Paper No. 857. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

OVERVIEW OF THE AISC NORTHRIDGE MOMENT CONNECTION TEST PROGRAM

T.A. SABOL[1], M.D. ENGELHARDT[2], R.S. ABOUTAHA[3], and K.H. FRANK[2]

[1]Englekirk & Sabol, Inc., 2116 Arlington Avenue, Los Angeles, CA 90018 USA
[2]Ferguson Structural Engineering Laboratory, University of Texas, Austin, TX 78712 USA
[3]Department of Civil Engineering, Georgia Institute of Technology, Atlanta, GA 30332 USA

ABSTRACT

Eighteen full-scale beam-column connections were tested using repetitive cyclic loading simulating earthquake demand. Specimens representing pre-Northridge and modified connections consisting of cover plates, upstanding ribs, or side plates were tested. Modified connections reduced demand on beam to column flange weld and shifted plastic hinges from face of column. Connections designed to prescriptive standards contained in 1991 Uniform Building Code suffered brittle failures at plastic rotations less than 0.05 radian while 10 of 12 modified joints developed plastic rotations from 0.025 to 0.05 radian without substantial strength loss. Unacceptable performance noted in two modified connections where steel divots were pulled from column face in a brittle failure or premature failure of a cover plated weld occurred. Marginal performance was obtained from the side plated connections is attributed to an unsuccessful detail compensating for unequal beam and column flange widths. Conclusions include: existing welded steel moment frames joints are susceptible to premature failure and unacceptable performance under moderate earthquake loads; modified joints that shift plastic hinge from face of column and reduce demand on welds are effective alternatives to pre-Northridge connections; and careful attention to proper welding technique and process is essential to achieve acceptable levels of plastic rotation.

KEYWORDS

Northridge Earthquake, steel moment frames, beam-column connection damage, weld damage

INTRODUCTION

Over 200 steel buildings sustained damage to beam-column joints in ductile steel frames during the 1994 Northridge Earthquake. The precise extent of damage and its causes are still being examined, but the structural engineering profession is being asked to evaluate methods of repairing damaged joints and designing reliable joints for ductile steel frames for on-going and future projects. This paper presents a brief summary of testing program sponsored by the American Institute of Steel Construction (AISC) shortly after the Northridge Earthquake.

Figure 1 schematically illustrates six basic failure attributes including cracks at the frame girder bottom flange, divots of steel removed from the face of the column flange, cracks in the frame column; cracks in the shear tab or shearing of web bolts; failures at the frame girder top flange welds; and cracks in the beam bottom flange.

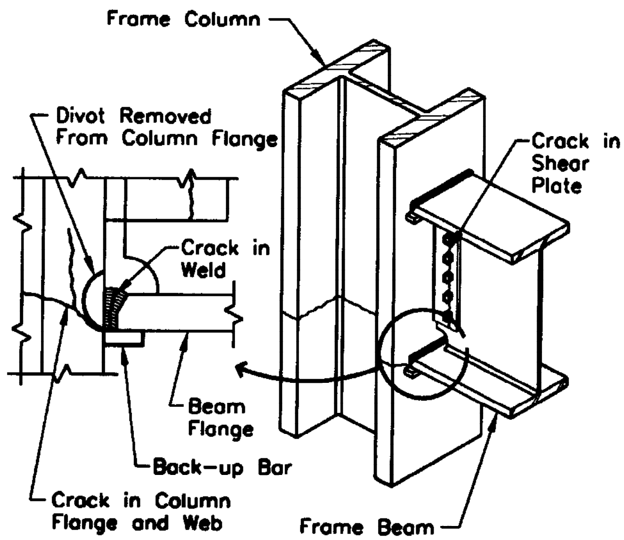

Fig. 1 . **Moment Frame Connection Damage Attributes**

One early study comparing bolted web-welded flange joints to fully welded joints was reported in the early 1970's (Popov and Stephen., 1972). This original research, using W18x50 and W24x76 beams which are quite a bit smaller than those found in most ductile steel frames, showed that while the bolted web-welded flange joints did not perform as well as the fully welded joints, they did develop substantial inelastic deformation. In spite of the erratic performance suggested by this research and subsequent testing programs (Popov *et al.*, 1986, Tsai and Popov.,1988, and Engelhardt and Husain, 1993), the conclusions drawn from the initial research, the improved economy of the bolted web-welded flange connection, and the structural engineering profession's increasing reliance on welding lead to widespread use of this connection.

Recent tests of welded flange-bolted web ductile frame connections (Engelhardt and Husain, 1993) and a re-examination of similar tests stretching as far back as 1972, suggest that these welded connections show a large variability in performance, maximum attainable plastic rotation, and weld failure rates. The observed post-earthquake condition of the failed connections appears to mirror the results of the least reliable connections in almost every aspect except for the horizontal cracks in the columns, which have not been widely reported in the literature. Since the earthquake, a number of potential problems inherent in the connection detailing, the welding materials and procedures used to execute the connection, and the advisability of a current design practice favoring a smaller number of high capacity frames have been expressed. Suggested factors contributing to this joint damage are summarized in Table 1.

THE AISC TESTING PROGRAM

Since the Northridge Earthquake, a number of testing programs have been undertaken within the United States. The three major programs are those sponsored by the AISC, the SAC Joint Venture, and the National Science Foundation. In addition, there have been a number of other, limited tests conducted in conjunction with specific projects. The earliest of these testing programs to be completed was the AISC program initiated under the guidance of the AISC Advisory Committee on SMRF Research (Engelhardt and Sabol, 1994). The goal of the AISC testing program was to conduct an *immediate* series of large scale tests to develop preliminary guidelines for improved beam-column connections for new construction. Tests were conducted on single cantilever specimens, as shown in Figure 2. Slowly applied cyclic loads were applied at the tip of the cantilever. Beam tip displacement was increased until connection failure occurred, or the limits of the testing apparatus were reached. Test specimen performance was judged primarily on the level of inelastic deformation achieved in the beam prior to connection failure. All test specimens were constructed of W36x150 beams of ASTM A36 steel, and either W14x455 or W14x426 columns of ASTM A572 Gr. 50 steel.

A number of different connection details were investigated in the test programs. The connections incorporated what were intended to be improvements in both welding and in connection design. Two replicates of each specimens were constructed by two different structural steel fabricators in order to gain some confidence in the repeatability of results.

Conventional Connections

The first connection detail investigated was the conventional welded flange-bolted web detail, designed in accordance with the seismic detailing provisions of the 1991 Uniform Building Code (ICBO, 1991). The detail for this specimen is similar to the geometry shown in Figure 1. Although the conventional construction detail was used, several improvements were incorporated in the welding, including removal of the backup bars

Table 1. Possible Factors Contributing to Connection Damage

Welding Related Factors
Inadequate welding workmanship and inspection
Lack of adherence to written Welding Procedure Specifications
Notch effect created by left-in-place backup bars
Detrimental effects of left-in-place weld extensions (tabs)
Use of weld metal with low notch toughness

Design Related Factors
Overstress of beam flange groove weld and surrounding base metal region caused by inadequate participation of bolted web connection in transferring bending moment
Uneven distribution of stress across the width of the beam flange groove weld
Highly restrained areas within the joint developing biaxial states of tension, thereby inhibiting ductile material response
Increase in bottom flange stress and strain caused by the composite floor slab
Panel zone deformation and column flange curvature
Use of deep frame girders

Material Related Factors
Actual yield stress of ASTM A36 beams considerably in excess of 36 ksi
Inadequate through-thickness strength or ductility of column flanges
Inadequate notch toughness of column material
High values of yield to tensile strength ratios (F_y/F_u)

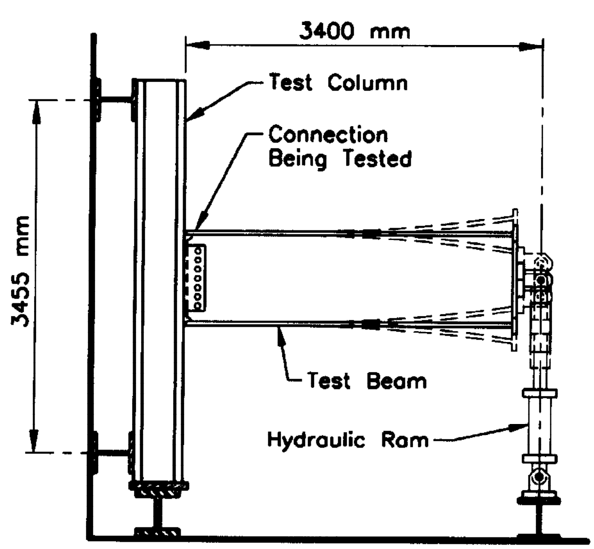

Fig. 2. Test Set-up

and weld extensions (tabs), combined with close attention to welding workmanship (AWS, 1994). Welding was accomplished by the self-shielded flux cored arc welding process (FCAW-SS). The electrode used for beam flange groove welds was classified as E70T-4, typical of past field welding practice for this connection. This electrode is characterized by very high deposition rates, but can result in weld metal with rather low toughness and ductility.

Both replicates of this specimen showed poor performance, developing only very limited ductility in the beam prior to connection failure. Failure of both replicates occurred by sudden fracture at the beam flange groove welds, with the fractures occurring near the weld-column interface. No welding workmanship defects were visible on the fracture surfaces. The unsatisfactory performance of these specimens suggests that improving welding, by itself, may not assure satisfactory connection performance. It has been conjectured that better performance could have been achieved if a different welding electrode or different welding process had been chosen. Figure 4 plots the results of these unsuccessful tests as dashed lines, showing maximum plastic rotations of approximately 0.005 radian. Numerical results are presented in Table 2.

The second connection detail investigated in the AISC program was an all-welded connection. It was similar to Figure 1, except that the beam web, rather than being bolted, was welded directly to the column flange. Past test programs have shown better performance from all-welded connections, as compared to welded flange-bolted web details. This better performance has been attributed to the improved ability of the welded web connection to transfer bending moment at the connection, thereby reducing stress on the beam flange welds. Both replicates of this connection detail showed poor performance, with fractures occurring at the beam flange groove welds early in the loading history for the specimens.

Modified Connections

The remaining connection details tested were classified as modified connections where the beam flanges were reinforced with cover plates or vertical ribs. Figure 3 illustrates representative geometries from the testing program.

Figure 3a illustrates the basic configurations of cover plate connections. The assumption behind the cover plate is that it reduces the demand on the weld at the column flange and shifts the plastic hinge away from the column face. In the figure shown, the bottom cover plate is rectangular, and sized slightly wider than the beam flange to allow downhand fillet welding of the joint between the two plates. Two specimens using triangular plates at the bottom flange, similar to the top flange, have also been tested but did not demonstrate any performance advantages.

Six of eight connections tested were able to achieve plastic rotations of at least 0.025 radian and up to 0.05 radian, as shown in Figure 4 (solid line). These tests were performed using heavy column sections which forced nearly all of the plastic rotation into the beam plastic hinge; very little column panel zone deformation occurred. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength. One specimen achieved plastic rotations of 0.015 radian when a brittle fracture of the CJP weld occurred. This unacceptable behavior may be a result of a weld that was not executed in conformance with the specified Welding Procedure Specification. The second unsuccessful test specimen achieved plastic rotations of 0.005 radian when a section of the column flange pulled out. Numerical results are presented in Table 2.

Figure 3b illustrates the basic configuration of connections with upstanding ribs. The two connections tested demonstrated the ability to achieve levels of plastic rotation in excess of 0.025 radian. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength; however, strength loss occurred more quickly than with the cover plated specimen. The tests were terminated when a slow tear of the bottom flange occurred at the tips of the ribs.

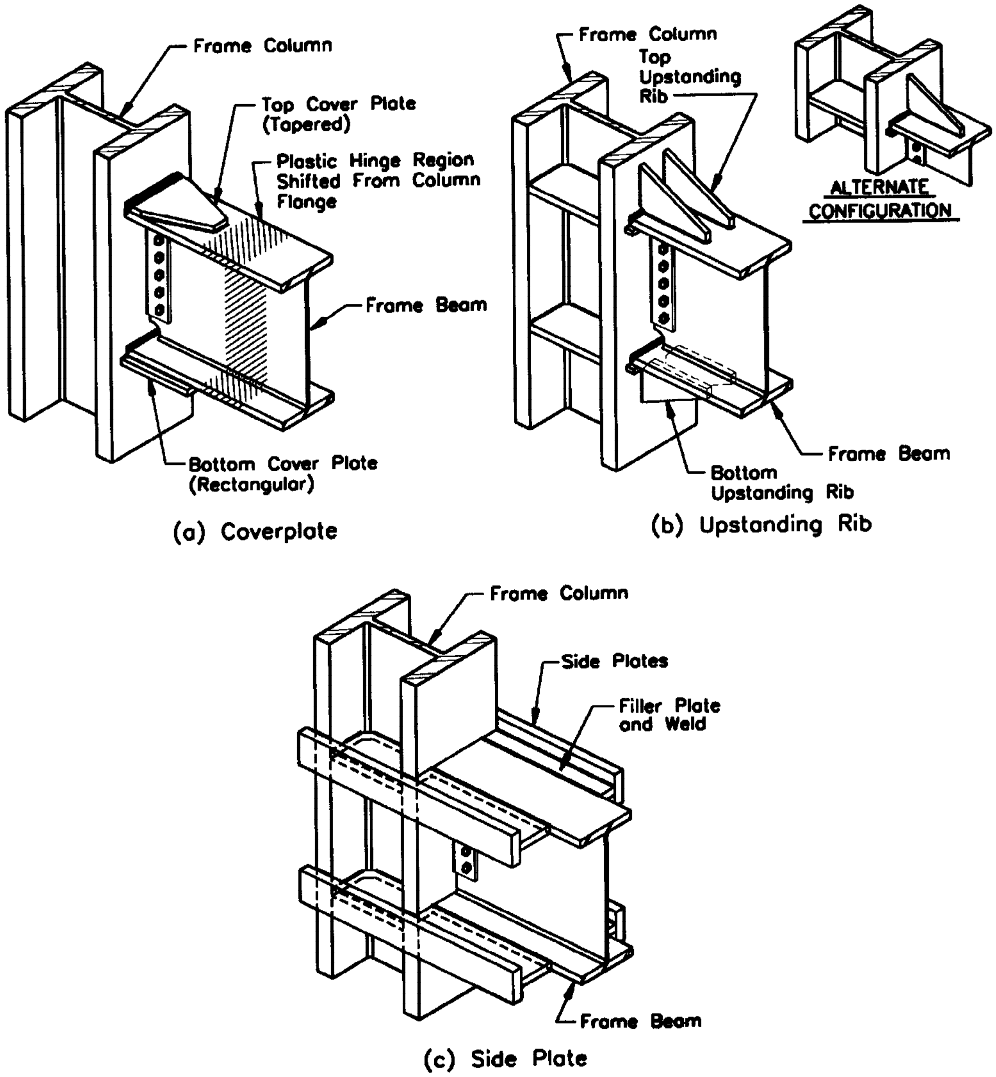

Fig. 3. Modified Beam-Column Connections Tested

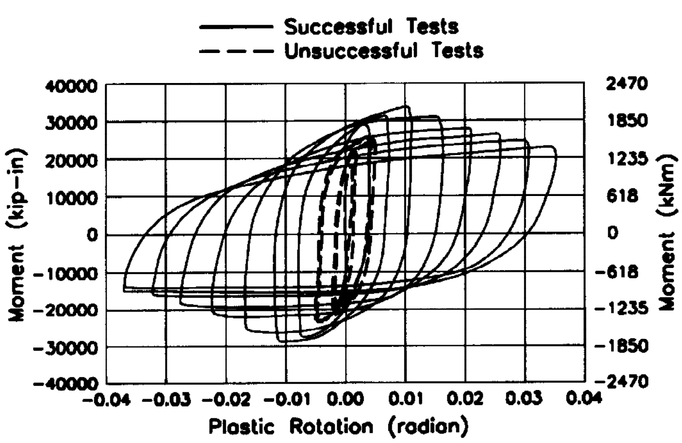

Fig. 4. Representative Moment-Rotation Curves for Unsuccessful and Successful Test Specimens

Table 2. Summary of Test Specimen Performance

Spec. No.	Fig. Ref.	Brief Description of Failure	Beam Plastic Rotation	Total Dissipated Energy (Joule)	Overall Assessment of Performance
1	1	Brittle fracture at top flange weld near weld-column interface	0.5 cycles at 0.005 rad.	77,500	Very poor
1B	1	Brittle fracture at bottom flange groove weld near weld-column interface	3 cycles at 0.005 rad.	182,400	Poor
2A	1*	Brittle fracture at top flange groove weld primarily within weld metal	0.5 cycles at 0.0025 rad.	34,200	Very poor
2B	1*	Brittle fracture at top flange groove weld primarily within weld metal	0.5 cycles at 0.009 rad.	296,400	Poor
3A	3a	Brittle fracture at top flange and cover plate groove weld near weld-column interface	0.5 cycles at 0.015 rad.	438,900	Poor
3B	3a	Gradual deterioration in strength , flange and web buckling, tear of bottom flange at cover plate	1 cycle at 0.025 rad.	1,915,200	Very good
5A	3a	Gradual deterioration in strength , flange and web buckling, tear of bottom flange at cover plate	2 cycles at 0.025 rad.	2,245,800	Very good
5B	3a	Brittle fracture within column flange (divot removed from flange)	1.5 cycles at 0.005 rad.	182,400	Poor
6A 6B	3b	Gradual deterioration in strength , flange and web buckling, tear of flange at cover plate	2 cycles at 0.025 rad. 1.5 cycles at 0.03 rad.	2,194,500 2,536,500	Very good Very good
7A 7B 8A 8B	3a	Gradual deterioration in strength , flange and web buckling, tear of fillet weld connecting cover plate to beam flanges	2 cycles at 0.035 rad. 0.5 cycles at 0.05 rad. 2 cycles at 0.035 rad. 2 cycles at 0.035 rad.,	2,907,000 2,907,000 2,998,200 2,941,200	Excellent Excellent Excellent Excellent
9A	3c	Brittle fracture of side plates	2 cycles at 0.015 rad.	866,400	Fair
9B	3c	Gradual fracture along connection of side strap to top and bottom flanges	3 cycles at 0.015 rad.	832,200	Fair

* Detail is similar to Figure 1 but connection used fully welded web in lieu of bolted web.

The failure of one cover plated specimen resulting a divot pulled from the face of the column suggested a detail to eliminate loading the column in the through-thickness direction. This was accomplished by eliminating the CJP welds at the girder flange and by shifting the plastic hinge from the column face. Figure 3c illustrates how the tension and compression forces are transferred from the girder flange through side plates into the column flange using fillet welds. Such a mechanism to provide direct connection between the column panel zone and the beam flanges must consider the unequal width of the beam and column flanges. A filler consisting of plate and weld metal was used to provide a load path between the girder flange and side plates. A tapered transition zone was used in an attempt to minimize the effect of the abrupt termination of the side plate.

These specimens achieved plastic rotations of only 0.015 radian and fractures developed within the weld connecting the beam flange to the side plates. Failure of the shear tab, and, finally, failure of the side plates themselves followed the initiation of these weld fractures. It is believed that the unsatisfactory behavior of these specimens was related to the detail used to increase the width of the beam flange to be equal to that of the column flange using a combination of a filler bar and welding. It has been suggested that a flat filler plate to transfer the forces, shown as an alternate to 3c, may perform better.

DESIGN CONSIDERATIONS

The results of the testing program suggest a number of design considerations to produce more reliable connection performance. Application of much of the currently available design guidance results in cover plates that are far thicker than those used in the successful test specimens and may result from a "conservative" interpretation of available information. Since there has been very limited testing of connections designed in accordance with these guidelines, it is not known if the presence of oversized CJP welds or overly long cover plates result in degraded performance.

In developing the analytical model of the frame, it is necessary to assume a realistic location of the plastic hinge. Based on an examination of the test specimens, it is likely that the effective plastic hinge is located beyond the tip of the reinforcing cover plates but within an additional length of one-fourth of the frame girder depth. Other, stiffer connection designs may shift the plastic hinge location further from the face of the column, a condition that should be reflected in the analytical model.

An effective yield stress in the beam must also be assumed. The moment reported in Figure 4 for the successful test specimen was calculated at the face of the column multiplying the applied load by the length of the cantilever (centerline of loading ram to face of column). This approach would not be appropriate to describe the plastic hinge moment for the modified connections, which is not located at the column face, because it will overestimate the actual moment at the plastic hinge, and, therefore, the stress at yield. However, the amplification of the moment from the plastic hinge to the face of the column should be reflected in the analytical model, but this amplified moment is not necessarily the moment obtained from calculations based on the total length of the cantilever times the applied load.

The presence of flange buckling when the plastic hinge forms appears to limit connection strength increases resulting from strain hardening. Arbitrary increases in assumed demand from strain hardening should be verified based on the level of strain achieved in the plastic hinge for the anticipated plastic rotation.

CONCLUSIONS

Beam/column connections with configurations corresponding to the prescriptive requirements of 1991 UBC (bolted web and welded flanges) do not develop adequate amounts of plastic rotation to insure acceptable performance in a large earthquake. This occurred despite efforts to insure a high level of workmanship and better than normal welding techniques and when removal of back-up bars, additional reinforcing fillets, fully

Reinforced connections using cover plates or upstanding ribs that shifted the plastic hinge away from the face of the column and the CJP weld performed better than connections that did not shift the plastic hinge. Sizing of the cover plate should keep the region of post-elastic behavior as close to the column face as possible to reduce the demand imposed on the weld and column panel zone. The cover plate should also have a minimum thickness to reduce the size of the CJP weld.

Strict adherence to an approved Welding Procedure Specification (WPS) and proper welding technique appear essential for reliable performance. The AISC testing program suggests that it is difficult for even an experienced welder to produce reliable welds when they deviate from an approved WPS. Experienced inspectors and non-destructive testing appear to be unable to detect welding practices that ultimately compromise the performance of welds that look acceptable but that were not executed in conformance with an approved WPS.

The AISC testing program did not examine directly the effect of column stiffeners, special welding techniques or processes (e.g. use of small diameter electrodes or SMAW vs. FCAW). Nevertheless, it is argued that providing column stiffeners, a minimum level of weld metal toughness and removal of potential stress risers in the joint will be shown to be beneficial.

ACKNOWLEDGMENT

The writers gratefully acknowledge financial support provided by the following organizations: the American Institute of Steel Construction, the National Science Foundation, and the J. Paul Getty Trust, and numerous donations from private companies involved in the steel construction industry. The opinions expressed in this paper are those of the writers, and do not necessarily reflect the views of the organizations noted above.

REFERENCES

AWS (1994). *Structural Welding Code, ANSI/AWS D1.1.* American Welding Society, Miami, FL.
Engelhardt, M.D. and A.S. Husain (1993). Cyclic-Loading Performance of Welded Flange-Bolted Web Connections. *Journal of Structural Engineering.* 119, 12.

Engelhardt, M.D. and T.A. Sabol (1994). Testing of Welded Steel Moment Frames in Response to the Northridge Earthquake. In *Northridge Steel Update Number 1.* American Institute of Steel Construction, Chicago, IL.

ICBO (1991). *Uniform Building Code, 1991 Edition.* International Conference of Building Officials, Whittier, CA.

Popov, E.P. and R.M. Stephen (1972), Cyclic Loading Performance of Welded Flange-Bolted Web Connections, *Bulletin 21.* American Iron and Steel Institute, Washington, D.C.

Popov, E.P., N.R. Amin, J. Louie, and R.M. Stephen (1986). Cyclic Behavior of Large Beam-Column Assemblies. *Engineering Journal.* American Institute of Steel Construction, Chicago, IL.

Tsai, K.C. and E.P. Popov (1988). Steel Beam-Column Joints in Seismic Moment Resisting Frames. Report Number UCB/EERC- 88/19. Earthquake Engineering Research Center, Berkeley, CA.