DEVELOPMENT OF POST-NORTHRIDGE STEEL MOMENT CONNECTIONS

Bozidar STOJADINOVIC[1], Subhash C GOEL[2] And Kyoung-Hyeog LEE[3]

SUMMARY

Fractures of steel moment connections in recent earthquakes sparked an unprecedented research effort to find out why these connections failed and how to design better, earthquake-resistant connections. A summary of the work on fully-restrained steel moment connections done at the University of Michigan is presented in this paper.

A comprehensive test series on pre-Northridge steel moment connections, organized by the SAC Joint Venture, demonstrated that these connections are vulnerable. Even in laboratory conditions, they suffered brittle failures, usually stemming from the beam flange complete joint penetration welds.

Development of a new connection design in the SAC Steel Program focused on deploying weld fracture mitigation measures, which comprise the use of notch-tough weld metal and improvements in weld detailing practice. The original connection configuration was not changed. A series of parametric tests of SAC post-Northridge connections was done at the University of Michigan. On average, the specimens developed a total plastic rotation of 0.015 radians. All specimens fractured in the same way: beam flanges cracked in the zone between the weld and the end of the beam access hole. The response of these connections did not satisfy the AISC pre-qualification criteria for special or intermediate moment frames.

A comprehensive suite of finite element analyses of steel moment connection models, conducted at the University of Michigan, showed that boundary restraints significantly affect the stress flow in the connection. The restraints produce a self-equilibrated stress field that overloads the flanges and causes excessive local flange bending. This overload condition is a potential source of connection failures.

Thus, fracture mitigation measures alone may not be sufficient for agood connection design. Connection configuration must also be changedto mitigate flange overload. Two new connection configurations weredeveloped and tested at the University of Michigan. One is anindirect beam connection which utilizes cover plates and verticalribs. The other is the Free Flange connection. Tests of these connections show that they should perform very well in real earthquakes.

INTRODUCTION

It has been almost five years since we learned that traditional fully restrained steel beam-to-column moment

[1] Asst Professor, Dept of Civil and Env Eng, The University of Michigan, Ann Arbor, MI, USA, e-mail: boza@umich.edu

[2] Professor, Dept of Civil and Environmental Eng, The University of Michigan, e-mail: subhash@umich.edu

[3] Transportation Engineer, California Dept of Transportation, Oakland, CA 94623, e-mail: kyoung-hyeoglee@dot.ca.gov

resisting frames. Much work has been done since the 1994 Northridge and 1995 Hyogo-Ken Nanbu earthquakes to understand the behavior of steel moment connections and to devise new, earthquake-resistant designs. A summary of the work on steel moment connections done at the University of Michigan is presented in this paper.

PRE-NORTHRIDGE DESIGN

Before the 1994 Northridge earthquake fully-restrained steel moment connections (Figure 1) were designed following the AISC Manual of Steel Construction. The flanges of the beam were welded to the column using complete joint penetration welds. The weld metal strength was chosen to over-match the strength of the connected base metals. There were no weld metal toughness requirements. The beam web was bolted to the shear tab with the intent to transfer the shear generated at the nominal moment capacity of the beam. This prescriptive design was developed in late 1960's and early 1970's and was widely used in steel moment-resisting frame structures.

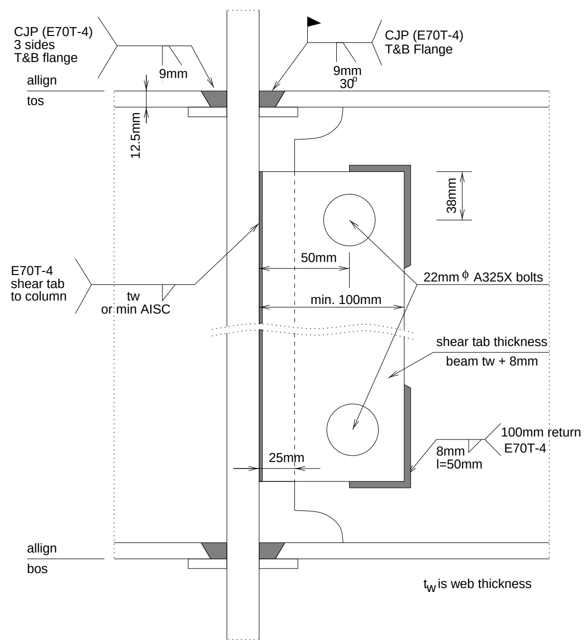

Figure 1: Pre-Northridge connection detail.

US-designed pre-Northridge fully-restrained steel moment connections suffered serious damage during the 1994 Northridge earthquake [FEMA-267 95]. They exhibited a multitude of failure modes. Almost all of these modes have been reproduced in laboratory conditions during the first phase of the SAC Joint Venture Steel Project [SAC 96]. Most of the failures were very brittle as they occurred before significant yielding in any of the connection elements. In a great majority of cases, connection failure initiated at the complete joint penetration weld between the beam and column flanges. The crack at the root of this weld propagated either through the weld, the beam heat affected zone, or through the column in a very quick fracture event.

Even though the engineering community was shocked by this outcome, the small plastic rotation capacity of the pre-Northridge connection design is not surprising. A statistical analysis of the 1970–1993 cyclic tests data shows that these connections have a mean plastic rotation capacity of only 0.009 radian [Fry 98].

POST-NORTHRIDGE DESIGN

The search for a new, earthquake-resistant steel moment connection proceeded in two ways: through independent proof-tests and through the SAC Joint Venture. The SAC Joint Venture effort focused on the beam flange complete joint penetration welds and the mitigation of their brittle fracture. Different measures, such as the use of notch-tough electrodes, weld root reinforcement, improved welding practices, more rigorous weld quality control, and the use of steel strength data consistent with the properties of today's steel [FEMA-267A 97], were implemented in a SAC post-Northridge connection design (Figure 2).

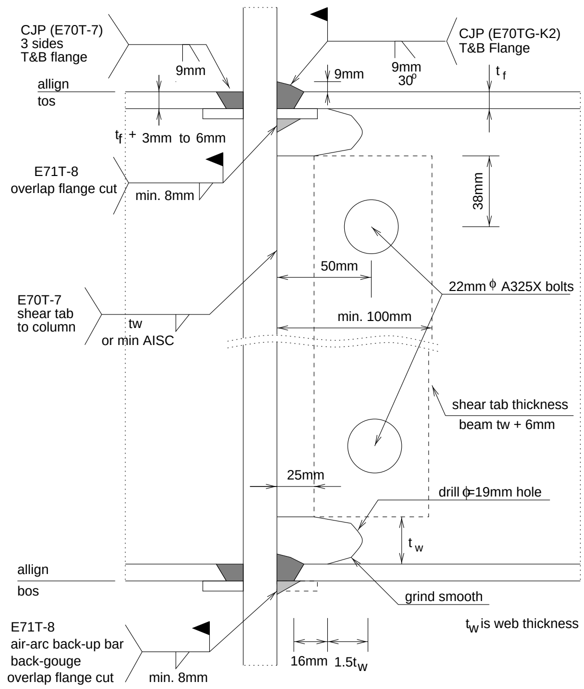

Figure 2: Post-Northridge connection detail.

This connection design was tested in a comprehensive parametric test program conducted at the University of Michigan [Stojadinović 00]. The parameters under investigation were: 1) connection size; 2) beam yield strength; and 3) panel zone strength. A pair of nominally identical specimens was tested for each parameter combination. The list of specimens is shown in Table 1. The specimens were divided into three groups by connection size: the small specimens featured a W24x68 (610 mm deep) beam, the medium ones had a W30x99 (770 mm deep) beam, while the large ones had a W36x150 (920 mm deep) beam. Specimen pairs 3, 5 and 7 had a panel zone whose strength conformed with the AISC guidelines. The panel zone of Specimen pair 4 was under-designed, while the panel zone of Specimen pair 6 was over-designed. Beam yield strength was varied between the specimens in pairs 4, 5 and 6.

The tests were prepared and conducted following the SAC Joint Venture Test Protocol [SAC-BD 97]. Each T-shaped specimen was a model of an exterior moment connection without a floor slab (Figure 3). The inflection points were assumed at beam midspan and column mid-heights, where pin supports were placed. A gradually increasing drift-based cyclic displacement history was applied at beam midspan in a quasi-static manner until specimen resistance in both directions diminished below 60% of its peak resistance.

The behavior and the failure of all tested specimens was very similar, despite the differences among them. Invariably, the panel zones of the specimens (except for Specimens pair 6) yielded first, followed by yielding of the beam flanges. Yielding propagated between one quarter and one half of beam depth along beam flanges before specimen failure. Failure of the tested connections occurred when one of the beam flanges fractured. In this test series, there was no bias towards either the top or the bottom flange fracturing first. All flange fracture occurred in the base metal. The crack was located in the area between the weld fusion zone on the outside flange surface and the end of the access hole cut on the inside flange surface (Figure 4). This crack propagated in two phases, by ductile tearing in the middle of the flange first, followed by brittle fracturing towards the edges.

The rotation capacities of the tested specimens are shown in Figure 5. The mean total connection rotation was 0.026 radian, while the mean total plastic rotation was 0.015 radian. The mean beam contribution to the total plastic rotation was 0.009 radian, regardless of the beam size or strength. The balance of the connection plastic rotation was provided by the plastic deformation of the panel zone, which depends on the panel zone strength. Thus, the rotation capacity of the SAC post-Northridge connection does not

Table 1: **Post-Northridge specimens (coupon yield/ultimate stress, MPa, for flanges and webs of the sections).**

specimen	beam			column			
$\sigma_y(static)/\sigma_u$	W36x150	W30x99	W24x68	W14x257	W14x176	W14x145	W14x120
3.1 (fl)			313/465				317/466
3.1 (web)			339/477				343/472
3.2 (fl)			313/465				317/466
3.2 (web)			339/477				343/472
4.1 (fl)		353/481				329/476	
4.1 (web)		380/494				333/467	
4.2 (fl)		293/430				329/476	
4.2 (web)		323/447				333/467	
5.1 (fl)		353/481			358/502		
5.1 (web)		380/494			354/505		
5.2 (fl)		293/430			358/502		
5.2 (web)		323/447			354/505		
6.1 (fl)		343/475		336/503			
6.1 (web)		389/503		412/504			
6.2 (fl)		281/417		336/503			
6.2 (web)		296/425		412/504			
7.1 (fl)	288/439			333/487			
7.1 (web)	325/444			305/472			
7.2 (fl)	288/439			333/487			
7.2 (web)	325/444			305/472			

meet the plastic rotation requirements for either special or intermediate steel moment frames [AISC 97].

STRESS FLOW ANALYSIS

The performance of the tested SAC post-Northridge connection shows that the use of weld fracture mitigation measures alone may not be enough to improve th connection performance to the level prescribed by current seismic design codes in the US. Severe local flange deformation in the span between the weld and the end of the access hole, observed in all tested specimens, may be another reason for the reduction in connection rotation capacity.

The source of flange local deformation was investigated in a study of the stress flow in fully-restrained steel moment connections, recently completed at the University of Michigan [Lee 97]. This study was done using a shell-element finite element model of a moment connection with a standard pre-Northridge configuration. The main finding stemming from a suite of parametric analyses was that the principal stress flows are redirected in the vicinity of the connection. The stresses concentrated towards the flanges, leaving the web of the beam practically stress-free. A truss analogy model of the connection stress flow, similar to the strut-and-tie model used for reinforced concrete structural elements, is shown in Figure 6. It simply shows that the flanges of the beam carry the moment couple forces and the connection shear force. This shear force produces local flange bending.

The underlying causes of the observed stress redirection are the deformation restraints imposed on the beam and the column by the configuration of the connection [Lee 00]. Following St. Venant's principle, these restraints produce a self-equilibrated stress field in the connection. This stress field, superimposed on the stress field produced by bending and shear away from the connection, redirects the stresses toward the flanges.

NEW CONNECTION DESIGNS

New, earthquake-resistant, steel moment connection designs should incorporate both weld fracture and flange overload mitigation measures. These connections should, also, be rational to design and economical to build. Two new connection configurations have been devised and tested at the University of Michigan.

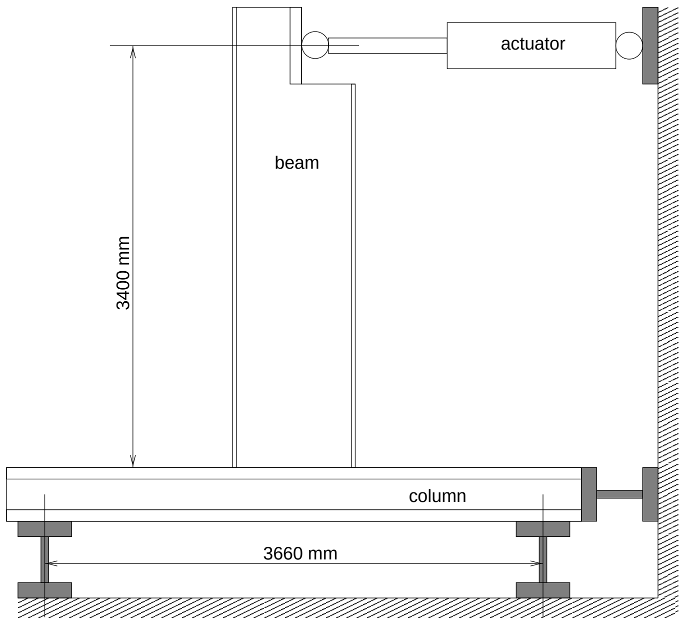

Figure 3: Connection test setup at the University of Michigan.

The first connection, called the Michigan connection, is an indirect, reinforced, fully-restrained connection (Figure 7). The beam and the column are not directly connected to reduce deformation restrains on them. Instead, they are indirectly connected using reinforcing elements. The cover plates are designed to transfer the moment couple while the vertical ribs, moulded into a K-shaped shear tab, are designed to take the connection shear force, in accordance with the truss model of connection stress flow. Details of the cover plate and vertical rib welds are designed to reduce local stress concentrations. To date, one prototype test and two proof-tests of the Michigan connection have been completed. These tests showed that the Michigan connection fulfills the requirements for special steel moment-resisting frames in regions of high seismicity [Goel 97].

The second new connection developed at the University of Michigan is the Free Flange connection (Figure 8). The free flange connection design is, also, based on a fundamental understanding of the stress flow in fully-restrained steel moment connections. However, the principal design goals are to reduce local bending of the flanges and to redirect the stress flow in the connection. Both goals are achieved by changing the relative shear stiffness of the flanges and the shear tab. By making a single cutback in the beam web, the flanges are made free to deform and less stiff in shear. Simultaneously, a thick trapezoid-shaped shear tab, fillet-welded to the beam web, provides the required shear stiffness and strength. All connection welds are made using notch-tough weld metal and improved weld practices, developed during the SAC post-Northridge connection tests. To date, seven proof-tests of the Free Flange connection have been successfully completed. Ultimately, they demonstrated that the Free Flange connections can be used in special steel moment-resisting frames in regions of high seismicity.

CONCLUSION

Fully restrained steel moment connections have been traditionally designed using a prescriptive design procedure. This procedure was invalidated by brittle fractures in these connections discovered after the 1994 Northridge earthquake. Until today, a widely accepted new design that satisfies the requirements for earthquake resistance has not been developed.

The work toward developing new earthquake-resistant steel moment connections conducted at the University of Michigan was presented in this paper. The fundamental finding stemming from this work is that successful earthquake-resistant connection should be designed through a combination of weld fracture and flange overload mitigation measures. This combination must be based on the fundamental understanding of the effect of boundary restraints on the stress flow in the connection.

The end result of the University of Michigan work are two practical and rational designs of fully-restrained

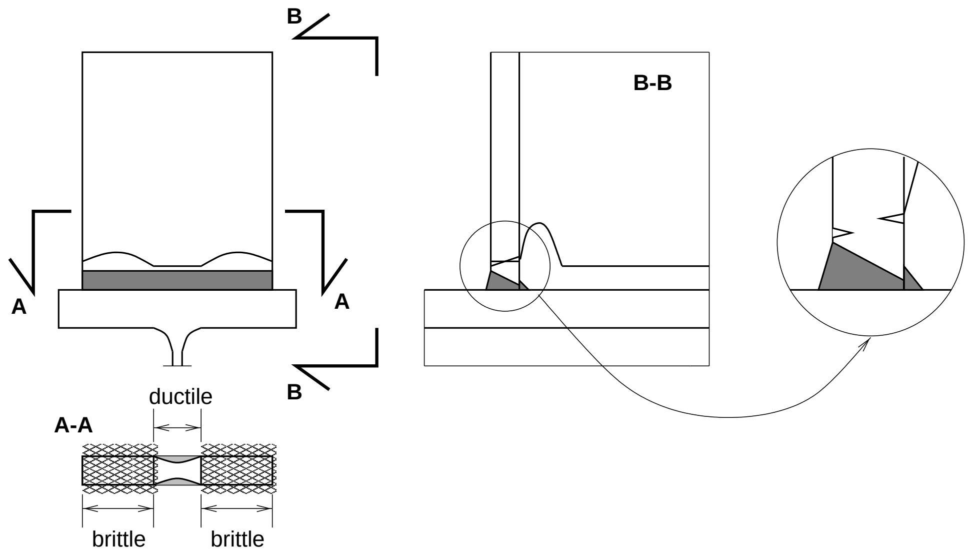

Figure 4: Geometry of the flange crack in post-Northridge connections.

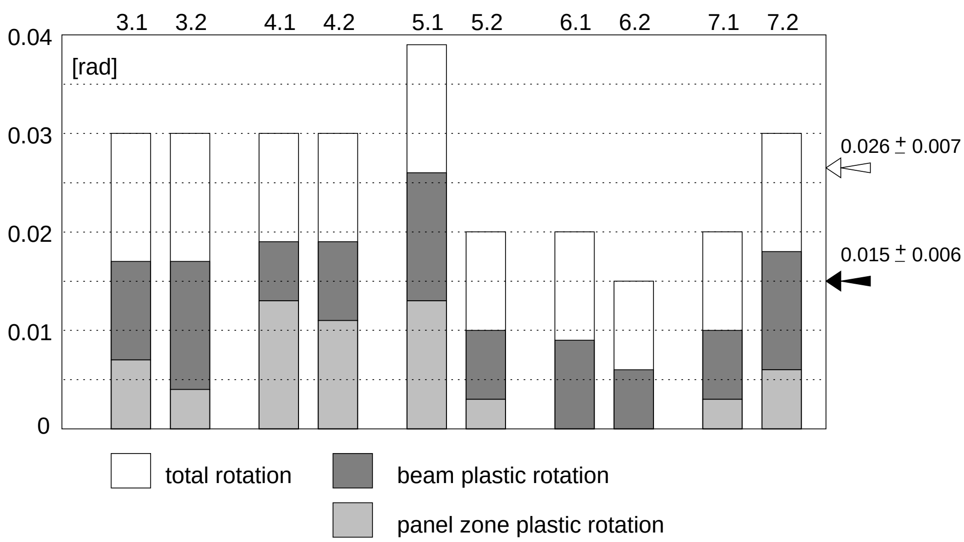

Figure 5: Post-Northridge connection rotation capacity summary.

steel moment connection. The Michigan connection and the Free Flange connection satisfy the AISC seismic proof-test requirements [AISC 97] and should behave very well in real earthquakes. They should restore designers' confidence in the special moment-resisting frame structural system.

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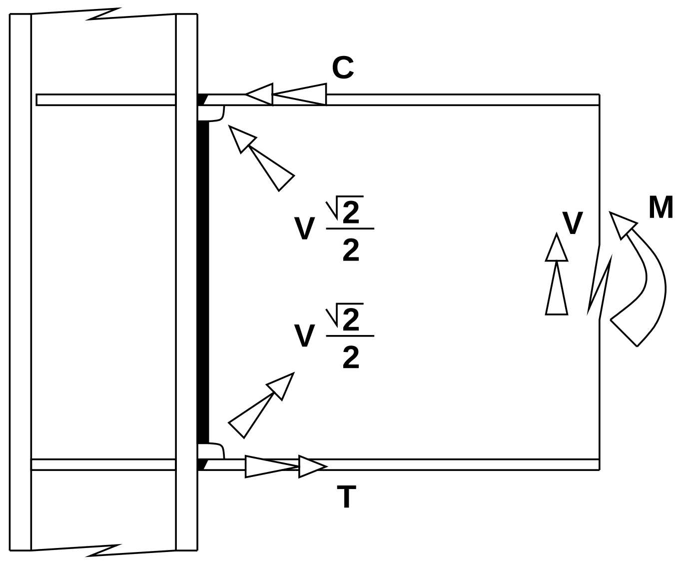

Figure 6: Truss model of the stress flow in steel moment connections.

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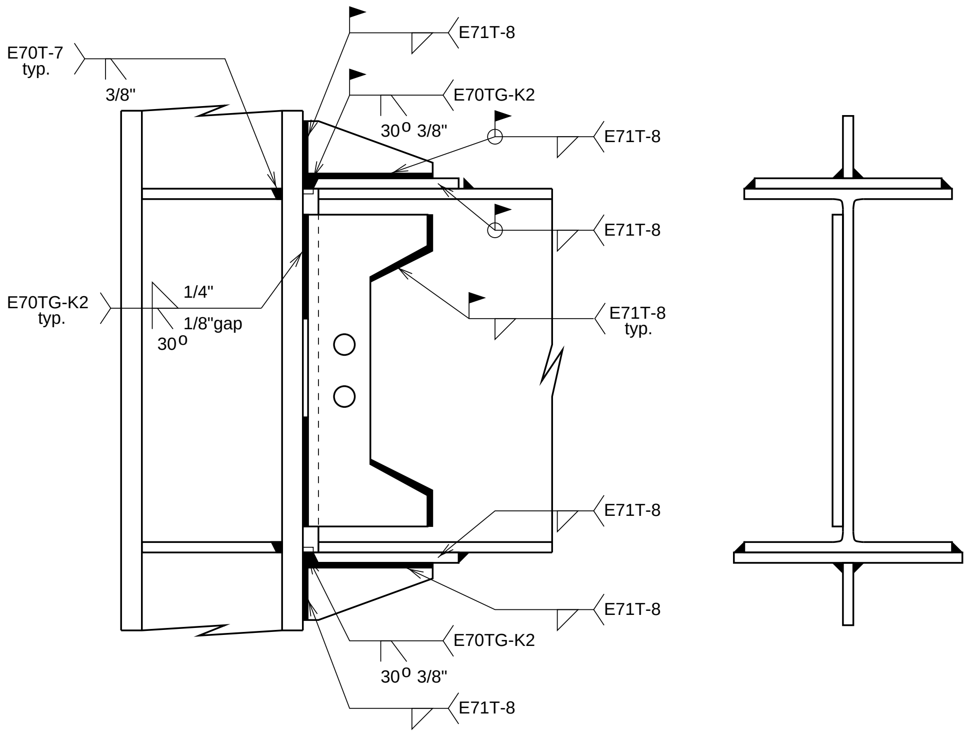

Figure 7: Michigan moment connection.

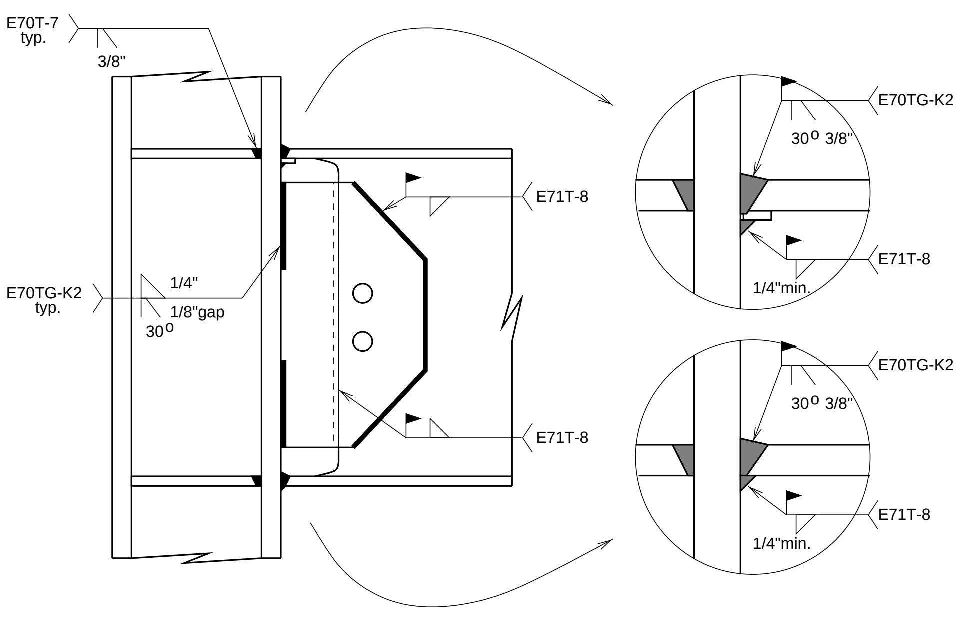

Figure 8: Free Flange moment connection.