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The Use of Steel Beams to Couple Concrete Walls

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ABSTRACT

The use of steel coupling beams, with their ends embedded in reinforced concrete walls, is investigated as an alternative to reinforced concrete coupling beams. Full-scale reversed cyclic loading tests of portions of reinforced concrete walls coupled with steel beams were carried out at McGill University. "Shear-critical" steel coupling beams, designed to exhibit large amounts of ductility and energy absorption were studied as an alternative to diagonally reinforced concrete coupling beams. The behaviour of "flexure-critical" coupling beams, as an alternative to conventionally reinforced concrete beams, was also investigated.

The design and analysis of coupled wall systems with steel coupling beams is placed in the context of the 1995 National Building Code of Canada and the 1994 CSA Standard A23.3, Design of Concrete Structures.

INTRODUCTION

In order for steel beams to be used to couple reinforced concrete walls it is necessary that they be capable of developing sufficient ductility and energy absorbing capabilities. The 1995 NBCC permits the use of a force modification factor, R, of 3.5 or 4 for ductile "partially" and "fully" coupled ductile wall structures, respectively.

It has been shown that the ductility requirements for coupling beams are inversely proportional the degree of coupling provided by the system. The degree of coupling is a function of the relative stiffness and strength of the beams and walls. Saatcioglu et al. (1987) showed that the displacement ductility demand of coupling beams will range between about 6 and 16 in order for system ductilities of 4.0 to be achieved.

Several researchers have investigated novel approaches for improving the ductility and energy absorption of reinforced concrete coupling beams. For span-to-depth ratios less than about 2, specially detailed diagonal reinforcement, developed by Paulay and Binney (1974), has been shown to significantly improve the reversed cyclic loading response. Shiu et al. (1978) confirmed the improved behaviour of diagonally reinforced beams over conventionally reinforced beams. However these tests demonstrated that for larger span to depth ratios (2.5 and 5) the diagonal reinforcement was not as efficient due to its lower angle of inclination and hence reduced contribution to shear resistance.

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Steel link beams serve as the primary energy absorbing elements in eccentrically braced frames, a role similar to that played by coupling beams in coupled wall systems. Research (e.g., Roeder and Popov, 1978, Malley and Popov, 1983a and 1983b, and Engelhardt and Popov, 1989) has shown that steel link beams in eccentrically braced frames, particularly when they are designed to yield in shear while remaining elastic in flexure, can be detailed to provide excellent ductility and energy dissipating characteristics.

The capacity of the embedment of the steel beam into the walls must be sufficient to develop the probable capacity of the coupling beam. This may be achieved by designing the embedment using the approach developed by Marcakis and Mitchell (1980) for precast connections (PCI, 1985 and CPCI, 1987).

This paper summarizes the results of an experimental programme at McGill University investigating the use of steel link beams, with their ends in embedded in concrete walls, to provide ductile coupled wall systems.

"SHEAR-CRITICAL" SPECIMENS

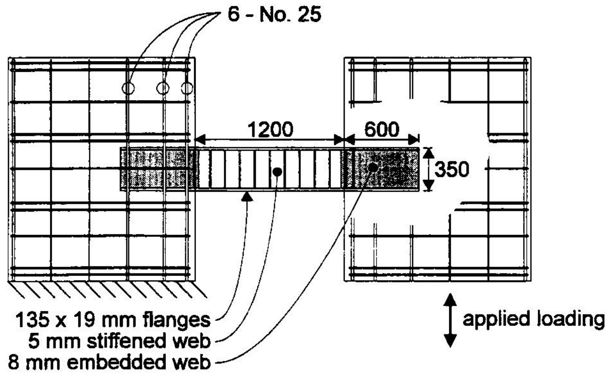
Specimens S2 and S3 (see Figs 1(a) and 2(a)) were designed to yield in shear while remaining elastic in flexure, without significant distress in the concrete embedment regions. The detailed design procedure for these "shear-critical" coupling beams and their embedments is given by Harries et al. (1993). Specimen S2, having a span-to-depth ratio of 3.43, approaches the practical upper limit for "shear-critical" design. In order to satisfy the capacity design criteria, this specimen had to be fabricated as a built-up section with web stiffeners. Specimen S3 had a span-to-depth ratio of 1.29, and was fabricated from a stiffened Class 1 rolled section.

Each specimen was tested to failure under reversed cyclic loading in the McGill University Coupled Wall Testing Apparatus (Harries et al. 1993). The observed hysteretic responses of Specimens S2 and S3 are shown in Figs 1(b) and 2(b). The hysteretic response of each specimen shows large stable hysteretic loops through ductility levels of 10 times the yield displacement ($10\delta_y$). The responses are typical of ductile steel link beams, of comparable span-to-depth ratios, designed to yield in shear (Engelhardt and Popov, 1989).

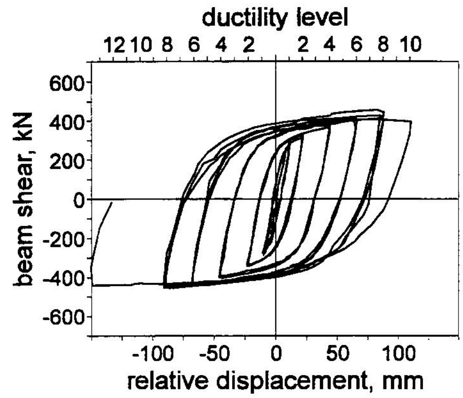
Specimen S2 experienced only a 20% reduction of load carrying capacity at a ductility level of $10\delta_y$ and maintained this capacity when loaded monotonically to a ductility level of $-16\delta_y$. The controlled web buckling of Specimen S2, indicated that the desired ductile shear yielding was achieved (see Fig. 1(c)).

Specimen S3 experienced a web tearing failure at a ductility level of $10\delta_y$. Such a failure is typical of shorter shear links in eccentrically braced frames (Malley and Popov, 1983b). It is clear from Fig. 2(c) that the response of Specimen S3 was dominated by ductile shear yielding.

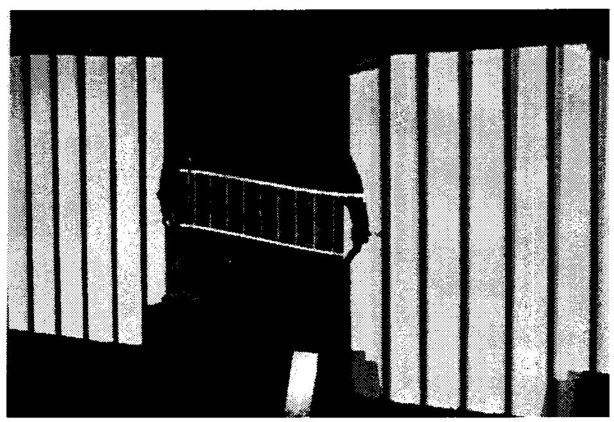
Figure 3(a) shows the cumulative energy absorption and Fig. 3(b) gives the cumulative energy absorption normalised by the cumulative energy absorption at yield for the specimens tested. Both "shear-critical" specimens absorb significant amounts of energy. The efficiency of Specimen S3, with a smaller span-to-depth ratio, is evident in Fig. 3(b).



(a) Details of Specimen S2

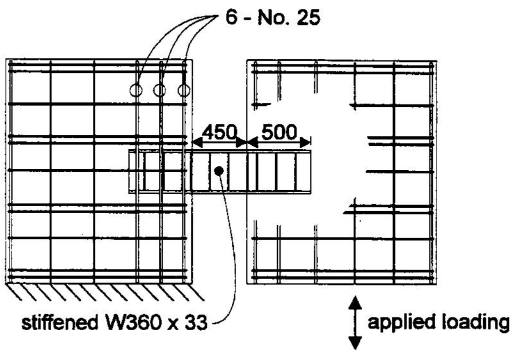


(b) Hysteretic behaviour of Specimen S2

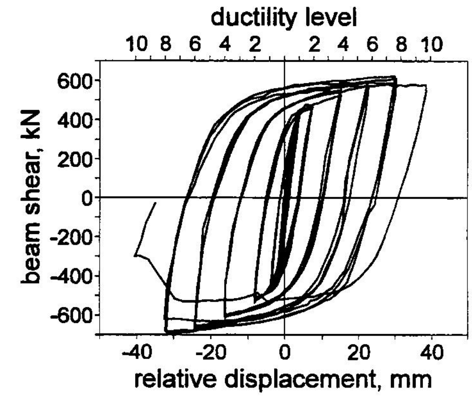


(c) Specimen S2 at the end of testing

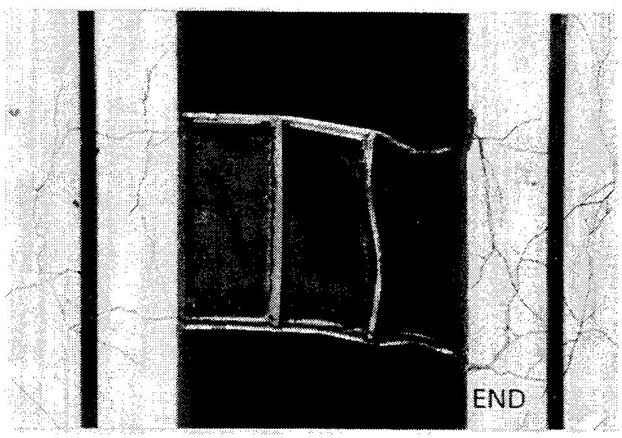
Figure 1 "Shear-critical" Specimen S2



(a) Details of Specimen S3



(b) Hysteretic behaviour of Specimen S3



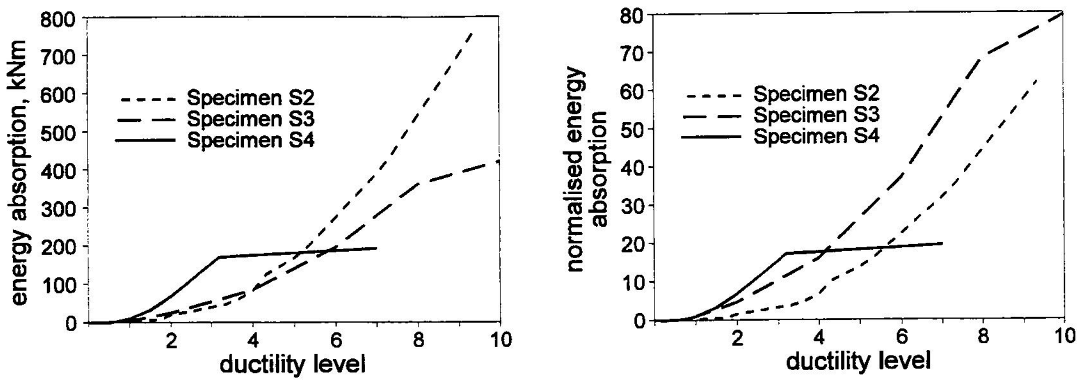
(c) Specimen S3 at the end of testing

Figure 2 "Shear-critical" Specimen S3

"FLEXURE-CRITICAL" SPECIMEN

Specimen S4 (see Fig. 4(a)), having a span-to-depth ratio of 3.43, was designed as a "flexure-critical" coupling beam. Selecting a Class 1 rolled section without web stiffeners ensured that flexural yielding occurred before shear yielding.

The reversed cyclic response of Specimen S4 (Fig. 4(b)) shows relatively large, stable hysteretic loops throughout the response history, typical for steel beams yielding in flexure.



(a) cumulative energy absorption

(b) energy absorption normalised by energy absorption at yield

Figure 3 Cumulative hysteretic energy absorption of Specimens S2, S3 and S4

No significant strength or stiffness degradation was noted through cycling to a ductility level of $3\delta_y$. Further monotonic loading to a ductility level of $-7\delta_y$ resulted in only a 20% drop in the load carrying capacity. Figure 4(c) shows the flexural hinges which formed near the face of each wall.

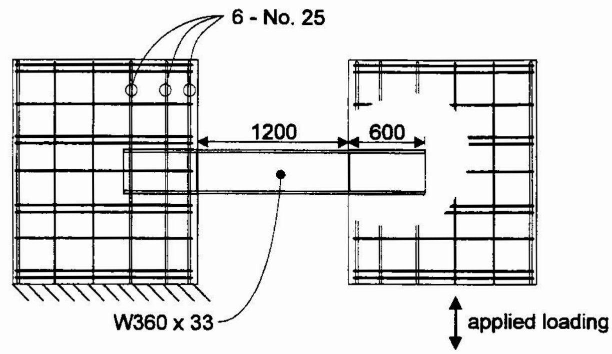
This "flexure-critical" specimen exhibits greater energy dissipation than the "shear-critical" only up to ductility levels of 3. Local flange buckling in the hinge region of the "flexure-critical" specimen limits the energy absorption, while the "shear-critical" specimens continue to absorb large amounts of energy at much higher ductility levels (see Fig. 3).

DESIGN USING NBCC

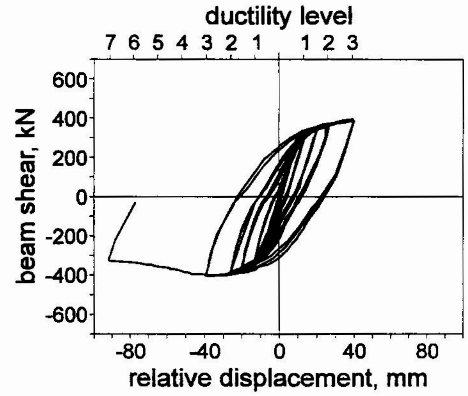
The degree of coupling of walls is the percentage of the total base overturning moment resisted by the couple produced by the axial compression and tension in the walls resulting from shears in the coupling beams. In CSA A23.3-94 (CSA, 1994), Clause 21.1 defines a "ductile coupled wall" as one "*where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shear in the coupling beams*". Similarly, a "ductile partially-coupled wall" is defined as one whose degree of coupling is less than 66%.

The CSA A23.3 Standard allows a force modification factor, R, of 4.0 to be used for ductile coupled walls, while a force modification factor of 3.5 is permitted for partially coupled walls. For comparison, ductile flexural walls (ie: cantilever walls) are assigned a force modification factor of 3.5. These differences take account of the improved reversed cyclic loading response and energy dissipating characteristics when a number of beams, carrying significant shears, form inelastic hinges.

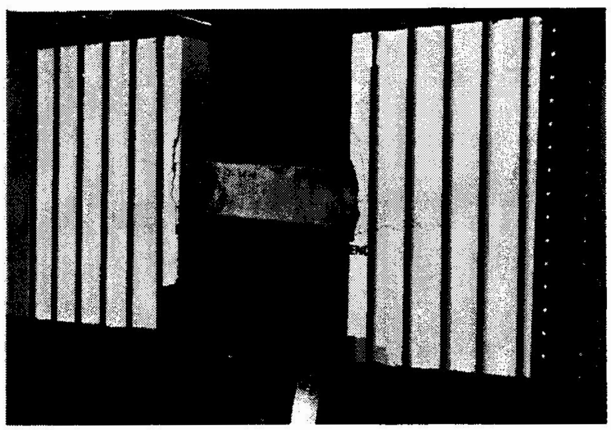
Clause 21.5.8 requires that diagonal reinforcement be used in coupling beams unless the factored shear stress is less than $0.1(\ell_u/d)\sqrt{f'_c}$ and the span-to-depth ratio, ℓ_u/d, is greater than 4.0. The factored shear stress in the beam is limited to $1.0\sqrt{f'_c}$. Conventionally reinforced coupling beams must satisfy the requirements of Clause 21.3 for ductile beams.



(a) Details of Specimen S4



(b) Hysteretic behaviour of Specimen S4



(c) Specimen S4 at the end of testing

Figure 4 "Flexure-critical" Specimen S4

Diagonal reinforcement, however has been shown to have significantly reduced efficiency for span-to-depth ratios greater than about 2.0 (Shiu et al., 1978).

Figure 5 shows the equivalent elastic damping coefficient, ß, for the three steel coupling beam specimens, the diagonally and conventionally reinforced concrete coupling beams tested by Shiu et al. (1978) and a steel "shear link" in an eccentrically braced frame tested by Engelhardt and Popov (1989). The elastic damping coefficient, defined in Fig. 5, is a measure of the coupling beam's ability to dissipate energy. It is clear that the "shear-critical" steel coupling beams are capable of dissipating as much energy as a well designed "shear link", and significantly greater energy than reinforced concrete coupling beams. The

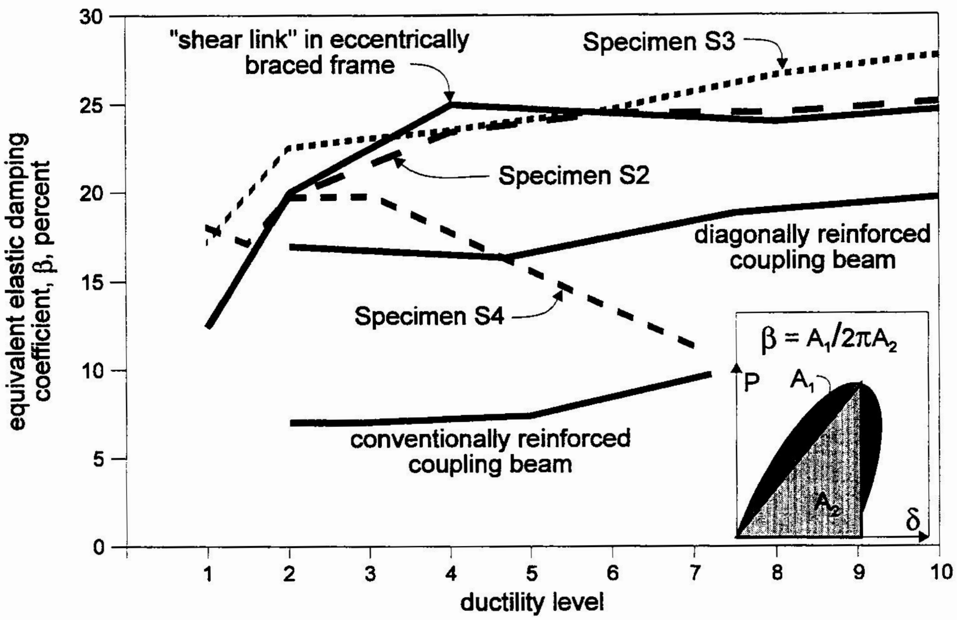


Figure 5 Equivalent elastic damping coefficients for Specimens S2, S3 and S4, reinforced concrete coupling beams (Shiu et al., 1978) and steel "shear link" in eccentrically braced frame (Engelhardt and Popov, 1989)

"flexure-critical" steel coupling beam, Specimen S4, while not as efficient as the "shear-critical" beams, was capable of absorbing larger amounts of energy than the conventionally reinforced coupling beam. With hinging, however the efficiency of the flexural beam decreased, although it still had larger hysteretic damping than the conventionally reinforced concrete coupling beam. The excellent energy dissipation provided by steel coupling beams is applicable over a wider range of span-to-depth ratios than either conventionally or diagonally reinforced coupling beams.

CONCLUSIONS

This experimental programme has demonstrated the following:

i) "Shear-critical" steel coupling beams are suitable for use in ductile coupled wall systems as they offer larger ductilities and energy absorption than diagonally reinforced coupling beams.

ii) "Flexure-critical" steel coupling beams are suitable for use in ductile partially-coupled wall systems since they provide greater energy absorption than conventionally reinforced coupling beams.

ACKNOWLEDGEMENTS

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