ENERGY DISSIPATING PRE-CAST COUPLING BEAMS

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Introduction

The good performance of structural systems either of shear walls only or in combination of shear walls led the designers to the use of shear wall systems. The failures in shear wall systems under seismic excitations usually concentrate on the coupling beams; and on the shear walls at the further deformation steps. (1, 2)

Investigations on the reinforced concrete beam reinforcement detailing revealed that conventional beam detailing results in poor performance in case of coupling beams. Experimental work on the behavior of coupled shear wall systems led to the development of diagonal beam reinforcement for the coupling beams. At present, detailing with diagonal reinforcement has been widely practiced in many earthquake codes including the most recent Turkish Earthquake Code (3)

Experimental and analytical studies on the coupling beams reported in the literature generally concentrate on the ductile design and detailing of these beams

The behavior of ductile steel coupling beams used instead of reinforced concrete coupling beams was also investigated by several researchers (4, 5). It has been reported that (5), properly designed and detailed steel coupling beams can exhibit excellent ductility and energy absorption. The prefabrication of the steel coupling beams provides improved quality control in addition to the advantages such as elimination of considerable amount of site work and simplified formwork. The steel coupling beams reported in the literature have two connection types to the reinforced concrete shear walls. In the first type (5), the steel beams were cast with the walls, having an embedded portion in the shear walls. In the second type (4), the steel beams were fit to their location after the concrete walls were cast, and the connections of the coupling beams to the shear walls were semi-rigid, designed by using either bolted or welded connections. The cast-in-place steel coupling beams were reported to be designed to remain elastic in flexure while undergoing significant web shear yielding to maximize the ductility and the energy absorption. For the case of on-site semi rigid connection of the steel coupling beams and the shear walls, the designers were also aimed at the shear yielding of the connection in order to dissipate maximum amount of energy. The semi-rigid connections of steel beams (4) were reported to be beneficial over the cast-in place ones, since they were replaceable if they yield under high seismic excitations.

Specimen Design and Testing Procedure

The idea of using "energy dissipating pre-cast reinforced concrete coupling beams" stemmed from the advantage of the semi-rigid connections of the steel coupling beams in energy absorption and the ease of construction of pre-cast concrete. Besides, the energy dissipated through cracking of concrete in addition to the energy dissipated at the connections was considered to be an added advantage.

The investigations on the beam-to-column connections in pre-cast concrete structural systems reveals that the initial stiffness of the connection mainly depends on the connection geometry and the tolerances made for the lack of fit etc. at the site. Beyond the limit of the gap, the connection behavior is governed by the material strength, i.e. the crushing and shear strength of concrete and the tensile strength of the reinforcement used at the connection. As in the case of cast-in-place connections, the ductility of the semi-rigid pre-cast connection is mainly a function of the ductility of the reinforcement (6).

The four reinforced concrete shear-wall-to-coupling-beam sub-assemblies tested in this research project were designed to be approximately as full size specimens. In the experimental investigation, the main variable was the type of the connection between the shear wall and the coupling beam.

Two types of pre-cast connections were used, namely WELD and ROD (Figure 1). In the specimen WELD, the steel strips protruding from the pre-cast reinforced concrete coupling beam were sandwiched between the two steel angles fixed to the shear wall and welded to each other. In the specimen ROD, however, the strips and the angles were connected via two 20 mm diameter steel rod passing through the pre-drilled holes on the strips and on the angles. The gap (about 105mm) between the coupling beam and the pre-cast shear wall was filled with High Strength Concrete (HSC) grout (f'_c=65-70 MPa). On the other hand concrete compressive strength values of the shear walls and the coupling beams were around f'_c=25 MPa, which was in Normal Strength Concrete (NSC) range.

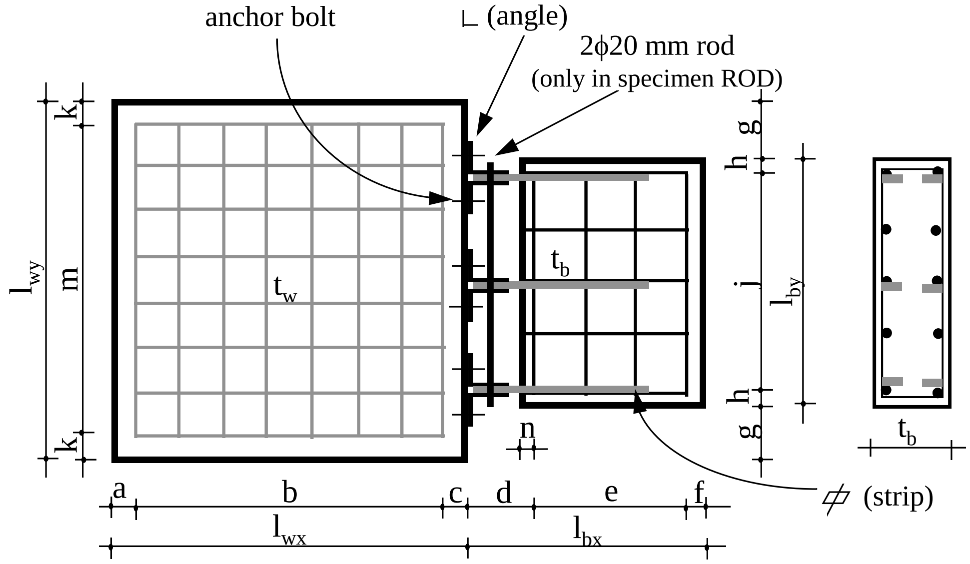

FIGURE 1 - Dimensions and Reinforcement Detail of Specimens WELD and ROD

Two cast-in-place specimens, namely MONO1 and MONO2 were also tested for comparison purposes. Specimen MONO1 was a pilot test in order to check the capacity and functioning of the loading and measuring systems, thus specimen MONO2 was used for further comparisons. The geometric and the material dimensions of all the specimens tested are given in (Table 1). The yield strength of ϕ10 (10mm in diameter) and ϕ14 rebars are 507MPa and 471MPa respectively.

TABLE 1 - Dimensions of Specimens (with reference to FIGURE 1)

	MONO1	MONO2	WELD	ROD
t_w	200	200	200	200
t_b	200	160	160	160
l_{wx}	1070	1200	1200	1200
l_{wy}	1060	1200	1200	1200
l_{bx}	675	600	600	600
l_{by}	752	500	500	500
a	60	35	35	35
b	8ϕ14@940	10ϕ14@1130	10ϕ14@1130	10ϕ14@1130
c	70	35	35	35
d	50	105	105	105
e	5ϕ10@595	4ϕ10@465	4ϕ10@465	4ϕ10@465
f	30	30	30	30
g	154	350	350	350
h	30	30	30	30
j	6ϕ10@692	5ϕ10@440	5ϕ10@440	5ϕ10@440
k	40	35	35	35
m	8ϕ14@980	10ϕ14@1130	10ϕ14@1130	10ϕ14@1130
n	-	-	5	5
angle	-	-	L80*80*10	L80*80*10
strip	-	-	2*3*□25*10	2*3*□50*10
Anchor Bolts : M16, St37, $L_{anchorage}$=160 mm, 2 bolts on each angle				
Connecting Rod: ϕ20, St37				

The four shear-wall coupling beam subassemblies (MONO1, MONO2, WELD, ROD) were tested under increasing reversed cyclic loading. Loads were applied in an incremental manner until failure. A displacement based loading regime was applied after the yield point. The loading setup and the specimens were positioned in the horizontal plane as shown in Figure 2. The shear wall was firmly fixed to the strong floor by means of four threaded steel rods. The loading was performed by using two 600 kN capacity manually driven hydraulic rams. The specimens were instrumented by electronic sensors such as a Load Cell and several LVDT's.

Experimental Results

The reversed cyclic tip point load versus measured net tip deflection graphs of monolithic and pre-cast specimens are given in Figure 3. The loading steps in specimens MONO1 and MONO2 was a predetermined percentage of the calculated failure load, where the calculated failure load was the minimum of the shear and the flexural capacities of the coupling beam. On the other hand, the loading step for specimens WELD and ROD until the experimental yield load was 20kN in either direction. A displacement controlled loading regime was applied in all of the specimens after the yield point.

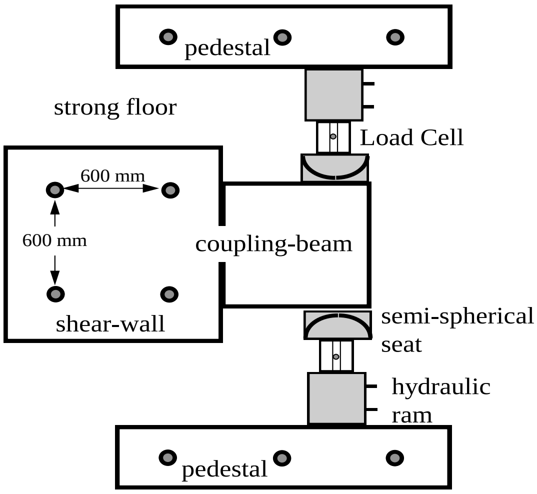

FIGURE 2 - The Loading Setup of the Specimens

Specimen MONO1 failed in a brittle manner. The load capacity of the section dropped down suddenly with increasing deflections. The reversed load capacity of this specimen in the last cycle was nearly 40% of the experimental yield load capacity observed in the previous cycle (Figure 3-a). Specimen MONO2 failed in a relatively ductile manner. The load capacity of the specimen in the next reversed loading right after yielding was approximately equal to the section capacity at the time of first yielding. The proceeding load cycle in the direction of the first yield was about 80% of the yield capacity (Figure 3-b). Pinching is not observed in the hysteretic curves of the monolithic specimens. The cracks were concentrated on the beam and along the joint line at the time of failure. Cracking on the shear wall was not observed on either monolithic specimen.

The load deflection behavior of specimen WELD, shows a ductile type of failure (Figure 3-c). Specimen could carry loads approximately equal to the experimental yield load until nearly two times the yield deformation. The drop of load in the next cycle after yielding was in the range of 10%. The cracks at failure were concentrated both in the coupling beam beyond the HSC region, and along the joint line. Different from that of the monolithic specimens, cracking was also observed in the shear-wall. Compared to the monolithic specimens, pinching type of deformation was observed in the load deflection curve.

The failure behavior of specimen ROD was relatively sudden. The failure occurred when the width of cracks on the shear wall suddenly started to increase. The line of cracks on the shear wall in specimen ROD was following the line of tips of anchor bolts in the shear wall. The specimen failed suddenly, and the proceeding load capacity in the direction of the first yield load was approximately 35% of the yield load. The pinching type of deformation was also observed in specimens ROD (Figure 3-d). The cracks in the prefabricated sub-assemblies spread into the shear-walls.

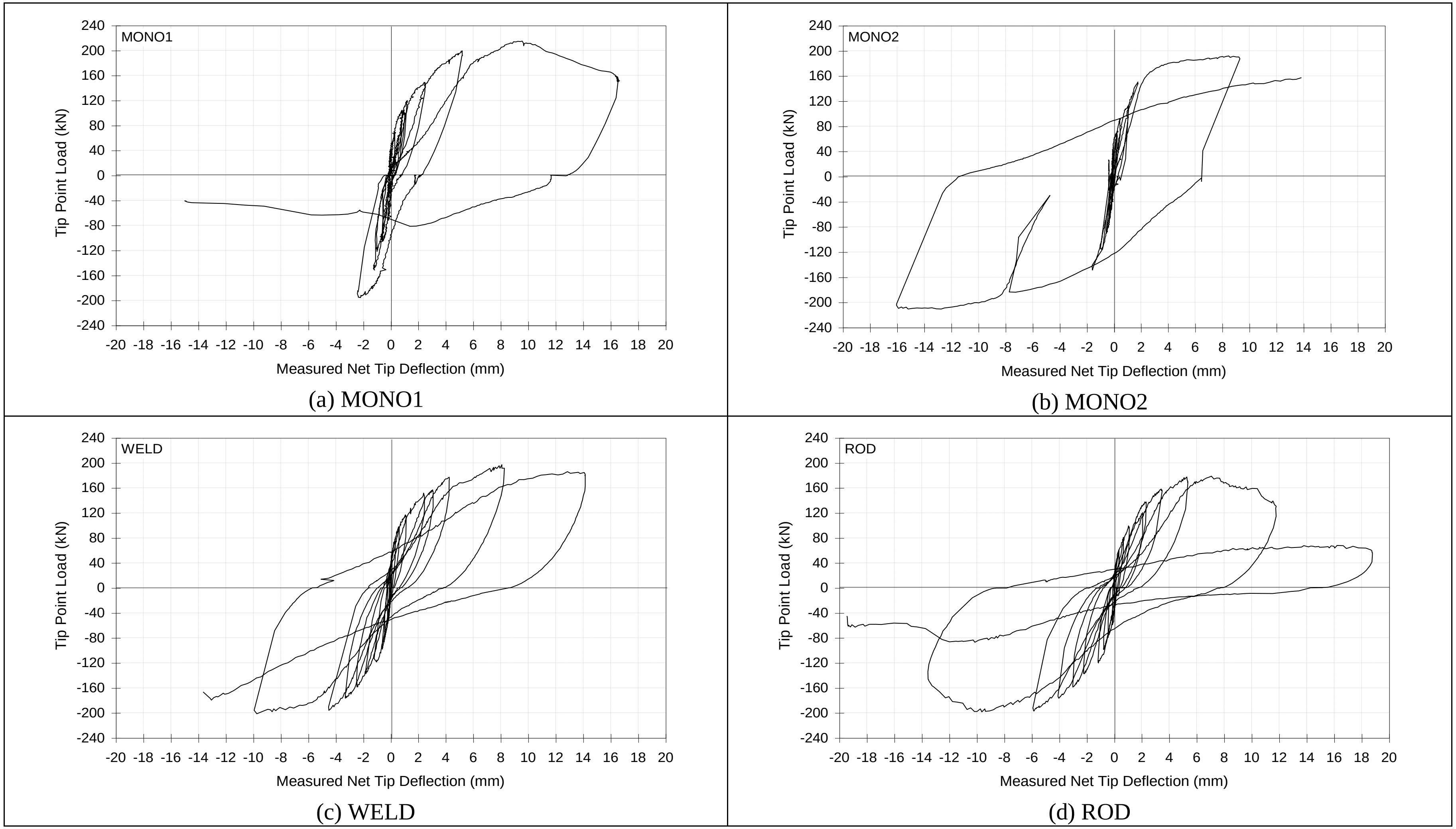

Figure 3 - Reversed Cyclic Load - Measured Net Tip Deflection Graph of Specimens

Behavior of Test Specimens

In specimen MONO2, all cracking was confined to the coupling beam, since the beam is uniform and the anchorage length of the beam reinforcement is long enough to prevent a premature slip deformation. The flexural capacity of the shear wall is very high in comparison to that of the coupling beam. In specimen WELD, the flexural capacity of the NSC coupling beam is above the flexural capacity of the HSC wet joint. In this specimen, the first yield load capacity of the anchor bolts were above the yield load capacity of the wet joint but below the yield load capacity of the coupling beam. Therefore, the failure was confined in the joint region. The failure cracks in specimen ROD were confined to the region on the shear wall connecting the HSC wet joint to the shear wall. The section capacity at the HSC wet joint was well above the first yield load of the anchor bolts. Yielding of the anchor bolts caused a rapid deterioration of bond strength resulting in slip outs of the bolts and concentration of cracks in the shear wall.

The reversed cyclic tip point load versus measured net tip deflection plots are given in Figure 3. Although , only the reversed cyclic load-deflection curve is enough to comment on the suitability of the connection for ductile seismic design, the reversed cyclic load versus, the tip deflection due to slip at the joint, the tip deflection due to shear deformation at the joint and the tip deflection due to rotation at the joint plots of all specimens were also investigated in order to identify the main mode of energy dissipation. Accordingly, in all the specimens, least energy was dissipated in the flexural rotation mode. In this mode, the prefabricated specimens dissipated more energy compared to the monolithic specimens. Monolithic specimens dissipated more energy in shear deformation mode compared to their joint slip mode. On the other hand, the energy dissipated due to shear deformation at the joint was higher in comparison with the monolithic specimens. When all the deformation modes namely; slip, shear, and flexural rotation deformations considered, all the specimens dissipated least energy in flexural rotation mode and in every mode, the energy dissipated by the prefabricated specimens was higher in comparison to the monolithic ones.

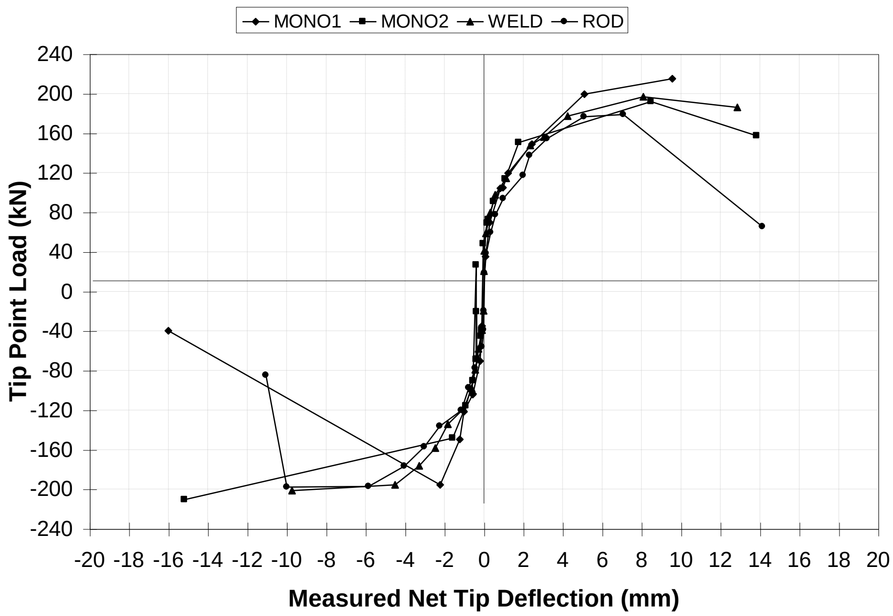

FIGURE 4 - Comparison of Envelope Curves

The load versus measured net tip deflection curves given in Figure 3, clearly show the differences between the behavior of monolithic and prefabricated specimens. The stiffness degradation in the monolithic specimen is slight; on the other hand, the stiffness degradation of the prefabricated specimens is noticeable. However, from the comparison of the envelope curves given in Figure 4, it can be concluded that the behavior of the prefabricated specimens in terms of energy dissipation and ductility is comparable to the monolithic specimens.

Conclusive Remarks

From the limited number of experimental and analytical investigations performed on the reversed cyclic behavior of the monolithic and prefabricated energy dissipating coupling beams, the following conclusions can be stated:

1- The main energy dissipation mode in monolithic specimens is the shear deformation mode.

2- Stiffness degradation and pinching effects are more pronounced in the prefabricated specimens.

3- Pre-cast specimens could carry the yield load well beyond the yield displacement.

4- The behavior of the prefabricated specimens in terms of energy dissipation and ductility is comparable to the monolithic specimens.

5- The failure zone and crack concentration regions can easily be identified from the comparison of the flexural yield capacity of the coupling beam, connection region and the anchor bolts.

6- In the prefabricated specimens, the failure of the steel strips connecting the coupling beam to the shear wall results in concentration of the cracks in the connection region, whereas the failure originating from the yielding hence slip-out of the anchor bolts causes cracking and failure in the shear wall.

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