

WELDMENT COMPONENT TEST FOR STEEL MOMENT CONNECTIONS

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SUMMARY

A common failure mode for welded steel moment resisting frames is fracture of the weldments, as evinced by the performance of these structures in the 1994 Northridge earthquake. Since 1994, seismic design guidelines in the United States have incorporated requirements for cyclic experiments on full-scale beam-column subassemblages to evaluate the capacity of steel moment connections, including the weldments. The cost of these experiments limits designers to a small database of pre-qualified connection details (i.e. combination of beams and columns). This study presents a weldment component test (WCT) on a considerably smaller—but still full-scale—specimen that emulates the stress/strain demands observed in the weldments of a moment connection. The WCT and the full-scale beam-column subassemblage experiment are simulated using the finite element software package ABAQUS. Comparison of stress/strain results from corresponding models indicates that the WCT accurately represents actual response of weldments in moment frames. Finally, a WCT is designed to benchmark against existing data from beam-column subassemblage tests. The costs associated with the new WCT are considerably less than those associated with existing experimental procedures. Overall, the WCT provides a valuable tool for predicting weldment resistance to brittle fracture and final mode of failure.

INTRODUCTION

The welded steel moment resisting frame (MRF) is essentially the cornerstone of seismic load resistance in steel construction. It provides a designer with a minimal skeleton upon which to construct a building, relying on the connections' moment capacity rather than diagonal bracing or shear walls to resist the lateral loads. For economic reasons, structural engineers have gone from designing highly redundant moment frame systems with moderate beams and columns to designing fewer frames with large members to resist seismic loads [1]. Very limited testing was done prior to the 1994 Northridge earthquake to support such a change in design philosophy. As a result, over 100 steel MRF buildings experienced fractures in the beam flange-to-column flange weldments.

The purpose of this study is to attempt to provide some flexibility to the structural engineer who designs steel MRFs by developing a weldment component test (WCT) that is much more affordable and much less

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time consuming than full-scale subassemblage tests. A history of testing on welded steel MRFs is presented and the shortcomings of the existing experimental programs are pointed out. An alternative to the existing testing programs is presented and compared to existing methods from the standpoints of both characterization of behavior and economics.

HISTORY OF WELD EVALUATION IN STEEL MOMENT RESISTING FRAMES

Several experimental studies were performed prior to 1994 to validate the performance of welded steel MRF connection details. Two examples of standard MRF connection specimens used in cyclic static experiments are shown in Figure 1. Popov & Pinkney [2] and Popov & Bertrero [3] performed the first tests of MRF connection details in the United States, using small beams and columns compared to those typically used in buildings. The authors observed satisfactory performance of the connections, disregarding specimens that performed poorly, reportedly because of low quality welds. Engelhardt & Husain [4] contradicted previous studies and predicted the failures in the Northridge earthquake, reporting that experiments on full-scale single-sided welded steel MRFs resulted in brittle fracture of the welds. A number of explanations have been offered for the difference in behavior between the pre-Northridge structural connections and the small-scale specimens tested previously. Among these are that (1) small welds perform differently than larger welds and (2) the yield and ultimate strength of some grades of steel has changed over the past twenty-five years [5].

After the Northridge earthquake, the SAC Joint Venture, funded by FEMA, initiated the first phase of an experimental and analytical study to evaluate the performance of the pre-Northridge connection detail and to develop methods for retrofitting damaged steel MRFs to resist seismic loading. The SAC Joint Venture [6] summarized a series of full-scale experiments on steel MRFs with both undamaged and repaired pre-Northridge connection details. All authors reported brittle fracture of the beam flange-to-column flange weldments in pre-Northridge connections, often in the elastic range of loading. Kaufmann & Fisher [7] performed detailed failure analyses of fractured welds taken from buildings in Southern California and experiments described by the SAC Joint Venture [6]. The authors concluded that fracture typically initiated at the weld root of the beam flange-to-column flange weldment as a result of leaving the backing bar in place. The authors also found the fracture toughness to be inadequate. Kaufmann & Fisher [8] and Xue et al. [9,10] developed a pull-plate test of the beam flange-to-column flange weldment (Figure 2). The experiment involved applying a tensile static or dynamic load to the specimen. The authors tested specimens welded according to pre-Northridge standards and concluded that the weldments performed poorly, often exhibiting brittle fracture in the elastic range of loading. Increasing the load rate further degraded the performance of the pre-Northridge welds. The authors also tested specimens designed to have higher fracture toughness in the welds and suggested several alternatives to the welding process used prior to 1994. As a result of these studies, AISC [11] adopted the requirement of full-scale subassemblage testing (Fig. 1) to "pre-qualify" all connections used in new construction in seismic zones.

The SAC Joint Venture has also supported a second phase of research to develop and evaluate new design methods for steel moment resisting frames. The experiments performed during phase 2 were in-tended to give structural engineers a database of "pre-qualified" connection details for use in seismic design. Stojadinovic et al. [12] and Ricles et al. [13] investigated the performance of welded steel MRFs (unreinforced flange connections) with higher quality weld material as suggested by Kaufmann & Fisher [8] and Kaufmann [14]. The authors reported better overall performance of these specimens, with failures occurring in the weldment after the frame has undergone moderate inelastic deformations. Also among the welded moment frame connection details studies sponsored in Phase II of the SAC Joint



a) Single-sided steel MRF subassemblage



b) Double-sided steel MRF subassemblage

Figure 1. Schematic drawing of typical specimens used for cyclic static experiments on steel MRF connections. The pins, rollers, and load points are designed to coincide with moment inflection points in a MRF subjected to lateral loads. The inset in (a) shows the weldment where failure is expected to occur in pre-Northridge connection details for both types of subassemblage. The stress gradient on section A-A is plotted in Figure 3a.



Figure 2. Schematic drawing of a pull-plate test of the beam flange-to-column flange weldment in a steel MRF (Kaufmann and Fisher, 1995b; Xue et al., 1996a, 1996b; Kaufmann, 1997).



(c) Difference between MRF subassemblage test and pull-plate test

Figure 3. Contour plots of axial normal stresses (in MPa) for section A-A in the inset of Figure 1a for (a) the MRF subassemblage test and (b) the pull-plate test.

Venture are the free flange connection [15,16,17], the welded flange plate connection [18], and the reduced beam section connection [19,20,21].

COMPARISON OF THE PULL-PLATE TEST AND THE SUBASSEMBLAGE TEST

Kaufmann & Fisher [8] and Xue et al. [9,10] developed a pull-plate test that has provided valuable insight into the performance of welded connections during the 1994 Northridge Earthquake. The pull-plate test arrangement shown in Figure 2 provides an inexpensive method to evaluate the general quality of the weld, but the experimental failure loads do not necessarily correlate to loads observed in the real situation. Finite element analyses of full-scale MRF subassemblage tests indicate that the axial stress acting normal to a cross section of the weld varies through the depth (Figure 3a). Similar analyses of the pull-plate test result in a relatively uniform distribution of axial stress through the depth of the weldment (Figure 3b). The difference between the two is shown in figure 3c. Note the substantial difference between the stresses at the bottom center of the beam flange where fracture most frequently initiated in the 1994 Northridge Earthquake. Although there is disagreement between the stresses in the pull-plate test and the MRF subassemblage test, there is generally good agreement between the two-tests when predicting cleavage fracture [22]. Correlating failure loads between the two tests for specimens that yield, however, is not trivial. In the following sections, a WCT that closely matches the weldment stress demand observed in full-scale subassemblage tests is described and evaluated.

WCT SPECIMEN GEOMETRY AND APPLIED LOADING

The WCT specimen geometry was selected to approximate the beam, column and panel zone stresses observed in full-scale subassemblage specimens. Figure 4 shows a preliminary drawing of the specimen along with the corresponding reaction frame. To mitigate end effects and simplify analysis of the panel zone, the specimen comprises two T-sections fabricated by cutting the W-sections used in the actual connection along the neutral axis. The length of the two T-sections must be large enough such that beam theory accurately describes the stress state in the beam and column (i.e. end effects are negligible). The machining of the weld access hole and the welding are to be performed according to standard practice. The remaining geometric considerations (rigid supports, load application) shall be considered on a case-by-case basis and will depend heavily on the assumptions used to resolve equilibrium. The following paragraphs give one example for designing a WCT specimen to represent a double-sided full-scale steel MRF subassemblage specimen (Fig. 1b).

One approach to designing a WCT specimen is to resolve force equilibrium in the beam, column, and panel zone based on simplifying assumptions. Using this method, the panel zone (column web in the connection region) is assumed to act purely in shear and the beam and column are assumed to behave according to Euler-Bernoulli beam theory along the free surfaces (semi-rigid supports in Fig. 4a). The panel zone equivalent reaction force, RPZ, can then be written as:

$$R_{PZ} = \frac{P(Lh - Ld_b - hd_c) \sqrt{d_b^2 + d_c^2}}{2Ld_b d_c}$$
(1)

and the direction of the applied force, θ_{PZ} , is described as:

$$\theta_{PZ} = \tan^{-1} \frac{d_c}{d_h} \tag{2}$$

where d_c is the column depth, d_b is the beam depth, P is the applied load at the column tip, and L and h are the length (roller support-to-roller support) and the height (pin support-to-point of load application)



Figure 4. Preliminary concept of the proposed new WCT: (a) WCT specimen and (b) selfequilibrated reaction frame.

respectively of the double-sided subassemblage. The beam equivalent force, R_b , has the following horizontal and vertical components:

$$R_{b,h} = \frac{Phd_b}{2I_bL} \left(\frac{L}{2} - \frac{d_c}{2} - l_b \right) \left[t_{f,b} b_{f,b} \left(1 + \frac{t_{f,b}}{d_b} \right) + t_{w,b} \left(\frac{d_b}{2} - t_{f,b} \right) \left(\frac{1}{2} - \frac{t_{f,b}}{d_b} \right) \right]$$
(3)
$$R_{b,v} = \frac{Ph}{2L}$$
(4)

where I_b is the moment of inertia of the beam, l_b is the length of the beam in the WCT, $t_{f,b}$ is the beam flange thickness, $b_{f,b}$ is the beam flange width, and $t_{w,b}$ is the beam web thickness. Similar equations for the column follow:

$$R_{c,v} = \frac{Pd_c}{2I_b} \left(\frac{L}{2} - l_c\right) \left[t_{f,c} b_{f,c} \left(1 + \frac{t_{f,c}}{d_c}\right) + t_{w,c} \left(\frac{d_c}{2} - t_{f,c}\right) \left(\frac{1}{2} - \frac{t_{f,c}}{d_c}\right) \right]$$
(5)
$$R_{c,h} = \frac{P}{2}$$
(6)

where I_c is the moment of inertia of the column, l_c is the length of the column in the WCT, $t_{f,c}$ is the column flange thickness, $b_{f,c}$ is the column flange width, and $t_{w,c}$ is the column web thickness. The beam and column equivalent applied loads are applied at the location of the resultant tensile force caused by bending ($R_{b,h}$ and $R_{c,v}$). The force and moment equilibrium equations are then:

$$\sum F_{x} = R_{b,h} - R_{c,h} - R_{PZ} \sin \theta_{PZ} = 0$$
(7)

$$\sum F_{y} = R_{b,v} - R_{c,v} + R_{PZ} \cos \theta_{PZ} = 0$$
(8)

$$\sum M = (R_{PZ} \sin \theta_{PZ}) l_c + R_{b,v} (l_b + d_c) - R_{b,h} (l_c - d_b + y_b) - R_{c,v} (d_c - y_c) = 0$$
⁽⁹⁾

For simplicity, the remainder of this study will focus on the specific case of a double-sided steel MRF subassemblage test similar to those performed at Texas A&M University and the University of Texas at Austin [19]. These specimens consist of two W36x150 beams welded to a W14x398 column (the RBS details are not considered in this study). The subassemblage has a height, h, of 3708 mm (144 inches) and a length, L, of 7620 mm (300 inches). The corresponding WCT specimen has a beam length, l_b , of 457 mm (18 inches) and a column length, l_c , of 711 mm (28 inches). Substituting the above values into equations 1-9 results in satisfaction of equilibrium.

The final design of the WCT test specimen requires consideration of experimental limitations. The above method of resolving force equilibrium based on simplifying assumptions illustrates the potential for developing a WCT specimen based on real loading, but displacement control is much easier to implement than load control. The following section illustrates displacement control in a finite element analysis with the geometry defined according to the force analysis performed in this section.

FINITE ELEMENT ANALYSES

A finite element model of full-scale beam-column welded MRF subassemblages used in experiments was developed to predict measurements of response. The finite element models were generated using MSC/PATRAN [23], solved using ABAQUS [24], and post-processed using ABAQUS CAE [24]. The global models consisted of between 24,334 and 30,158 nodes and between 23,972 and 29,804 S4R four-node reduced-integration shell elements. The node at the center of the column web at the bottom of the

and menes).		
*MA **	ATERIAL,	, NAME=STEEL-PL
*EL 300 **	ASTIC, T 00.,	YPE=ISO 0.3
*PL	ASTIC	
54.5	5, 0).
62.7	7, 0.0	02
69.0), 0.0	04
73.9	9, 0.0	06
77.	5, 0.0	08
80.2	2, 0.1	10
82.1	1, 0.1	12
83.3	3, 0.1	35
84.3	3, 0.1	15

 Table 1. ABAQUS Command Lines Defining Inelastic Stress-Strain Curve for Steel (units of kips and inches).

column was restrained in the x and y directions and nodes at the center of the beam web at the free end of each beam were restrained in the y direction only. The nodes on the right column flange at the top of the column were displaced in the x direction. All elements within five inches of the roller supported ends of the beams and within five inches of the pinned end and the displaced end of the column were treated as approximately infinitely stiff elastic material to prevent inelastic behavior at these locations from affecting the response of the frame. All finite element models contained nonlinear material properties, shown in Table 1 (as ABAQUS input). A local submodel, comprising between 14,577 and 17,372 nodes and between 9,787 and 13,008 C3D8R eight-node reduced-integration three-dimensional brick elements, was created to refine the stress information for the weldment.

A model of the WCT, similar to the local submodel described above, was also created. The semi-rigid endplates were modeled using approximately infinitely stiff elastic material and the beam and column were modeled using the inelastic material described in Table 1. Based on the points of load application defined above, displacements were extracted from the local submodel and applied to the WCT specimen. Weldment stress results from these preliminary analyses are compared in contour plots shown in Figure 5. The stress fields appear to be very similar in the two contour plots indicating that the WCT might successfully emulate weldment failure in full-scale steel MRF subassemblage tests.

COMPARISON OF SPECIMENS

Based on the feasibility of a WCT that closely matches the stress state in a full-scale subassemblage test, some comparisons of the two specimens are made to justify further development of the WCT. First, the costs associated with the two experiments are compared by considering the following factors: capital investment, materials, and specimen fabrication. Each of the factors is discussed below.

Capital investment involves purchasing major equipment necessary to perform the experiments over time. For both experiments, hydraulic actuators are required to impart loads to the specimen. Full scale subassemblage tests use actuator(s) with large load capacity and large stroke (1870 kN and +/- 381 mm for the specimen defined above), whereas the WCT will use two actuators with larger load capacity and



Figure 5. Contour plots of the von Mises equivalent stress (in MPa) in the weldment region from preliminary finite element analyses of (a) a full-scale subassemblage test and (b) a WCT. Figure 5c shows a contour plot of the difference in von Mises equivalent stress (in MPa) between Figures 5a and 5b along plane A-A.

small stroke (~6700 kN and +/- 51 mm). Full-scale subassemblage tests require a laboratory with a reaction wall and/or reaction floor. In the case of a reaction wall and floor, hydraulic actuators can be connected from the specimens directly to the wall and floor. Otherwise, reaction frames are required to impart load from the specimen to the reaction wall and/or floor. The WCT requires only a self-equilibrated reaction frame (Fig. 4b). The costs associated with constructing or renting space in a lab facility with a reaction wall and/or floor more than offset whatever cost differential is associated with the actuators making the WCT more economical in terms of capital investment.

Material and specimen fabrication costs are considerably lower for the WCT because of the smaller size. Full-scale subassemblage specimens are on the order of 6 to 8 times larger and heavier than the WCT specimens (3708 mm x 7620 mm and 37 kN as opposed to 787 mm x 813 mm and 5.5 kN). Since the cost of steel shapes is approximately proportional to the weight (\$.75 to \$.80 per pound depending on the shape in the United States), the material for seven WCT specimens can be purchased for the same price as the material for one subassemblage specimen. The major cost in fabricating the two specimens is in welding. Both specimens require full penetration groove welds at the beam flange-to-column flange connections, but the subassemblage specimen will have two to four welds as opposed to one in the WCT specimen. Additional tack welds are required for stiffeners in the beams and column at the points of load application to avoid web crippling in the subassemblage specimens. In the WCT specimens, groove welds will connect the end plates to the beam and column, but these welds will be installed under optimal

conditions due to the size of the specimens. The welding costs are expected to be similar for the two specimens. Overall, the materials and fabrication costs for the WCT will be considerably less than those for the subassemblage test.

In addition to the above economic considerations, several other factors motivate the development of the WCT. First, the WCT assures failure in the weldment. In reality, failure may occur due to in-stability and fracture away from the weldment, especially in the case of connections with reinforced welds or RBS details. Isolating the weld allows the designer to optimize the weld strength relative to the rest of the frame. Second, the WCT allows researchers to use smaller scale specimens to evaluate other failure modes. Fracture does not scale because the tiny defects that serve as points of crack initiation are not controlled during manufacture of steel shapes and welding. Local and global buckling, on the other hand, are scalable. By testing small-scale models with over strength welds, researchers could potentially impose realistic earthquake loads on small-scale structures using shake tables and evaluate the strength of the welds separately. This type of comprehensive analysis would be a valuable tool for evaluating global stability of steel structures.

CONCLUSIONS

The weldment component test (WCT) can be used to evaluate the weldment capacity in a welded steel moment resisting frame. Based on a finite element analysis, the weldment stresses predicted in the WCT closely match the weldment stresses predicted in a full-scale subassemblage test.

Among the benefits associated with the WCT are cost and the scalability of corresponding tests on structures and their components. One WCT will cost considerably less and require less time than a similar full-scale subassemblage test. Furthermore, by using the WCT to evaluate weldment strength, researchers can potentially use small-scale models with overstrength welds to evaluate other failure modes.

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