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Concrete walls coupled by ductile steel link beams

K.A. Harries, W.D. Cook, R.G. Redwood & D. Mitchell
McGill University, Montréal, Canada

ABSTRACT: Preliminary experimental results are reported on the response of embedded steel coupling beams linking reinforced concrete shear walls. The excellent performance under reversed cyclic loading, together with the ease of construction, demonstrate the viability of this alternative form of construction. Steel coupling beams, designed for large ductility and energy absorption, are comparatively simpler to construct than their reinforced concrete counterparts. The use of steel coupling beams enables larger ductilities and larger amounts of energy absorption to be attained compared with conventionally reinforced or diagonally reinforced concrete coupling beams. Preliminary design and detailing guidelines, to ensure that the steel coupling beams exhibit significant ductility and energy absorption, are presented.

1 INTRODUCTION

Following the 1964 Alaskan earthquake, considerable attention was devoted to improving the response of reinforced concrete coupling beams. Experimental work, under the direction of Paulay, led to the development of design guidelines for diagonally reinforced concrete coupling beams (see Santhakumar, 1974, Park and Paulay, 1975 and Paulay, 1986). These diagonally reinforced concrete coupling beams offer improved ductility and energy absorption over traditionally reinforced concrete coupling beams.

The National Building Code of Canada (1990) determines the design lateral seismic base shear by dividing the elastic base shear by a force modification factor, R. The force modification factor varies from 1.5 to 4.0 for reinforced concrete structures, reflecting the ability of the structure to dissipate energy through inelastic action. For ductile flexural wall systems, R is 3.5. For the design of coupled ductile flexural wall systems, the Canadian concrete design code (CSA, 1984) requires that in-plane shear and flexure in coupling beams be resisted by diagonal reinforcement. However, for coupling beams having relatively large span-to-depth ratios, subjected to low shear levels, the CSA code permits the use of well-detailed longitudinal and transverse reinforcement for these critical members.

This paper presents the preliminary results of a research program investigating the feasibility and response of reinforced concrete shear walls coupled by ductile steel beams. The primary objective of this program is to develop design and detailing guidelines to enable steel coupling beams to develop large levels of ductility and significant energy absorption capabilities.

2 DESIGN CONSIDERATIONS

The use of steel coupling beams to connect reinforced concrete walls has the following potential advantages:

1. Properly designed and detailed steel coupling beams can exhibit excellent ductility and energy absorption.
2. The prefabrication of steel coupling beams provides improved quality control and eliminates a considerable amount of on-site labour.
3. Formwork can be significantly simplified.

Research on the response and design of steel link beams, in eccentrically braced frames, provides valuable guidance for achieving extremely high levels of ductility and energy absorption in these critical elements (see Malley and Popov, 1983 and Engelhardt and Popov, 1989). The Canadian steel design code (CSA, 1989) provides design provisions for eccentrically braced frames including design and detailing of ductile link beams based on the SEAOC (1988) code provisions.

The response and design of embedments of steel sections in reinforced concrete has been investigated by Marcakis and Mitchell (1980) and forms the basis for the PCI (1985) and CPCI (1987) design recommendations for precast concrete embedments.

The lessons learned from research on the response of ductile steel link beams, in eccentrically braced frames, and on precast concrete embedments provide useful guidance for the design and detailing of ductile steel coupling beams.

3 DESCRIPTION OF TEST SPECIMENS

Two full-scale test specimens having steel coupling beams embedded in reinforced concrete walls were tested under reversed cyclic loading. The steel coupling beam and the reinforced concrete embedment region were designed in accordance with the Canadian steel design code (CSA, 1989) provisions and the CPCI (1987) design guidelines, respectively. The clear spans of the coupling beams between the walls were designed and detailed as ductile steel link beams in eccentrically braced frames. These beams were designed to remain elastic in flexure while undergoing significant web shear yielding to maximize the ductility and energy absorption. The embedments of the coupling bear in the reinforced concrete walls were designed suc that the full capacity of the coupling beam could b attained.

Figure 1 shows the coupling beam and wa reinforcement of Specimen 1. The coupling beam had web stiffeners in order to control web and flang instability (see Fig. 2).

The uniformly distributed wall reinforcement wa chosen to conform with the minimum amounts o reinforcement required for walls (CSA, 1984). Th concentrated reinforcement at the edges of the wall was chosen to control the crack opening along th coupling beam flange-to-wall interface. The details o the wall reinforcement are given in Fig. 3.

4 TESTING SETUP AND PROCEDURE

Figure 4 shows the details of the test setup. Th reinforced concrete walls are post-tensioned to two beams with one wall held fixed by post-tensioning to the reaction floor. The other wall is loaded, through the loading beam, in a reversed cyclic manner. 'Load control' was used up to general yielding of the coupling beam, while 'deflection control' was used thereafter. Three cycles of loading were applied at each load or deflection increment. Positive (upwards) loading is applied using the positive loading ram above the reaction floor, while negative (downwards) loading is applied by tension rods and rams beneath the reaction floor. The loading beam is kept parallel to the fixed beam by adjusting with a levelling ram.

Load cells and LVDTs were used to monitor applied loads and resulting deflections. The coupling beam was heavily instrumented over its clear span to

Figure 1. Overall view of reinforcing cage and coupling beam of Specimen 1.

determine flange and web strains. Electrical resistance strain gages were used to monitor strains in the longitudinal and transverse reinforcement in the embedment regions. Surface targets and strain gages enabled the determination of concrete strains in the embedment regions.

5 EXPERIMENTAL RESULTS

Figure 5 shows the shear versus displacement response of Specimen 1.

First yielding occurred in the web of the coupling beam at a shear of 250 kN and a deflection of 6.3 mm. General yielding, in shear, occurred in the web at a load of 303 kN at a deflection, δ_y, of 12 mm. The specimen exhibited excellent hysteretic response with cycling up to $\pm 8\delta_y$. The peak load of -409 kN was attained at a deflection -76 mm. At the end of testing, the specimen was loaded monotonically to a peak deflection of 123 mm, corresponding to about $10\delta_y$. An overall view of the specimen, after testing, is shown in Fig. 6.

After achieving the maximum capacity, the peak loads attained did not drop below 80% of this capacity. Significant shear yielding of the coupling beam was observed. Progressive spalling of the concrete resulted in an increase in the clear span of the coupling beam, leading to increased moments in the beam. Spalling also led to progressive loss of confinement along the embedment, eventually causing web crippling due to the concrete bearing reaction. It was evident, upon removal of the coupling beam from the walls (see Fig. 7), that shear yielding in the web had progressed into the embedded region, and that significant web crippling had occurred. The yielding of the web of the embedded steel, under cyclic loading, resulted in an outwards 'ratchetting' movement of the embedded steel member from the concrete. This, together with the spalling of the cover concrete, resulted in a reduced embedment length and a corresponding

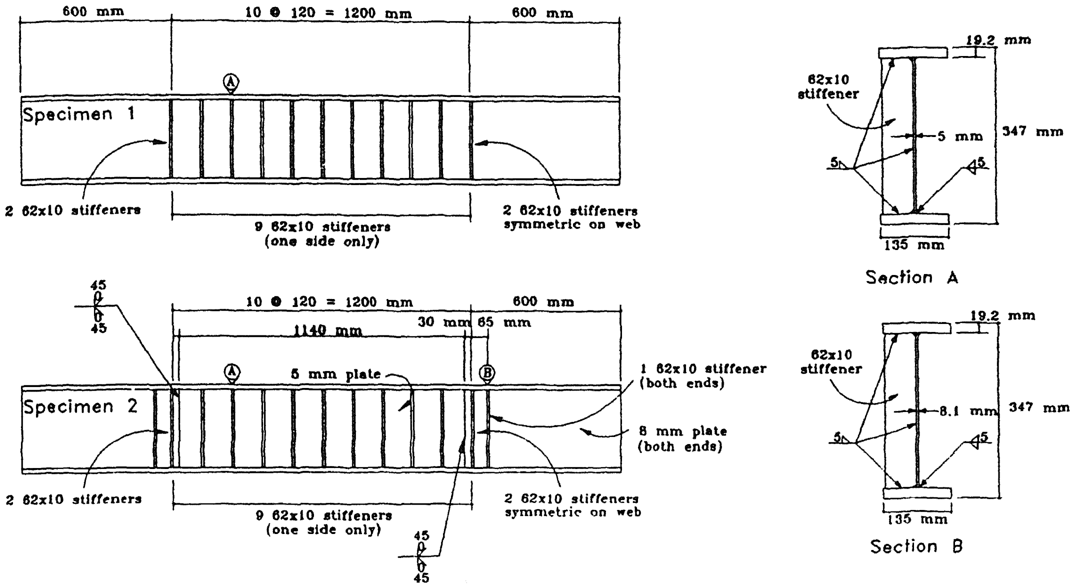

Figure 2. Details of coupling beams.

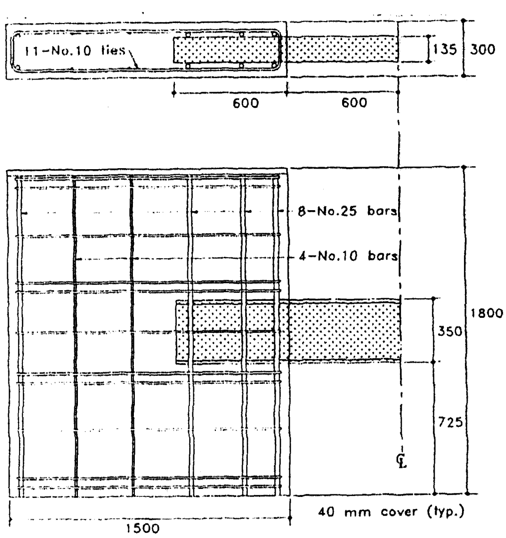

Figure 3. Details of reinforcement.

increased clear span.

In order to improve the overall response, Specimen 2 was designed to anticipate the effects of cover spalling and to prevent yielding of the steel beam over its embedment length. As can be seen in Fig. 2, a thicker web plate and additional stiffeners were provided in the embedded regions.

Figure 8 shows the shear versus displacement response of Specimen 2. First yielding occurred in the web of the coupling beam at a shear of 230 kN, while general yielding in the web occurred at a load of 274 kN at a corresponding deflection, δ_y, of 11 mm. The specimen exhibited excellent hysteretic response with cycling up to $\pm 10\delta_y$. The peak load of -446 kN was attained at a deflection of -90 mm. At the end of testing, the specimen was loaded monotonically to a peak deflection of -150 mm, or about $14\delta_y$. Figure 9 shows an overall view of this specimen after testing.

This specimen demonstrated its ability to maintain a significant load level (at least 80% of maximum load) upon cycling after general yielding. The web in the clear span exhibited significant shear yielding with tension field action at large ductility levels. Web buckling between the stiffeners resulted in the stiffeners bending in double curvature. Despite the progressive spalling of the concrete, strain measurements on the embedded web indicated that it remained essentially elastic. Upon removal of the coupling beam from the walls (see Fig. 7), it was clear that the web of the embedded region had not experienced any significant yielding and that the 'ratchetting' effect was minimized. This specimen demonstrated excellent ductility and hysteretic response while maintaining its load carrying ability. The response achieved is similar to that exhibited by ductile link beams in eccentrically braced frames.

6. COMPARISON OF RESULTS

Figure 10 compares the applied shear versus displacement envelopes for Specimens 1 and 2 for

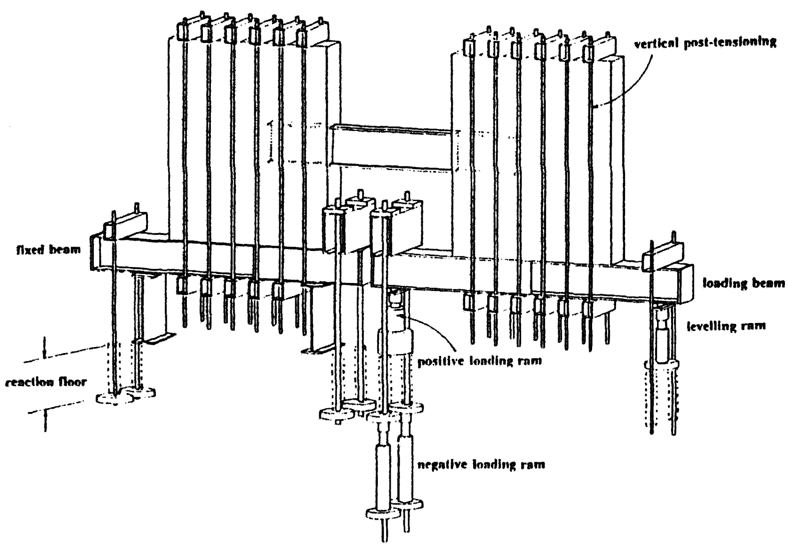

Figure 4. Test setup.

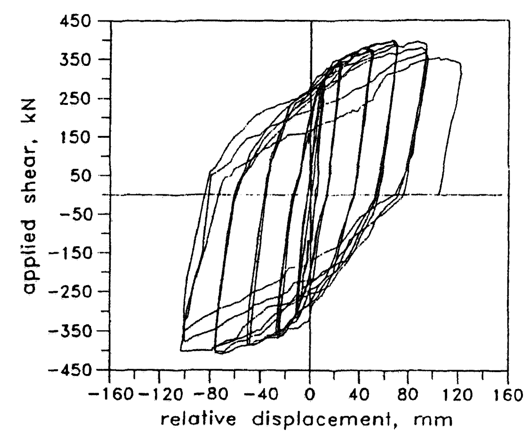

Figure 5. Applied shear versus relative displacement for Specimen 1.

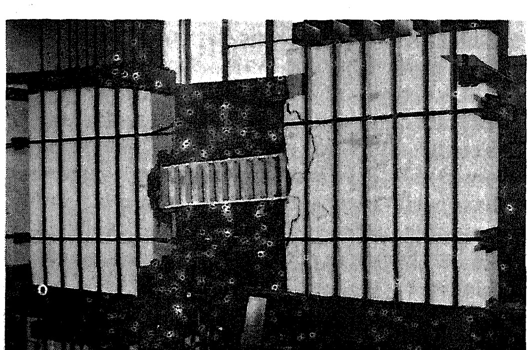

Figure 6. Overall view of Specimen 1 after testing.

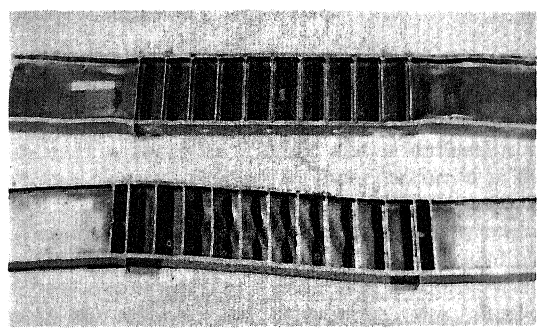

Figure 7. Residual deformations in coupling beams of Specimens 1 (top) and 2 (bottom) after removal from walls.

the response up to and including a displacement ductility level of 8. As can be seen, Specimen 2 achieved higher shear forces in the coupling beam, despite a slightly lower yield stress of the web material (302 MPa versus 320 MPa). Additionally, Specimen 2 exhibited larger post-yielding stiffnesses and larger amounts of energy absorption.

Figure 11 shows the variation of the equivalent elastic damping coefficient, β, for different displacement ductilities. This coefficient, β, is defined in Fig. 11 and is shown for both specimens for the first and third cycles at each ductility level. The reduction in β between the first and third cycles illustrates the reduction of damping with cycling. Specimen 2, not only displays larger values of damping, but also exhibits a smaller decay of damping with cycling.

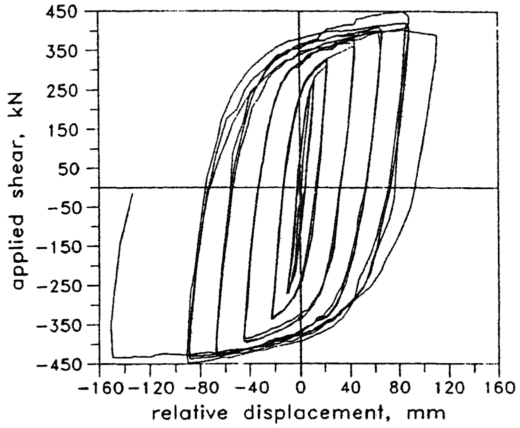

Figure 8. Applied shear versus relative displacement for Specimen 2.

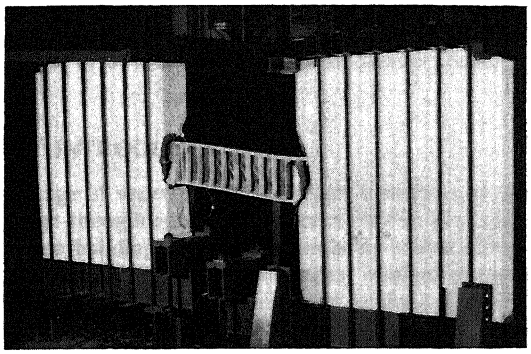

Figure 9. Overall view of Specimen 2 after testing.

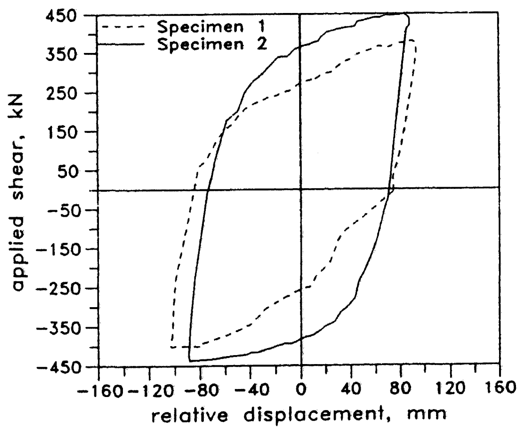

Figure 10. Shear versus displacement envelopes for the response of Specimens 1 and 2 up to a displacement ductility of 8.

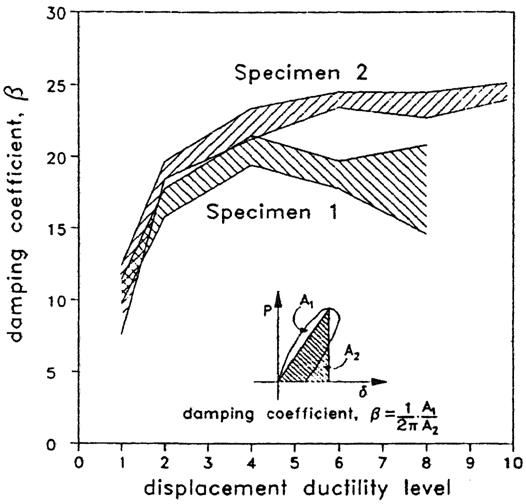

Figure 11. Hysteretic damping coefficient versus displacement ductility of Specimens 1 and 2.

7. CONCLUSIONS

These preliminary test results have led to the following conclusions:

1. It is possible to achieve excellent ductility and energy absorption by carefully designing and detailing the steel coupling beams and the reinforced concrete embedment regions.

2. In coupling beams having small to medium span-to-depth ratios, the excellent response, similar to that of ductile link beams in eccentrically braced frames, can be achieved provided that the following measures are taken:

   a. The coupling beam, in the clear span, is designed and detailed to remain elastic in flexure.

   b. The web in the clear span is stiffened to prevent web or flange instabilities, enabling significant shear deformations to develop beyond yielding.

   c. Additional stiffeners are provided inside the embedment, in the region of expected cover spalling.

   d. If the web of the coupling beam is strengthened within the embedment, even greater ductility and energy absorption can be attained.

3. In order to ensure that the coupling beam can perform in the required ductile manner, the reinforced concrete embedment regions must be designed as follows:

   a. The embedment must be designed for a shear and moment corresponding to the development of the full capacity of the coupling beam.

   b. In calculating the embedment strength for the applied shear and moment, a reduced embedment length must be used to account for cover spalling.

c. Vertical reinforcing bars, placed near the face of the embedment must provide adequate control of the cracking at the coupling beam flange-to-concrete interface.

4. The use of prefabricated steel coupling beams resulted in simpler formwork and construction as compared to reinforced concrete coupling beams.

These preliminary tests demonstrate that steel coupling beams linking reinforced concrete walls are a viable alternative to diagonally or traditionally reinforced concrete coupling beams. The experimental evidence suggests that larger levels of ductility and energy absorption can be achieved by using ductile steel coupling beams instead of reinforced concrete coupling beams. This research is continuing in order to develop specific design and detailing requirements for these ductile systems.

ACKNOWLEDGEMENTS

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