Seismic Connection Designs for Retrofitting Steel Moment Frames

Allen, Jay[1], Richard, Ralph M.[2], Partridge, James[3]

ABSTRACT

The "pre-Northridge" SMRF connection is fundamentally flawed. This is because its geometrical properties result in very high gradients in stiffness at the beam flange/beam web/column flange/column web junction in this connection. This results in large stress and strain gradients at the beam flange and column flange weldment that are caused by the out-of-plane flexing of these flanges that generate large local curvatures in these elements. The RBS (dogbone) and cover plate connections have these same geometrical properties and similar large stress and strain gradients. By separating the beam flange from the beam web with a beam web slot in the vicinity of the weldment, the beam flange curvature at the weldment is greatly reduced so that the stress and strain gradients in the beam flange and weld are also very significantly reduced.

In summary, the SSDA Slotted Beam design: (1) develops the full plastic moment capacity of the beam; (2) moves the plastic hinge region in the beam away from the face of the column; (3) results in nearly uniform tension and compression stresses and strains across and through the beam flanges from the face of the column to the end of the slot; (4) eliminates the lateral-torsional buckling mode that occurs in non-slotted beams; and (5) enhances ductility by reducing the beam flange/weld vertical shear and the residual weld stresses in the connection elements. Moreover, the slotted beam web concept may be used in retrofitting existing SMRF connections.

SALVAGING THE "PRE-NORTHRIDGE" STEEL BEAM TO COLUMN CONNECTION

Extensive damage to steel beam-to-column moment connections in several hundred buildings was caused by the 17 January 1994 earthquake in Northridge, California. These connections, used in Moment Resisting Frames (MRF), were fabricated with the beam flanges attached to the column flanges by full penetration welds and with the beam webs bolted to single plate shear tabs. Fractures originated in the MRF connection welds and/or in the beam and column base metal in an unexpected premature and brittle manner during the Northridge earthquake. Consequently, it has been concluded that this field-welded, field-bolted connection, described in the Uniform Building Code 2211.7.1.2 (UBC, 1994, pp 2-361) and shown in the American Institute of Steel Construction ASD and LRFD design manuals, is fundamentally flawed and should not be used in new construction. This conclusion, stated in the Structural Engineers Association of California's seismic structural design Blue Book (SEAOC, 1996, C706 Commentary), was determined through: (1) post-earthquake surveys of connection damage and modes of fracture; (2) a review of literature on historic laboratory tests that indicated significant failure rates during these tests (usually attributed at the time to poor specimen fabrication); (3) recently performed full scale connection tests that indeed failed premature and brittle fractures, and (4) recent finite element analyses and test results that show large stress and strain concentrations and gradients horizontally across and vertically through the beam flange/welds.

[1] President, The Allen Company/Seismic Structural Design Associates, Laguna Niguel, California
[2] Professor Emeritus, University of Arizona, Principal, Seismic Structural Design Associates, Tucson, Arizona
[3] President, Smith Emery Company, Principal, Seismic Structural Design Associates, Los Angeles, California

FINITE ELEMENT ANALYSES OF THE "PRE-NORTHRIDGE" CONNECTION

Presented here are finite element model results made to determine the stress, strain and force distributions in the pre-Northridge MRF connections using the ATC-24 (Applied Technology Council) connection test protocol assembly that is shown in Figure 1.

This moment frame subassembly, that is "pin" supported at the top and bottom of the column and loaded with a laterally supported beam tip load, was designed by the ATC to provide a reasonable simulation of seismic loading for the connection. Shown in Fig 2 is close-up view of one of the finite element meshes used in the connection analyses. Several mesh sizes were evaluated to ensure that the high stress and strain gradients in the beam flange and weldment were properly obtained (Richard, et al., 1995). These high fidelity models usually consisted of about 10,000 four node plate bending elements having about 10,000 degrees of freedom. Solid element sub-models used to evaluate internal stress and strain distributions in the beam and column flanges and in the welds comprised 80,000 elements and 100,000 degrees of freedom. The results of the plate and solid models generally were in agreement to within ten percent.

Shown in Figure 3 and 4 are the top, middle and bottom surface flexural stress and strain distributions, respectively, at the face of the column of the tension flange of a W36X150 beam connected to a W14X311 column using a baseline (pre-Northridge) connection with 1" thick fully extended column continuity plates. The nominal (strength of materials) flexural beam flange/weld stress at the column face is 50 ksi. However, this linear finite element analysis shows very large stress and strain gradients horizontally across and vertically through this tension flange. The maximum flexural stress is 176 ksi tension at the top and center of the flange whereas at the bottom and center of this tension flange the flexural stress is 30 ksi compression. These stresses are based upon a linear analysis so that the stress and stress gradient intensities are much larger than those obtained by an elastic-plastic stress analysis. However, the opposite is true for the strain and strain gradients which become much higher and much more severe as the beam flange becomes plastic. This very important ductility demand, which is required to accommodate stress redistribution in the MRF connection, has not been addressed. In low cycle fatigue research and design, Neuber's Theorem (Neuber, 1961) is widely used to evaluate both the stress and strain distributions in notched specimens. This theorem postulates that the product of the stress and the strain concentration factors evaluated in the elastic or inelastic range is equal to the square of the elastic stress concentration factor:

$$\mathbf{K_{stress} \times K_{strain} = (K_{stress\ elastic})^2}$$

For example, if the elastic stress concentration factor is, say, 4.0, and the actual stress concentration factor due to material inelastic behavior reduces to 1.0 (uniform flange stress), then the actual strain concentration factor at this stress level is 16.0! It is apparent that a great deal of ductile behavior is needed to meet this strain requirement. If the loading is dynamic, this puts even more adverse requirements on the ductility of the connection behavior. Typical elastic stress concentration factors in the beam flange/weld in the pre-Northridge, dogbone, and cover plate connections range from 4.0 to 5.0.(Allen, et al.,1995) and depend primarily upon the ratios of the beam flange width to column flange width and beam flange thickness to column flange thickness. Note that in cover plate designs, the beam flange/plate weld is often about the same thickness as the column flange thickness which creates a large HAZ in the column flange.

A second linear analysis of this assembly was made without the 1" fully extended continuity plates in the W14X311 column. This design modification changed the above maximum stresses at the center of the top and bottom surfaces to 197 ksi tension and 20 ksi compression, respectively. This relatively small increase in the maximum stress (about 10%) indicates that continuity plates in MRF columns are not as effective under seismic (lateral) frame loading where they act primarily in shear as they are under frame gravity loading where they act in tension.

These severe stress gradients, horizontally across and vertically through the beam flange/weldment, as shown in Figures 3 and 4, are a result of out-of-plane flexing of both the beam and column flanges. The resultant moment of this transverse stress distribution results in beam flange prying action on the face of the column. This prying action can initiate the following connection fracture modes: a divot pullout in the column flange; a tensile weld fracture at the column face; a fracture of the beam flange near the weld or in the vicinity of the web weld access hole boundary; or a "k line" fracture in the column web opposite the beam flange. All of these fracture modes have been observed both in the field (Youseff, et al.,1995) and in laboratory tests of pre-Northridge connections (Engelhardt, et al., 1995). For example, a cover plate connection, that has these similar large stress and strain gradients and flange distortions, was tested at the University of Texas and fractured in a brittle manner due to a divot pullout in the column flange. An examination of this fracture by the Swinden Laboratories, Moorgate, England, indicated the final fracture initiated in the heat affected zone (HAZ) of the column flange. The initiation region, that was at the high stress and strain location, was in the column flange and was characterized by a flat fracture appearance and consisted of a mixture ductile microvoid fracture and fine grained cleavage (Harrison, 1995).

SLOTTED BEAM WEB CONNECTION DESIGNS

After comprehensive elastic, plastic, and buckling analyses using finite element models coupled with ATC protocol testing of modified designs, Seismic Structural Design Associates, Inc. (SSDA), developed proprietary slotted beam connection designs that essentially salvage the pre-Northridge connection (Richard, et al., 1997). These designs may be used for both new construction and retrofitting existing buildings. This connection, shown in Figure 5, uses horizontal beam web slots, a fully welded and bolted single plate shear tab, and a beam web to column flange weldment. Also, a ductile weld material is specified.

The SSDA Slotted Beam Web connection designs reduce the Stress Concentration Factor (SCF) at the beam-to-column flange interface from values typically ranging from 4.5 to 5.5 in non-slotted beams down to a typical value of about 1.4 in slotted beams. This is accomplished by providing a much more uniform beam flange/weld stress and strain distribution. Shown in Figure 6 are the top and bottom surface flexural stresses in the tension flange for the above connection with 18" beam web slots in the W36X150 beam, and for comparison the stress distributions for the pre-Northridge and Reduced Beam Section (RBS) connections (the RBS is also called the "dogbone" connection). This linear analysis shows that the slotted beam maximum top and bottom surface stresses at the center of the flange/weld have been reduced to 75 ksi tension and 50 ksi tension, respectively. This results in a very dramatic reduction in the beam flange prying action and in the ductility demand of this connection.

The large strain gradients, that exist in the pre-Northridge, Reduced Beam Section (RBS) or dogbone, and cover plate connection designs also result in large strain rate variations in the beam flange/weld region under dynamic loadings. High strain rates increase the yield strength of structural steels Collins (1993). These high rates can initiate brittle fracture modes in connections that would behave in a ductile manner under low strain rates.

Shown in Figure 7 are the shear force distribution at the face of the column for the assembly. Both the baseline and dogbone connections have large vertical shear forces (50% of the total shear) in the beam flanges which demonstrates that in contrast to classical beam theory, the stress distribution change drastically in the vicinity of the connection. This results in very high stress and strain concentrations in the flanges and flange welds. In the slotted beam, however, virtually all the vertical shear (97%) is carried by the beam web and shear plate which is in agreement with usual design practice.

Another very important attribute of the SSDA slotted beam design is that it allows the beam flanges and beam web to buckle independently as shown by the elastic buckling modes in Figures 8 and 9. This circumvents the beam lateral-torsional buckling mode shown in Figure 10 that occurs in all non-slotted beams (Zekioglu, et al.,1997). The beam web slots eliminate the torsional moments and torsional stresses in the beam flanges and welds at the column flange that result from this buckling mode. Tests of non-slotted beams showed that these test specimens typically exhibited a considerable twist and strength degradation that accompanied lateral-torsional beam buckling as exemplified in Figure 10. Testing was often stopped when lateral-torsional bucking occurred in order to avoid test fixture and equipment damage (Kaufman, et al., 1996). One mode of connection failure reported in these tests was an initial fracture of the beam flange/weld at one tip that then progressed to the other tip. This fracture mode indicates the role of the torsional moment and stresses in this mode of buckling. Recent ATC-24 tests of RBS (dogbone) beams at the University of California at San Diego used lateral bracing at the reduced beam section to prevent rapid degradation of beam strength. Measured lateral forces at the brace point at the reduced beam section were of the order of 3% to 6% of the flange forces or 10% to 20% of the applied ATC-24 test beam tip load (Zekioglu, 1997, p. 36)

ANALYTICAL AND ATC-24 TEST CORRELATIONS

Shown in Figure 11 are the results of an elastic, plastic, and plastic buckling analysis of a 12 ft. W33X201 slotted beam using a bilinear stress-strain curve (E = 30,000 ksi; Ep = 300 ksi). The plastic buckling load for this cantilever beam with a yield stress of 42 ksi is approximately 250 kips whereas its first mode linear elastic buckling load, shown in Figure 8, is 1240 kips. Typical analyses such as this and ATC-24 test results demonstrate that the SSDA beam web slots do not reduce the strength of the beam. Moreover, finite element analyses have shown that these slots did not reduce the elastic stiffness of the ATC-24 assemblies. The beam web slot lengths are the smaller of: (1) 1.5 times the beam flange width measured from the column face or (2) the length of the beam web plastic hinge length measured from the end of the shear plate. The latter length criterion usually governs in short beam spans.

A plot of the cyclic hysteresis curve for the W14X500 column and W36X280 slotted beam ATC-24 assembly is shown in Figure 12. A plastic rotation of 3.7% exceeded the current recommended SAC Guidelines minimum of 3%. Shown in Figures 13 and 14 are the yielded and hinged W36X280 Beam at -7 and +7 delta yield, respectively. These two figures show the buckled 1.57" lower beam flange for the downward loading followed by the straightening of this flange during the upward loading. SSDA has successfully completed a total of seven ATC-24 protocol tests using columns ranging from W14X176 to W14X550 with beams ranging from W27X94 to W36X280. None of these assemblies experienced the lateral-torsional buckling modes that are typical of non-slotted beam and column assemblies.

Two "retrofit" slotted beam assemblies were fabricated using the "pre-Northridge" E70-T4 weld material which now has been banned because of it low ductility and notch toughness. These test specimens exhibited excellent moment-plastic rotation behavior by attaining over 5% of plastic rotation. This indicates that the "pre-Northridge" SMRF connections lack of ductility may have been primarily due to the large stress and strain gradients in the beam flange/welds that also result in large strain rate variations in these connections.

COMPLETED AND CURRENT PROJECTS

SSDA has completed or is currently completing connection designs for seven new steel moment frame buildings. These include the eight story St. Francis Medical Center (OSHPD Approved) in Los Angeles, CA, and the Gateway West twenty story office building in Salt Lake City, UT. Thompson and La Brie Structural Consulting Engineers, Pasadena, CA, is the engineer of record for the former, and Reaveley Engineers and Associates, Inc., Salt Lake City, for the latter. Additional new projects under contract with Culp & Tanner, Inc., Lake Forest, CA, are a nine story office tower in Las Vegas, NV and a two story addition to the John Ascuaga's Nugget Hotel & Casino in Reno, NV. New projects pending include a six story office tower in Long Beach, CA and a five story office tower in Los Angeles, CA. The slotted connection cost is between $400 and $900 per connection, depending upon beam and column sizes, above the "pre-Northridge" connection.

Four buildings in the Los Angeles area have been or are being retrofitted using the SSDA Slotted Beam connection. Included is a ten story office building in Burbank, CA, where 194 "pre-Northridge" connections were repaired as required and retrofitted at an average cost of $2700 each, and a three story medical office building in Woodland Hills, CA, where 97 "pre-Northridge" connections were repaired and/or retrofitted at a cost of $3000 per connection. The latter project costs includes the repair of 22 connections and the retrofitting of 97 connections. Web access holes were made in 32 of these connections to make the beam web to column flange weldment without removing the building skin. The above costs include removal and replacement of fireproof materials and cleanup and all design and contractor costs. Additionally, a 3 story medical office building in Granada Hills, CA and a 3 story office building in Thousand Oaks, CA are under contract.

REFERENCES

Allen J., Partridge, J., Radau, S., and Richard, R., "Ductile Connection Designs for Welded Steel Moment Frames", Proceedings-Structural Engineers Association of California, 64th Annual Convention, October, 1995.

American Institute of Steel Construction, AISC Northridge Steel Update I, October, 1994.

Collins, J.A., "Failure of Materials in Mechanical Design (2nd ed.)", New York, John Wiley & Sons, 1993.

Englehardt, M.D., Sabol, T.A., and Frank, Karl., "Testing Connections", Modern Steel Construction, AISC, p 36, May, 1995.

Harrison, P.L.,"Examination of Two Moment Resisting Frame Connections Utilizing a Cover Plate Design", British Steel Technical Report SL/EM/R/S1198/18/95/D, 20 March, 1995.

Kaufmann, E., Ming Xue, Le-Wu Lu, Fisher, J., "Achieving Ductile Behavior of Moment Connections", Modern Steel Construction, AISC, p. 30, January, 1996.

Neuber, H., H., "Theory of Stress Concentration for Shear Strained Prismatic Elements with Nonlinear Stress-Strain Law", Jr. of Applied Mechanics, Vol. 28, Series E, No. 4, December, 1961.

Richard, R., Partridge, J., Allen, J., and Radau, S., "Finite Element Analysis and Tests of Beam-to-Column Connections", Modern Steel Construction, AISC, Volume 35, No. 10, p. 44-47, October, 1995.

Richard., R., Allen, J., and Partridge, J., "Proprietary Slotted Beam Connection Designs", Modern Steel Construction, AISC, p. 28-33, March, 1997.

"SEAOC Blue Book", Structural Engineers Association of California Seismic Design Code, 1996.

"Uniform Building Code", Volume No. 2, Structural Engineering Design provisions, pages 2-361, 1994.

Youseff, N., Bonowitz, D., Gross, J.L., " A Survey of Steel Moment Resisting Frames Affected by the 1994 Northridge Earthquake", Rep. No. NISTIR 5625, NIST, Gaithersburg, MD.

Zekioglu, Atila, Mozaffarian, H., Le Chang, King., and Uang, C-M, "Designing After Northridge", Modern Steel Construction, AISC, p. 36, March, 1997.