Current Steel Structures Research

REIDAR BJORHOVDE

After more than five years of assembling and evaluating structural steel research projects at universities and other institutions around the world, it has become very clear to me that significant efforts are taking place in a number of countries. Although American steel research for buildings and bridges was preeminent for many years, and especially during the period from 1930 to 1980, it is amply evident that the developments in many countries have produced wide-ranging results with major implications for engineering and the construction industry. Most importantly, the worldwide community of researchers and engineers exchange ideas and findings to ensure that advances will continue to take place. Differences occur in interpretations and developments in the form of design criteria and construction practices, but by and large, the end results are effectively the same.

As a result, a new feature is being introduced for these steel structures research papers. In addition to the regular project descriptions from individual institutions, each paper will highlight the work at two or three major steel research universities—the "steel schools" of the world. The descriptions will not discuss all of the current projects at the school. Rather, a selection of studies will provide a representative picture and demonstrate the school's importance to the efforts of industry and the profession.

This issue of *Engineering Journal* provides the first in this new series, featuring the work at two North American universities. It reflects their ongoing, central roles in the research activities of the United States and Canada and their long-time impact on the design standards of the two countries. Both schools are well known: Lehigh University in Bethlehem, Pennsylvania, and the University of Alberta in Edmonton, Alberta. Researchers at the two universities have been very active for many years, as evidenced by their participation and leading roles in the standards development committees and by large numbers of outstanding technical papers, reports and conference presentations.

References are provided throughout the paper, whenever such are available in the public domain. However, much of the work is still in progress, and in some cases reports or publications have not yet been prepared for public dissemination.

Reidar Bjorhovde, Research Editor for *Engineering Journal,* 5880 E. Territory Ave., Suite 202, Tucson, AZ 85750-1803. E-mail: rbj@bjorhovde.com

LEHIGH UNIVERSITY

Development of Improved Welded Moment Connections for Earthquake-Resistant Design: This study was conducted with the sponsorship of the SAC Joint Venture and the Pennsylvania Infrastructure Technology Alliance. Steel shapes for the test specimens were donated by the Technical Committee on Structural Shapes. Professor James M. Ricles has been the director of the project.

The project was undertaken as a major assessment of the effects on the cyclic (seismic) ductility of welded unreinforced moment connections of the weld metal, the geometry of the weld access hole, the form of the beam web attachment to the column, the continuity plates, and the panel zone strength. Using finite element evaluations as well as full-scale physical tests, the results of the former were used to develop 11 connections for the testing program (Ricles et al., 2003). As illustrations of some of the findings, Figure 1 shows the setup for the connection tests and Figure 2 shows the effective plastic strain contours in the vicinity of the weld access hole.

Commenting on some of the key findings, the researchers note that:

1. Several studies have shown that the geometry of the weld access hole is critical to the performance of the connections. The difference between the original configuration and the modified access hole is emphasized by a short, flat portion on the beam flange. Figure 2 shows that the modified shape allows for a larger portion of the flange to develop plastic strains, hence an increased ductility for the connection as a whole.

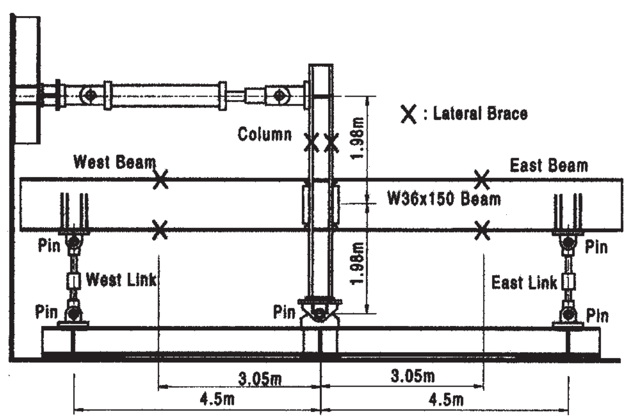

Fig. 1. Test setup for connections (Courtesy of Professor James M. Ricles).

2. A panel zone designed on the basis of the column web yield capacity gives a connection that performs better than specimens that also incorporate the contribution of the column flanges.

3. Shear studs should not be placed in the plastic hinge region of the beam, since this reduces the fracture toughness of the beam flange material.

4. Continuity plates are not always needed. The connections that were tested without such plates and without shear studs in the plastic hinge region of the beam performed very well, developing plastic story drifts larger than 0.04 radian.

Seismic Behavior of Reduced Beam Section Moment Connections to Deep Columns: This study was conducted with the sponsorship of AISC and the Pennsylvania Infrastructure Technology Alliance. Materials for the test specimens were donated by Arcelor International America, Nucor Vulcraft Group and Lincoln Electric Company. Professor James M. Ricles has been the director of the project.

A number of other projects have addressed the issues of reduced beam section (RBS) connections and the performance of frames with such connections. This project focused on frames using deep beam shapes as columns, which is not an uncommon feature of certain structures in seismic areas. Finite element analyses were used to perform parametric studies with three-dimensional RBS models in special perimeter moment-resisting frames subjected to inelastic monotonic and cyclic loading (Zhang and Ricles, 2006a; 2006b). The parameters were (1) the beam-to-column connection type, (2) the column shape, (3) the composite floor slab, (4) the strength of the panel zone, and (5) the beam web slenderness. Six full-scale frame subassemblages were tested; the general appearance of the test setup is similar to what is illustrated in Figure 1.

Among the unique performance characteristics of these types of frames is the fact that under certain conditions and for some of the parameters, the column will exhibit significant twisting. Figure 3 illustrates this phenomenon. Specifically, the twisting occurs as a result of out-of-plane movement of the RBS compression flange. But it was also demonstrated that the composite slab has a very significant influence on the response of the subassemblage, to the effect that it restrains the top flange of the beam and therefore reduces the lateral displacement of the RBS connection and consequently the magnitude of the column twist. In addition, the slenderness of the beam web is an important contributor to the lateral displacement of the beam flange in the connection. Figure 4 shows the finite element results for the column twist as a function of the beam web slenderness and the influence of the slab, using a range of beam shape sizes. Thus, the amount of twist is small and essentially constant for all magnitudes of beam slenderness when the slab effect is taken into account. In the absence of the slab, the twist is quintupled as the beam slenderness goes from that of a W36×135 to that of a W36×256.

Horizontally Curved Tubular Flange Girders: This project has been sponsored by the Federal Highway Administration and the Pennsylvania Infrastructure Technology Alliance, with High Steel Structures, Inc., Lancaster, Pennsylvania, as a project partner. The project director has been Professor Richard Sause.

Recognizing the fact that complex highway geometries offer significant challenges for the bridges that have to be built, curved girder bridges have become very common over

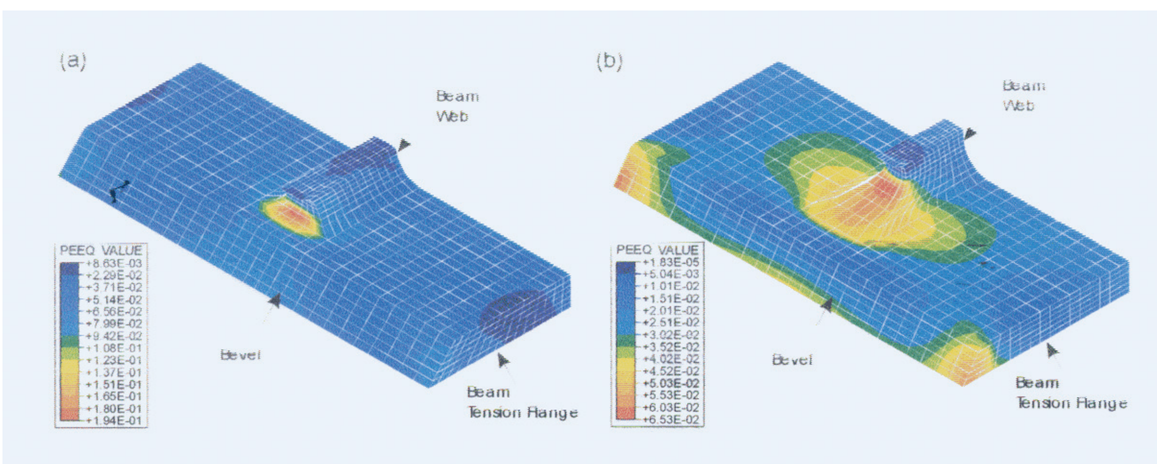

Fig. 2. Plastic strain contours for (a) traditional weld access hole and (b) modified weld access hole (Courtesy of Professor James M. Ricles).

the past several years. One of the major issues for curved girders is the lateral stability during construction as well as during service. Traditional rolled beams and plate girders have flanges that offer low torsional stiffness. The concept of built-up girders with tubular flanges was developed in recognition of the stiffness offered by the tubular shape, in addition to the fact that such flanges could add strength as well as bending stiffness for the girders. Figure 5 illustrates the concept.

Recent analytical studies have demonstrated the potential of such girders (Dong and Sause, 2009, 2010). A testing program for curved tubular flange girders is now under way, focusing first on one-half scale individual girders and now also on a two-thirds scale two-girder bridge, as shown in Figure 6. The two-girder bridge test aims at determining the behavior and strength under simulated construction and service conditions; the response characteristics of individual girder will be assessed in comparison to regular curved plate

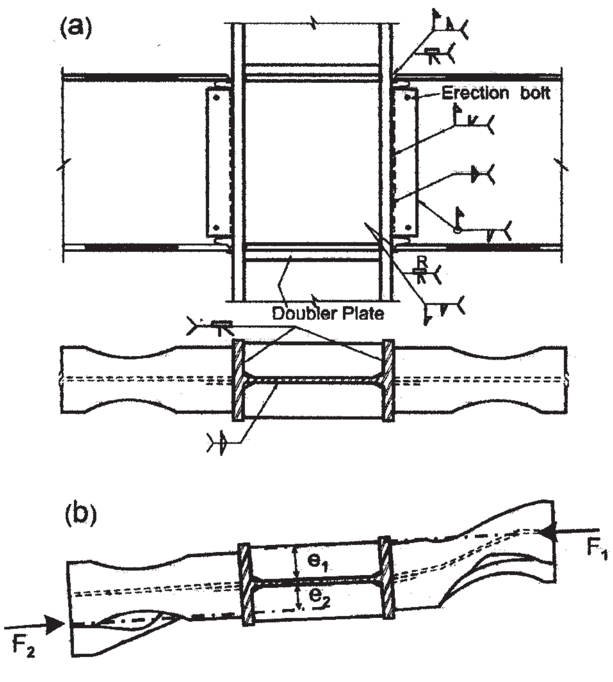

Fig. 3. (a) Details of the RBS connection; (b) column twisting that occurs when a plastic hinge forms in the RBS (Courtesy of Professor James M. Ricles).

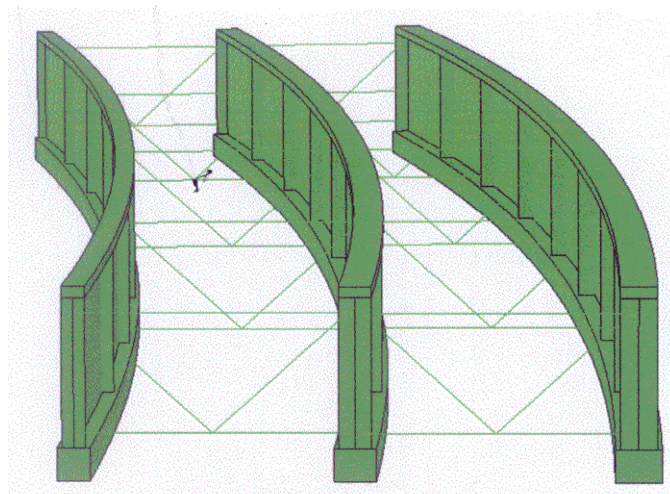

Fig. 5. Curved hollow flange girder system (Courtesy of Professor Richard Sause).

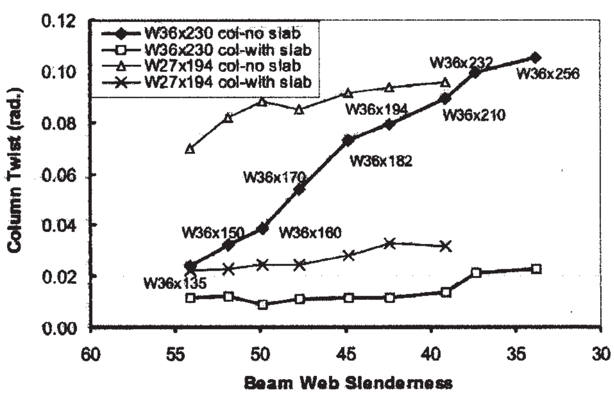

Fig. 4. Influence of slab, beam size and web slenderness on the magnitude of the column twist. Column sizes are shown in top left corner. (Courtesy of Professor James M. Ricles).

Fig. 6. Test specimen for a two-thirds scale curved tubular flange bridge structure (Courtesy of Professor Richard Sause).

girders. The results to date confirm the predicted stiffness and strength, with substantially smaller stresses, deflections and cross-sectional rotations than those of curved plate girders. Further, a multi-girder tubular flange bridge requires fewer cross frames and smaller frame members.

Damage-Free Seismic-Resistant Self-Centering Concentric Braced Frames: This project has been sponsored by the NEES Program of the National Science Foundation and by AISC. Professors James M. Ricles and Richard Sause have been the project directors.

Concentrically braced frames (CBF) offer relatively simple and economical structural systems. However, in view of the limited drift capacity of such types of frames and their tendency to suffer seismic damage, the self-centering CBF system was developed to provide structures that would undergo limited damage. At the same time the system offers the opportunity for improved repair and restoration after an earthquake (Sause et al., 2006; Roke et al., 2009).

The unique behavior of these self-centering frames (SC-CBF) is represented by the rocking of the frame at the base. The structure is stabilized by vertical high-strength post-tensioning (PT) bars that extend over the full height of the frame. Under low-level earthquakes the frame behaves as traditional CBF; under high lateral loads the PT bars provide a stabilizing, restoring system that limits the frame displacements and the damage that otherwise would occur.

Extensive nonlinear dynamic analyses have been performed to assess the behavior of the frame, and at this time a large-scale frame test is under way. Figure 7 shows the multistory frame (yellow elements within the test setup), and Figure 8 shows the uplift that takes place at the base. The physical response therefore mirrors the analytical predictions.

The researchers have developed an improved design method that also incorporates nonlinear dynamic procedures. It takes into account the internal forces associated with yielding of the post-tensioning bars as well as the forces associated with higher modes of dynamic response.

UNIVERSITY OF ALBERTA

Steel Plate Shear Walls with Partially Encased Composite Columns: This project has been sponsored by the Natural Sciences and Engineering Research Council (NSERC) of Canada and the Canam Group. Professor Robert G. Driver has been the project director.

Steel plate shear walls are now being used in a number of structures in seismic areas. One of the problems of these otherwise very efficient and ductile lateral load-resisting systems has been the stability and strength of the columns (boundary elements) of the walls. Studies have demonstrated that partially encased composite columns, with concrete placed between relatively thin flanges and web of built-up H-shapes (Chicoine et al., 2002; Prickett and Driver, 2006), may offer a suitable solution for the columns of the shear walls. It is noted that the thin flanges are connected by closely spaced steel bars to prevent local buckling. These bars can be seen in Figure 9.

A series of three large-scale shear wall tests are in the process of being conducted. The intent is to establish the behavior, ductility and failure mode of the shear wall and the columns. The second of the test walls utilized a modular system, omitting the moment connections to have a more

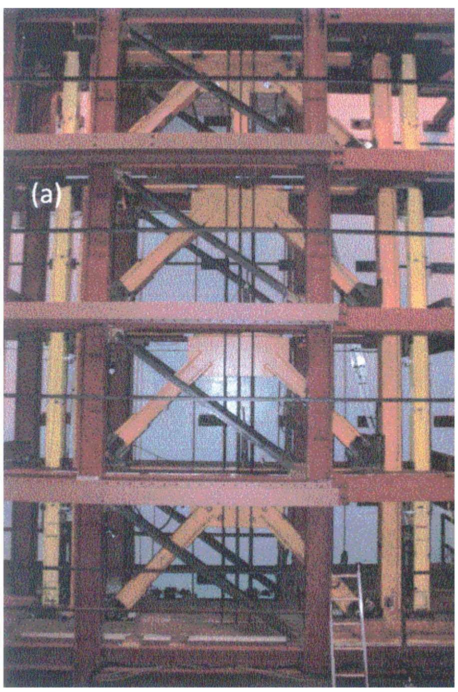

Fig. 7. SC-CBF test setup. Yellow elements in the picture represent the braced frame (Courtesy of Professors James M. Ricles and Richard Sause).

Fig. 8. Rocking displacement at the base of a column of the SC-CBF (Courtesy of Professors James M. Ricles and Richard Sause).

economical solution. Figure 9 shows this test specimen before the concrete has been placed between the flanges of the columns.

The first test used the regular solution with beam-to-column moment connections; it demonstrated very ductile response characteristics but also identified certain connection detailing issues. Figure 10 shows this test in progress, with the usual buckling shape of the shear wall.

Design recommendations are currently being developed. In particular, special attention is paid to the detailing needs of the walls.

Repair of Fatigue Cracks in Steel Structures: This project has been sponsored by the Natural Sciences and Engineering Research Council (NSERC) of Canada and Syncrude Canada Ltd. Professor Gilbert Y. Grondin has been the project director.

Fatigue cracks occur regularly in structures such as bridges, industrial structures, offshore structures and mining equipment. Downtime due to repair efforts can be long and costly, and the repairs are not always effective. Under the best of circumstances, the designer should be in a position to specify what repair method and procedures should be followed, and even whether repairs should be attempted in the first place. The latter is recognition of the fact that such repairs often make the situation worse. Nevertheless, practical repairs need to be done, with a realistic expectation that they will be successful. Very often the designer will evaluate what was done in the past for similar cracking details, but such an approach is neither effective nor fully reliable.

Various repair techniques are being evaluated by the researchers at the University of Alberta, including assessments by fracture mechanics, reliability methods and similar advanced approaches. The variables include material properties, detail geometries, initial flaw conditions and loading conditions. Possibly two of the most common repair methods involve hole drilling and hole drilling with hole expansion. For the case of hole drilling alone, a hole is placed at the crack tip, on the assumption that this will arrest the crack propagation. Such has been successfully used in many cases. On the other hand, hole drilling and a small amount of hole expansion has been found to be very effective. Thus, expanding the hole at the crack tip by a mere 3% increases the fatigue life by a factor of 4 to 5. This is illustrated in Figure 11, where fatigue life versus stress range has been determined for holes with and without expansion. The correlation with reliability analyses of various forms is very good (Josi, 2010).

SWISS FEDERAL INSTITUTE OF LAUSANNE

Crack Propagation in Tubular Joints under Compressive Loading: This project has been sponsored by the Swiss National Science Foundation (SNF) with test materials provided by Vallourec & Mannesmann. Professor Alain Nussbaumer has been the project director.

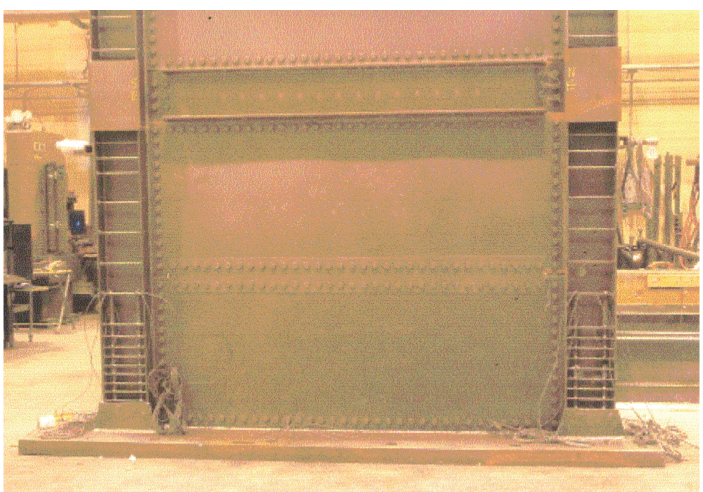

Fig. 9. Modular shear wall specimen before concrete placement in between the flanges of the columns (Courtesy of Professor Robert G. Driver).

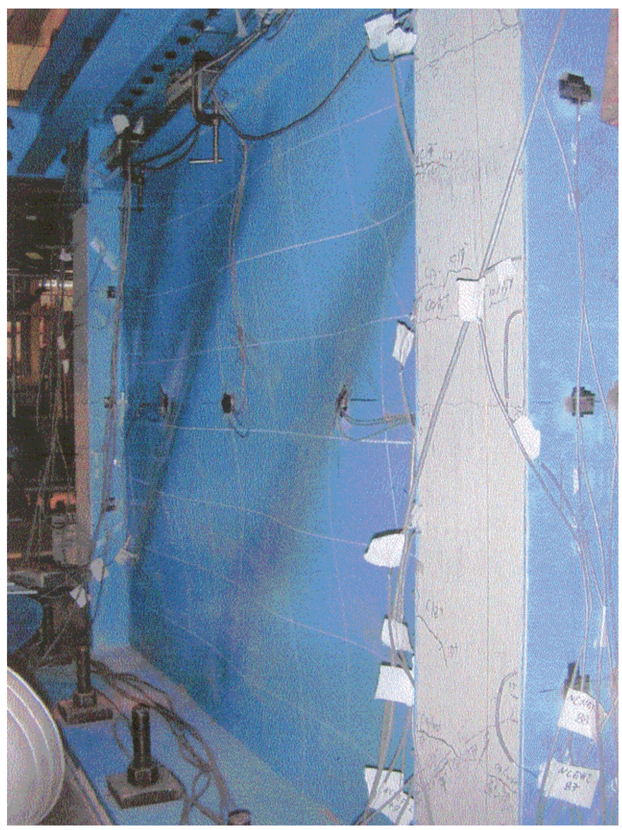

Fig. 10. Shear wall with partially composite columns (Courtesy of Professor Robert G. Driver).

Large scale tubular truss beams have been tested under constant amplitude fatigue loading. The trusses were 30 ft (9 m) long and 6 ft 8 in. (2 m) high, as shown in Figure 12. The truss members were circular hollow sections (CHS) in S355 steel (50 ksi yield stress). The truss itself was planar only, and the connections to be tested were all of the K-configuration. The primary aim was to evaluate the fatigue behavior of compression-loaded joints; specifically, where the chord is in compression, one diagonal is in compression and one diagonal is in tension.

As found in other studies, fatigue cracks developed in the compression joints as a result of the high tensile residual stress that was produced by the welding of the connection. The cracks started at the weld toe and their propagation was monitored by checking the alternating current potential drop. The residual stress measurements were made by hole drilling as well as neutron diffraction. Additional observations have been made regarding the crack initiation location, the crack propagation speed, and the formation of additional (secondary) cracks. As expected, the final failure took place when the tension member cracked. Further tests and analyses are being conducted, with the aim of providing improved fatigue design criteria and fatigue life predictions (Acevedo and Nussbaumer, 2009).

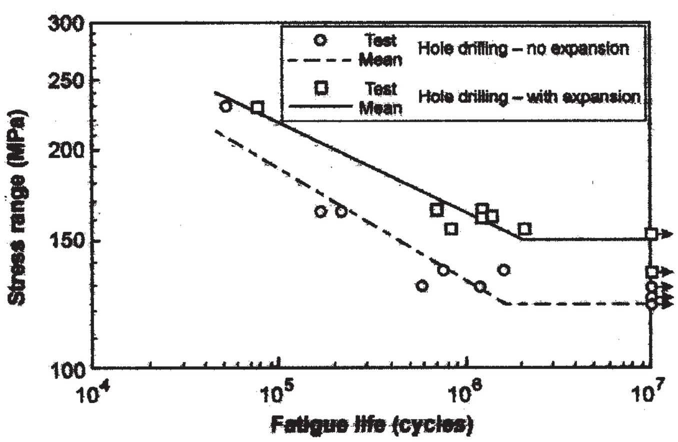

Fig. 11. Fatigue life test data for hole drilling with and without hole expansion (Courtesy of Professor Gilbert Y. Grondin).

UNIVERSITY OF LJUBLJANA

Bending-Shear Interaction in Plate Girders: This project has been conducted at the University of Ljubljana in Ljubljana, Slovenia, with Professor Darko Beg as the director.

Studies have shown that the interaction between bending and shear may not be significant. As a result, the design criteria of Eurocode 3 (EC3) appear to be very conservative (CEN, 2005). The key question is whether it is correct to use the same moment-shear interaction approach for longitudinally stiffened and unstiffened plate girders. A major parametric study was conducted to arrive at a realistic approach. At this stage it has been determined that the interaction is negligible for girders with high web slenderness; the opposite is correct for girders with low slenderness. Further, the EC3 criteria are shown to be safe for high slenderness girders; the opposite is true for girders with low slenderness. Figure 13 demonstrate these observations, with the left portion showing the results for girders with a web thickness of 6 mm (¼ in.) and the right portion for girders with a web thickness of 10 mm (⅜ in.). All of these girders had a web slenderness of 1,500. The flange *b/t* value was 300 with a thickness of 20 mm (¾ in.). The vertical stiffener spacing was 1,500 mm (5 ft). There was one longitudinal stiffener.

An extensive investigation is currently under way to determine the importance of web imperfections. The initial results show very limited influence in the context of moment-shear interaction.

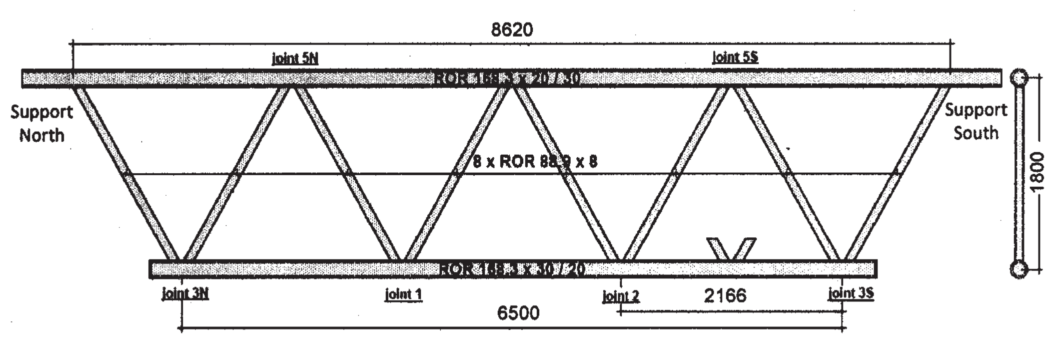

Fig. 12. Planar truss with circular hollow section members for fatigue testing of K-joints (Courtesy of Professor Alain Nussbaumer).

REFERENCES

Acevedo, C. and Nussbaumer, A. (2009), "Study on Crack Propagation in Tubular Joints under Compressive Fatigue Loadings," *Fatigue Design 2009*, Cetim, Senlis, France.

CEN (Comité Européen de Normalisation) (2005a), *Eurocode 3—Design of Steel Structures—EN 1993-1*, CEN, Brussels, Belgium.

Chicoine, T., Tremblay, R., Massicotte, B., Ricles, J.M. and Lu, L.-W. (2002), "Behavior and Strength of Partially-Encased Composite Columns with Built-Up Shapes," *Journal of Structural Engineering*, ASCE, Vol. 128, No. 3, pp. 279–288.

Dong, J. and Sause, R. (2009), "Flexural Strength of Tubular Flange Girders," *Journal of Constructional Steel Research*, Vol. 65, No. 3, pp. 622–630.

Dong, J. and Sause, R. (2010), "Finite Element Analysis of Curved Tubular Flange Girders," *Engineering Structures*, Vol. 32, No. 1, pp. 319–327.

Josi, Georg (2010), "Reliability-Based Management of Fatigue Failures," Ph.D. dissertation, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Prickett, B.S. and Driver, R.G. (2006), "Behaviour of Partially Encased Composite Columns Made with High Performance Concrete," *Structural Engineering Report No. 262,* Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Ricles, J.M., Mao, C., Lu, L.-W. and Fisher, J.W. (2003), "Ductile Details for Welded Unreinforced Moment Connections Subject to Inelastic Cyclic Loading," *Engineering Structures*, Vol. 25, pp. 667–680.

Roke, D., Sause, R., Ricles, J.M. and Gonner, N. (2009), "Damage-Free Seismic-Resistant Self-Centering Steel Concentrically Braced Frames," in *STESSA 2009, 6th International Conference on Behavior of Steel Structures in Seismic Areas*, Philadelphia, Pennsylvania, August, pp. 3–10.

Sause, R., Ricles, J.M., Roke, D., Seo, C.-Y. and Lee, K.-S. (2006), "Self-Centering Seismic-Resistant Steel Concentrically Braced Frames," in *STESSA 2006, 5th International Conference on Behavior of Steel Structures in Seismic Areas*, Yokohama, Japan, August, pp. 85–90.

Zhang, X. and Ricles, J.M. (2006a), "Experimental Evaluation of Reduced Beam Section Connections to Deep Columns," *Journal of Structural Engineering*, ASCE, Vol. 132, No. 3, pp. 346–357.

Zhang, X. and Ricles, J.M. (2006b), "Seismic Behavior of Reduced Beam Section Moment Connections to Deep Columns," *Journal of Structural Engineering*, ASCE, Vol. 132, No. 3, pp. 358–367.

ACKNOWLEDGMENTS

Assistance has been provided by ISSRA members Professor Darko Beg, University of Ljubljana in Ljubljana, Slovenia and Professor Alain Nussbaumer, Swiss Federal Institute of Technology in Lausanne, Switzerland.

Significant special assistance has been extended by Professors Richard Sause and James M. Ricles, Lehigh University, Bethlehem, Pennsylvania, and Professors Robert G. Driver and Gilbert Y. Grondin, University of Alberta, Edmonton, Alberta, Canada.

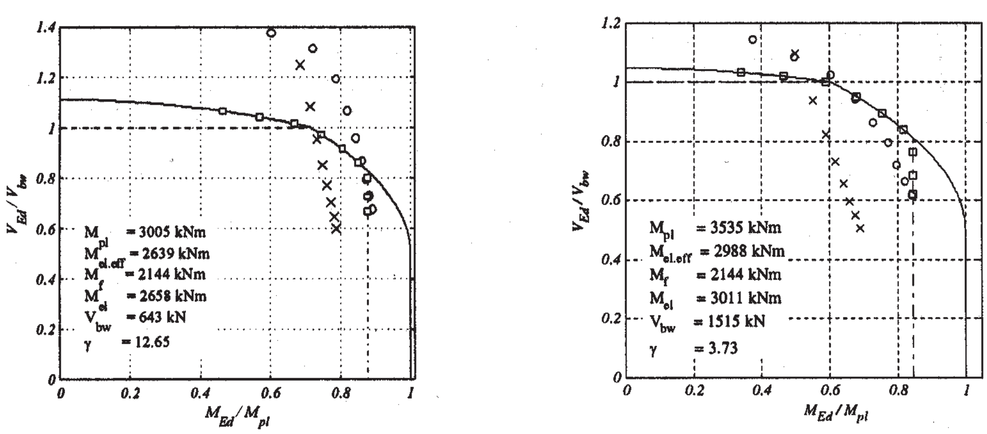

Fig. 13. Moment-shear interaction data for longitudinally stiffened plate girders (Courtesy of Professor Darko Beg).