Current Steel Structures Research

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The ultimate use of the results of structural engineering research work should be their incorporation in some form, in design standards or building codes. This is often regarded as the highest level of peer recognition, since the groups that decide what goes into such essential documents are composed of outstanding researchers and practicing engineers. Since strength and performance of structures should not be expected to vary from one locale to another throughout the world, it might also be anticipated that design codes could be very similar. Unfortunately this is rarely so, for a number of reasons. Basic philosophies may vary, and fundamental attitudes toward risk and safety have profound effects on contemporary thinking and the evolution of what is perceived as practical and desirable. Recent developments may reflect state-of-the-art approaches, but to gain access and incorporation into what the profession regards as the preferred solutions takes a great deal of time and extensive evaluations. This is as it should be, because the codes and standards have to be beyond the vagaries of professional politics. As such, the design requirements reflect proven records of safety and performance and economy.

The issues mentioned in the previous paragraph are particularly thorny for the engineers who venture into the international arena because their designs have to satisfy local requirements and preferences. Familiarity with one country's codes and regulations is difficult enough, and aiming to practice in vastly different locales demands skills that go far beyond common engineering prowess. Yet there are numerous companies, some of which occupy substantial staffs and capabilities, that practice in many areas of the world. Successful projects bring novel techniques to the firms and their home offices, which positions them uniquely for preferential treatment in many regions. This has made for a changing business climate in all respects, and the global perspective is essential for such assignments. Familiarity with international codes is critical, and at the same time it must be recognized that the bottom line is really that numerical results of designs will almost always be similar, regardless of the standard that has been used. Thus, designs based on the American or the European codes, to mention two of the most prominent sets of design criteria that adhere to similar philosophies, will usually turn out to be close. This is in spite of the significant differences in their approaches to tolerances and mathematical apparatus (Bjorhovde, 2006).

The strength and behavior of connections under monotonic and cyclic loads are addressed in one of the projects presented in this paper. One of the interesting features of this study is the hybrid nature of the connections, whereby elements of different steel grades are used in the connections, to achieve improved performance as well as economy. Such thinking has also gone into examinations of the behavior of tubular (HSS) members in structures, for which two major projects have recently been initiated in Europe. Structural stability and the assessment of first- and second-order effects are examined in a project that aims at improving how these parameters are treated, using an energy-based solution. Last, but not least, a major research effort has focused on how to extend service life and performance for a range of different types of welded joints and structures.

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 many cases reports or publications have not yet been prepared for public dissemination.

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MEMBERS AND CONNECTIONS

Performance of Dual-Steel Connections of High Strength Components under Monotonic and Cyclic Loading: This study has been conducted at the Polytechnic University of Timisoara in Timisoara, Romania, with Professor Dan Dubina as the director.

As defined by the researchers, a dual-steel structure is for all practical purposes a hybrid assembly, where mild- to medium-strength steel is used for the elements that are intended to provide energy dissipation through ductility, stiffness and strength. Higher-strength steel is used for the elements that are to provide stability and strength following the force redistribution that will take place along with the formation of plastic hinges in the structure. In this context the grades of steel that will provide for the energy dissipation through ductility are typically 36 and 50 ksi (235 and 350 MPa) yield stress materials; the grades of steel to be used for the post-plastic hinge structural elements are required to have a yield stress of 65 to 100 ksi (460 to 700 MPa) (Dubina et al., 2007). The very high strength of these components is chosen to ensure that they will remain elastic during the entire structural response. The dual-steel principles have also been extended to connections, considering ductile and brittle detailing (Dubina et al., 2008).

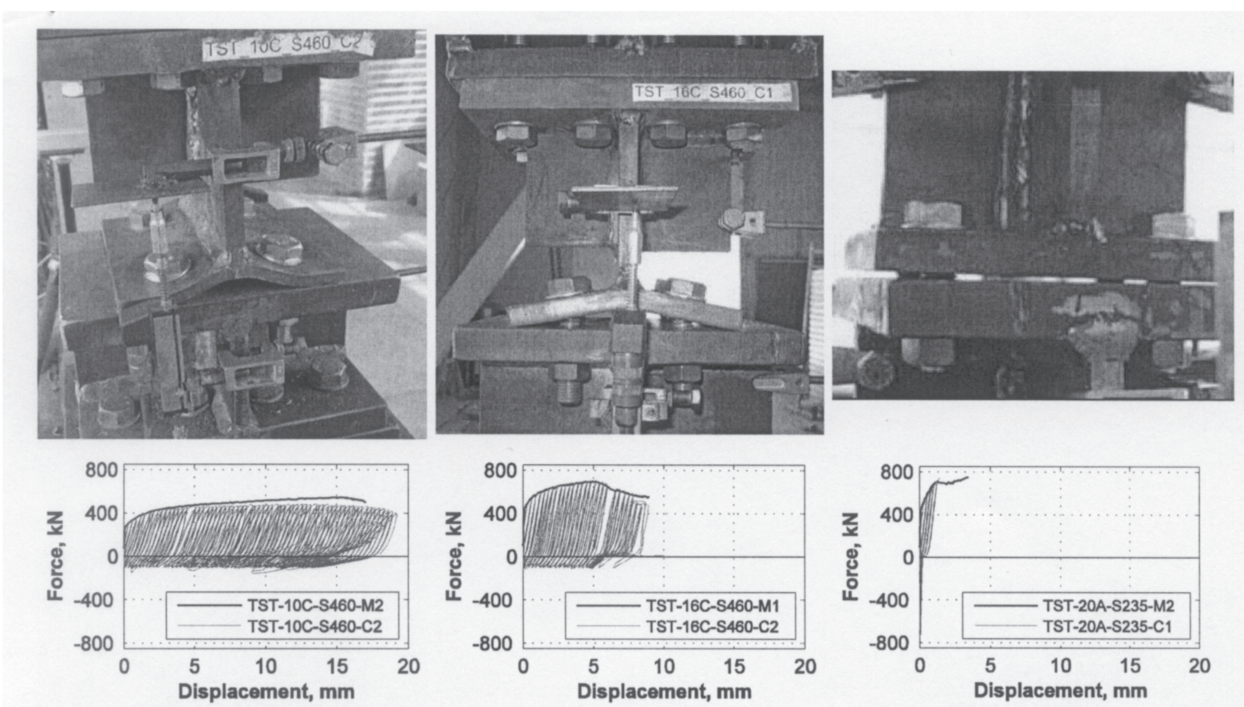
Reflecting design and fabrication preferences in Romania, a substantial testing program for welded built-up T-stub connections was developed. As will be seen, the end-plate thicknesses in particular are much smaller than what is commonly used in the United States for moment connections; end-plate shear connections are almost never used in American practice. Using test specimens modeling end-plate moment connections with a ⅝-in. (15-mm) beam web in 36-ksi steel and end-plates thicknesses of 5⁄16, ⅜, ½, ⅝ and ¾ in. (8, 10, 12, 16 and 20 mm) in 65- and 100-ksi steel, a total of 18 different connections were developed. The end plates were bolted to the test frame, using ¾-in. Grade 8.8 bolts (same as ASTM A325 bolts). Six of the specimen types had two-sided web stiffeners (Type A), another six had one-sided stiffeners only (Type B), and the final six had no web stiffeners (Type C). Three test specimens of each type were made, one to be tested monotonically and two to be tested cyclically. The project thus included a total of 54 specimens.

Part of the aim of the testing program was to verify the design models of Eurocode 3 (CEN, 2005a) and Eurocode 8 (CEN, 2005b), including the detailed connection design requirements as well as the fatigue criteria. Fatigue was considered in combination with the high cyclic tests that were performed.

Figure 1 shows the appearance of two of the Type C (with 65-ksi end plates) [parts a (⅜-in. end plate) and b (⅝-in. end-plate)] and one of the Type A (with a ¾-in., 36-ksi end plate) connections, along with the monotonic and cyclic load-deformation diagrams. The monotonic curves mirror the hysteresis curves of the cyclic tests. It is also noted that full reversal of the load from tension to compression was not feasible for the unstiffened (Type C) and the one-sided-stiffener (Type B) configurations, due to local buckling considerations.

In general, the specimens exhibited adequate ductility, and the limit states predicted by the component model of Eurocode 3, Part 1-8, were observed for all of the tests, including the cyclic ones. It is interesting to note that the specimens with the thickest end plates (¾-in.) did not display as high ductility as the other specimens, even for the tests that had 36-ksi end plates. Finally, the fatigue performance of all of the connections was in agreement with requirements of Eurocode 3, Part 1-9.

Performance-Based Approaches for High Strength Tubular Columns and Connections Subjected to Seismic and Fire Loads: This project is a three-year (2008–2011) joint effort among the University of Liège in Belgium, the University of Trento in Italy and the University of Thessaly in Greece. The industrial partners are Centro Sviluppo Materiall and Stahlbau Pichler from Italy. Funding is provided



(a) (b) (c)

Fig. 1. Dual-steel end-plate connection tests (courtesy of Professor Dan Dubina).

by the Research Fund for Coal and Steel of the European Commission. Professor Jean-Pierre Jaspart of the University of Liège is the project coordinator.

The project aims at developing capacity design criteria for HSS members and structures as is commonly done in Europe for other types of cross-sections for seismic design. The work will also extend to performance under fire conditions. Full-scale fire tests will be conducted for circular tubes (CHS), along with column tests, column base tests and wide-flange beam-to-tubular column connection tests. Since the project got under way in late summer 2008, no results are available at this time. However, the researchers and their industrial partners feel that the construction industry will benefit significantly once complete data and coherent methods of analysis have been developed.

Design and Integrity Assessment of High Strength Tubular Structures for Extreme Loads: This project is another three-year (2008–2011) joint effort among the University of Liège in Belgium, the University of Trento in Italy and the University of Thessaly in Greece. The industrial partners are Centro Sviluppo Materiall, Stahlbau Pichler and Instituto de Soldarua e Qualidade (Institute for Welding and Quality) from Italy; Fundacion ITMA from Spain; and Korrosions och Metallforskningsinstitutet (Institute for Corrosion and Metal Research) from Sweden. Funding is provided by the Research Fund for Coal and Steel of the European Commission. Professor Jean-Pierre Jaspart of the University of Liège is the project coordinator.

The project is obviously tied very closely to the other HSS study, just discussed. Some specific tests to be conducted are tension-loaded circular bolted flange connections for CHS. Simple design rules for such joints will be developed, along with suitable software.

ANALYSIS OF STEEL STRUCTURES

Imperfections for Global Analysis of Frames: EC3 Drawbacks and Energy-Based Procedure: This is a collaborative research project involving the University of Navarra in Pamplona and the University of Cantabria in Santander, both in Spain. Professor Eduardo Bayo of the University of Navarra is the project director and lead researcher.

Just like the 2005 AISC *Specification* (AISC, 2005) and soon to appear 2010 AISC *Specification,* through the use of the direct analysis method, Eurocode 3 (EC3) (CEN, 2005) promotes the use of direct global analysis of steel frames for stability and strength in addition to the various traditional approaches. Although the 2005 AISC *Specification* gives the advanced procedure in Appendix 7, it is anticipated that this approach will be the preferred solution for future applications (to be given in Chapter C of the 2010 edition). In EC3, the advanced method requires that local as well as global imperfections be defined to allow for stability checks without going to individual member evaluations. EC3 provides for two such methods, as follows:

Method 1: The local and global imperfections are uncoupled, and the global (frame) imperfection is defined as an amount of initial sway and the local imperfection is defined as an initial bow.

Method 2: The local and global imperfections are derived on the basis of the shape of the elastic buckling mode of the structure, using a scaling technique that takes into account the relative stiffness of the frame.

The researchers have examined the magnitudes of the sway and bow that result from using the two methods for typical plane frames, and the findings are very different—"shockingly diverse," as observed by the authors (Serna et al., 2009). As an illustration, Figure 2 shows an elementary

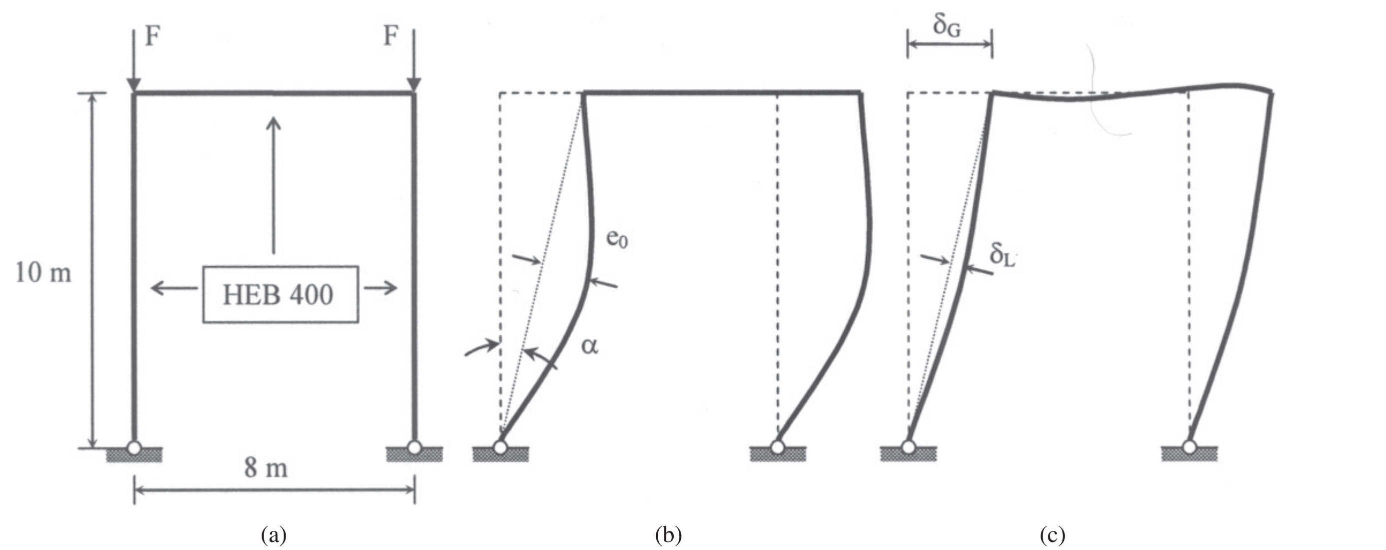


Fig. 2. Frame imperfections for Eurocode 3 Methods 1 and 2 (courtesy of Professor Eduardo Bayo).

one-bay, one-story rigid frame subjected to equal gravity loads at the top of the columns. Part b of the figure shows the imperfections that have been determined by Method 1, with an initial sway imperfection of *h*/300, corresponding to a displacement at the top of the column of 10/300 = 0.033. EC3 requires that the initial bow must be taken as *L*/300, where *L* is the column length, which corresponds to column buckling curve “a” in Figure 3 and an elastic analysis. (Note that Eurocode 3 uses five column curves [“multiple column curves”], designated as curves a_0, a, b, c and d. Figure 3 shows these column curves, along with the AISC column curve (the heavy line curve in the figure) as it was developed for the Specification (Bjorhovde, 2006). Resistance factors are not incorporated with the column curves shown in Figure 3.)

Consequently, for Method 1 the ratio between the maximum value of the bow and the displacement at the top of the column is 1.0. For the results of the analysis using Method 2 (i.e., the buckled frame shape shown in part c of Figure 2), the ratio between the two imperfections is 0.15, and this is independent of the absolute values of the imperfections. In addition to these discrepancies between the two methods, the researchers also note that it is very difficult to apply either of the two methods to three-dimensional frames. For Method 1, for example, EC3 does not indicate the appropriate direction of the imperfections that is necessary to arrive at the governing solution.

A new, energy-based procedure has been developed by the researchers; it is currently undergoing additional evaluations and testing (Serna et al., 2009). The procedure uncouples the imperfections. The global (frame) imperfection is defined on the basis of the sway buckling mode of the frame, using a scaling factor that is related to the relative global slenderness. This allows for the definition of the locations of the joints of the frame, and the local (member) imperfections then follow the non-sway buckling mode of the frame with a scaling factor that is related to this mode. The energy-based procedure will be further tested and evaluated for use with the design code.

FATIGUE-PERFORMANCE OF WELDED STRUCTURES

Extension of Service Life of Existing and New Welded Steel Structures: Known as the REFRESH project, this is a very large, 3½-year study focusing on the development of criteria and methods to increase the service life of fatigue-loaded welded structures. Emphasizing high-frequency post-weld improvement techniques for existing structures *in situ* as well as structures being designed and fabricated, the overall aim is to arrive at methods that are amenable to a dependable quality control system that can be interfaced with the design codes. Funded by the Federal Ministry of Education and Research in Germany, a number of universities and research organizations have been involved in the recently (June 2009) completed study. Overall management and direction of the project has been provided by the Production and Manufacturing Technologies Division of the Research Center of the University of Karlsruhe.

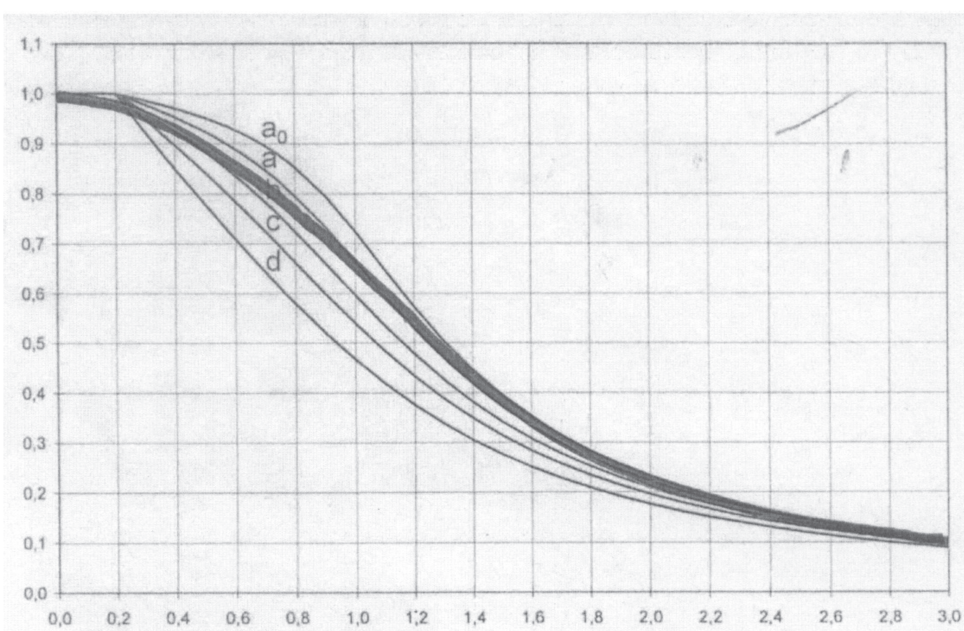


Fig. 3. Eurocode 3 column curves and the AISC column curve (courtesy of Reidar Bjorhovde [2006]).

The primary focus of the project is to examine the efficiency of high-frequency peening methods for the improvement of the fatigue resistance of welds and welded structures. Specifically, the two methods that have been evaluated in detail are high-frequency impact treatment (HiFIT) and ultrasonic impact treatment (UIT). Both methods focus on the use of hardened steel pins that hammer the weld toe with a frequency of 150 to 200 Hz, deforming the material plastically. This reduces or eliminates the notch effect, hence increasing the fatigue resistance of the weld, in spite of the increased surface hardness of the weld due to the peening.

The HiFIT-tool uses a single pin with a diameter of 3 to 4 mm (approximately 0.1 to 0.15 in.); the choice of diameter depends on the weld local geometry and the hardness of the toe. The UIT-tool uses an ultrasonic converter to develop the hammering effect, with three or four pins hammering the weld toe simultaneously. The pin diameter is 3 to 5 mm (0.1 to 0.2 in.), depending on the application. Figures 4 and 5 show close-ups of the equipment and the appearance of the weld toes after the peening has been completed.

Quality control methods are a major part of the project, pre- and post-peening, to assess weld hardness and residual stress levels. Hardness is measured by usual techniques; residual stresses are determined by hole-drilling or X-ray diffraction methods.

A broad range of fatigue tests have been conducted to verify the efficiency of the peening methods. The test parameters include steel material properties, plate thickness, specimen scale and notch details (butt welds and fillet welds), as well as transverse and longitudinal stiffeners. Fatigue crack location and behavior after initiation are monitored very carefully. The fatigue test results are currently being evaluated, but the initial findings show that the fatigue strength for butt welds and longitudinal stiffeners with S690 steel (for all practical purposes, the same as ASTM A514) can be doubled. The results are similar for both types of treatment. The cracks that developed following the HiFIT peening primarily took place in the base metal; these results are being further examined. For fillet-welded stiffeners in S690 steel the crack initiation moved from the weld toe to the root; for such cases the fatigue could also be doubled.

Since the REFRESH project was completed very recently (June 2009), detailed research reports and technical papers are currently not available.

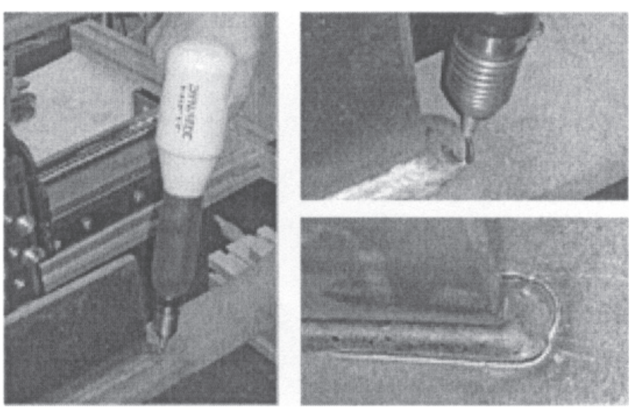


Fig. 4. Peening by high-frequency impact treatment (courtesy of Professor Alain Nussbaumer).

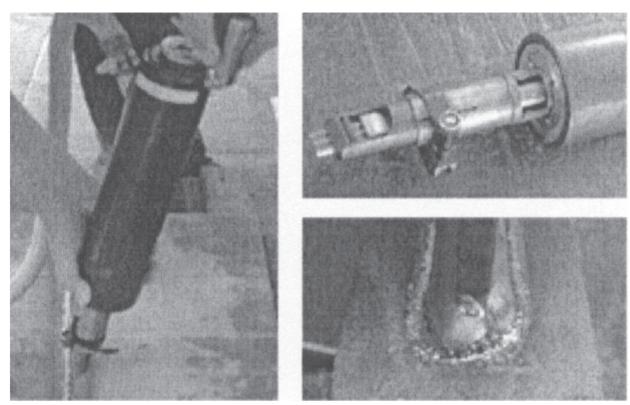


Fig. 5. Peening by ultrasonic impact treatment (courtesy of Professor Alain Nussbaumer).

REFERENCES

American Institute of Steel Construction (AISC) (2005), *Specification for Structural Steel Buildings*, AISC, Chicago, IL.

Bjorhovde, Reidar (2006), "International Steel Design Codes: Commonalities and Differences," in *Steel—A New and Traditional Material for Building*, Dan Dubina and Viorel Ungureanu, Eds., Taylor & Francis Group, London, U.K., pp. 3–13.

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

CEN (Comité Européen de Normalisation) (2005b), *Eurocode 8—Design Provisions for Earthquake Resistance of Structures—EN 1998-1*, CEN, Brussels, Belgium.

Dubina, D. and Dinu, F. (2007), "High Strength Steel for Seismic Resistant Building Frames," *Proceedings, Sixth International Conference on Steel and Aluminium Structures* (Ed. R. G. Beale), Oxford Brooks University, Oxford, U.K., pp. 133–140.

Dubina, D., Stratan, A., Muntean, N. and Grecea, D. (2008), "Dual-Steel T-Stub Behavior under Monotonic and Cyclic Loading," in *Connections in Steel Structures VI*, R. Bjorhovde, F. S. K. Bijlaard and L. F. Geschwindner, Eds., AISC, Chicago, IL, pp. 185–196.

Serna, M. A., Bayo, E. and Ibañez, J. R. (2009), "Imperfections for Global Analysis of Frames: EC3 Drawbacks and Energy Based Procedure," 7th EUROMECH Solid Mechanics Conference, Lisbon, Portugal, September 7–11.

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