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EXPERIMENTAL PROGRAM AND PROPOSED DESIGN METHOD FOR THE RETROFIT OF STEEL MOMENT CONNECTIONS

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SUMMARY

An experimental program was undertaken to evaluate methods to retrofit existing steel moment connections for improved seismic performance. Six full scale subassemblages were tested under cyclic loading. Typical pre-Northridge connections were retrofit either by the addition of a bottom flange reduced beam section (RBS) or by the addition of a welded bottom flange haunch. Retrofitted specimens were tested both with and without a composite floor slab. The tests showed poor performance of the bottom flange RBS retrofit when the existing low toughness welds were left in place. Somewhat improved performance was observed when the bottom flange RBS was combined with replacement of beam flange groove welds with higher toughness welds. The welded bottom haunch retrofit showed excellent performance on specimens with a composite slab, even though the existing low toughness beam flange groove welds were left in place. The presence of the composite slab appeared to help prevent fracture of the existing top flange weld in the haunch retrofit. Previous test data was referenced to determine allowable stresses on existing welds with varying states of weld improvement. This information was used to develop a consistent method for designing a connection retrofit of either haunch or RBS designs. The design method is applicable to composite or bare steel beam sections. Design is based on limiting the applied average stresses at the column face to critical weld stress values which vary depending on the degree of weld modifications. These methods could be applied towards new construction as well as a retrofit. the design method was found to predict previous test results for 19 haunch and 26 RBS specimens.

INTRODUCTION

In response to steel moment resisting frame connection failures in the 1994 Northridge Earthquake, major testing programs were initiated to establish safe connection detail standards. The purpose of this project was to evaluate two promising new construction details for use in the retrofit of existing structures. These two techniques were i) the addition of a welded haunch at the beam bottom flange, and ii) the addition of a reduced beam section (RBS), also known as a dogbone, cut at the beam bottom flange. These techniques were believed to be among the most promising for retrofit based on past experimental data. The welded haunch has shown good performance in a limited number of tests for repair and retrofit of existing connections (Shuey and Engelhardt 1996, Uang and Bondad 1996) but has not been previously tested with a slab. The RBS connection has shown good performance in new construction applications (Chen and Yeh 1994, Engelhardt et all 1996, Iwankiw and Carter 1996, Tremblay et al 1997), but has not been tested for retrofit applications. Both the RBS and haunch retrofits investigated in this program require little or no modification to the beam top flange, thereby minimizing the need to remove or alter the floor slab. This would make them well suited to retrofit applications. The project had two components, experimental testing of full scale specimens and the development of a design method for these retrofits. Acceptance criteria of 0.02 radian of plastic rotation was used in accordance with Gross et al (1999).

EXPERIMENTAL TEST SETUP AND SPECIMENS

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Tests were performed on full sized interior joint subassemblages. Points of inflection were assumed at column story mid-heights and at beam mid-spans. A typical story height and beam span were assumed. The overall test frame schematics can be seen in Fig 1.

The test specimens were chosen to be representative of building construction details in common use prior to the Northridge Earthquake, and to not duplicate specimens investigated elsewhere. Beams were W30X99 sections of A36 steel. Columns were W12X279 sections of A572 Grade 50 steel to provide strong column, weak beam action and to provide for a strong panel zone. Three pairs of specimens were tested. Each pair consisted of a bare steel specimen and a similar specimen with a composite slab attached. In the first pair the RBS retrofit was investigated. In the second and third sets a haunch retrofit with slightly differing weld procedures was investigated. In keeping with the actual construction sequence the specimens were assembled as an "original connection", and then retrofitted. Specimens of pre-Northridge design with W30X99 beams were tested previously as part of the SAC program ("Connection" 1997). These previous results were referenced as a benchmark for the performance of the retrofit specimens. Overall specimen details are compiled in Table 1. Beam materials came from four separate heats of steel with average measured stresses of: static yield of 329 MPa (47.7 ksi), dynamic yield of 345 MPa (50.0 ksi) and dynamic ultimate of 452 MPa (65.6 ksi)

The "original" connection was designed, detailed, and constructed in a manner typical of mid 1970's pre-Northridge building construction.

The RBS retrofit method required some special considerations. First, discussions with fabricators indicated that cutting an RBS in the beam top flange in the presence of a floor slab would likely be difficult and costly. Consequently, an RBS cutout was provided in the bottom flange only. Second, the flange area reduction was limited to a maximum of 50 percent of the total flange area due to concerns over the stability of the beam should larger reductions be provided. RBS specimens (DB1 and DB2) were therefore reduced with a radius cut contour with a 50 percent flange reduction at the bottom flange only.

The haunch retrofit (HCH1, HCH2, HCH3 and HCH4) consisted of welding a wide flange section into the area of intersection of the bottom beam flange and column flange. Sizing of the haunch was chosen to replicate details tested by Uang and Bondad (1996).

One of each pair of similar specimens included a 2440 mm (8 ft) wide composite slab. The goal was to observe the effects of a typical building slab on composite connection performance. Detailing was representative of past construction practice in California and was recommended by practicing engineers. Metal decking was oriented perpendicular to the beams and lightweight concrete was used. The number and location of shear studs was chosen to be representative of existing buildings. These shear studs do not provide fully composite action as defined by the AISC LRFD code. They do, however, provide the capacity of the expected maximum compressive force in the concrete slab, estimated to be 1.3fc times the effective slab area in contact with the column flange, per Du Plessis and Daniels (1972). Specimens DB1, HCH1, and HCH3 were bare steel specimens, while DB2, HCH2, and HCH4 were composite. Concrete compressive strengths on the day of testing were 33.7, 42.3, and 22.2 MPa (4883, 6132, and 3220 psi) for specimens DB2, HCH2, and HCH4 respectively.

Welds in the "original" connection were made using the self shielded flux core arc welding (SS-FCAW) process with a 3 mm (0.120 in.) diameter E70T-4 electrode. Backing bars and weld tabs were left in place for the "original" connection. All "retrofit" welds were made using SS-FCAW with a 1.8 mm (0.072 in.) E71T-8 electrode.

EXPERIMENTAL RESULTS

Results of the experimental program are included. The load versus column tip deflection as well as story drift is plotted for several specimens in Fig. 2. Photographs of selected specimens after testing are shown in Fig. 3. Specimens HCH1 and HCH2 results were very similar to HCH3 and HCH4 respectively. A summary of the test results is presented in Table 2.

Overall Performance

The bare steel bottom flange RBS (DB1) exhibited the poorest performance of all specimens tested. Bottom flange groove welds for both beams failed within the existing low toughness weld metal, near the weld-beam interface, at low levels of total plastic rotation (0.006 and 0.009 radian). The fractures appeared to initiate at the center of the flange, near the beam web cope. This specimen did not provide any increase in performance over a non-retrofitted connection.

The composite bottom flange RBS (DB2) exhibited a marked improvement over specimen DB1, achieving beam plastic rotations of 0.020 radian. Both connections, however, still failed by fracture of the bottom flange groove welds. During the 120 mm (4.80 in.) load cycles, fractures initiated at the bottom cope holes and propagated along the bottom edge of the beam web with each cycle. This was followed during the next cycle at 120 mm (4.80 in.) by the fracture of the bottom flange groove weld in both beams. The fractures initiated at the center of the beam flanges, near the beam web cope. Inspection of the weld fracture surfaces revealed some rather large slag inclusions not detected by ultrasonic testing which may have contributed to the weld failures. Specimen DB2 sustained much higher levels of plastic rotation than DB1, likely due to a substantial benefit from the higher toughness weld metal (composite slab effects were also involved.) Once the bottom flange welds of DB2 failed, the behavior was extremely poor and degraded substantially during later load cycles. Although the specimen obtained beam plastic rotations meeting or exceeding the 0.020 radian of plastic rotation acceptance criteria, the weld fractures occurred at levels very close to this acceptance criteria. Variations in slab details, such as stronger concrete, more reinforcing steel, or steel decking oriented in the other direction, may cause earlier fractures. Therefore, the connection detail tested must be viewed with caution.

Three of the four bare steel bottom haunch connections (specimens HCH1 and HCH3) failed by fracture of the existing E70T-4 top flange welds at total plastic rotations in the range of 0.012 to 0.023 radian. The similar behavior of specimen HCH3 confirmed that that the haunch retrofit is vulnerable to fracture at the existing low toughness top flange weld, even when precautions are taken to ensure that the existing weld contains no rejectable defects. The fractures appeared to initiate at the edge of the beam flanges. Little deterioration in the overall strength of the specimen was observed until the fracture propagated across the full flange width. Significant local buckling and lateral torsional buckling of the beams as well as some twisting of the column was observed in the latter cycles of the test. After weld fracture, the haunch specimens (HCH1 and HCH3) showed a significantly higher residual strength than the RBS specimen (DB1). The fourth connection (south beam of HCH1) did not fail by fracture, but simply deteriorated gradually due to local and lateral buckling. These specimens performed better than DB1 although the bottom haunch retrofit may still be vulnerable to fracture at the existing low toughness top flange welds. These results suggest that if existing welds are not replaced with higher toughness weld metal as part of a connection retrofit, then the haunch may provide a greater improvement in connection performance as compared with the bottom flange RBS.

With the addition of a composite floor slab (specimens HCH2 and HCH4) the connection behavior was excellent. All four connection beam flexural capacities deteriorated gradually due to local and lateral buckling of the beams. The top weld fractures of specimens HCH1 and HCH3 were prevented. Testing of both specimens was stopped due to testing limitations, with total plastic rotations of 0.028 to 0.055 radians.

Peak rotations were often associated with substantial loss of load carrying capacity. All connections which achieved 0.020 radian of total plastic rotation , (with the exception of the north beam of HCH3), sustained in excess of 80 percent of the peak attained moment when reaching this critical rotation.

Slab Effects and Stud Failures

During the testing of the composite specimens, failure of shear studs at the welds often occurred. Typically, shear stud welds severed, although at least one shear stud sheared completely through the stud itself. Although the specimens provided only partially composite action when gravity loading was considered, the specimens were provided with sufficient shear stud capacity to withstand a maximum concrete compressive force equal to 1.3f’c times the effective concrete area for lateral loads (width of column flange times slab thickness above the steel decking). This slab compressive force was not generally attained prior to the shear stud failures. It was not possible to conclusively identify the cause of shear stud failures in the specimens. These stud failures may indicate an inadequacy in the shear stud strength provisions of the AISC specification when applied to cyclic loading, poor weld quality for the studs in the specimens, or additional forces on the studs due to twist of the specimens.

The composite slab provided a slight increase in initial elastic stiffness, and increased positive moment capacity of ten to twenty percent as compared to the bare steel specimens. Negative moment capacities were often slightly increased by the composite slab, but this effect was not apparent in all specimens. Diagonal cracks propagating from both the column flange face and the inside edges of the far column flange were seen. It may therefore be speculated that maximum concrete compressive forces may not only be increased beyond fc, but that the effective width of the compression zone at the column face is wider than the column flange. The slab also influenced local and lateral instabilities of the specimens. Twist and distortions were much less severe in composite specimens. It appeared that top flange buckling was also controlled by the slab, and was significantly less severe than in the bare steel specimens. This, in part, may explain the absence of top flange weld failures in the composite specimens. The weld failures in both HCH1 and HCH3 appeared to initiate at the edge of the beam flange in contrast to the RBS weld fractures which appeared to initiate at the center of the welds. It is possible that beyond the brittle fractures which occurred in Northridge and in specimen DB1, the next weld failure mode may be due to low cycle fatigue from high amplitude distortions associated with local buckling of the flange. The presence of a slab appeared to control this behavior very well at the top flange.

DESIGN METHOD

In addition to the experimental tests, results of previous studies were referenced in the formulation of a design method for haunch and RBS retrofits. A simplified design model basing design on overall section properties to calculate average stresses across sections was used. By picking a simplified model, and then calibrating test results to this model, it is possible to develop overall criteria which incorporate the uncertainties related to numerous factors. This approach can therefore account for factors which are not completely understood, or which would be otherwise difficult to include. Northridge field observations found that fractures often occurred with little or no apparent inelastic deformations of the beam sections (Youssef et al 1995). Based on this, it seems that critical weld stresses would occur in, or close to, the elastic range of loading. An elastic stress would then be appropriate, therefore:

$$F_w = \frac{M}{S_x} \quad (1)$$

where:

F_w = reference average weld stress

M= applied moment

S_x = elastic section modulus of the gross section

Sixty-two bare steel specimens from studies on typical pre-Northridge connections were referenced in the literature (Tsai and Popov (1989) specimens 3, 5, 7, 9, 11, and 13-18, Popov and Bertero (1973), Anderson and Linderman (1991) specimens 13, 14 and 20, Tsai et al (1994), Engelhardt and Husain (1993), Popov et al (1986), “Connection” (1997) specimens 1-14). Critical specimens had original E70T-4 (low toughness) electrode welds with backing bars and end tabs left in place. Peak stresses which were maintained were slightly over 30 ksi. Similar estimates were made for other weld improvements and will be included in a later paper.

When new welds are provided utilizing E71T-8 or other moderate toughness electrodes with a full weld inspection and removal of backup bars and end tabs, there is relatively little data on allowable stresses. However, when RBS specimens were tested for new construction, and the RBS “fuse” designed to limit the moment at the face of the column to 0.85 to 0.95 of the plastic moment, the connections have performed very well (Iwankiw and Carter (1996), Engelhardt et al (1996), Popov (1996), Tremblay et al (1997)). These limits were very close to the ratio of yield moment to plastic moment for these specimens. It is therefore assumed that one can safely reach the yield moment and associated stress with these welds, (extreme fiber of weld at yield of base material).

The design procedures were compared to previous haunch and RBS test results. Only specimens which were attached to the strong axis of an I shaped column were included, as allowable weld stresses were calibrated to this type of connection. For the cases of top and bottom flange haunches or RBS flange reductions, the equations were modified accordingly.

All previously tested haunch specimens with weld failures were shown by the method to have overstressed welds at those locations and all specimens which failed to reach 0.020 radian of plastic rotation were shown to be inadequate. Successful specimens were shown to be adequate by the design method when allowable weld stresses were considered, however the column through thickness stress restrictions of Interim Guidelines

Advisory No. 1 (1997) would indicate the designs to be inadequate due to overstressing the column in the through thickness direction. For specimens with beam yield stresses approaching those of the columns the designs can be significantly affected by this criteria which does not appear to correlate with test data.

The design method shows several previously tested RBS specimens which exhibited acceptable behavior to be slightly overstressed (Iwankiw and Carter (1996), Engelhardt et al (1996)). These specimens were tested as new construction details and were therefore welded initially with the moderate toughness electrode. Removal of existing welds prior to re-welding with a moderate toughness electrode, (as would likely be the case in an existing building to provide a comparable section), has the potential of altering the base metal properties. In addition, the new construction designs provided continuity plates and often had fully welded beam web connections which may not be provided in a retrofit. Therefore, the apparent conservatism of the design method would be warranted. All specimens which failed to meet the acceptable total plastic rotation requirement of 0.020 radian are shown to be highly overstressed at the welds.

The column through thickness requirements of the Interim Guidelines Advisory No. 1 (1997) were exceeded in practically all RBS specimens, with only one divot fracture reported. This fracture (Iwankiw and Carter (1997) specimen DBT2B) occurred beyond the acceptance criteria rotation and at a much higher assumed stress than the allowable value. For specimens with beam yield stresses approaching those of the columns, the designs can be significantly affected by this criteria which does not appear to correlate with test data. Once again, the column through thickness guidelines therefore appear overly restrictive and should be modified.

CONCLUSIONS

A large inventory of buildings exist which contain connection detailing similar to those found to be inadequate in the Northridge Earthquake. To provide an adequate level of safety, many of these structures would need to be retrofit to some degree. The two retrofit techniques investigated in this program were chosen because they were believed to represent potentially effective and economical means to increase the plastic rotation capacity of existing pre-Northridge welded flange – bolted web moment connections. Experimental testing to evaluate potential retrofit details and the influence of a composite slab was performed. Typical pre-Northridge connections were retrofit with either a bottom flange RBS or a welded bottom haunch. For each retrofit technique, an effort was made to minimize welding or design modifications at the beam's top flange, in order to minimize the need to remove portions of the floor slab. A method was provided for designing a haunch or RBS retrofit for an existing connection. An acceptable solution can be provided by altering the size of haunch section or RBS flange reduction, or the degree of weld modification.

The use of the bottom flange RBS, by itself, did not provide an improvement in connection performance. In order to achieve an improvement in plastic rotation capacity, it was necessary to remove the existing low toughness E70T-4 weld metal at the top and bottom beam flange groove welds, and re-weld the beam with a higher toughness electrode. When replacement of the weld metal was combined with the bottom flange RBS in a composite specimen, plastic rotations of about 0.020 radian were achieved. These connections, however, still failed by fracture of the beam flange groove welds.

RBS connections for new construction differ significantly from the RBS retrofit details tested here. The tested RBS cutout permitted substantially less moment reduction at the face of the column compared and less extensive detailing compared to typical RBS connections tested for new construction applications. These results do not indicate inadequacy of the details or methods for new construction.

The welded bottom haunch specimens tested generally showed better performance than the RBS specimens. In the welded haunch specimens, no modifications were made to the existing beam flange groove welds. Three of the four bare steel haunch specimens failed by fracture at the existing E70T-4 top flange welds at plastic rotations ranging from 0.012 to 0.023 radian. However, after fracture of an existing beam flange weld, the rate of strength deterioration of the haunch specimens was significantly less than that for the RBS specimens.

With the addition of a composite slab, the haunch specimens showed outstanding performance. Of the four composite haunch connections tested in this program, none experienced a fracture at the existing beam flange groove welds. Rather, the strength of these specimens deteriorated gradually due to local and lateral buckling of the beams without failure of the connection. These connections all developed in excess of 0.030 radian of plastic rotation without connection failure. At total plastic rotations of 0.020 radian, beam moments exceeded 80 percent of the peak values achieved in the tests. The composite specimens retrofitted with a bottom haunch therefore showed performance comparable to new construction standards, even though the existing low toughness E70T-4 beam flange groove welds were left in place. Similar performance was observed in bottom haunch retrofit tests by Uang et al (1998). Consequently, when a composite floor slab is present, the use of a welded bottom haunch appears to provide significantly better structural performance than the RBS retrofits tested in this program.

A design method based on a simplified elastic analysis was developed. From previous test data from 62 specimens of Pre-Northridge connection design, estimates of allowable average stresses on beam flange groove welds were estimated. The design method developed using these allowable weld capacities was compared to existing test data of 19 haunch and 26 RBS specimens. Good agreement was found between acceptable or unacceptable designs and specimen behavior.

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Table 1: Specimen Details

Specimen	Type of Retrofit	Modification to Beam Flange Welds		Composite or Bare Steel
		Top Flange	Bottom Flange	
DB1	Bottom Flange Dogbone	None	Backing Bar and Weld Tabs Removed	Bare Steel
DB2	Bottom Flange Dogbone	E70T-4 Completely Removed Reweld with E71T-8 Weld Tabs Removed, Backing Bar Left in Place with Seal Weld to Column	E70T-4 Completely Removed Reweld with E71T-8 Backing Bar and Weld Tabs Removed	Composite
HCH1	Bottom Haunch	None, Flaws Left in Place in North & South beams	None, Flaws Left in Place in South beam	Bare Steel
HCH2	Bottom Haunch	None, Flaws Left in Place in North & South beams	None, Flaws Left in Place in South beam	Composite
HCH3	Bottom Haunch	None	None, Flaws Left in Place in North and South beams	Bare Steel
HCH4	Bottom Haunch	Weld Tabs Inadvertently Removed	None, Flaws Left in Place in North beam	Composite

Table 2: Test Results

Specimen	Brief Description of Failure		Total Plastic Rotation*	
	North Beam	South Beam	North Beam	South Beam
DB1	Fracture of Bottom Flange Weld	Fracture of Bottom Flange Weld	0.009 radian	0.006 radian
DB2	Fracture of Bottom Flange Weld	Fracture of Bottom Flange Weld	0.020 radian	0.020 radian
HCH1	Fracture of Top Flange Weld	Gradual Deterioration in Strength Due to Local and Lateral Buckling	0.012 radian	0.044 radian
HCH2	Gradual Deterioration in Strength Due to Local and Lateral Buckling	Gradual Deterioration in Strength Due to Local and Lateral Buckling	0.030 radian	0.030 radian
HCH3	Fracture of Top Flange Weld	Fracture of Top Flange Weld	0.023 radian	0.013 radian
HCH4	Gradual Deterioration in Strength Due to Local and Lateral Buckling	Gradual Deterioration in Strength Due to Local and Lateral Buckling	0.050 radian	0.050 radian

*Note: 1) Total plastic rotation is computed with respect to the centerline of the column.

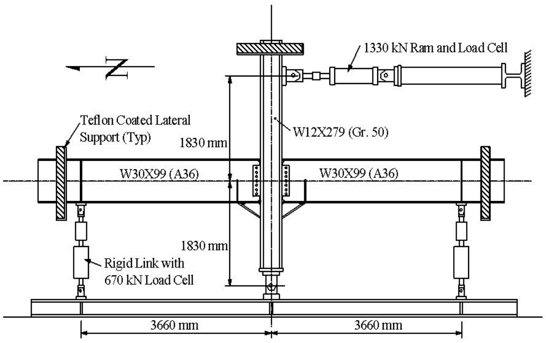


Figure 1: Test Setup

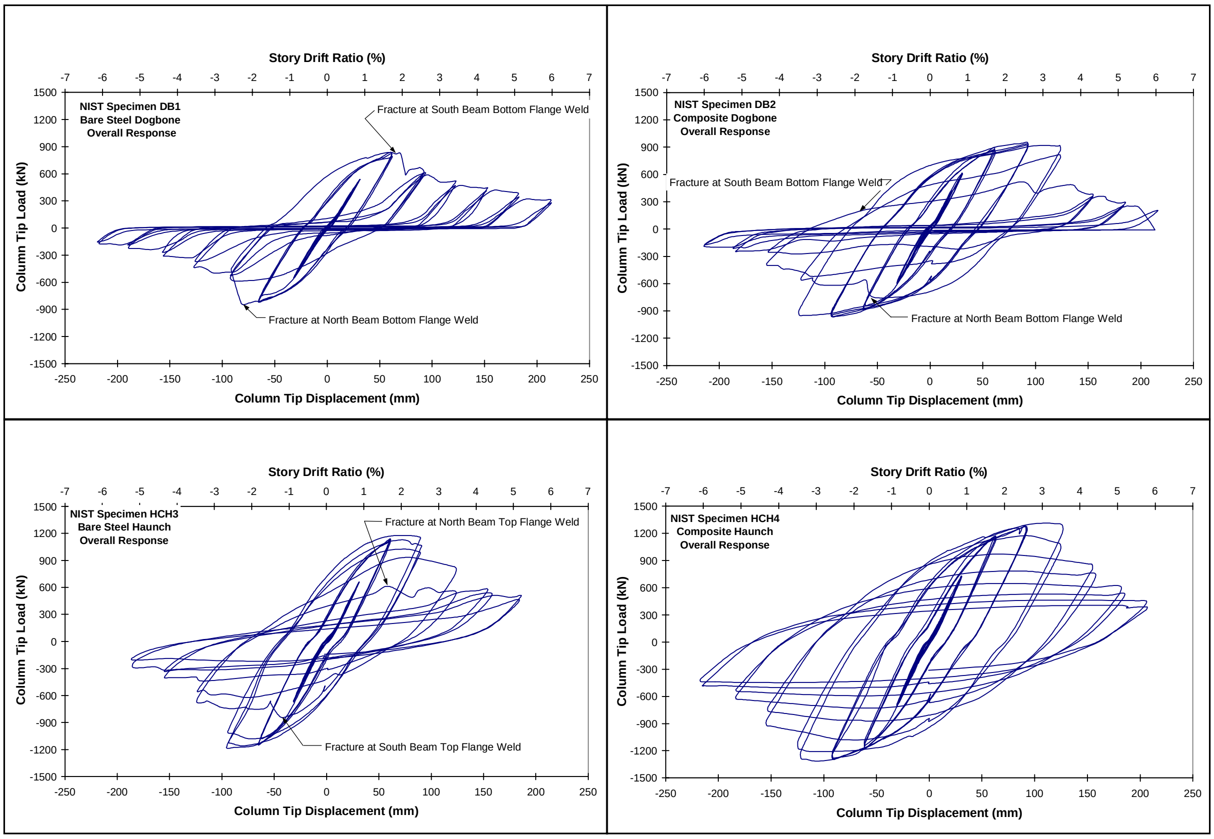
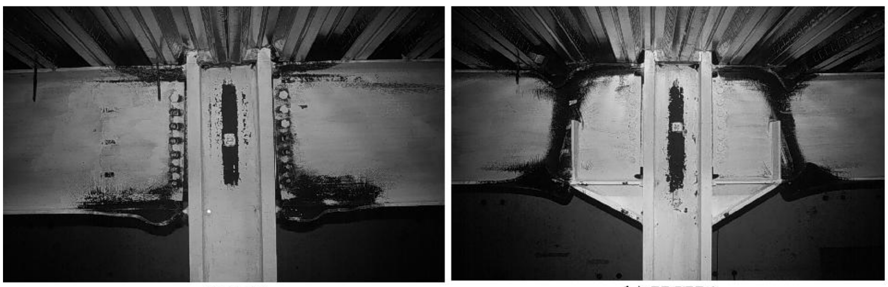


Figure 2: Specimen Load Vs. Story Drift/Tip Deflection



a) DB2 b) HCH4

Figure 3: Specimens After Testing