Current Steel Structures Research

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Structural steel research in American academic institutions and laboratories has been very active for many years. Considering the number of projects that have recently been completed or currently are under way in the United States, it has been decided to add information about ongoing work in the United States to the series of papers that address current steel structures research. This article therefore includes results from two American projects, along with a number of international efforts. Depending on the subject matter, some of the findings may be applied very quickly to actual construction projects, and other results will take some time before entering the design and fabrication realm. This is especially the case for criteria that are intended for the steel design Specification.

As has been the case for many years in all areas of the world, behavior and strength of connections continue to be among the most active research areas. So is composite construction, for buildings as well as bridges, and some of these studies are expanding the current knowledge base as well as design considerations very significantly. New structural elements and forms have especially been considered by European bridge designers, including the use of stainless steels for certain applications.

It is also important to note that higher strength steels are becoming more common in practical applications, including 100 ksi (690 MPa) steels and certain very high strength bolts. Some studies in Europe in particular are examining expanded and realistic uses of such materials.

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

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CONNECTIONS

Beam-to-Column Connections with Beams of Unequal Height: This is a major research project that involves physical tests and analytical evaluations of two-sided beam-to-column connections, using two primary grades of steel for the members (Jordão, da Silva, and Simões, 2006). The work is being done at the University of Coimbra in Portugal, with Professor Luis da Silva as the director.

Beam-to-column connection design by state-of-the-art European practice is based on the so-called component method. This approach allows for a detailed assessment of the various limit states that govern connection strength and behavior (Weynand, Jaspart, and Steenhuis, 1996; CEN, 2005). The component method guided the researchers at the University of Coimbra in developing the test specimens and the loading protocol. The need for the study was based on the fact that the current Eurocode 3 only addresses equal beam depths for internal beam-to-column connections. The researchers have noted that the more complex stress states in the panel zone and different local limit states for unequal depth beams may necessitate revised criteria.

As shown schematically in Figure 1, a total of 13 full-scale connections have been planned, using the steel grades S355 (very similar to ASTM grades A992 and A572 Gr. 50) and S690 (very similar to ASTM A514). Two specimens from each group, for a total of eight, will use S690 steel; the remaining five specimens will have S355 steel. The steel grades for the beams are actually not important, since the specimens were designed with a focus on the panel zone. The column shape is therefore critical, with a web thickness of 10 mm (close to ⅜ in.). The shapes that are used for the specimens are European types, as follows.

Beam 1 (left) of Groups 2, 3, and 4: HE200B (Groups 2 and 4) and IPE 400 (Group 3)

Beam 2 (right) of all Groups: IPE 400

Column for all Groups: HE240B

The column length for all specimens is 3000 mm (10 ft). The single-sided (right-hand) beam elements of Group 1 are 1300 mm (approximately 50 in.) long, as are the right-hand elements of all the other specimens. The beam left-hand beam elements in the other groups are 708 mm (28 in.) for Groups 2 and 4 and 1300 mm (50 in.) for Group 3.

Some of the members are relatively shallow, at least by North American practice. Thus, the HE200B and HE240B sizes are 200 mm (8 in.) and 240 mm (9.5 in.) deep; the IPE 400 is 400 mm deep (16 in.). There are no exact counterparts among American wide-flange shapes. In the case of the S690 steel, for which there are no rolled shapes (as is also the case in the United States for A514 steel), welded built-up shapes of the same dimensions as the rolled shapes will be used.

Specimen E1.1 (see Figure 1) is unique in the test series, in that it only connects the beam to the column by flange welds. The beam web is not connected to the column at all. This specimen layout was specifically chosen to examine the behavior of the panel zone in the vicinity of the beam-to-column flange points. This is a reflection of the component method of design and how it assesses the effective lengths that are used to determine element buckling behavior.

The connections are fully welded (except E1.1, as noted above); although the researchers observe that the findings should be equally applicable to bolted connections. The loading protocols for the different specimens are indicated by the representative forces that are applied at the ends of the beams.

At this time, all of the five specimens using S355 steel have been tested, although the evaluations of limit states and other aspects of the tests are not yet available. But it is noted that specimens E1.1, E1.2 and E2.1 failed by web buckling. This is as would be expected. Additional test data will be forthcoming, including the eight specimens in S690 steel.

Design of Extended Shear Tabs: This study aimed at developing design criteria for extended shear tab connections, for use when simple connections are required to frame beams into the web of columns or into girders. It was conducted at the University of Wisconsin at Milwaukee, with Professors Donald Sherman and Al Ghorbanpoor as the project directors.

Figures 2 and 3 illustrate the types of connections that were examined. The unstiffened extended shear tab of Figure 2 offers the advantage of a minimum amount of welding, along the tab edge connected to the column or girder web

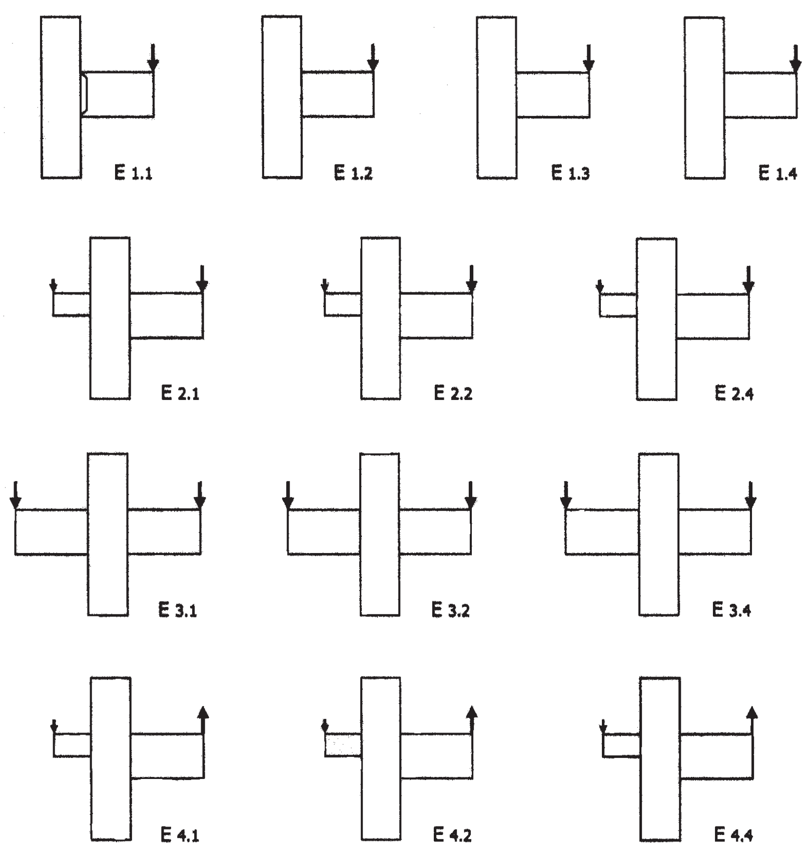


Fig. 1. Schematic illustration of connections and loading.

only. It also provides for easy erection of the beam in the structure. The stiffened extended shear tab of Figure 3 has the advantages of a significantly stiffer support configuration, through the use of stiffener plates that are welded to the column flanges, or by having the top of the shear tab welded to the top flange of the girder. Clearly, the ease of erection of the basic shear tab does not apply to the stiffened extended connection, and the amount of fabrication is more complex. However, there are numerous cases where the stiffened connection offers the only practicable solution.

The researchers ran a total of 27 full-scale tests, examining unstiffened and stiffened connections with three and five bolts in a single row, and subsequently with six and eight bolts. Weld configurations were as have been described above. In addition, four tests were performed to evaluate the potential influence of snug-tight bolts in slotted holes, as well as the use of a single stiffening plate at the top of the shear tab.

Although these connections offer significant practical options, no design criteria had been developed prior to this study. The work aimed at providing requirements for the strength and behavior of the connections, including the governing limit states and especially the limit states that are unique to the extended connections. Finally, the detailed design procedure that was prepared takes into account what is used for the common shear tab connections, and extends it for the unique conditions of the extended case. The procedure also recognizes that the stiffened extended shear tab is the preferred type of connection. The unstiffened alternative is not recommended.

SEISMIC RESPONSE OF CONNECTIONS

Evaluation of the Plastic Rotation Capacity of Beam-to-Column Connections: This project has been undertaken at Nagoya Institute of Technology in Nagoya, Japan, with Professor Tetsuro Ono as the director. Focusing on the need to assess rotation capacities of beam-to-column connections while considering the properties of the steel in different areas of the joint, the researchers have configured a finite element model of the beam and the connection areas, as shown in Figure 4. Expressions have been developed to incorporate local buckling as well as fracture, and the correlation with known performance data for a range of different connections has been found to be excellent. The solution also incorporates criteria that consider failure modes as functions of the width-to-thickness ratios of the flange and the web of the shape that is used for the beam.

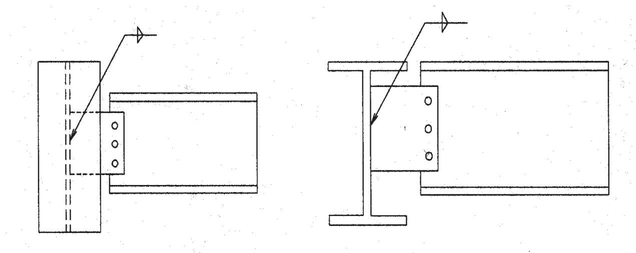


Fig. 2. Unstiffened extended shear tab connections.

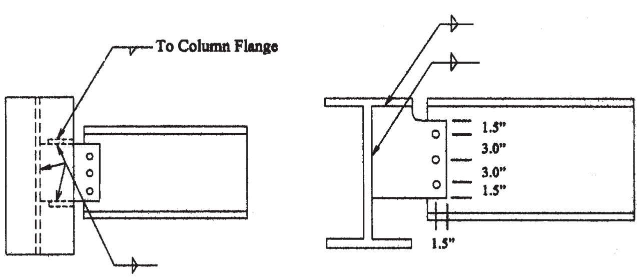
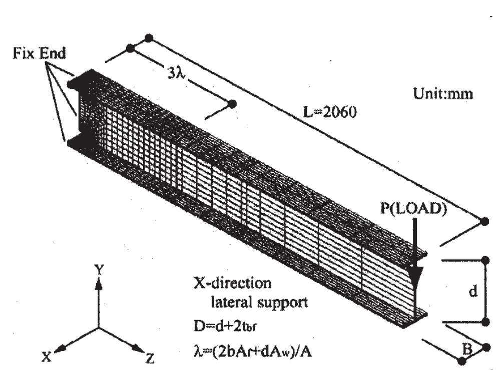
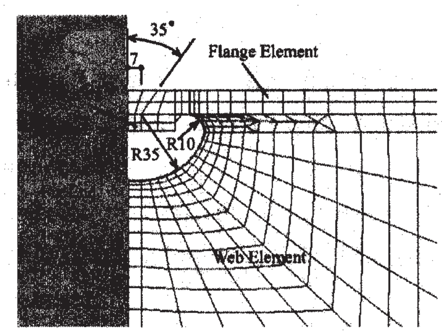


Fig. 3. Stiffened extended shear tab connections.



(a)



(b)

Fig. 4. Modeling of beam and connection detail for plastic rotation capacity.

This study was awarded the 2005 JSSC Prize of the Japanese Society for Steel Construction.

Experimental Behavior of Standardized End-Plate Connections under Arbitrary Cyclic Loads: This work is being conducted at the University of Coimbra in Coimbra, Portugal, with Professor Luis da Silva and Pedro Nogueiro as the directors (Nogueiro and da Silva, 2006). It is part of a larger scale European effort to develop standardized connections with known strength, stiffness, and rotation capacity characteristics. In some respects, this is comparable to the American development of prequalified connections for use in seismic design of structures.

A total of 13 specimens are to be tested, using steel grade S355 (50 ksi yield stress) for all. The specimens use 10-ft-long pinned-end columns with approximately 4-ft-long beams cantilevering from the connection region. Four connections will be tested in monotonic fashion; the other nine will be tested dynamically according to a loading protocol that specifies the number of elastic and inelastic cycles. Specifically, Regimen 1 used increasing amplitude cycles in the elastic range and constant amplitudes reflecting two times the yield rotation in the plastic range. These are small amplitudes. Regimen 2 is the same for the elastic range; in the plastic range the constant amplitudes reflected five times the yield rotation. Regimen 3 used random cycle amplitudes. These protocols are quite different from the ATC criteria used for cyclic testing in the United States.

A typical specimen is shown in Figure 5. The beam and column shapes are medium-size European sections, with HE320A or HE320B columns (approximately 13 in. deep) and IPE 360 (14 in.) or HE280A (11 in.) beams. These are relatively shallow, at least by American standards, but European steel structures tend to use much more slender elements than what is common in the United States. The end plate is fairly thin (18 mm ≈ 11⁄16 in.), but typical for such elements. The flange thickness of the HE320A is 5⁄8 in. (15.5 mm); the web thickness is 3⁄8 in. (9 mm). It is interesting to note that the bolts are 24 mm Grade 10.9, which effectively corresponds to 1 in. ASTM A490 bolts.

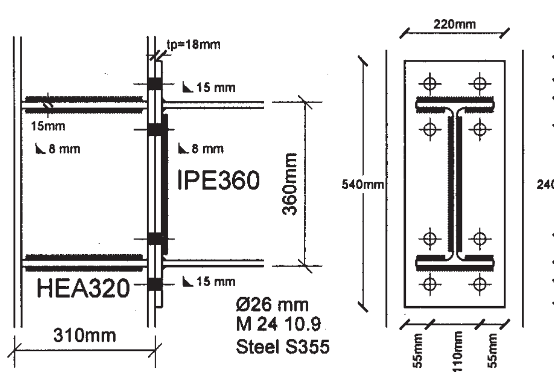


Fig. 5. Typical European standardized end-plate connection.

At this time, one monotonic and three cyclic tests have been completed. The monotonic test failed by fracture of the end plate in the interface between the weld and the base metal, at the top flange; the column flange did not fail. The cyclic tests failed in the same fashion, and these offered significant energy absorption by the panel zone in particular. This is as would be expected. The specimen tested by cyclic Regimen 1 failed after 82 cycles with a maximum rotation of 17 mrad. The large number of cycles is most likely due to the relatively small amplitude. Specimens 2 (cyclic Regimen 2) and 3 (random cycles, Regimen 3) failed after 22 and 28 cycles, respectively, with maximum rotations for both around 20 mrad.

Moment-Rotation Characteristics of Slender Portal Frame Connections: Portal frames are used a great deal in many areas, but maybe especially in various European countries. In the United States these structures are most often designed and fabricated as preengineered buildings. Nevertheless, these are very important structures for numerous applications, and research efforts on frame stability, girt and purlin design, roof systems and certain of the connection components of these frames are under way in several locales. The project described here has been carried out at the Technical University of Timisoara in Romania, under the leadership of Professor Dan Dubina.

Focusing on the use of Class 3 and 4 sections, as defined by Eurocode 3 (CEN, 2005), these are cross sections that are classified as noncompact and slender by American criteria. The most critical part of the portal frame insofar as moment characteristics are concerned is the corner joint, as shown in Figure 6. The details of the joint reflect current practice in Romania.

Seismic conditions in Romania require significant ductility and rotation capacity of most structural elements. In the case of slender components such as the shapes that are used for the portal frames, the issue becomes very complex. This is in part because of the slenderness of the members and in part because of the shape and plate thickness of the panel zone at the knee.

Physical tests were conducted first to determine the monotonic response of the knee connection, and cyclic loading tests were then performed to assess ductility and rotation capacity. Three different joint designs were examined, with two test specimens for each. One specimen was tested for monotonic load, the other for cyclic conditions. The steel grade was S275, with a specified minimum yield stress of 40 ksi (275 MPa).

The governing limit state for the monotonic tests was the same for all three specimens: buckling of the interior (lower) flange of the rafter, along with web buckling of the panel zone. The correlation with finite element analyses was very good. The limit states of the cyclic test specimens were similar: lower flange local buckling and shear buckling of the panel zone, along with distinct bending of the end plate that connects the rafter to the column. The hysteretic loops demonstrate a certain amount of "pinching," as would be expected for such slender components. The maximum rotation capacity was approximately 30 mrad.

FABRICATION ISSUES

Making Bolt Holes in the Fabrication of Structural Steel: Much research has dealt with bolts, bolt capacities, and bolt installation practices, but relatively little attention has been paid to the important fabrication issue of making the holes for the bolts. Current practices around the world include punching, subpunching and reaming, drilling, oxygen cutting, and plasma cutting. Preferences depend on the individual company and its equipment, but economy and efficiency in handling of the pieces in the shop are critical factors. In the United States, punching is generally the preferred approach, although material thickness is a major factor. Drilling is typically very accurate, but this requires more time. Oxygen and other thermal cutting practices can be acceptable, but this depends very much on the operator of the cutting equipment.

Professor James Swanson of the University of Cincinnati undertook studying the hole-making practices in the United States to assess strength and ductility of connections made by the different methods. The study also examined the impact of the microcracks that might form during punching and oxygen cutting, for example, as well as the potential for strain aging in the material after thermal operations.

Briefly, it was determined that there is no reduction of the connection or member strength when the holes are made by punching, punching and strain aging, and oxygen cutting. On the other hand, the ductility is significantly reduced for punched holes, although beams with such holes in the flanges were generally capable of reaching the fully plastic moment before fracture occurred. The study did not address the performance characteristics of beams with very large flange area reductions due to bolt holes, nor did it look at fatigue issues.

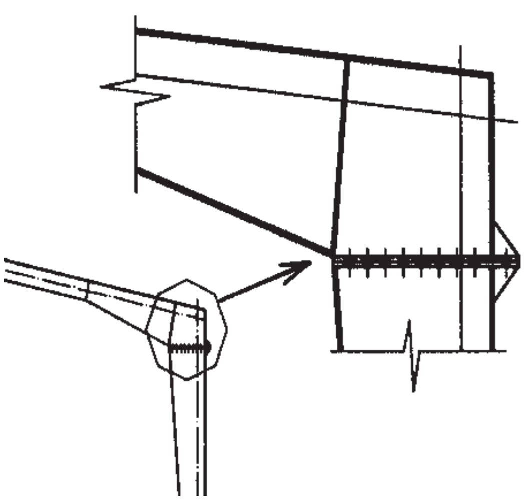


Fig. 6. Typical corner detail of portal frame.

COMPOSITE MEMBERS

Partially Encased Composite Columns with High Performance Concrete: This project addresses the strength and behavior of welded built-up columns with relatively thin plate elements, having concrete cast between the flanges of the shape. The study is a joint effort between the University of Alberta in Edmonton, Alberta, the École Polytechnique in Montreal, Quebec, and Le Groupe Canam, Boucherville, Quebec, Canada, with Professor Robert Driver of the University of Alberta as the director.

The thin plate elements of these built-up shapes require regularly spaced links between the flanges in order to prevent premature local plate buckling. The Canadian steel design standard CSA S16-01 incorporated requirements for this type of column in the 2001 edition but limited the strength of the concrete to 5.8 ksi (40 MPa) (CSA, 2001). In addition, the standard permitted the columns for use only as axially loaded members. The current project aims to provide performance data for axially and eccentrically loaded columns, as well as having high-performance concrete with strengths up to 8.7 ksi (60 MPa).

Eleven full-scale column tests were conducted, with 18 in. × 18 in. (450 mm × 450 mm) cross-section members in 50-ksi (CSA 350W grade)-yield stress steel. Two of the specimens had the lower strength (5.8 ksi) concrete; the other nine used the higher strength (8.7 ksi) material. Due to placement and vibration difficulties, the concrete was self-consolidating. Four of the columns were tested eccentrically. Finally, steel fiber reinforcement was used for the concrete of some of the axially loaded columns.

The results to date demonstrate that the columns with high-strength concrete behave like the members with the lower strength material, and further that the design criteria of the standard can be used, but they are very conservative. Interaction diagrams were developed to determine the capacities of the eccentrically loaded members, and the correlation between tests and theory is very good. Finally, the steel fibers had a moderate positive effect.

The project is now looking at the development of a suitable finite element model, to provide realistic assessments of the behavior and strength of the columns up to and past the maximum (peak) strength. The aim is to have such columns used in construction as parts of steel plate shear walls, for which much of the original development took place in Canada.

ANALYSIS OF PLATED STRUCTURES

Semi-Analytical Buckling Strength Analysis of Stiffened Plates: This multiyear project has been conducted at the University of Oslo in Oslo, Norway, with Professor Jostein Hellesland as the project director. The study also has involved close collaboration with Det Norske Veritas (DNV), Oslo, and the Norwegian University of Science and Technology (formerly the Norwegian Institute of Technology) in Trondheim, Norway.

The importance of offshore and ship structures and design approaches for plated structures provided the impetus for the investigation. Specifically, the traditional design equations are not amenable to irregular geometries and stiffener arrangements. Further, finite element analyses tend to be very complex and somewhat removed from the practical treatment of the problems.

On this background, so-called semi-analytical approaches have become common. They are much more efficient, computationally speaking, and therefore lend themselves to being incorporated into computerized design code formats. Such codes have become the preferred approach for ship and offshore structures, for example, and they are now part of DNV rules. The approach is suitable for extension to bridge girders, for example, as illustrated by the model shown in Figure 7. This represents the lowest buckling mode of a plate (for example, an orthotropic deck or a girder web) with two inclined stiffeners.

The project is now examining plates with arbitrary stiffener arrangements, plates with constant or stepwise variable thickness, and various interior and exterior edge restraint conditions. The implications for practice of this highly complicated problem are very significant.

BRIDGE STRUCTURES

Analysis and Design of Rehabilitated Built-Up Hybrid Compression Members: This study has been conducted at the University of Western Ontario in London, Ontario, Canada, with Professor F. Michael Bartlett as the project director.

The investigation was prompted by the fact that compression members in older bridge structures in Canada appear to be deficient, especially when analyzed in accordance with current requirements (Shek and Bartlett, 2006). This type of situation is not unusual, but for the slender compression members that were used extensively for certain bridge types before 1960 the issue is significant. Figure 8 shows column curves for bridge structures under the Canadian Standards Association bridge standards of 1952 (WSD = working stress design), the safety factor of 1.5 times the 1952 requirement, and the limit states (LSD = LRFD) curve from the 2000 bridge standard. The discrepancy is significant for slenderness ratios larger than approximately 70, for which the 1952 standard without the safety factor gives design strengths well above the limit states curve. The 1952 curve by itself lies

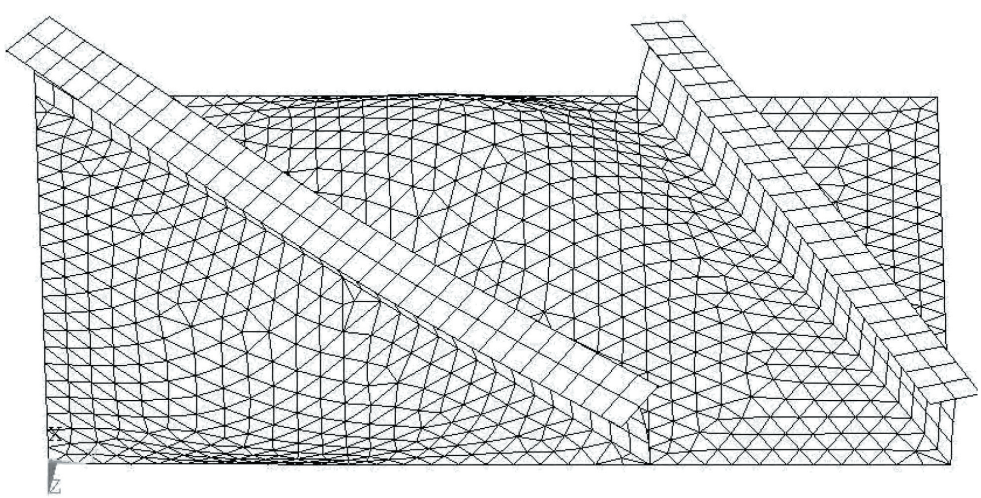


Fig. 7. First buckling mode of a plate with two inclined stiffeners.

above the 2000 LSD curve for slenderness ratios larger than about 130.

The compression members obviously would have to be strengthened, and the research team developed a hybrid model, as shown in Figure 9. Using cover plates that are welded to the members in the field, the analysis and design procedure would have to take into account such effects as locked-in dead load stresses. It would also have to account for the influence of residual stress, differences in material strength between the original shape steel [yield stress 33 ksi (228 MPa)] and the plate steel, and geometric imperfections.

A "refined numerical analysis method" (RNMA) was developed for the hybrid members, and a parametric study was conducted for a range of column and plate sizes. The accuracy was very good, and a simplified version of the RNMA was developed for practical design usage. It was found that

- The locked-in stresses do not cause a lowering of the buckling strength about either axis of the members for slenderness ratios less than 33 and more than 90. In between these limits, the strength decreases as the locked-in stresses increase, but the changes are not significant.

- The cover plates make the strong axis buckling strength insensitive to the level of residual stress in the W-shape. However, the weak axis strength is strongly influenced by the cover plates and the welding residual stresses, especially for narrow plates and for smaller (lighter) shapes.

- For maximum efficiency, the cover plates should be as wide as possible for the column and its flange width. This primarily affects the weak axis strength.

- For the preliminary design, the effects of the locked-in stresses can be ignored. Further, the capacities of the plates and the shape are computed separately and then added, to give a reasonable starting magnitude for the hybrid member strength.

- If the cover plates are at least ⅔ of the flange width, the slenderness of the shape and the plates can be based on the radius of gyration of the full hybrid member.

Use of Stainless Steel for Welded Bridges in Aggressive Environments: This project is a joint effort between the University of Liège in Liège, Belgium, the Technical University of Aachen in Aachen, Germany, the French steel company Industeel Creusot and two Italian engineering firms. Professor René Maquoi from Liège is the director.

The prime objective of the study is to assess whether it is possible and economically feasible to extend the life of bridges in aggressive environments through the use of certain high-strength grades of stainless steel. Specifically, these are the very high chromium, so-called duplex steels, with a chemistry designation as 22Cr5Ni3MoN. The nominal yield stress for the representative grade is 70 ksi (480 MPa).

Since the behavior of stainless steels differs significantly from the common (carbon-manganese and HSLA) structural steels, part of the project involves the development and definition of welding technologies and consumables, appropriate structural details, and assessment of test results in any form. Extensive corrosion and fatigue tests are planned, as are plate buckling tests for suitable structural elements. Finally, in addition to developing design criteria for incorporation into Eurocode 3, detailed life cycle cost

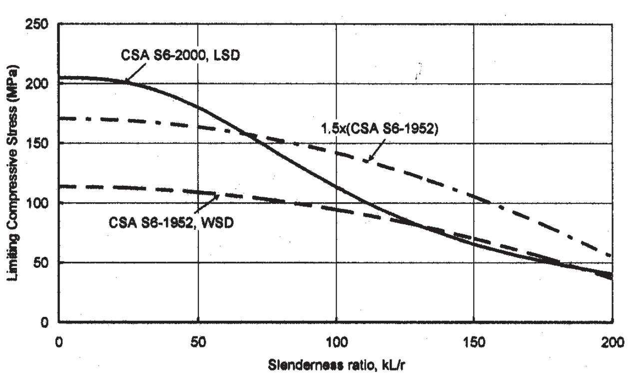


Fig. 8. Column curves from various Canadian bridge standards.

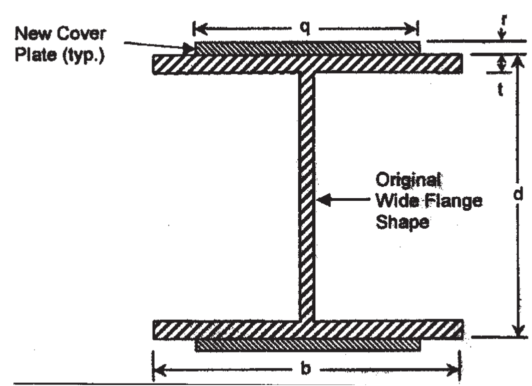


Fig. 9. Hybrid compression member.

analyses will be performed. In effect, the project aims at developing a new form of bridge technology that is suitable for very demanding service conditions.

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