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Ductility and Deformability of Coupling Beams in Reinforced Concrete Coupled Walls

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Abstract

The response of coupling beams in coupled wall structures is discussed. It is demonstrated through a review of large-scale experimental investigations of coupling beam behaviour and analytic studies of coupled wall behaviour that often the beam ductility demand exceeds the expected available ductility. As a result, it is possible that coupled wall structures will not behave as desired in the course of a significant seismic event. Practical limits on the allowable degree of coupling are proposed as a remedy to this apparent deficiency. Additional design and analysis issues are discussed including reduced section properties and wall overstrength requirements.

Introduction

In earthquake resistant design, it is recognised that critical structural elements, those whose yield strength is likely to be exceeded in the event of a severe earthquake, should be designed and detailed to exhibit large ductilities and stable hysteretic responses. Doing so enables the structure to achieve its maximum potential seismic resistance through efficient distribution of internal forces. Furthermore, large ductility capacities permit a structure to dissipate significant amounts of energy through movement as a mechanism.

Structures that resist lateral forces through a combination of structural systems, such as wall-frame structures or coupled wall structures, will often exhibit significantly different ductility demands in each component of the system. Design standards address global ductility capacities by allowing a reduction in lateral design forces. However, little attention is paid to the local ductility demand or capacity of sub-assemblages which may significantly exceed the global ductility demand.

In a structural system where lateral forces are resisted by a combination of systems, the more flexible systems will exhibit a lesser ductility demand than the stiffer systems within the structure. In the case of a wall-frame structure, the ductility demand on the wall elements will exceed those on the frames (Bertero et al., 1991). Similarly, in a coupled wall structure, the "frame" action of the coupling beams, that is: the axial forces in the walls resulting from shear in the beams, is typically stiffer than the flexural response of the individual wall piers. Thus, the coupling beams exhibit greater ductility demands than the walls.

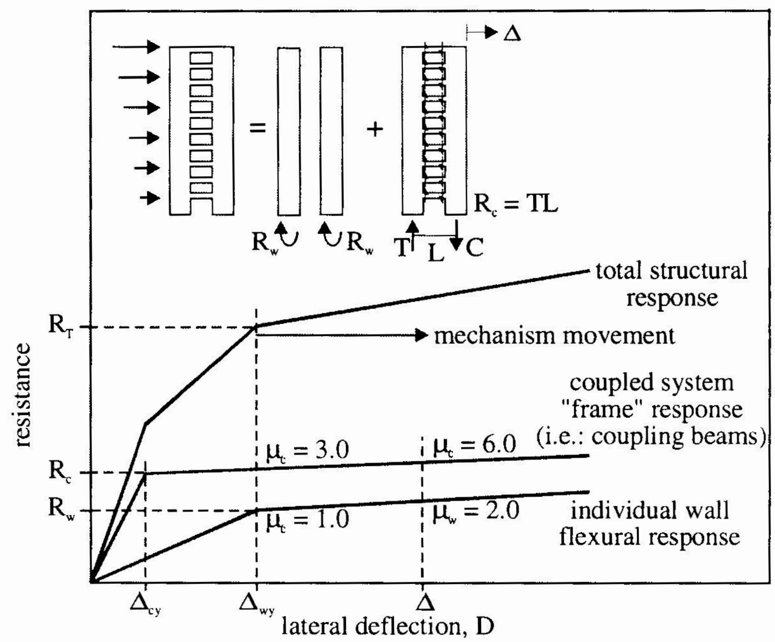

Figure 1. Idealised lateral response of coupled wall structure.

Figure 1 shows the idealised response of a coupled wall structure as the sum of the individual pier flexural responses and the "frame" response of the coupling action provided by the beams. To allow the structure to move as a mechanism under its maximum potential strength, R_T, the coupling beams must already have undergone significant inelastic deformations. As the structure continues to behave in a ductile manner, the ductility demand on the coupling beams exceeds that of the walls. This figure also illustrates the difference between

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deformability and ductility. While the walls exhibit greater deformability, their ductility ratio, defined as the ratio of the ultimate deformation to that at yield, is smaller than that of the beams. If, for instance, the beams are unable to achieve the ductility ratios demanded of them, a premature failure occurs. That is to say the potential strength of the walls, R_w, and therefore the potential strength of the entire structure, R_T, is not realised.

Degree of Coupling

The degree of coupling is defined as the portion of the structure base overturning moment resisted by the axial compression and tension couple in the walls resulting from shear in the coupling beams. As this report investigates the results of many nonlinear analyses performed under different circumstances, undoubtedly with some different analysis assumptions, for the sake of comparison, a uniform procedure for determining the theoretical degree of coupling is useful. Therefore, a theoretical degree of coupling, doc, is determined for a two wall system (i.e.: two resultant axial loads) with parallel coupling beams as (Stafford-Smith and Coull, 1991):

$$\mathrm{doc} = \frac{200}{(k\alpha H)^2}\left(1 + \frac{\sinh(k\alpha H) - k\alpha H}{\cosh(k\alpha H)}\sinh(k\alpha H) - \cosh(k\alpha H) + \frac{(k\alpha H)^2}{2}\right) \tag{1}$$

$$k\alpha H = \sqrt{\frac{12\, n\, E_b I_b L^2}{\ell^3 h E_w I_w}\left(1 + \frac{A_w I_w}{A_1 A_2 L^2}\right) H^2} \tag{2}$$

where A_w = sum of areas of individual wall piers, i.e.: $A_w = A_1 + A_2$;
E_b and E_w = Young's modulus of coupling beams and walls, respectively;
h and H = storey height and overall height of core wall, respectively;
I_b = moment of inertia of the coupling beam;
I_w = sum of the moments of inertia of the individual wall piers, i.e.: $I_w = I_1 + I_2$;
ℓ = span of coupling beam;
L = lever arm between wall centroids; and,
n = number of coupling beams at each level.

The factor kαH, is a measure of the relative stiffness of the walls behaving as two independent flexural walls to that of the walls behaving as a single unpierced wall. That is, if kαH = 0, the structure will behave as a pair of independent flexural walls. As kαH increases, the response approaches that of a single wall pier. The first term under the radical in Equation 2, k^2, is a measure of the relative stiffnesses of the coupling beams and walls. The second term, α^2, is a measure of the relative flexural to axial stiffness of the walls. Descriptions of these factors can be found in Stafford-Smith and Coull (1991).

Table 1. Reduced member stiffnesses.

Member	stiffness value used
wall piers	0.70EI and 0.70EA
Coupling Beams	
conventionally reinforced	0.20kEI $k = (1+3(h/\ell)^2)^{-1}$
diagonally reinforced	0.40kEI $k = (1+3(h/\ell)^2)^{-1}$
steel Harries et al., 1997	0.60kEI $k = (1+(12EI\gamma/\ell GA))^{-1}$

The actual inelastic degree of coupling observed for a structure will typically be less than the theoretical value predicted. Local inelasticities at the beam-wall interfaces, redistribution of forces between coupling beams and from the tension wall to the compression wall and strain hardening effects all contribute to reducing the theoretical degrees of coupling. Reduced section properties, accounting for cracking and loss of stiffness due to cycling can be used to determine a more appropriate value for the theoretical degree of coupling. Table 1 shows the reduced section properties used for the computations in this study.

Ductility Capacity of Coupling Beams

Table 2 summarises experimentally observed ductility capacities of conventionally (traditional longitudinal reinforcement only) and diagonally reinforced concrete coupling beams and "shear" and "flexure critical" steel coupling beams as demonstrated in large-scale structural tests. Only tests having reasonably well detailed coupling beams have been included in Table 2 (many research programmes include details that would be deficient under current standards).

Table 2. Experimental ductility capacities exhibited by coupling beams.

Researcher	span	Ductility ultimate	Ductility sustained
Conventional Reinforcement span = ℓ/h			
Bristowe, 1998	3.6	3.0 - 5.0	2.0 - 4.0
Harries et al., 1996	3.0	3.5	2.0[1]
Paulay, 1971	1.3	3.0	2.0
Shui et al., 1978	2.5	7.8	3.8
	5.0	10.0	7.4
Santhakumar, 1974	1.25	6.0	2.0[1]
Tassios et al., 1996	1.0	3.1	1.6
	1.67	2.9	2.6
Diagonal Reinforcement span = ℓ/h			
Paulay and Binney, 1974	1.3	11.6 - 4.5	3.3 - 5.2[1]
	1.03	6.3	6.0[1]
Santhakumar, 1974	1.25	11.8	5.4[1]
Shui et al., 1978	2.5	9.0	7.9
	5.0	10.2	8.9
Tassios et al., 1996	1.0	5.2 - 5.6	2.7 – 2.9
"Shear Critical" Steel Coupling Beams span = $\ell/(M_r/V_r)$			
Engelhardt and Popov, 1989	1.83	11.0	6.9
	1.98	6.3	4.8
Harries et al., 1997	1.52	13.1	7.7
	1.11	9.4	7.5
Malley and Popov, 1983	1.71	7.4 - 14.4	6.0 - 8.8
"Flexure Critical" Steel Coupling Beams span = $\ell/(M_p/V_p)$			
Engelhardt and Popov, 1989	2.44[2]	9.6	8.9
	2.44	10.3	2.8
	2.37[2]	21.2-23.8	11.3-12.2
	3.84	7.9 – 9.3	3.3 – 3.5
	2.59	4.8	3.9
	4.31	6.3	4.8
Harries et al., 1997	2.97	6.9	3.0

[1] no "sustainable" ductility capacity observed, value given is ductility at 80% of ultimate capacity.
[2] these beams where detailed as "shear critical"

The observed ductility entries correspond to the ultimate displacement ductility observed in the course of the test ("ultimate") and the ductility at which it would appear that the beam could sustain its load carrying ability through repeated cycles ("sustained").

Ductility levels achievable by conventionally reinforced concrete coupling beams are proportional to their span to depth ratios. Longer beams are better able to develop ductile plastic hinges as the shear transmitted across the hinge region is reduced. The transmission of high shear forces through a region expected to behave as a ductile hinge results in rapid deterioration of the hinge region and an eventual sliding shear failure (Paulay and Bull, 1979). This effect was clearly evident in the specimens tested by Bristowe (1998).

Greater ductility levels are achievable with diagonally reinforced concrete coupling beams. The sustainable ductility levels achieved are relatively uniform and do not appear to be effected by the beam span to depth ratio. It has been suggested (Shui et al., 1978) that diagonally reinforced coupling beams are not practical for span to depth ratios exceeding about 2.5. Larger span to depth ratios necessitate lower angles of inclination for the steel resulting in a decreased ability of the section to resist shear. The improved ductility capacity observed by Shiu et al. (1978) result from the inclusion of additional, well detailed, transverse reinforcement, resulting in improved shear capacity. Such details are not required in current design standards (such as CSA A23.3-94), where only minimum transverse reinforcement is required in diagonally reinforced coupling beams.

"Shear critical" steel coupling beams are those that behave in a dominantly shear mode of behaviour, remaining elastic in flexure. Similarly, "flexure critical" steel beams are those that behave in a dominantly flexural mode of behaviour. With the exception of the tests conducted by Harries (1995), the programmes report on tests intended to investigate the behaviour of link beams in eccentrically braced frames, structural systems which are analogous to coupled wall systems (Harries, 1995). Only results from coupling beam and link beam tests where the observed responses were governed by the beam, rather than the embedment or connection conditions, have been included in Table 2.

Ductility capacities exhibited by "shear critical" steel beams are relatively uniform. Typically, web buckling and eventual rupture will dictate the failure criteria. Therefore, sustainable levels of ductility must be below that where low-cycle fatigue rupture of the web is likely. As such, although ultimate monotonic ductilities exceeding 14 are observed, sustainable ductility levels of only about 7 are likely. Ductility capacity available from "flexure critical" steel beams is dependent upon the stiffener detailing. Sustainable levels of ductility are most often governed by the need to maintain flange stability. Provision of stiffeners beyond the expected plastic hinge regions appears to ensure flange stability through many cycles at elevated ductility levels.

Table 3. Ductility demands predicted from coupled wall analyses.

Researcher	n[1]	Degree of Coupling		Ductility Demand	
		EQ. 1	anal.	wall	beam
Aktan et al. 1982	15	62.8	66.8	1.4	10.0
Fintel and Ghosh, 1982 and 1980	16	79.6	65.7 - 69.5	1.5 - 2.4	11.5
	31	70.3	61 - 73.1	1.0	9.9 - 12.3
Guizani and Chaallal, 1995	6	60.0			11.4
	12	76.7			6.2
	12	63.4			8.4
	20	75.9			5.9
Harries et al., 1998	18	43.4	44	1.6	2.9
	18	79.2	62	2.7	6.6
	18	50.5	49	1.8	3.1
	18	67.3	58	2.1	3.3
Pekau and Cistra, 1989	26	80.6	60.6	2.6	11.0
Saatcioglu, 1986	20	33.5	58.3	4.0	4.5
	20	60.9	61.9	2.3	11.5
	20	68	63.5	4.5	17.0
Santhakumar, 1974	7	76.4	50.6	3.4	8.3
	7	74.3	55.8	3.0	6.0
	7	80.9	57.9	5.7	11.8
Srichatrapimuk, 1976	14[2]		60.0		14.5
	18[3]		54.9		8 - 24

[1] number of storeys.
[2] North exterior wall of Mt McKinley apartment building subject to 1964 Anchorage ground motion.
[3] Banco de America Building subject to 1972 Managua ground motion.

Ductility Demand in Coupled Wall Structures

Table 3 summarises the predicted ductility demands for beams and walls determined from analyses of coupled wall structures. All of the analyses are of planar wall systems, except those reported by Harries et al. (1998) and Fintel and Ghosh (1980) which are double channel-shaped core structures. All of the analyses presented assume fixed conditions at the base of the walls (i.e. infinite soil stiffness and no up-lift). Flexible foundations will increase the effective degree of coupling, by reducing the stiffness of the wall piers, and therefore the ductility demand on both the walls and coupling beams (Pekau and Cistra, 1989).

The degrees of coupling reported correspond to the theoretical degrees of coupling determined from Equation (1). The analytical degree of coupling is determined from the results of the analyses.

Both wall and beam ductility demands are presented in Table 3 because each model uses different ground motion records and peak ground motion parameters. The wall ductility demands, therefore, give an indication of the severity of loading to which the model was subject. The prototypes reported by Harries et al. (1998), for instance, were designed and detailed according to the Canadian Standard CSA A23.3-94, Design of Concrete Structures, using wall overstrength parameters prescribed therein. These structures were subjected to the "maximum credible earthquake", that is 1.5 times the NBCC prescribed peak horizontal ground velocity (for Vancouver, BC). The ground motions were further factored to reflect the inability of the two-dimensional inelastic model to include torsional effects. The resulting increase in prescribed ground motion intensity is proportional to the wall ductility demand predicted. As such, the expected wall ductility demands, for credible ground motion intensities, should fall in the range of 1.5 to 2.5.

It is clear that some of the analyses reported in Table 3 represent extreme ground motions (cases where wall ductilities exceeding 3.0 are predicted). Research (such as that reported in Paulay and Priestley, 1992) has shown that well detailed ductile flexural walls are capable of achieving and sustaining ductility levels in the vicinity of 3.0.

The ductility demand values given represent the peak values predicted for each analysis. Little data is available indicating the number of inelastic cycles the walls and beams are expected to endure. Certainly, the peak ductility demand will not be predicted for each inelastic cycle. For instance, it was predicted that the coupling beams of the Mt. McKinley building underwent about 15 to 30 (depending on their location in the structure) inelastic excursions during the course of the 1964 Anchorage earthquake (Srichatrapimuk, 1976). Only a few of these excursions approached the peak ductility demand reported.

Limitations on the Degree of Coupling

Figure 2 summarises the coupling beam ductility demands predicted by the analyses reported. Shown as shaded regions in Figure 2 are the ranges of sustainable ductility capacity available for each type of concrete and steel coupling beam. As discussed above, individual excursions to very high ductility levels are likely in the course of a seismic event. It is likely that steel coupling beams will exhibit a more favourable response in these cases, being better able to absorb the energy of these extreme excursions.

Evident in Figure 2 is the relationship between the degree of coupling and the predicted coupling beam displacement ductility demand. As the degree of coupling increases the beams and their associated coupling action becomes stiffer relative to the flexural actions of the individual wall piers. As a result, the ductility demand on the beams increases. Therefore, since there are practical upper limits governing the ductility capacity of coupling beams, there are corresponding degrees of coupling beyond which the particular methods of coupling are no longer appropriate. Conservative values of these practical limits to the degree of coupling may be taken as (see Figure 2):

(i) for conventionally reinforced concrete coupling beams the degree of coupling should not exceed 50%;
(ii) for diagonally reinforced concrete coupling beams the degree of coupling should not exceed 55%;
(iii) for "shear critical" steel coupling beams the degree of coupling should not exceed 60%; and,
(iv) for "flexure critical" steel coupling beams the degree of coupling should not exceed 65%.

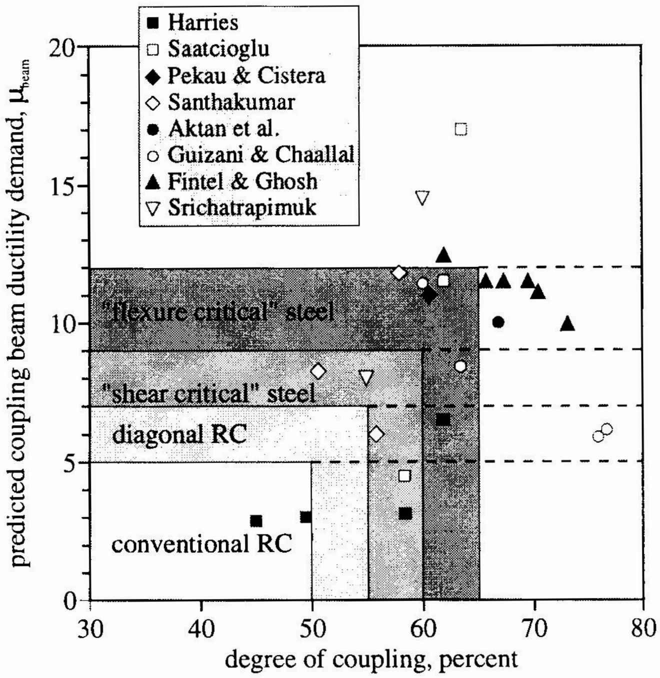

Figure 2. Relationship between degree of coupling and ductility demand and proposed limits.

As has been discussed above, the determination of the degree of coupling should be based on effective section properties. As such, considerable judgement is required in determining appropriate reduction factors for coupling beams and flexural walls. The assumptions of reduced stiffness will significantly effect the design of the coupled wall system. For instance if the predicted degree of coupling is too high, the wall design capacities will be reduced and the beam design capacities will increase. This may lead to an undesirable structural response where the walls will act as the primary energy dissipating component, rather than the coupling beams. Furthermore, high degrees of coupling result in large tensile forces in the walls, significantly reducing their flexure and shear capacity as well as their stiffness. For these reasons, restricting the degree of coupling in design will allow the structure to be more confidently modelled and to dissipate energy in a more desirable manner.

Wall Overstrength

In order to ensure the desired seismic response of coupled wall systems, the walls are provided an overstrength. This overstrength is based on the total probable moment capacities of the coupling beams. That is, the walls should have sufficient capacity to allow all of the coupling beams to achieve their probable capacities. As an example, the wall overstrength factors for the four prototype structures considered by Harries et al. (1998) are in the range of 1.5 to 1.6. Despite this, significant inelastic rotations were predicted at the base of the walls (see Table 3). These overstrength factors appear just adequate to offset the significant force redistribution between wall piers resulting from the variation of axial load, and thus stiffness, brought about by the coupling action of the beams.

Theoretically, if very "sharp pencil" coupling beam designs were used, the wall overstrength could be reduced to about 1.27 where reinforced concrete coupling beams are used and about 1.12 where steel coupling beams are used. If reduced overstrength factors were used, greater inelastic demands would be predicted at the base of the walls (since the wall capacities would be reduced). This further increase of the ductility demand on the walls will result in an increase in the ductility demands on the beams which have already been shown to approach the practical limits for the sections being used. In addition, the walls may exceed their limit of sustainable capacity. The New Zealand Standard NZS 4203 – *General Structural Design and Design Loadings for Buildings* addresses this issue by imposing a minimum value for wall overstrength factor equal to 1.5. This provision should be investigated for adoption in North American practice.

Conclusions

The nature of the response of coupled walls is such that very large deformations are imposed on very stiff coupling beams, resulting in large ductility demands in these beams. It has been shown that in many cases the predicted displacement ductility demand of coupling beams is greater than the experimentally demonstrated available ductility of these beams. This conclusion has been drawn based on coupling beams and wall piers having, what is considered today, good seismic detailing.

Based on these observations, the desired mode of response of coupled wall structures, and the variability of analysis assumptions, limits on the degree of coupling permitted ranging from 50% to 65%, based on the type of coupling beam used are proposed. Additionally, in order to further ensure that the coupling beams act as the primary energy dissipating component of a coupled wall system, a lower limit on the wall overstrength factor of 1.5, as is used in New Zealand, is recommended.

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