (3). The expected shear in the coupling beams continues to increase with kαH and the distribution of coupling beam shear forces becomes less uniform with respect to the height of the structure.

High shear in coupling beams may be a critical factor in design in as far as coupling beams are typically relatively short and have a correspondingly steep moment gradient. More significantly, nonuniform shear demand over the height of the structure can also negatively impact the design of the wall system. For example, Canadian design practice (CSA 1994), clearly states that the piers of a coupled wall system must be designed for forces resulting from all of the coupling beams reaching their nominal capacities (all coupling beams yielding). At high degrees of coupling, this may be a very restrictive requirement. The Canadian code mitigates the restrictiveness of this requirement somewhat by specifically permitting a redistribution of forces between coupling beams of up to 20% provided the total capacity does not fall below the total demand. A similar requirement for considering the nominal capacity of all beams is implied in the *Commentary* of ACI 318 (2002), although it would appear as though the designer is given more discretion in this case. There is no discussion of redistribution in the ACI code.

Parametric Analysis

Assumptions of Parametric Analysis

In order to conduct the analysis and assessment of structural response, a number of additional assumptions are necessary. The structural response will be assessed based on its conformance to the requirements of the 2000 International Building Code (IBC 2000). The following additional assumptions were made:

Table 2. IBC 2000 ELF Procedure Natural Periods and Base Shears

n	W (kN)	$T=1.2(0.049)H^{3/4}$ (s)	C_s (kN)	$V=C_sIW$
6	60,000	0.59	0.178	13,315
9	90,000	0.80	0.131	14,736
12	120,000	0.99	0.106	15,835
18	180,000	1.34	0.078	17,524
24	240,000	1.66	0.063	18,831
30	300,000	1.96	0.063	23,438

Note: 1 kN=225 lbs.

1. The prototype structure is assumed to be located in downtown Seattle in a location having a Site Class D. The mapped spectral accelerations are $S_S=1.50$ and $S_1=0.50$.
2. For the purpose of determining an upper bound for the base shear, the Importance Factor, I, is assumed to be equal to 1.25.
3. The Global Response Modification Factor, R, is assumed to be equal to 6.
4. The Deflection Amplitude Factor, C_d, is assumed to be equal to 4.5.
5. The equivalent lateral load is idealized as a triangularly distributed load [see Fig. 2(a)].

Elastic Analysis Procedure

The objective of this study is to identify parameters that will permit an accurate initial estimate of the global behavior of a coupled system, the local behavior of the coupling beams and the interaction between the global and local behaviors. As such, the response parameters of interest are lateral displacement, interstory drift, and coupling beam deformations as measured by the chord rotation over the length of the beam. The analyses proceeded in the following manner for each structure:

Step 1: Using Eq. (9), determine the value of the triangularly applied load, $p(z/H)$, resulting in the maximum allowable roof displacement $y_H=0.02H/R$ (2% drift limit).

Step 2: Calculate the values of interstory drift, Δ_n, corresponding to the applied load $p(z/H)$.

Step 3: If necessary, scale the solution such that $\Delta_{\max}$ does not exceed the maximum allowable interstory drift, $\Delta=0.02h/R=12$ mm (0.5 in.). Thus the magnitude of the triangularly distributed load becomes $p=p_{\text{from step 1}}\,12\text{ mm}/\Delta_{\max}$. This is the applied load which results in the structure attaining either a maximum interstory or overall drift of 2%.

Step 4: The equivalent elastic base shear is found to be $V=0.5pH$. Because some of the prototype structures are very stiff, the base shear determined in Step 4, which assumes that the 2% drift limit has been met, may be exceptionally large. In addition to limiting the response to the allowable drift limit, responses are further limited by an upper bound base shear. This base shear limit is that determined by the IBC 2000 equivalent lateral force (ELF) procedure

$$V=C_SIW \tag{12}$$

where C_S=seismic response coefficient (IBC 2000); I=Importance Factor (IBC 2000), assumed to be equal to 1.25; and W=seismic weight of the building, previously defined as 10,000 kN per floor. Table 2 summarizes the ELF procedure and resulting upper limits applied to the base shear determined in Step 4. In this way, the magnitude of the applied triangularly distributed load, p, is found such that all the following conditions are met:

1. The roof drift does not exceed 2% of the height of the structure;
2. The maximum interstory drift does not exceed 2% of the story height; and
3. The base shear does not exceed that determined using the ELF procedure of IBC 2000.

Step 5: Having determined the value of p, calculate all beam deflections δ from Eq. (2).

Step 6: The maximum coupling beam chord rotation, $\varphi_{\max}$, is found from the maximum beam deformation, $\delta_{\max}$, found in Step 5

$$\varphi_{\max}=\frac{C_d\delta_{\max}}{L_b} \tag{13}$$

where L_b=length of the coupling beam.

Coupling Beam Ductility Demand and Yield Displacement

For the sake of comparison between prototype structures and experimentally determined coupling beam behaviors, the coupling beam displacement ductility demand, μ_b, is determined. The displacement ductility is the chord deformation found in Eq. (13), $\varphi_{\max}$, divided by that corresponding to yield of the coupling beam, φ_y.

The yield displacements of the coupling beams, δ_y, were determined from a plane section analysis of the beams; the chord deformation, φ_y, is found by dividing δ_y by the length of the beam, L_b. This analysis was carried out using the program RESPONSE-2000 (Bentz and Collins 2000). It was assumed that the beams had a longitudinal reinforcing ratio, $\rho=0.02$, for both the top and bottom steel. For evaluation purposes, it is assumed that all beams were detailed in accordance with Chap. 21 of ACI 318 (2002), thus some beams would have diagonal reinforcing while some would be conventionally reinforced (conventional reinforcement is shown in Fig. 1). This distinction will be important later when assessing the likely performance of the coupling beams. Table 3 summarizes the assumed coupling beam details and chord rotations at yield. Also shown in Table 3 is the likely reinforcing arrangement—diagonal or conventional—based on ACI 318 (2002). requirements.

Results of Parametric Study

Table 1 provides a summary of the dimensions of the prototype structures considered. In the parametric study, the values of k range from 1.01 to 1.12. The values of α range from 0.46×10^{-4} to 5.52×10^{-4} m^{-1}. The resulting values of $k\alpha H$ range from 1.1 to 36.4, corresponding to doc values of 19% to 93%. It is noted that most extreme values in this study do not represent architecturally practical structures but are included to capture the full range of response.

Fig. 6 shows the range of response parameters obtained from the elastic analyses of the prototype structures. In this figure the prototype structures are grouped along the horizontal axis by the number of stories, n. Within each group, the structures are further arranged according to the depth of their coupling beams, d_b. In these figures, data represented by filled circles represent structures whose response is limited by the 2% drift limit—that is, the base shear required to cause a limiting story drift is less than the base shear arrived at using the IBC 2000 ELF method. The data represented by open circles is limited by the base shear, V—that is, the calculated storey drifts for these structures, at the IBC 2000

Table 3. Values of Yield Deformations for Coupling Beams

	700×400 mm beams			1000×400 mm beams		
	ρ=0.02 (8–30 M bars top and bottom)			ρ=0.02 (8–35 M bars top and bottom)		
Beam length, L_b (mm)	Arrangement	δ_y (mm)	ϕ_y	Arrangement	δ_y (mm)	ϕ_y
1,200	Diagonal	3.0	0.0025	Diagonal	2.4	0.0020
1,500	Diagonal	4.4	0.0029	Diagonal	3.4	0.0023
2,000	Diagonal or conventional	7.0	0.0035	Diagonal	5.4	0.0027
2,500	Diagonal or conventional	10.2	0.0041	Diagonal or conventional	7.8	0.0031
3,000	Conventional	14.0	0.0047	Diagonal or conventional	10.4	0.0035
3,500	Conventional	18.2	0.0052	Diagonal or conventional	13.6	0.0039

Note: 25.4 mm=1 in.

base shear load level, are below 2%. This distinction is made clear in Fig. 6(a) discussed below. Finally, the larger data points, labeled "further analysis," represent the prototype structures chosen for further analysis. The selection of these structures will be discussed further below.

Fig. 6(a) clearly illustrates the limits described in Steps 3 and 4 of the elastic analysis procedure described above. All of the data points shown (open and closed circles) correspond to the value of the seismic resposnse coefficient, C_s, obtained by simply limiting the maximum interstory drift to 2% (Step 3). Clearly, for short structures having stiff beams and walls, the base shear required to obtain this drift limit is very large. As such, the base shear limit was imposed (see Table 2 and Step 4). The effect of this is that all of the open circle data points in Fig. 6(a) are reduced to the line shown for V. All data presented other than that in Fig. 6(a) reflect this reduction.

Coupling beam ductility demand, based on chord rotation is shown in Fig. 6(b). As expected, for tall structures whose response is limited by interstory drift, the beam ductility is moderate and generally falls near or below the selected value of C_d $=4.5$. Similarly, beam ductility demand for taller structures whose response was limited by base shear also fall in this range. The beam ductility demand of shorter structures, however, increases significantly despite most of these structures having interstory drifts well below the 2% limit.

Coupling Beam Ductility Capacity

In a previous paper (Harries 2001), the author proposed practical limits to the degree of coupling in order to control the ductility

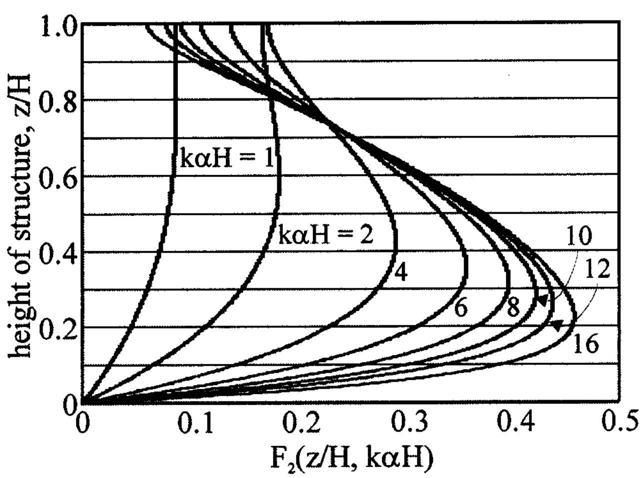

Fig. 6. Response parameters from elastic analysis

demand in the coupling beams. In light of the present study, it is clear that additional parameters enter into the determination of coupling beam ductility demand and simply restricting the doc is not sufficient, particularly for shorter structures. Additionally, based on a review of available experimental data (Harries 2001), the author has identified sustainable levels of displacement ductility for various well-detailed coupling beams: five for conventionally reinforced concrete coupling beams, seven for diagonally reinforced concrete coupling beams and up to 12 for steel coupling beams. Based on these levels of sustainable ductility, and considering the results shown in Fig. 6(b), it can be seen that well-detailed reinforced concrete coupling beams are likely to be able to provide sufficient levels of ductility in tall and midrise structures.

Coupling Beam Shear Capacity

Ductility capacity of the coupling beams notwithstanding, the shear stress carried by concrete beams must also be considered. ACI 318 (2002) limits the shear stress to $0.67\sqrt{f_c'}$ (MPa units) and $0.83\sqrt{f_c'}$ ($8\sqrt{f_c'}$ and $10\sqrt{f_c'}$ in psi units) for conventionally and diagonally reinforced concrete coupling beams, respectively.

Fig. 7 plots the coupling beam ductility demand against the average coupling beam shear demand. The average coupling beam shear is determined as the sum of the axial forces in one wall, T, divided by the number of stories, n. This value would represent the ideal case of all coupling beams having the same capacity and yielding simultaneously. Coupling beam shear varies over the height of the structure (see Fig. 5), therefore the maximum coupling beam shear in any structure is greater than the average presented. In this parametric study, the shear demand in the critical coupling beam varied from 1.2 to 23.8 times the average coupling beam shear demand. The average increase in shear demand for the critical coupling beam was 1.78 times the average shear demand.

Also shown in Fig. 7 are the regions of acceptable behavior for conventionally and diagonally reinforced concrete coupling beams. These regions are bounded by the ACI 318 limits to shear stress and the sustainable ductility limits described previously. It is clear that, while most of the prototype structures fall within ductility limits, many exceed shear stress limits. Indeed, if one considers the critical coupling beam in each structure, only 18.6 and 7.4% of the structures considered satisfy the shear stress limits for diagonally and conventionally reinforced concrete coupling beams, respectively.

It can be shown (Brienen and Harries, unpublished, 2003) that when one includes the effects of code-prescribed torsion, redundancy factors and material resistance factors, very few reinforced concrete coupling beam designs will satisfy the requirements of

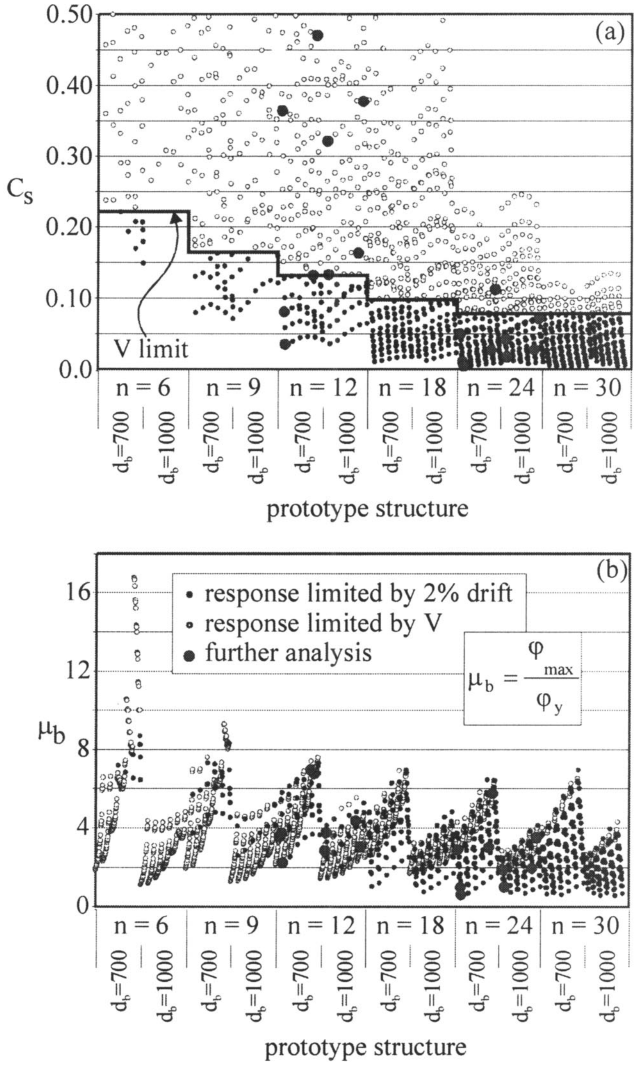

Fig. 7. Coupling beam ductility and average shear demand

ACI 318. Often designers assume very strong concrete and very low beam stiffness in their analyses in order to make the coupling beams acceptable based on strength. Unfortunately, such assumptions result in excessive ductility demands.

Finally, it can also be demonstrated (Brienen and Harries, unpublished, 2003) that the practical design of diagonally reinforced concrete beams is not possible for shear stresses greater than about $0.50\sqrt{f_c'}$ $(6\sqrt{f_c'})$. In all but the very deepest beams, it is simply not possible, from a constructability standpoint, to provide sufficient diagonal reinforcement while respecting concrete cover, development, confinement and bundling requirements.

Effective Section Properties

The previously discussed analysis used gross section properties (see Table 4) in an elastic analysis. While the authors feel that this is valid in the large parametric study to gain an understanding of global and local behavior, this would not be the case in the design of individual structures.

There are a number of standard assumptions used in estimating the effective stiffness of a concrete element for use in analysis. In this section, the recommendations presented in three national concrete design codes, those of the U.S. [ACI 318 (2002)], Canada [CSA A23-3 (1994)] and New Zealand (NZ 3101 1995), are investigated for their effect on the elastically predicted behavior of coupled walls.

As indicated in Fig. 6, a number of the prototypes have been selected for further parametric study and eventual nonlinear analysis. The prototypes selected represent a range of parameters, two heights (12 and 24 stories) and represent architecturally practical core walls. Eighteen prototypes were selected, nine having beam dimensions appropriate for conventional reinforcement and nine for diagonal reinforcement. The 18 selected prototypes have values of k ranging from 1.03 to 1.08 and values of $k\alpha H$ ranging from 2.3 to 36.1.

All 18 prototypes were subject to an additional five elastic analyses each using different code-prescribed effective stiffness values. These analyses were conducted using the effective wall and beam properties given in Table 4. It is noted that the NZ 3101 code has different effective property recommendations based on the global ductility level, μ, considered. The ACI 318 and CSA A23 codes provide only a single recommended value irrespective of structural performance considered.

Impact of Use of Effective Properties on Response Parameters

A summary of the effect that the assumed reduced section properties have on the response parameters is presented in Table 5. In each case, the values presented in Table 5 are ratios of the calculated parameter with respect to the parameter determined using the gross section properties.

Table 4. Effective Section Properties Recommended by Various National Concrete Codes

					NZS 3101		
Member	Parameter	Gross section	ACI 318	CSA A23-3	$\mu=1.25$	$\mu=3$	$\mu=6$
Compression wall in flexure	I_2	EI_2	$0.70EI_2$	$0.80EI_2$	EI_2	$0.70EI_2$	$0.45EI_2$
Tension wall in flexure	I_1	EI_1	$0.35EI_1$	$0.50EI_1$	EI_1	$0.50EI_1$	$0.25EI_1$
	$I=I_1+I_2$	$2.0EI_1$	$1.05EI_1$	$1.30EI_1$	$2.0EI_1$	$1.2EI_1$	$0.70EI_1$
Compression wall axial	A_2	EA_2	EA_2	EA_2	EA_2	$0.90EA_2$	$0.80EA_2$
Tension wall axial	A_1	EA_1	$0.35EA_1$	$0.50EA_1$	EA_1	$0.75EA_1$	$0.50EA_1$
	$A=A_1+A_2$	$2.0EA_1$	$1.35EA_1$	$1.50EA_1$	$2.0EA_1$	$1.65EA_1$	$1.3EA_1$
Conventionally reinforced beams	I_c	EI_c	$0.35EI_b$	$\frac{0.20EI_b}{1+3(d/L_b)^2}$	$\frac{EI_b}{1+5(d/L_b)^2}$	$\frac{0.70EI_b}{1+8(d/L_b)^2}$	$\frac{0.40EI_b}{1+8(d/L_b)^2}$
Diagonally reinforced beams	I_c	EI_c	$0.35EI_b$	$\frac{0.40EI_b}{1+3(d/L_b)^2}$	$\frac{EI_b}{1.7+1.3(d/L_b)^2}$	$\frac{0.70EI_b}{1.7+2.7(d/L_b)^2}$	$\frac{0.40EI_b}{1.7+2.7(d/L_b)^2}$

Table 5. Effect of Effective Section Properties on Response Parameters

Parameter	Coupling beam reinforcement	Ratio of parameter to that calculated using gross section properties				
				NZS 3101-1995		
		ACI 318-02	CSA A23-3-94	$\mu=1.25$	$\mu=3$	$\mu=6$
k	Both	1.001	0.999	1.000	≈0.98	≈0.97
$k\alpha H$	Conventional	≈0.90	0.55	0.85–1.07	0.86–0.95	0.85–0.95
	Diagonal	1.15–1.41	0.77	≈0.95	≈0.94	≈0.93
doc	Conventional	0.89–0.99	0.57–0.86	≈1.00	≈1.02	≈1.04
	Diagonal	1.01–1.05	≈0.97	≈0.98	≈1.01	≈1.02
Beam ductililty demand, μ_b	Conventional	1.05–1.38	2.00–4.50	0.88–1.33	1.43–1.61	1.73–2.01
	Diagonal	0.52–0.79	1.5–1.75	1.04–1.10	1.09–1.40	1.17–1.66
Beam shear demand	Both	0.44–1.02	0.27–0.97	0.94–1.01	0.62–0.99	0.37–0.99

As is expected when reduced section properties are used, displacements, and thus ductility demand, particularly on the coupling beams, increase. Similarly, the average shear demand on the beams is reduced, however the shear demand is still generally observed to be greater than code-prescribed limits as discussed previously.

Although the general impact of code-prescribed effective section properties is expected, it is interesting to contrast these recommendations. As can be seen in Table 5, the modeling recommendations of CSA A23 result in a substantial reduction in the parameter $k\alpha H$, and thus in the doc, as compared to that calculated using gross section properties or those determined from other code assumptions. The NZ 3101 assumptions, on the other hand, result in very little change to these parameters. Thus, the elastic analysis of the same structure based on assumptions of these codes will result in significantly different assumed behavior. The coupling beam ductility demand found in the CSA-based analysis will be substantially greater than that found in the NZ-based one. More importantly, the coupling beam shear demand found in the CSA analysis will be lower than that determined by NZ. These differences have implications on design philosophy and particularly in attempts to develop international codes and performance based specifications.

Finally, the ACI recommendations do not differentiate between conventional and diagonally reinforced coupling beams. The predicted behavior of these prototypes suggests that the assumed stiffness reduction of $0.35EI_b$ for coupling beams is greater than appropriate if correlation with other recommendations is considered. Indeed, common U.S. practice is often to use $0.15EI_b$ and $0.30EI_b$ for conventional and diagonal reinforced coupling beams, respectively. These values are more consistent with those calculated using the CSA or NZ recommendations.

Conclusions

An extensive parametric analysis of coupled wall behavior was conducted. Using elastic analysis and gross section properties, the role of representative geometric parameters in the response of coupled structures has been illustrated. The effect of using various code-prescribed reduced section properties is also discussed. The critical role of the coupling beam design has also been illustrated.

It is established that coupled wall behavior may be described using the geometric parameters k and α, given in Eqs. (5) and (6), the overall height of the structure, H and the product of these three parameters, $k\alpha H$. These parameters may be used to obtain a basic prediction of coupled wall behavior early in the design process—when only basic geometry is known.

While it is well recognized that increasing the degree of coupling improves the global performance of a structure, incremental improvement is negligible once $k\alpha H$ exceeds a value of approximately 5. When one considers local coupling beam design, increasing $k\alpha H$ beyond approximately 5 produces greater demands on the critical coupling beams without a corresponding improvement in the performance of the structure. The selection of wall pier parameter k also affects the global performance of the structure. A more flexible wall system increases the coupling, thus reducing the moment demand on the individual piers, but also results in greater lateral displacements of the structure. Based on the continuous medium method and practical limits to the value of k, attaining a doc greater than 70% is inefficient from a structural response standpoint and attaining a doc greater than 80% is impractical.

In using the response parameter $k\alpha H$ to investigate the behavior of a coupled wall, the use of appropriate effective section properties is critical in determining the structural behavior. The selection of reduced section properties for the coupling beams has a considerable impact on the predicted shear and deformation demands. Different effective properties should be used for conventionally and diagonally reinforced beams.

Coupling beam ductility demand will often exceed the practical limits of sustainable ductility for reinforced concrete coupling beams. Taller and more flexible structures whose designs are limited by interstory drift considerations will exhibit coupling beam deformation demand that can be accommodated by well-detailed reinforced concrete beams. Shorter and stiffer structures may be candidates for more ductile steel coupling beams. Finally, the results of this parametric study demonstrate that little structural benefit is obtained by coupling short (six and nine story) stiff wall piers.

Ductility capacity of the coupling beams notwithstanding, the average shear demand on the concrete coupling beams is shown to often exceed the ACI 318 (2002) limits for shear stress in reinforced concrete coupling beams. Additionally, the shear demand in the critical beam of the system exceeds the average demand by a factor whose average is 1.8. The difference between the maximum coupling beam shear demand and the average demand is reduced with lower values of $k\alpha H$. Finally, when one includes the effects of code-prescribed torsion, redundancy factors and material resistance factors, very few reinforced concrete coupling beam designs will satisfy the requirements of ACI 318. This result, in addition to constructability issues associated with coupling beams, suggests either the use of steel coupling beams or the adoption of performance-based design methods for overcoming the code-prescribed limitations of concrete coupling beams.

Notation

The following symbols are used in this paper:

A = sum of cross sectional areas of individual wall piers A_1 and A_2;
A_b = cross sectional area of coupling beams;
b_w = width of wall piers;
C_d = deflection amplitude factor from IBC (2000);
C_s = seismic response coefficient from IBC (2000);
d_b = depth of coupling beams;
doc = degree of coupling defined in Eqs. (1) and (4);
E = modulus of elasticity;
f'_c = compressive strength of concrete;
G = shear modulus;
H = overall height of structure;
h = story height;
I = sum of moments of inertia of individual wall piers I_1 and I_2;
I = importance factor from IBC (2000) (Table 2 only);
I_b = gross moment of inertia of coupling beams;
I_c = effective moment of inertia of coupling beam defined in Eq. (7);
k = geometric parameter defined in Eq. (6);
L_b = length of coupling beams;
L_w = distance between centroids of wall piers;
L_1, L_2 = length of individual wall piers;
M_w = moment in individual wall pier;
N = axial force in wall piers;
n = number of stories;
$n(z)$ = axial forces in coupling media;
p = magnitude of top of triangular load;
$q(z)$ = shear in coupling media;
R = global response modification factor from IBC (2000);
S_S, S_1 = mapped spectral accelerations for 0.2 and 1 s, respectively;
T = period of structure from IBC (2000);
V = IBC (2000) seismic base shear;
W = seismic weight of building;
y_H = lateral displacement at roof;
z = vertical dimension of structure;
α = geometric parameter defined in Eq. (5);
Δ_n = interstory drift at floor n;
δ = relative displacement of coupling beam ends;
δ_y = yield displacement of coupling beam ends;
λ = shape factor;
μ_b = coupling beam ductility demand;
ρ = reinforcement ratio of coupling beams;
φ = curvature of coupling beams; and
φ_y = yield curvature of coupling beams.

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