Moment Connections

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Summary

This session will address the following topics: 1. Current alternative details for steel moment-resisting frame (SMRF) connections in regions of high seismic activity. 2. Implications of Northridge SMRF connection failures for wind-controlled moment connection design. 3. New developments in extended end-plate moment connection design-use in seismic applications and use of snug-tight bolts.

SEISMIC PERFORMANCE OF BOLTED END-PLATE MOMENT CONNECTIONS

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ABSTRACT

Interest in bolted alternatives for moment connections in seismic areas has been rejuvenated since the Northridge earthquake. Seismic testing of bolted end-plate moment connections has been performed with promising results. This paper discusses results and associated developments that indicate bolted end-plate moment connections are a viable alternative in seismic moment frame construction.

INTRODUCTION

Moment end-plate connections are commonly used in metal building construction, but are not common in conventional construction. The primary reason seems to be long standing fabricator resistance to the connection because (1) fabrication techniques are somewhat more stringent due to the need for accurate beam length and "squareness" of the beam end, (2) fit-up problems caused by column out-of-squareness, and (3) end plates often warp due to the heat of welding. Fabricators who have experience with moment end-plate connections report that these difficulties can be overcome without much additional expense. In addition, end-plates are subject to lamellar tearing in the region of the top flange tension weld and the tension bolts are subject to prying forces, both of which are designer concerns. However, there are a number of significant advantages for end-plate connections, especially for seismic design: (1) All welding is done in the shop where back-up bars may not be required, welding process is more easily controlled, and inspection is easier. (2) The connection is relatively "soft" at the web-flange-end plate juncture which decreases the chances of beam flange-to-end plate weld fracture. (3) The load path at the connection includes more of the beam web than direct flange connections. And, (4) the beam flange force is distributed to a significant longer length of the column flange than for direct beam flange welded connections. Because of (2), (3), and (4), the load path from the beam tension flange to the column web is significantly less rigid than for direct beam flange-to-column flange welded connections. Thus, demands on the end-plate, weld material, and column flange are much less.

To demonstrate that moment end-plate connections are a viable connection for seismic moment resisting frames, a number of end-plate configurations were tested using the ATC-24 loading protocol (1). The original intent of the testing was to verify that design procedures developed for static loading, particularity the prediction of bolt prying forces, are satisfactory for seismic loading. Because of unexpected failures of beam flanges at weld access holes (rat holes), the investigation was expanded to study this phenomenon.

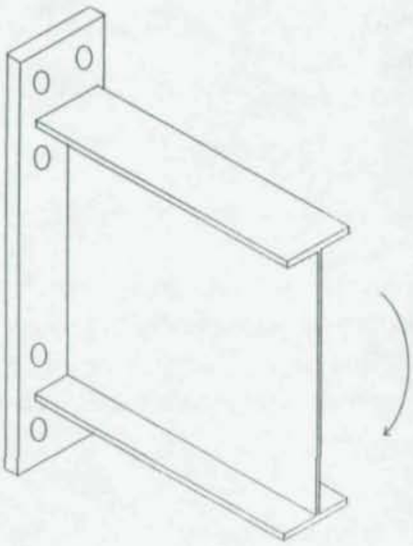


Figure 1. Four Bolt End-Plate

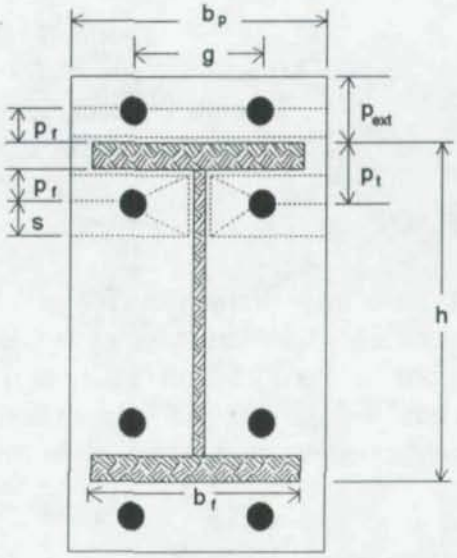


Figure 2. End-Plate Yield Line Pattern

This paper summarizes the study and results for the four tension bolt, unstiffened, extended moment end-plate connection shown in Figure 1.

END-PLATE CONNECTION DESIGN

The design procedures used to determine bolt size and end-plate thickness for the connections tested are based on methods found in the literature, but not those found in the AISC manuals (2, 3). End-plate thickness is determined from yield line analysis and the required bolt force resistance includes prying force effects. The required moment strength of each connection was taken as 1.1 times the plastic moment capacity of the beam, M_p, to account for possible over-strength of the beam material. Preliminary bolt size was then determined from

$$F_f = 1.10\, M_p / (d - t_f) \tag{1}$$

and

$$T_b = F_f / 4 \tag{2}$$

where F_f is the beam flange force, d is the beam depth, t_f is the beam flange thickness, and T_b is the required bolt resisting force. Using T_b, a bolt type (A325 or A490) and diameter is selected. The connection moment strength, M_{con}, is then calculated from

$$M_{con} = 4\,\phi T_n\ (d - t_f) \tag{3}$$

where T_n is the nominal tensile resistance of the bolt.

The required end-plate thickness is determined from the yield-line pattern shown in Figure 2 and

$$t_p = \left[\frac{M_u / F_{py}}{\left(\frac{b_p}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+s\right)\left(\frac{2}{g}\right)\right)\left(h-p_t\right)+\frac{b_p}{2}\left(\frac{h}{p_f}+\frac{1}{2}\right)}\right]^{1/2} \tag{4}$$

where the geometric terms are defined in Figure 2, F_{py} is the yield stress of the plate material and

$$s = \sqrt{b_p g}\Big/2 \tag{5}$$

After selecting a standard plate thickness the nominal strength of the connection due to the end-plate limit state, M_{plate}, is calculated from

$$M_{plate} = F_{py} t_p^2 \left[\left(\frac{b_p}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+s\right)\left(\frac{2}{g}\right)\right)\left(h-p_t\right)+\frac{b_p}{2}\left(\frac{1}{2}+\frac{h}{p_f}\right)\right] \tag{6}$$

Bolt prying forces are then estimated from a method suggested by Kennedy *et al* (4) and modified for end-plate connections (5,6). The magnitude of the bolt prying forces depends on the thickness of the plate relative to the thickness required for the given loading. If the actual end-plate is relatively "thick", the prying force, Q, is zero. If the relative thickness is "intermediate", the prying force is determined from

$$Q = \frac{0.5 F_f p_f}{2a} - \frac{\pi d_b^3 F_{yb}}{32a} - \frac{b_f t_p^2}{8a}\sqrt{F_{py}^2 - 3\left(\gamma F_f / b_f t_p\right)^2} \tag{7}$$

where

$$a = 3.682\,(t_p / d_b)^3 - 0.085 \tag{8}$$

except that (a) must be less than (p_{ext} - p_f) for the external bolts, and where d_b is the bolt diameter, and F_{yb} is the bolt yield stress. If the relative thickness is "thin", the prying force is maximum as determined from

$$Q_{max} = \frac{w' t_p^2}{4a} \sqrt{F_{py}^2 - 3\left(F'/w' t_p\right)^2} \tag{9}$$

where

$$F' = F_{11}/2 = \frac{t_p^2 F_{py}\left[0.85\left(b_f/2\right) + 0.80w'\right] + \left[\left(\pi d_b^3 F_{yb}\right)/8\right]}{4p_f} \tag{10}$$

The end plate category is determined as follows

if $t_p > t_1$, the end-plate exhibits thick behavior,
if $t_1 < t_p < t_{11}$, the end-plate exhibits intermediate behavior, and
if $t_{11} < t_p$, the end-plate exhibits thin behavior

where

$$t_1 = \sqrt{2.11 p_f t_f \frac{b_f}{b_p} \frac{F_{fy}}{F_{py}}} \tag{11}$$

$$t_{11} = \sqrt{\frac{\left(b_f t_f F_{fy} p_f - \frac{\pi}{8} d_b^3 F_{yb}\right)}{F_{yp}\left(0.425 b_p + 0.80w'\right)}} \tag{12}$$

and w' is the net width of the end-plate at a bolt line.

The required bolt resistance including prying force effects is then

$$B = F_f / 4 + Q$$

where Q is determined from the relative thickness of the end-plate. The strength of the preliminary bolt size is then checked using

$$B < T_n \tag{13}$$

and increased if Inequality 13 is not satisfied. Note that the strength reduction factor, ϕ, is not used in Inequality 13.

TESTING PROGRAM

Full scale testing was conducted using several different size A36 steel wide flange section and one built-up beam section of Grade 50 steel. All testing was conducted to failure, with failure determined by either local buckling of the beam flange after at least 18 ATC-24 loading cycles or fracture of the beam flange at the weld access hole or fracture of the beam web-to-flange weld for the built-up sections. The physical test setup used was a cantilevered beam connected to a column section as shown in Figure 3. Instrumentation for recording of test data included a load cell, displacement transducers to measure beam deflections, instrumented calipers to measure end-plate and column flange separation, a displacement transducer to measure end-plate slip parallel to loading, strain gauges to measure beam flange strain and instrumented bolts to measure bolt strain. Quasi-static or "slow cycle" loading was applied to the end of the cantilever beam using a hydraulic actuator. The loading history prescribed by ATC-24 (Guidelines 1992), as shown in Figure 4, was attempted for all tests. The ATC-24 loading procedures are specifically written for testing with slow cyclic load application which is less than the real-time loading of an earthquake. A major limitation of this testing methodology is that it severely distorts time and its material property effects. For the tests reported here, each load step consisted of three complete cycles. The first two load steps were with a peak deformation less than the first yield deformation, δy. The third load step was at the estimated yield deformation, δy. The fourth and subsequent sets were at multiples of δy, that is $2\delta y$, $3\delta y$, etc for the 4th, 5th and succeeding load steps. A completely successful test is when failure does not occur until after six load steps have been completed.

Table 1 summarizes the parameters for the four bolt extended end-plates tests. All of the hot-rolled beam sections and their end-plates were A36 steel; the built-up sections and end-plates were fabricated from A572 Gr 50 steel. The end-plate connections were designed assuming the actual yield stress of the A36 beams was 42 ksi and that of the A572 Gr50 plates was 55 ksi. The bolts were grade A325 or A490 and were fully tightened to either the prescribed pretension as listed in the AISC Specification (7) as determined from bolt strain measurements or by the turn-of-nut method. Weld access holes were used to fabricate eight of the fourteen specimens. Because, in early tests, beam flanges fractured prematurely at the weld access hole locations, a sub-study was conducted to experimentally and analytically determine the effect of weld access hole size on the ability of a beam section to sustain ATC-24 loading.

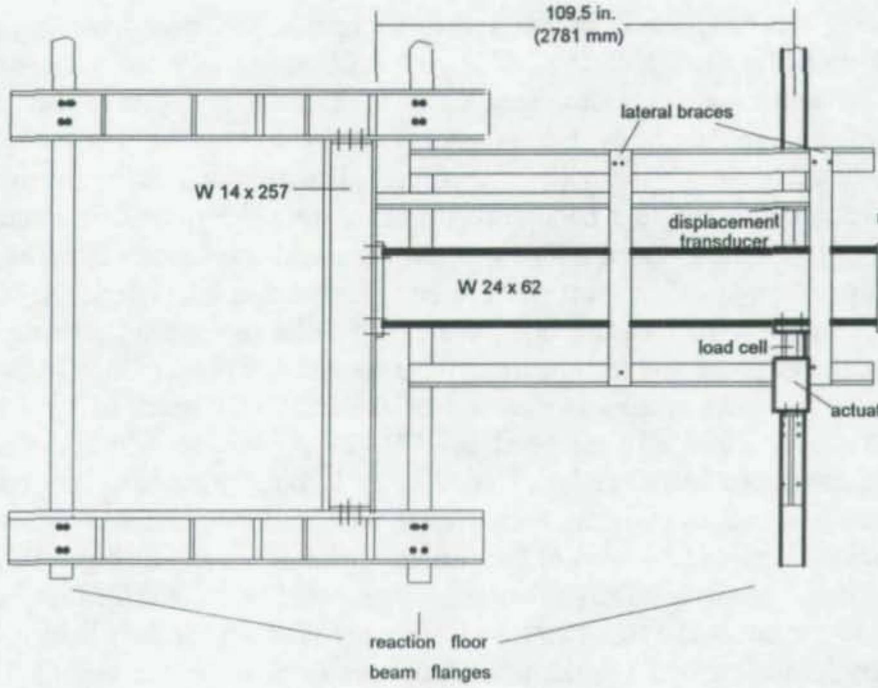


Figure 3. Test Setup

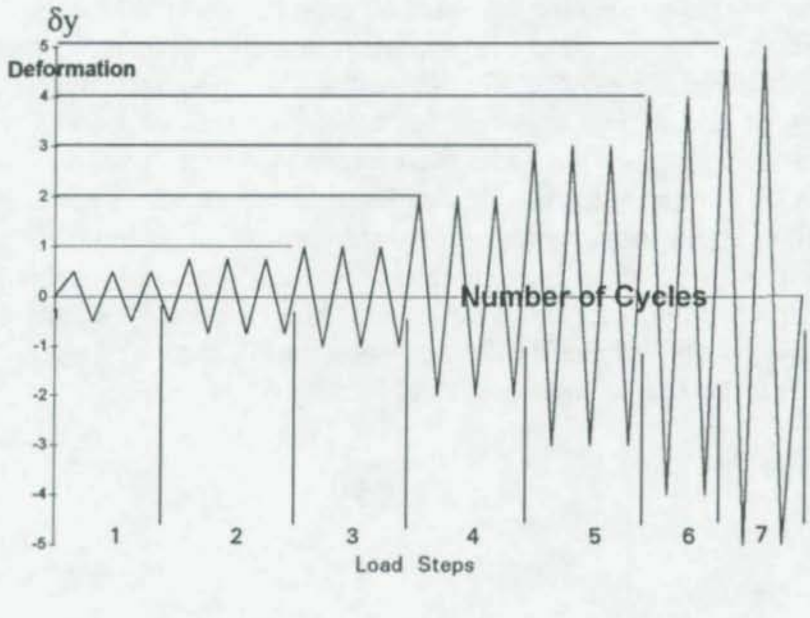


Figure 4. ATC-24 Loading History

TABLE 1
Four-Bolt Extended End-Plate Test Parameters

Test No.	Beam Section	Measured F_y, ksi	End-Plate b_p, in	End-Plate t_p, in	Bolt Diameter, in.	Bolt Type	Weld Access Holes
1/95	W 18x35	55.0	7.0	1.0	1.0	A325	No
4/95	W 18x35	55.0	7.0	1.0	1.0	A325	No
5/95	W 24x62	53.0	8.0	1.0	1.25	A325	Yes
8/95	W 24x76	58.0	9.0	1.25	1.25	A490	Yes
9/95	W 24x76	58.0	9.0	1.25	1.25	A490	Yes
RH-1B	W 24x76	58.0	9.0	1.25	1.25	A490	Yes
RH-2	W 24x62	47.0	8.0	1.0	1.25	A325	No
RH-2A	W 24x62	47.0	8.0	1.0	1.25	A325	Yes
RH-3	W 24x62	47.0	8.0	1.0	1.25	A325	Yes
BuS 1	Built-up	65.0	8.0	1.0	1.25	A490	Yes
BuS 2	Built-up	65.0	8.0	1.0	1.25	A490	No
BuS 1A	Built-up	63.0	8.0	1.0	1.25	A490	Yes
BuS 2A	Built-up	63.0	8.0	1.0	1.25	A490	No
1/96	W 24x62	47.0	8.0	1.0	1.25	A325	No

RESULTS OF TESTING AND ANALYTICAL STUDY

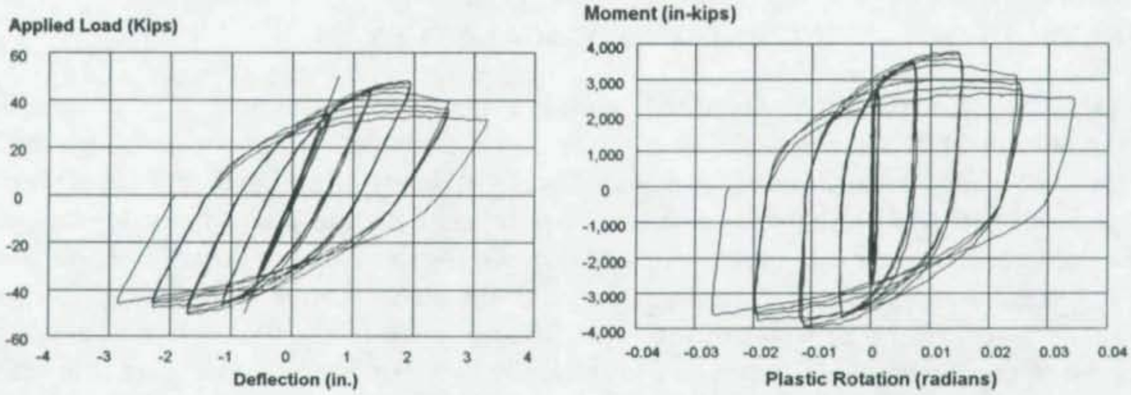
Table 2 summarizes the results of the four bolt extended end-plate tests. The table shows the number of cycles conducted in each test and the condition of the beam at the end of the test: flange local buckled, flange fractured, or beam web-to-flange weld fractured (built-up sections). Neither excessive end-plate deformation, beam flange-to-end plate weld, or bolt fracture was observed in any test. Of the fourteen tests conducted, six failed by beam flange fracture prior to the 20th cycle. Weld access holes were used to fabricate all six specimens. The web-to-flange web failed in three of the four built-up beam tests prior to completing the 21st cycle. The remaining five tests were stopped after completing at least the 19th cycle with the beam flanges exhibiting large local buckles near the end-plate. The latter tests are considered to be successful tests.

Figure 5 shows loading versus beam deflection, connection rotation, and inner and outer bolt force histories for Test 1/95, a typical successful test. The test was terminated after 19 cycles due to noticeable deterioration of the beam strength. The hysteresis loops shown are robust and the connection rotation exceeded 0.02 radians. As seen in the figure, there was a complete loss of bolt tension in the inner bolt and a substantial loss of bolt force in the outer bolt. However, these losses did not affect the strength of the connection.

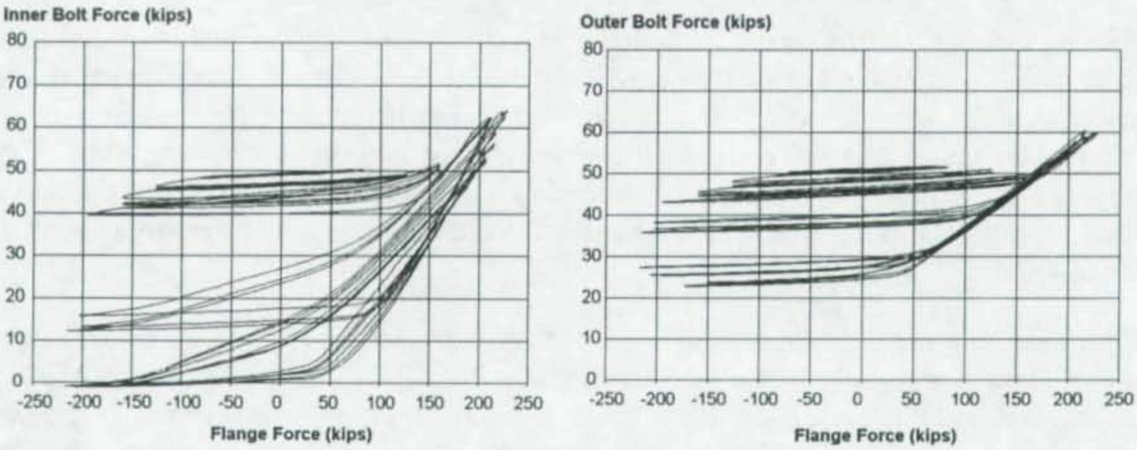
The four built-up beam specimens were obtained from a metal building manufacturer. Fabrication was typical for that industry in that web-to-flange weld was only on one side of the web except for approximately the first 12 in. from the end-plate were weld was placed on both sides. Standard practice in the industry is not to use weld access holes, however, two of the specimens were fabricated with them so that their effect could be studied. Premature web-to-flange weld fracture occurred in the first two tests conducted,

TABLE 2
Four-Bolt Extended End-Plate Test Results

Test No.	Beam Section	Weld Access Holes	Cycles	Remarks
1/95	W 18x35	No	19	Local Flange Buckling
4/95	W 18x35	No	20	Local Flange Buckling
5/95	W 24x62	Yes	15	Flange Fracture
8/95	W 24x76	Yes	14	Flange Fracture
9/95	W 24x76	Yes	21	Local Flange Buckling
RH-1B	W 24x76	Yes	16	Flange Fracture
RH-2	W 24x62	No	20	Local Flange Buckling
RH-2A	W 24x62	Yes	19	Flange Fracture
RH-3	W 24x62	Yes	15	Flange Fracture
BuS 1	Built-up	Yes	18	Web-to-Flange Weld
BuS 2	Built-up	No	16	Web-to-Flange Weld
BuS 1A	Built-up	Yes	19	Flange Fracture
BuS 2A	Built-up	No	20	Web-to-Flange Weld
1/96	W 24x62	No	21	Local Flange Buckling



a) Loading and Connection Rotation Histories, Test 1/95



B) Inner and Outer Bolt Forces, Test 1/95

Figure 5. Results from a Successful Test

Tests BuS 1 and 2. Test BuS 1A was conducted after web-to-flange weld was added on the unwelded side of the web outward from the end-plate to approximately the depth of the beam. During the 19th cycle of the test, a web-to-flange welded failed 4-6 in. from the end-plate. Test BuS 2A was also conducted after a web-to-flange was added to the unwelded side of the web. The specimen was fabricated without the use of weld access holes. Severe local flange buckling developed during the 15th cycle and a web-to-flange weld failed during the 20th cycle.

Because of the number of beam flange fractures at weld access hole locations, a series of tests were conducted to investigate the phenomenon. Test RH-1B was the last of three tests designed to investigate the effects of different length weld access holes on beam flange strain. The same specimen was used for all three tests. The beam was prepared with additional strain gauges on the beam flanges. The initial size of the weld access holes was 1 1/8 in. from end-plate to edge by 1 1/8 in. from beam flange to edge. The beam was subjected to 10 cycles of loading with the original size access holes. During the 10th cycle, beam flange yielding, but not strain hardening occurred. Testing was stopped and the weld access holes elongated along the flange, doubling the length to 2 1/4 in. The width from the flange to edge was not changed. At the end of five additional cycles, there appeared to be a slight reduction in beam flange strain. The weld access hole was then increased to three times its original length (3 3/8 in.) and two additional cycles were applied. The beam flange strains showed a slight increase from all gauges, ranging from 100 to 1,000 $\mu\varepsilon$. Because of the higher than expected yield strength of the beam material (58 ksi measured versus 36 ksi nominal), the strength of the bolts was not sufficient to complete the test. Instead, the beam flanges were "dog-boned" to reduce the strength and the test restarted. After 14 cycles, a small crack was noted in the beam flange near the end of the weld access hole. During the 16th cycle, the crack propagated through the entire beam flange and the testing was terminated.

To further investigate the length of weld access holes on beam flange strains, a limited finite element model study was conducted. The commercial software ABAQUS was used in a two step process. First, a geometric, three dimensional, non-contact model of the entire end-plate and beam was constructed. Then the beam loading versus deflection results of the model and actual data from W18x35 and W24x62 tests were compared to determine model validity. Once satisfactory results were obtained, the model was modified to study the effect of weld access hole length. The AISC Specification (7) requires that weld access holes be a minimum of 1 1/2 times the thickness of the beam web. The required minimum hole depth for each beam is less than approximately 5/8 in. To accommodate the AISC requirement, yet be realistic in size, 1 in. weld access holes were initially modeled in each beam web. For the W24x62 beam, two holes, 1 in. by 1 in. and 1 in. by 2in., were modeled. The results shown in Figure 6 indicate that flange strain increases with increasing weld access hole length. This result, plus unavoidable roughness caused by cutting of the hole, indicates the weld access holes are to be avoided.

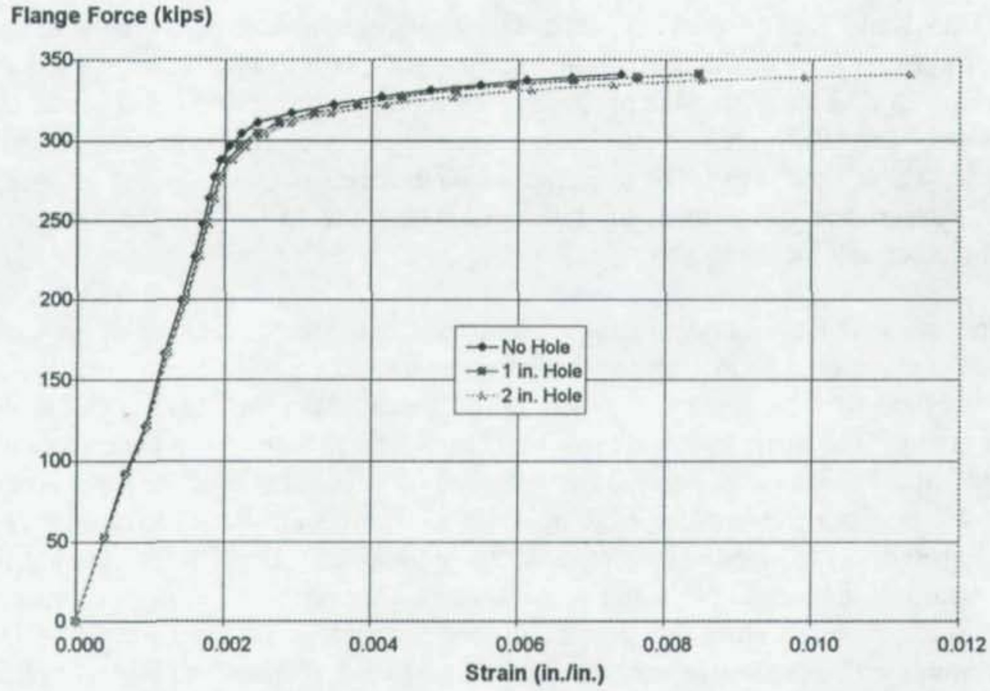


Figure 6. Model Flange Strain with Weld Access Holes (W24x62)

If a weld access hole is not used, radiographic inspection of the beam flange-to-end-plate weld directly over the beam web is not possible. However, the beam to end-plate connection is relatively "soft" at this location, and, therefore, small weld imperfections may not be detrimental to the connection strength. To test this hypothesis, specimen 1/96 (W24x62 beam) was fabricated with an inclusion in the form of a carbon block (1/8 in. by 1/8 in. by 1/4 in.) placed at the flange-web juncture before welding. The subsequent test was terminated after 21 cycles without weld failure.

SUMMATION AND RECOMMENDATION

As part of a broader research project, 14 four bolt, extended, unstiffened end-plate moment connections were tested. Both A325 and A490 bolts, and A36 and A572 Gr50 steel beams, with and without weld access holes, were used in the project. The procedure used to design the connections requires yield line analysis to determine the end-plate thickness and accounts for prying force effects for determination of the bolt size. The test specimens were subjected to the ATC-24 quasi-static loading protocol. The major findings from the effort are:

1. Properly designed moment end-plate connections are a viable connection for seismic moment frame construction.

2. Either A325 or A490 bolts may be used if the effects of prying forces are considered in the design.

3. Weld access holes are not recommended for moment end-plate connections. The presence of weld access holes greatly increases the chance of premature beam flange fracture.

4. When weld access holes are not used, radiographic inspection of the beam flange-to-end plate weld is not possible. It was shown with one test that this limitation is not a problem.

5. For built-up members, web-to-flange welds are required on both sides of the web for at least a distance equal to the depth of the beam from the face of the end-plate.

The true measure of a connection design is its performance during cyclic loading. All connections tested displayed excellent strength and ductility when flange fracture failures were eliminated. The connections were slightly stronger than predicted and their ductility was well above the 2% rotation desired.

ACKNOWLEDGMENTS

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