COLD-FORMED STEEL RESEARCH AT THE UNIVERSITY OF WATERLOO

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Abstract

Active cold-formed steel research started in the early 70s at the University of Waterloo with Profs. N.C. Lind, A.N. Sherbourne, J. Roorda, and R.M. Schuster, and their respective graduate students. More recently, Prof. L. Xu and his graduate students have also contributed in this area of research. Numerous publications have been documented in National and International sources over the past 35 years. Presented in this paper are the many noteworthy contributions that have been made as a result of the cold-formed steel research activities at the University of Waterloo. What is of particular interest is the number of design recommendations that have been adopted by National and International cold-formed steel design Standards and Specifications. With the recently formed Canadian Cold-Formed Steel Research Group, cold-formed steel research is very much alive at the University of Waterloo.

Introduction

The use of cold-formed steel as a structural building material dates back to the 1850s in the USA and the UK. Initially, the acceptance of cold-formed steel in the building construction industry faced difficulties because no published Specification for the design of cold-formed steel structures existed at that time. For such a Specification to be published, research had to be carried out. In 1939, the American Iron and Steel Institute (AISI) sponsored a research project at Cornell University under the direction of Professor George Winter with the objective of developing the necessary technical information that would lead to the development of the first cold-formed steel design Specification, which was published by AISI in 1946. This design Specification has been revised subsequently by AISI a number of times to reflect the technical developments resulting from the continuing research in the field of cold-formed steel. The Canadian Standards Association (CSA), following a number of subsequent editions, published the first cold-formed steel design Standard, S136, in Canada in 1963.

In comparison to hot-rolled structural steel members, cold-formed steel is uniquely different in that structural panel/deck sections and individual profile members are made of rather thin-coated steel plate material. This allows for the production of a great variety of geometric shapes, as can be seen in Figure 1. Most of the cold-formed steel shapes are produced economically by using roll-forming operations. Cold-formed steel structural members have one of the highest strength to mass ratios of any structural building material. The use of cold-formed steel in the construction industry in North America is steadily growing and gaining momentum, including cladding, roofing, composite decking, lightweight steel framing, corrugated steel pipe, storage racks, transmission towers and numerous other applications. Due to the thinness of the steel plate material, cold-formed steel products are subject to local buckling, resulting in more detailed and complicated calculations for the structural designer. Ongoing research in this field is extremely important in order to update and improve the governing cold-formed steel structural design Specifications and Standards.

The 1970s

Cold-formed steel research started in the early 70s at the University of Waterloo with Prof. Lind as one of the leading Canadian researchers.

Lind and Schroff (1971, 1975) dealt with the utilization of cold work of forming in cold-formed steel. The mechanical properties of cold-formed sections can be substantially different from those of flat sheet steel before the cold forming operation. When a piece of flat sheet steel is bent about a radius, the yield strength and tensile strength will increase as a result of this forming operation, but at the same time decreasing the ductility. Lind and Schroff (1971, 1975) used a linear strain-hardening

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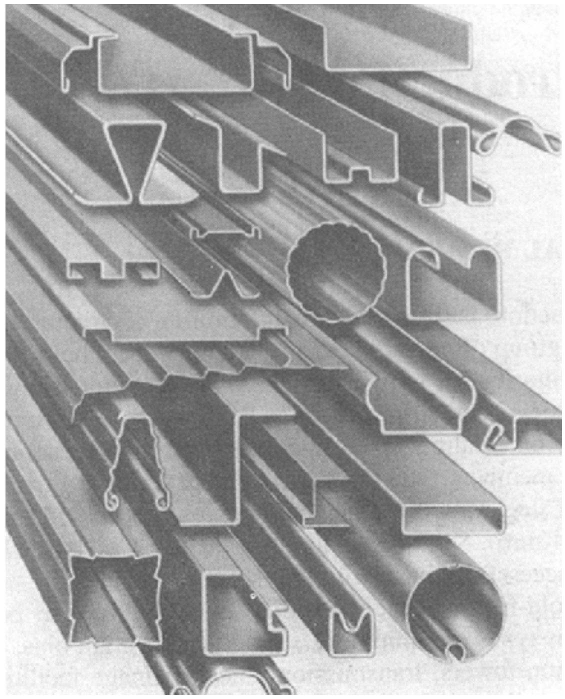

Fig. 1 Various Cold-Formed Steel Shapes (Yu 2000)

model and concluded that the increase in yield strength depends only on the inside bend radius ratio, r/t, and the hardening margin ($F_u - F_y$). Furthermore, to take the cold work strengthening into account, it is only necessary to replace the virgin yield strength by the virgin ultimate strength over a length of 5t in each 90° corner. The researchers concluded that the r/t ratio is not a significant parameter, which was established by examination of test data. Based on the work by Lind and Schroff (1971, 1975), the following simplified equation resulted:

$$F'_y = F_y + \frac{5D}{W^*}(F_u - F_y) \tag{1}$$

where F'_y is the calculated average tensile yield strength of the full cold-formed section of tension or compression members, or the full flange of flexural members. F_y and F_u are the tensile yield and tensile strengths of the virgin steel, respectively. D is the number of 90° corners – if other angles are used, D is the sum of the bend angles divided by 90°. W* is the ratio of the length of the centerline of the full flange of flexural members, or of the entire section of tension or compression members, to the steel thickness, t. Equation 1 was adopted by CSA Standard S136-74 – Cold Formed Steel Structural Members.

Another important work that was undertaken by Lind et al. (1971) and Lind et al. (1976) was investigating the effective width formula used at that time. Their study showed that, based on experimental evidence, the effective width formula for stiffened compression elements can be expressed independent of the flat width ratio, w/t, as follows.

$$b/t = 1.64\sqrt{\frac{E}{f_{max}}} \tag{2}$$

One can conclude that this expression is simpler and offers greater computational advantages, often eliminating the usual time-consuming iteration process with the existing effective width formula that is a function of w/t.

Venkataramaiah (July 1971) carried out an extensive experimental investigation on different edge stiffeners that can be used with cold-formed sections instead of the typical simple lip stiffener. The objective of his work was to obtain optimum shape and optimum size edge stiffeners for thin-walled compression elements. Another extensive experimental study was carried out by Venkataramaiah (August 1971), Lind and Venkataramaiah (1972) into the behaviour of straight-lipped and L-lipped simply-supported channel sections, eccentrically loaded, thin-walled short columns. It was concluded that the ultimate strength of eccentrically loaded thin-walled short channel columns can be expressed by a simple but general formula involving critical stress.

Schuster (1972) started his research at the University Waterloo by investigating composite steel-deck reinforced concrete slabs, in short, "composite slabs". In order for a steel deck to achieve the required composite action between the deck and concrete, the deck must be capable of resisting horizontal shear and prevent vertical separation between the concrete and deck. The most common of composite deck products in the marketplace today are decks that utilize a fixed pattern of embossments/indentations rolled into the deck, thus providing a mechanical interlocking device between concrete and deck so that the required composite action can be developed. These types of floor systems are typically used in high rise buildings because of their numerous inherent advantages, one of the major advantages being, the steel deck serves as a form for the concrete during the construction stage and it remains permanently in place as the positive reinforcement. Based on tests, "shear-bond" is the most common mode of failure with composite slab systems. Schuster (1972) developed the following shear-bond expression:

$$\frac{V_{uc}}{bd} = K_5 \frac{\sqrt{f_c'}d}{L'} + K_6\rho \tag{3}$$

where V_{uc} is the ultimate calculated transverse shear; b is the unit slab width; d is the effective slab depth; f_c' is the concrete compressive strength; L' is the shear span; ρ is the percent of steel; K_5 and K_6 are shear-bond coefficients that have to be obtained from a linear regression analysis of test data.

Lind (1973) investigated the buckling of multiple closely spaced intermediate longitudinal stiffened plate elements. When the stiffeners are spaced so closely that the compression sub-elements have no tendency to buckle individually, overall plate buckling of the entire element can occur. It was assumed in the 1963 edition of the CSA S136 Standard that the entire element was comprised of a rectangular plate having an equivalent steel thickness of t_s. While this was a simple design method, Lind (1973) developed a more comprehensive expression by treating the entire stiffened plate element as an orthotropic plate, resulting in the following equation:

$$t_s = t\left[\frac{w_s}{2\rho} + \left(\frac{3I_s}{\rho t^3}\right)^{1/2}\right]^{1/3} \tag{4}$$

where t is the plate thickness; w_s is the overall plate element width; I_s is the moment the inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis; p is the perimeter length of the multiple-stiffened element (between edge stiffeners).
Equation 4 is valid up to the initial buckling stress, however, it was demonstrated by Sherbourne et al. (1971), Sherbourne et al. (1972) that some post-buckling strength is available. Based on this, Eq. 4 is meant to also apply in the post-buckling range as an approximation, and it was adopted in the 1974 Edition of CSA S136.

In 1974, the Solid Mechanics Division published a special publication, entitled "Design in Cold Formed Steel", edited by Schuster. Contained in this document are a number of noteworthy papers dealing with the topic of cold-formed steel. Lind (Strength, Deformation and Design of Cold Formed Steel Structures), Lind (Design Procedures for Flexural Members), Lind (Recent Developments in

Cold Formed Steel Design Requirements), Lind (Additional Design Examples); Roorda (On the Buckling Behaviour and Design of Thin-Walled Beams and Columns); Schuster (Composite Steel-Deck Reinforced Concrete Floor Systems), Schuster (Current Design Criteria of Steel-Deck Reinforced Concrete Slabs), Schuster (Proposed Ultimate Strength Design Criteria for Steel-Deck Reinforced Concrete Slabs). Presented in this publication were important papers on cold-formed steel to date, providing structural engineers with the latest design information in cold-formed steel.

Schuster, in collaboration with Lind (1975) published the book, entitled, "Cold Formed Steel Design Manual", which was the first document of its kind in Canada, containing the 1974 Edition of CSA Standard S136 and its respective Commentary. The purpose of this book was to assist engineers, designers, manufacturers and educators in the design of cold-formed steel structures. Including in the book were numerous design examples, helpful design aids, and many useful tables.

Lind et al. (1975) investigated the economic benefits of the connection safety factor used in the CSA S136-74 Standard and the AISI – 1968 Specification. Using an analysis based on economic principles of equal marginal returns, they concluded that the current safety factors in CSA S136-74 and AISI-1968 are not far from the economic optimum.

Knab and Lind (1975) developed a rational, reliability based design criteria for determining allowable stresses for temporary (2–5 years) cold-formed steel buildings. The method involves comparing differences in reliability levels between permanent and temporary building design criteria. It was shown that allowable stresses can be established that take into account the temporary nature of the buildings and, at the same time, maintain comparable permanent building reliability levels.

Parimi and Lind (1976) the objective of this paper was to explain the new limit states design option in the CSA S136-74 Standard for cold-formed steel design. It was concluded that in the case of cold-formed steel design, the limit states cases could be reconstructed rather easily from the working stress design format. As well, it was found that the stress format was also quite suitable for the limit states design format of cold-formed steel, which allows for ease of calibrating the respective resistance factors. Probabilistic and deterministic approaches were used in the resistance factor calibrations, resulting in identical results.

Schuster (1976) presented a comprehensive overview of the state of the art of composite steel-deck concrete floor systems in the US and Canada, outlining the important inherent attributes of such floor systems in the construction industry. Also presented in the paper is an overview of the latest research findings to-date.

Lind (1978) carried out a study relating to the buckling strength of steel plate assemblies. The analysis is reduced to an eigenvalue problem of an ordinary differential equation that is solved by a Vianello-Stodola procedure, using the tabular numerical approach presented by Newmark for buckling analysis of columns. Two example problems are presented, illustrating the simplistic approach.

The 1980s

Schuster and Ling (1980) developed a new shear-bond expression for composite slabs on the basis of the mechanical interlocking capacity of concrete and steel deck within the shear span. Equation 5 is similar to *Eq. 3*, however, the strength of concrete and the percent of steel terms are no longer in the expression. Based on the available data, it was found that these two parameters are not important terms. The percent of steel parameter is not required since separate tests have to be carried out for each steel deck thickness anyway. See *Eq. 3* for the description of terms.

$$\frac{V_{uc}}{bd} = K_5 \frac{1}{L'} + K_6 \tag{5}$$

Equation 5 was adopted in 1984 by the Canadian Sheet Steel Building Institute (CSSBI) for the calculation of the shear-bond strength of composite slabs. A number of updated editions have since followed [Schuster and Trestain (2002)]. Schuster (1980) presented a paper on composite steel deck concrete floor slabs, highlighting the detailed development of *Eq. 5.*

Venkataramaiah et al (1980) did a study on the experimental determination and the statistical evaluation of the elastic modulus of elasticity of cold-formed sheet steel. In addition to some column compression data found in the literature, tensile coupon tests were carried out as part of this study. After a statistical study, it was found that the mean value of the elastic modulus of cold-formed steel is 30 071 ksi (207 332 MPa), which is greater than the value of 29 500 ksi (203 395 MPa) that is used by the AISI Specification and the CSA S136 Standard.

Wing and Schuster (1981, 1982, 1986) started to carry out research in the field of web crippling of cold-formed steel members, followed by numerous other studies over the years. This work dealt with web crippling and the interaction of bending and web crippling of multi-web cold formed-steel deck sections. One of the major reasons for this study was to correct some of the inconsistencies that existed in the two North American cold-formed steel design documents at that time. Since web crippling of cold-formed steel sections is an extremely complex analytical problem, experimental testing had to be carried out. Web crippling without the influence of bending is typically divided into the following four categories: 1) End one-flange loading (EOF), 2) Interior one-flange loading (IOF), 3) End two-flange loading (ETF), 4) Interior two-flange loading (ITF). Only with the IOF loading case was some moment present, however, the specimens were short enough so that the moment influence was kept to an absolute minimum. In the case of the interaction of bending and web crippling, the specimens were of larger length to account for different degrees of bending influence. The work by Wing on web crippling of multi-web cold formed-steel deck sections was adopted in the 1984 edition of CSA S136.

Schurter et al. (1982) carried out a study on three different cold-formed steel C-section type studs, i.e., one section had solid webs and the other two had lip-reinforced trapezoidal holes in the webs. The tests consisted of 1) Stub column tests and 2) Full scale wall assembly panel test that were subjected to compression load and combined compression and lateral load. The test results were compared to their respective calculated values, resulting in good correlations.

Seleim and Schuster (1982, 1985) developed a new shear-bond expression for composite slabs. The reason for this work was to reduce the number of composite slab tests required to establish the shear-bond capacity of any given composite floor system. Typically, four different steel deck thicknesses are produced by any given composite slab manufacturer. Using *Eq. 5* and having to test four composite slab specimens for each steel deck thickness [Schuster, Trestain (2002)], 16 composite slabs would have to be tested to establish the shear-bond capacity of any given composite floor system. This can be quite costly. The work by Saleim produced the following shear-bond expression, which has been adopted by the CSSBI [Schuster, Trestain (2003)]:

$$\frac{V_{uc}}{bd} = K_1 \frac{t}{L'} + K_2 \frac{1}{L'} + K_3 t + K_4 \tag{6}$$

where (t) is the steel deck thickness and K_1 to K_4 are shear-bond coefficients that have to be obtained from a multiple linear regression analysis of the test data. In this case, only eight slab specimens have to be carried out to establish the shear-bond capacity.

Schuster and Suleiman (1986) carried out a study on composite slabs subjected to repeated point loading. Composite slabs are sometimes subjected to repeated point loading, such as resulting from forklift trucks. In this study, experimental testing was carried out on single span and double span composite slabs, using only one particular company's composite slab product. To establish the ultimate capacity of the composite slabs, first static load tests were carried, followed by a number of respective repeated load tests subjected to cyclic loading. It was found that shear-bond was always the mode of failure, and the composite slabs were able to carry 75% of the static load up to 1.25 million cycles without failure.

McCuaig and Schuster (1988) did a similar study as Suleiman (1986), except the composite steel deck was from a different manufacturer. Based on these test results, the conclusions were that both simple and double span specimens were able to sustain repeated point loads of 55% of static ultimate for at least 1.25 million cycles and the mode of failure in all cases was by metal fatigue in the steel deck.

Schurter and Schuster (1986) were involved in investigating the shear-bond capacity of composite slabs relating to the surface conditions/coatings of the steel decks. Twenty pull-out tests involving four different metallic coatings and four different surface conditions were carried out. As well, six full-scale slab tests with two different metallic coatings (AZ150 Galvalume and Z275 galvanized) were performed. It was concluded that the shear-bond capacity of the AZ150 Galvalume steel slab was on average 32 % greater than the Z275 galvanized steel slab.

Schuster et al. (1986), Fox et al. (1986) and Schuster (1987, 1991) presented papers on the Canadian Standard for the Design of Cold-Formed Steel Structural Members, highlighting the new Limit States Design approach used in Canada.

The 1990s

McCuaig and Schuster (1990) performed resistance factor calibrations for the shear-bond failure mode of composite slabs. 196 test data, representing nine different product types, were used in the reliability calibrations based on a dead load factor of 1.25 and a live load factor of 1.5, as specified in the National Building Code of Canada. It was concluded that a resistance factor of 0.7 can be used for shear-bond of composite slabs, resulting in an average calculated safety index of 3.71, which corresponds well with the target safety index of 3.5.

Schuster (1991, 1992) presented a paper on lightweight steel framing in the 90s, focusing on the use of cold-formed steel in residential construction. In 1992, Schuster presented a paper, entitled "The 1989 Edition of the Canadian Cold Formed Steel Design Standard".

Dinovitzer et al. (1992) made some important observations pertaining to the CAN/CSA-S136-M89 cold-formed steel design Standard. In the case of partially stiffened compression elements, discontinuities in effective width estimates for sections with similar flanges and stiffeners were observed, resulting from a sudden change in the behavioural states. Recommendations are presented to rectify this discontinuity and at the same time simplifying the analysis procedure. These recommendations are now contained in the North American Design Specification. In addition, a simplification was presented for deflection determination of multi-web deck sections. A new plate buckling coefficient was developed to be used in the basic effective width expression.

Papazian et al. (1994) carried out an experimental investigation on multiple intermediate longitudinally stiffened deck profiles. The primary objective of this work was to substantiate the design approach contained in the CSA S136 Standard for closely spaced intermediate longitudinal stiffeners. It was the belief that this approach was quite conservative in that it only addressed the elastic plate buckling capacity. All tests were subjected to a uniform bending load using a vacuum chamber. It was concluded that indeed the ultimate capacities were conservative.

Schuster et al. (1995) undertook an investigation of perforated cold-formed steel C-Sections in shear. The reason for this work was because no specific design provisions were contained in the design Standards for perforated web elements subjected to shear, even though such sections are frequently found in practice. Experimental testing was carried out to substantiate the analytical formulation. The work resulted in a design approach that is now contained in the North American Design Specification.

Rogers and Schuster published four different papers in 1996, all resulting from the research by Rogers. 1) Effective Width of a Single Edge Stiffener Subjected to a Stress Gradient; 2) Interaction of Flange/Edge-Stiffened Cold Formed Steel C-Sections; 3) Cold Formed Steel Flat Width Ratio Limits, d/t and d_i/w; 4) Test Results and Comparison of Cold Formed Steel Edge Stiffened C and Z-sections. Every one of these papers resulted in meaningful design information.

Hancock et al. (1996) published a paper entitled, "Comparison of the Distortional Buckling Method for Flexural Members with Tests" and in 1997, Rogers and Schuster followed with a paper on the flange/web distortional buckling of cold-formed steel sections in bending.

Prabakaran and Schuster (1998) developed a new web crippling expression for cold-formed steel members that can be used for different geometric shapes and the four loading cases, and it is non-dimensional. The following expression for the nominal web crippling strength was adopted by the 94 edition of CSA S136 and the North American Specification for the Design of Cold-Formed Steel Structural Members:

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \quad (7)$$

where (t) is the web steel thickness; F_y is the design yield point of the steel; C, C_R, C_N and C_h are coefficients that have been established based on experimental data found in the literature. These coefficients are given in the North American Specification for the different geometric section types and load cases. This work by Prabakaran was actually completed in 1993.

Gerges and Schuster (1998) carried out an experimental investigation on the web crippling of single web cold-formed steel members subjected to end one-flange loading (EOF). The primary focus of this study was on single-web C-sections with large inside bend radius to thickness ratios. There was insufficient data in the literature to properly establish the web crippling coefficients. This study produced the additional data and the appropriate web crippling coefficients were established.

Acharya and Schuster (1998, 1998) undertook a study of hat-type sections with multiple intermediate longitudinal stiffeners subjected to uniform bending load. Similar to Papazian et al. (1994), additional tests were carried out in a vacuum chamber to complete the data pool for the establishment of a new plate buckling coefficient to be used with the basic effective width expression.

Fox and Schuster (1998) investigated the use of bearing stiffeners with C-section floor joists. Tests were carried out involving three different types of stiffeners, i.e., stud section, track section and a fully effective bridging channel section. The governing design Standards required that a stiffener must be fully effective, which is a restriction that is rather difficult to meet in practice. Based on this investigation, it was concluded that the governing design provisions for bearing stiffeners do not accurately predict the capacity of the stiffeners currently used in lightweight steel framing.

Schuster (1998, 1998) presented a paper on the advantages of having one North American Specification for the Design of Cold Formed Steel Structures. In the paper entitled, "Cold-Formed Steel - the Construction Material of the Future", Schuster presented

The 2000s

Sloof and Schuster (2000) investigated the yield strength increase of cold-formed sections due to cold work of forming. Extensive testing was performed to substantiate the simplified design approach developed by Lind and Schroff in the early 70s at the University of Waterloo, which is *Eq. 1*. This additional test data provided the needed information to conclude that *Eq. 1* is indeed the best predictor expression when compared to the more complex approach used by AISI.

Beshara and Schuster (2000) performed additional web crippling tests to obtain data of C- and Z-sections subjected to interior two-flange loading and end two-flange loading. In addition, new web crippling coefficients were generated using the data from this study and all of the available published data. Calibrations for resistance factors and factors of safety were also carried out (Beshara and Schuster, 2002). The results of this work have been adopted by the North American Specification for the Design of Cold-Formed Steel Structural Members.

Craig and Schuster (2000) carried out calibrations of the cold-formed steel shear equations in an effort to correct a discontinuity in the nominal shear equation that was being considered by the North American Specification for the Design of Cold-Formed Steel Structural Members. Again, the results were adopted by the Specification.

Fox and Schuster (2000) investigated the lateral strength of wind load bearing wall stud-to-track connections. Typically in this case, stud sections are framed into a channel track section, resulting in the possibility of web crippling at the end of the stud connection due to the action of the wind load. The governing cold-formed steel Design Standards/Specifications in North America do not cover this case specifically. The web crippling provisions are based on the member being supported by a rigid bearing plate. Based on this, testing was carried out to develop a design approach for this specific end condition load case. The recommended procedure recognizes two different observed failure modes, i.e., 1) web crippling of the stud and 2) punch-through of the track. The basic web crippling expression was used (*Eq. 7*) with different web crippling coefficients and a new expression was developed for the punch-through case. Again, the results were adopted by the North American Specification for the Design of Cold-Formed Steel Structural Members.

Fox and Schuster (2000, 2003) continued their work of 1998 to establish a simplified design approach for bearing stiffeners in cold-formed C-Sections. It was found that in all of the tests with stud or track stiffeners the failure mode was local buckling of the stiffener in compression. There are a number of variables affecting the strength of the assembly, but in general, the study found that a simple expression for the nominal bearing resistance of the stiffener types tested can be used.

Xu et al. (2000) carried out a study on the optimum design of cold-formed steel residential roof trusses. A computer-based optimal design approach for residential roof trusses was developed, using cold-formed steel C-sections. The truss design was based on the CSA S136-94 and the truss design guide published by the American Iron and Steel Institute and the Canadian Sheet Steel Building Institute. A generic algorithm was adopted to obtain the minimum cost design with due consideration to truss topology and member size simultaneously.

Xu et al. (2000, 2005) investigated the dynamic/vibration behaviour of floors with cold-formed steel joists. Both static and dynamic tests were conducted on cold-formed C-section floor joists with different span lengths based on different design criteria. The static tests were done to obtain the stiffness and the degree of load sharing between the joists, and the purpose of the dynamic tests was to establish the frequencies of the floor systems. To identify the critical parameters that contribute to the control of floor vibration, tests were also carried out on floors without attached ceiling materials, with different bridging and blocking patterns, and with different support conditions. Test results are presented in comparison with the analytical results obtained from different design methods.

Xu et al. (2001) carried out compressive tests of cold-formed corrugated steel curved panels. In the absence of a standard test protocol, presented in this paper are two types of compressive tests, i.e., 1) corner and flange-section tests, and 2) full-panel tests. The purpose of these tests was to investigate the influences of panel curvatures and transverse corrugations on the buckling behaviour of the cold-formed corrugated steel curved panels. The full section tests yielded consistent results, with the deviation of the individual test ultimate load less than 6% and 8% of the average ultimate load with regard to CSA S136-94 and AISI-96, respectively. For corner and flange section tests, the deviation of the ultimate load was generally less than 7%, except one group. Based on this, both test approaches and associated experimental set-ups could be regarded as reliable.

Xu and Cui (2002) undertook a study relating to the connection flexibility of cold-formed steel C-shape connections" Current design practice on cold-formed steel trusses assumes that web-to-chord connections of trusses are ideally pin-connected. The most recent revision of the *AISI Standard for Cold-Formed Steel Framing - Truss Design* permits the connection flexibility to be taken into account in the analysis and design of such trusses. However, no specific guidelines on how to incorporate this connection flexibility into the design process are provided, which is primarily due to the lack of information on the moment-rotation behaviour of such connections. This paper presents results from a series of tests on web-to-chord connections of cold-formed steel trusses. A mathematical model was developed to represent the behaviour of the connections. The objective of the investigation was to assess connection flexibility via the moment-rotation behaviour for the purpose of design of cold-formed steel trusses.

Tangorra et al. (2002) performed calibrations for resistance factors and factors of safety on cold-formed steel welded connections, using all of the published data. The North American Specification again adopted these results for the Design of Cold-Formed Steel Structural Members.

Wallace and Schuster (2002) carried out tests on bolted cold-formed steel tension member connections in bearing (with and without washers). Additional data was needed to complete the pool of data required to properly establish the best design method for bolted tension members failing in bearing. Comparisons were made with the two design methods used in North America and the resulting recommended the North American Specification adopted design approach for the Design of Cold-Formed Steel Structural Members. Calibrations for resistance factor and factor of safety were also carried out as part this study (Wallace et al., 2002).

Wallace et al. (2002) investigated the bending and web crippling interaction of cold-formed steel members. With the recent adoption of the new web crippling design approach *(Eq. 7)*, the web crippling and bending interaction expressions contained in the North American Specification for the Design of Cold-Formed Steel Structural Members need to be re-evaluated. In addition, changes in the bending strength calculation (effective web method) have been introduced, hence, possibly affecting the interaction evaluation. The Specification contains interaction expressions for single web geometry, I-section geometry, and two nested Z-shapes. Based on the recommendations of this study, the North American Specification for the Design of Cold-Formed Steel Structural Members adopted the design approach.

Wallace and Schuster (2004) carried out tests on web crippling of cold-formed steel multi-web deck sections subjected to end one-flange loading. The resulting data was virtually the only data in this category. New web crippling coefficients have been developed along with the respective calibrated resistance factors and factors of safety. And once again, the North American Specification has adopted the results of this work for the Design of Cold-Formed Steel Structural Members.

Conclusions

It has been clearly demonstrated in this paper that in the past 35 years a great deal of research in the field of cold-formed steel has been produced at the University of Waterloo. What is even more remarkable is, that so many of the resulting information has been adopted by North American and International cold-formed steel design Standards/Specifications. I have truly enjoyed having played a small part at this remarkable University over the past 35 years and I look forward to being around and involved in future research in cold-formed steel.

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