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STUDY OF MAJOR DESIGN PARAMETERS AFFECTING THE BEHAVIOUR OF COUPLED CORE WALL SYSTEMS

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SUMMARY

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, a 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.

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 [1] and displacement-based approaches to ensuring efficient wall-pier response [2].

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

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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:

$$doc = \frac{NL}{\sum M_w + NL} \tag{1}$$

where N is the axial load in walls due to shears in coupling beams;
L is the lever arm between centroids of wall piers; and,
M_w are the overturning moments resisted by individual wall piers.

The axial force couple (NL in Equation 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 [3–6]. 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 [7] 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 they are often presented [7]. A high degree of coupling is more practical for cases where the wall piers are flexible. This is illustrated by the wall structures presented by Guizani and Chaallal [8]. 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 [7].

It was concluded that the degree of coupling, alone, is not always a suitable parameter for predicting or defining expected coupled wall behavior [7]. 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 [9]. These analyses permitted the evaluation of the sensitivity of the structural response to various geometric parameters. The intent of this study was 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 are 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 the parametric study, only the coupled core wall of the structure is considered to contribute to the lateral resistance of the structure. The general prototype geometry for the parametric study is shown in Figure 1. The parameters investigated are provided in Table 1. For the initial parametric study [9], gross section properties were used for the wall piers and the coupling beam stiffness was only reduced to account for shear deformations.

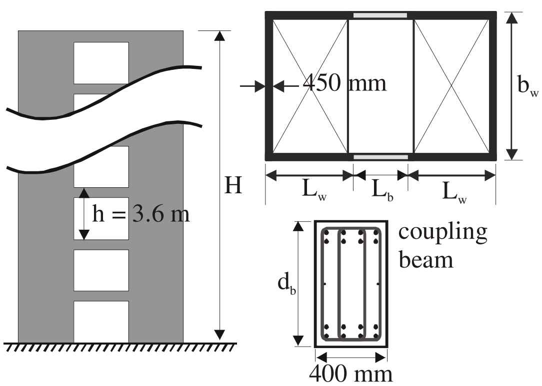

Figure 1 Prototype geometry.

Table 1 Geometric parameters considered.

parameter	values
number of storeys, n	6, 9, 12, 18, 24, 30
building height, H	21.6, 32.4, 43.2, 64.8, 86.4 and 108 m
length of wall pier, L_w	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

The prototype is a reinforced concrete double channel core wall with coupling beams spanning the flange wall toes. Both wall piers are identical. The walls have a uniform thickness of 450 mm over their entire height. Storey heights, h, are also constant at 3600 mm. The coupling beams are all 400 mm wide. For evaluation purposes, the coupling beams are assumed to have a longitudinal reinforcing steel ratio, ρ, equal to 0.02. It is assumed that the beams are detailed to satisfy the seismic requirements, Chapter 21, of ACI-318-02 [10]. The structure surrounding the core is assumed to be symmetric – torsion is not be considered in the initial investigation – and has a seismic weight of 10000 kN per floor. It is assumed that concrete having a compressive strength, f_c' = 30 MPa, and a modulus, E = 28.5 GPa, is 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 Figure 1) lateral resistance was investigated. It is acknowledged 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 assumed lateral loading on the prototype structures is a triangularly distributed load varying uniformly over the height of the structure, p(z/H). All internal forces, reactions and lateral displacements of the structure are found using the continuous medium method [11]. The continuous medium method 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 Stafford-Smith and Coull [12].

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 [13]:

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

Significance of Geometric Parameter kαH

In the continuous medium method, the structural geometry is defined by three parameters: k, α and H, the overall height of the coupled wall system. The parameters α and k are defined as:

$$k = \sqrt{1 + \frac{AI}{A_1 A_2 {L_w}^2}} \tag{3}$$

$$\alpha = \sqrt{\frac{12 I_c {L_w}^2}{{L_b}^3 h I}} \tag{4}$$

Where I is the sum of the moments of inertia of the individual wall piers ($I = I_1 + I_2$);
A is the sum of the areas of the individual wall piers ($A = A_1 + A_2$);
L_w and L_b are the length of the wall piers and coupling beam, respectively (see Figure 1)
h is the storey height; and,
I_c is the effective moment of inertia of the coupling beam accounting for shear deformations:

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 product of these parameters, kαH, 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 (kαH = 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 (kαH = ∞) the structure behaves as a single cantilever wall.

Typically, if kαH is less than 1, the structure is considered to have negligible coupling (doc < 20%) and behaves as an arrangement of linked walls. For values of kαH 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 beams. The relationship between kαH and the degree of coupling (doc) is shown in Figure 2.

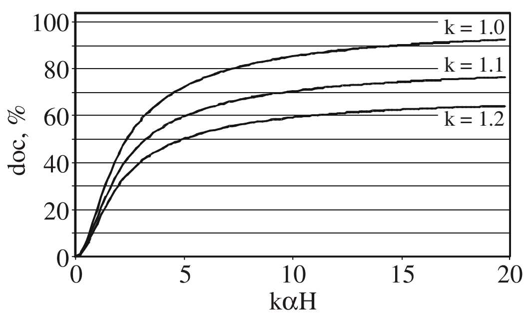

Figure 2 Degree of coupling determined from Equation (2).

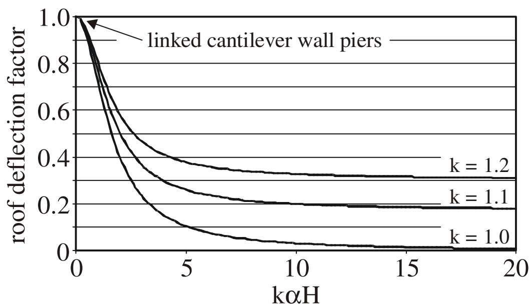

Figure 3 Effect of coupling action on roof deflection (adapted from Stafford-Smith and Coull [12]).

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 kαH (see Figure 2). Global structural deformation, represented by the roof deflection, shown in Figure 3 normalized by the roof deflection for a pair of linked cantilever walls, is also relatively unaffected beyond kαH = 8.

Figure 3 clearly demonstrates the advantages of coupling walls in order to control lateral displacements. At higher degrees of coupling (higher kαH) , the roof deflection falls below 33% of that expected if the same walls were simply linked, acting as a collection of individual cantilevers.

Although the global response of the structure remains relatively consistent at values of kαH greater than 5 (see Figures 2 and 3), once high levels of coupling are achieved, many of the local response parameters continue to be significantly affected by changes in kαH. Figure 4 shows the distribution of shear in the coupling beams as represented by the shear flow in the theoretical coupling continuum [12]. 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.

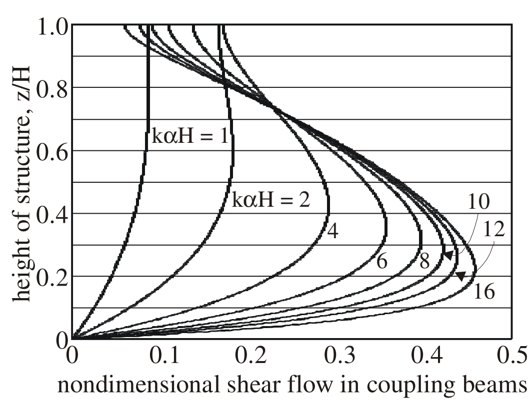

Figure 4 Effect of coupling action on shear forces in coupling beams (adapted from Stafford-Smith and Coull, [12]).

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, non-uniform shear demand over the height of the structure can also negatively impact the design of the wall system. For example, Canadian design practice [14] clearly states that the piers of a coupled wall system must be designed for the sum of the 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 318-02 [10], 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

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, interstorey drift, and coupling beam deformations as measured by the chord rotation over the length of the beam. The structural response was determined using the continuous medium method applying an idealized triangularly distributed load. The magnitude of the triangularly distributed load used in the continuous medium calculations was determined such that none of the following performance criteria were exceeded:

1. the roof drift does not exceed 2% of the height of the structure;
2. the maximum interstorey drift does not exceed 2% of the storey height; and,
3. the base shear does not exceed that determined using the equivalent lateral force procedure of 2000 International Building Code [15].

The following assumptions were made in carrying out the 2000 International Building Code [15] equivalent lateral force (ELF) procedure:

1. The prototype structure has a Site Class D ("stiff soil") with mapped spectral accelerations of $S_S = 1.50$ and $S_1 = 0.50$ (based on Seattle WA).
2. The Global Response Modification Factor, R, is assumed to be equal to 6.
3. The Deflection Amplitude Factor, C_d is assumed to be equal to 4.5.
4. For the purpose of determining an upper bound for the base shear, the Importance Factor, I, is assumed to be equal to 1.25.

Details of the analysis procedure are presented by Harries et al. [9].

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, was determined. The displacement ductility is the analytically determined chord deformation φ_{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 [16]. 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 Chapter 21 of ACI 318-02 [10], thus some beams would have diagonal reinforcing while some would be conventionally reinforced (conventional reinforcement is shown in Figure 1). This distinction is important when assessing the likely performance of the coupling beams.

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 x 10^{-4} to 5.52 x 10^{-4} m^{-1}. The resulting values of kαH 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.

Coupling Beam Ductility Capacity

As expected, for tall structures whose response is generally limited by interstorey drift, coupling beam ductility demand 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 coupling beam ductility demand of shorter structures, however, increases significantly despite most of these structures having interstorey drifts well below the 2% limit.

In a previous paper, Harries [7] proposed practical limits to the degree of coupling in order to control the ductility 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 [7] has identified sustainable levels of displacement ductility for various forms of well-detailed coupling beams: 5 for conventionally reinforced concrete coupling beams, 7 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 of the parametric study [9] it is concluded that well detailed reinforced concrete coupling beams are likely to be able to provide sufficient levels of ductility in tall and mid-rise structures.

Coupling Beam Shear Capacity

Ductility capacity of the coupling beams, notwithstanding, the shear stress carried by concrete beams must also be considered. ACI 318-02 [10] limits the shear stress to $0.67\sqrt{f_c'}$ (MPa units) and $0.83\sqrt{f_c'}$ for conventionally and diagonally reinforced concrete coupling beams, respectively.

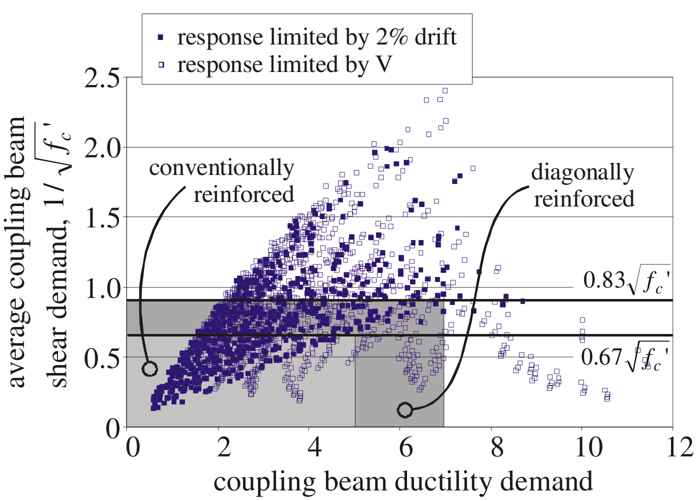

Figure 5 Coupling beam ductility and average shear demand.

Figure 5 plots the coupling beam ductility demand against the average coupling beam shear demand. The average shear is determined as the sum of the axial forces in one wall due to coupling divided by the number of storeys. This value represents the ideal case of all coupling beams having the same capacity and yielding simultaneously. Coupling beam shear can vary significantly over the height of the structure (see Figure 4), therefore the maximum coupling beam shear in any structure is greater than the average. 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 Figure 5 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 above. It is clear that, while most of the prototype structures fall within the ductility limits, many exceed shear stress limits. Indeed, if one considers the critical coupling beam in each structure, only 19% and 7% of the structures considered satisfy the shear stress limits for diagonally and conventionally reinforced concrete coupling beams, respectively.

It can be shown [17] that when one includes the effects of code-prescribed torsion [15], redundancy factors [15] and material resistance factors [10], very few reinforced concrete coupling beam designs will satisfy the requirements of ACI 318-02 [10]. 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 [17] that the practical design of diagonally reinforced concrete beams is not possible for shear stresses greater than about $0.50\sqrt{f_c'}$. In all but the very deepest beams, it is not possible, from a constructability standpoint, to provide sufficient diagonal reinforcement while respecting concrete cover, development, confinement and bundling requirements of ACI 318-02 [10].

EFFECTIVE SECTION PROPERTIES

The previously described analyses used gross section properties (see Table 2) 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 United States (ACI 318-02 [10]), Canada (CSA A23-3-94 [14]) and New Zealand (NZ 3101-1995 [18]), are investigated for their effect on the elastically predicted behavior of coupled walls.

Table 2 Effective section properties recommended by various national concrete codes.

Member Properties		Gross Section	ACI 318 [10]	CSA A23-3 [14]	NZS 3101 [18]		
					$\mu = 1.25$	$\mu = 3$	$\mu = 6$
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	$0.35EI_b$	$\frac{0.40EI_b}{1+3(d/L_b)^2}$	$\frac{EI_b}{1.7+1.3(d/L_b)^2}$	$\frac{0.70EI_b}{1.7+2.7(d/L_b)^2}$	$\frac{0.40EI_b}{1.7+2.7(d/L_b)^2}$

A subset of 18 prototype structures were 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. Of the 18 prototypes selected, nine having beam dimensions appropriate for conventional reinforcement and nine for diagonal reinforcement. The eighteen selected prototypes have values of k ranging from 1.03 to 1.08 and values of kαH 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 as given in Table 2. It is noted that the NZ 3101 code [18] has different effective property recommendations based on the global ductility level, μ, considered. The ACI 318-02 [10] and CSA A23-3 [14] 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 3. In each case, the values presented in Table 3 are ratios of the calculated parameter with respect to the parameter determined using the gross section properties.

As 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 the code-prescribed limits discussed previously.

Table 3 Effect of effective section properties on response parameters.

Parameter	**Coupling Beam Reinforcement**	**Ratio of parameter to that calculated using gross section properties…**				
		ACI 318 [10]	**CSA A23-3 [14]**	**NZS 3101 [18]**		
				μ = 1.25	**μ = 3**	**μ = 6**
k	both	1.00	1.00	1.00	≈ 0.98	≈ 0.97
kαH	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	1.50–1.75	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

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 3, the modeling recommendations of CSA A23 result in a substantial reduction in the parameter kαH, and thus in the doc, as compared to that calculated using gross section properties or those determined from other code assumptions. The NZ 3101 [18] 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 a CSA A23-based analysis [14] will be substantially greater than that found in a NZ 3101-based analysis [18]. More importantly, the coupling beam shear demand found in the CSA A23 analysis will be lower than that determined by NZ 3101. These differences have implications on design philosophy and particularly in attempts to develop international codes and performance based specifications.

Finally, the ACI 318-02 [10] recommendations do not differentiate between conventional and diagonally reinforced coupling beams (Table 2). The predicted behavior of these prototypes suggests that the assumed stiffness reduction of $0.35EI_b$ for coupling beams is insufficient 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 A23-3 [14] or NZ 3101 [18] recommendations.

Conclusions

An extensive parametric analysis of coupled wall behavior was conducted. Using elastic analysis and gross section properties, it is established that coupled wall behavior may be described using the geometric parameters k and α, given in Equations 3 and 4, respectively, the overall height of the structure, H, and the product of these three parameters, kαH. 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 recognized that increasing the degree of coupling improves the global performance of a structure, incremental improvement is less significant once kαH exceeds a value of approximately 5. When one considers local coupling beam design, increasing kαH beyond approximately 5 produces greater demands on the critical coupling beams without a corresponding improvement in the performance of the structure. The selection of the wall pier parameter k (Equation 3) 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 likely impractical.

In using the response parameter kαH 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 interstorey 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 (6 and 9 storey) 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 318-02 [10] 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. This factor is reduced with lower values of kαH, resulting in a more uniform coupling beam shear demand over the height of the structure. 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 [17] for overcoming the code-prescribed limitations of concrete coupling beams.

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