Flexural Resistance of Cold-formed Steel Built-up Box Sections of Subjected to Eccentric Loading

L. Xu

P. Sultana

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L. Xu[1] and P. Sultana[2]

Abstract

In cold-formed steel building construction, there are several applications where built-up box sections made of a C-shape nested with a track section, with screw fastenings, are used to resist loads induced in a structural member; when a single section is not sufficient to carry the design load. The cold-formed steel box section may be subjected to eccentric loading when the web of one of the sections receives the load and transfers it through the connection to another section. There may be an unequal distribution of load in cold-formed steel built-up box assemblies loaded from one side. In the current North American Specification for the Design of Cold-Formed Steel Structural Members (CSA, 2002), there is no guideline or design equation to calculate the flexural capacity of this type of section. Cold-formed Steel Framing Design Guide (AISI, 2002) has recommended that the moment resistance and moment of inertia of the built-up sections can be taken as the sum of the two components; based on deflection compatibility of the components. However, this design approximation has yet to be justified by experimental or numerical study especially for the case of eccentric loading. Therefore, a research project involving finite element analysis was undertaken to investigate the flexural behaviour of built-up box sections assembled from cold-formed steel C-shape and track sections when subjected to eccentric loading. The proposed finite element model of the built-up box sections was validated with the tests carried out by Beshara and Lawson (2002). The ultimate moment capacities obtained from the finite element analysis were then compared with the predictions from the current design method; in order to

[1] Associate Professor, The Canadian Cold-formed Steel Research Group, Department of Civil and Environmental Engineering, University of Waterloo, Waterloo, Ontario, Canada.
[2] Former Research Assistant, The Canadian Cold-formed Steel Research Group, Department of Civil and Environmental Engineering, University of Waterloo, Waterloo, Ontario, Canada.

asses its suitability. Parametric studies were carried out to identify the factors affecting the flexural capacity of built up cold-formed steel box sections.

Introduction

Cold-formed steel (CFS) sections such as C-shape and track sections are extensively used in low- and mid-rise residential and commercial building construction in North America. While single sections are not sufficient to sustain applied loads, built-up sections made of back-to-back C-shapes or a nested C-shape with a track section forming a box section, are normally used to carry heavier loads. For example, the built-up box girders or headers are commonly used for floor or wall openings as structural members to support floor joists which are connected to the web of one component of the built-up box assembly as shown in Figure 1. There may also be an unequal distribution of the load in the built-up box section and the section may also be subjected to torsional moments when loaded from one side. Unequal load distribution can potentially lead to a reduction in capacity compared to the sum of the capacities of the individual components that make up the built-up section. As a result, the resistance contributed by the component of the built-up section which is not directly connected with the floor joists, is affected the efficiency of the fasteners in transferring load and possibly other factors. The current North American Specification for the Design of Cold-Formed Steel Structural Members (CSA, 2001) does not provide any guideline on this issue. The Cold-Formed Steel Framing Design Guide (AISI, 2002) suggests that the moment of resistance and inertia of built-up sections can be approximated as the sum of that of the individual components. Addressing these problems presents an interesting challenge for the designer and more research is required to understand the flexural behaviour of CFS built-up box sections subjected to eccentric loading.

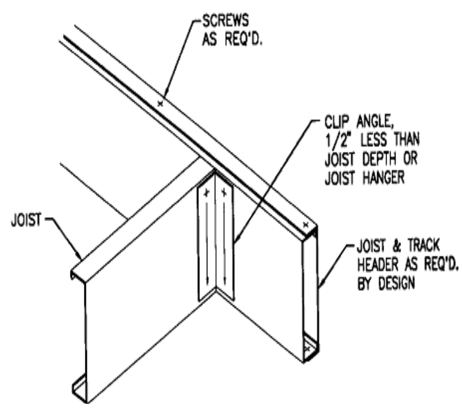


Figure 1: Joist to Joist-header Assembly (CSSBI, 1994)

The objective of this study is to understand the flexural behaviour of CFS built-up box sections subjected to eccentric loading, and to verify whether the current design practice for calculating the moment capacity of the of CFS built-up box section is conservative or not. The built-up box section studied herein is made from nesting a C-shape with a track section, with self-drilling screws fastened at both the top and bottom flanges. The C-shape receives the applied load first, and then transfers the load to the track section through the self-drilling screws. A finite element (FE) model is developed using the ANSYS program to determine the ultimate moment capacity of CFS built-up box sections. After that, parametric studies are conducted using FE analysis to identify the factors affecting the moment capacity of the built-up box section.

Previous Experimental Investigation

A thorough literature review of previous work related to the flexural capacity of CFS built-up box sections subjected to eccentric loading was conducted. As a result, very little information was found. Serrette (2004) investigated the flexural performance of CFS built-up box rafters under eccentric loading. The built-up box sections were made with two face-to-face C-shapes, with a track section cover at the top and bottom flanges of the C-shapes respectively. The tests revealed that failure of the rafters under the eccentric loading condition ultimately resulted from twisting. The analytically computed capacities of the tested box assembles were compared with the test values. The cumulative strength of the box members was computed based on the assumptions that there is no composite flexural action between the components and that lateral buckling is restrained. The limited test data suggests that the eccentric loading and the mechanism of load transfer from the directly loaded C-shape member to the adjacent C-shape member induces twist in the box assemblies. The edge loaded box assemblies were able to resist at most 85-90% of their calculated fully braced flexural capacity.

Beshara and Lawson (2002) conducted internal tests to evaluate the impact of varying the location of connection screws on the behaviour of built-up box sections. Two types of built-up box assemblies were tested. The first type assembly is nested a C-shape with a standard track section to form a box section while in the second assembly the standard track section is replaced with a proprietary product, named TradeReady® rim track (TD) featured with unequal flanges. The assemblies are fastened with self-drawing screws at locations of flanges and lips, respectively. All specimens tested by Beshara and Lawson were fabricated by nesting a C-Shape with either a standard or a TD track section. The cross-section dimensions and yield strengths of the specimens are listed in Table 1. The C-shape has punched out holes spaced at 4 ft. (1219 mm), and the dimension of the hole are 1-1/2 in. (38 mm) by 4 in. (102 mm). The C-Shape and track section are fastened with #10-16 HWH T-3 self-drilling screws placed 12 in. (305 mm) on center.

The test assembly consisted of two parallel CFS built-up box specimens with span lengths of 10 ft. (3048 mm). Two 3 ft. (914) mm long cross–member beams framed into webs of the specimens through hot-rolled steel angle brackets connected the specimens. A single row of #12 self-drilling screws connected each angle bracket to the web of the C-shape specimen, defining the two vertical lines of load application along the depth of the web. The lines of loading were spaced 32 in. (813 mm) apart. The load was applied at the centre of a load distribution beam, loading each cross member equally and creating a region of constant bending moment between the two lines of load application on both specimens. The distance between the supports of specimens and the line of loading was 44 in. (1118 mm).

Table 1 Component cross-section dimensions and yield strengths

Section	F_y (MPa)[1]	Thickness (mm)[2]	Depth (mm)	Top flange (mm)	Bottom flange (mm)	Lip (mm)
C-shape	349	1.61	254	76.2	76.2	25.4
Standard track	307	1.44	254	31.8	31.8	
TD track	417	1.39	254	31.8	63.5	

[1] 1 MPa = 0.145 ksi.; [2] 1.0 mm = 0.0394 in.

Investigation with the load applied on the C-shape side and track section side were conducted. It was found that the moment capacities of the assemblies from the tests were considerably less than the capacities calculated by adding the individual moment capacity of the C-shape and track section as suggested by CFS Framing Design Guide (AISI, 2002). Based on the results of the test series, Beshara and Lawson (2002) recommended that the nominal moment capacity of the built-up box sections should be considered equivalent to 75% of the combined nominal capacities of its components evaluated based on the Specification for the Design of Cold-Formed Steel Structural Members (AISI 1996) .

Mode of Finite Element Analysis

The FE model was developed to simulate the tests conducted by Beshara and Lawson (2002). Instead of simulating the whole test set up, initially, a half of the specimen was modeled to take advantage of symmetry. The analysis results show that the lateral bracing between the two built-up box specimens can sufficiently represented by setting the lateral displacement U_x=0 at the location of bracing. Therefore, the model is further simplified as a quarter of the test setup as that shown in Figure 2. The Shell181 element in ANSYS was selected to model the C-shape and track sections, while the effect of screws has been accounted for by coupling translational and rotational degrees of freedom of the global x, y, and z-directions. For regions around the holes and supports, refined meshes were created to account for stress concentrations. The corner inside bend radius of CFS C-shape and track section is taken as two times of steel thickness. As CFS end stiffeners were used at support locations to prevent web crippling, the stiffeners were modeled by creating Shell181 elements that overlapped the web in the location of the stiffener, and a bond contact was defined to model the influence of the stiffener retaining the web deformation of the specimens.

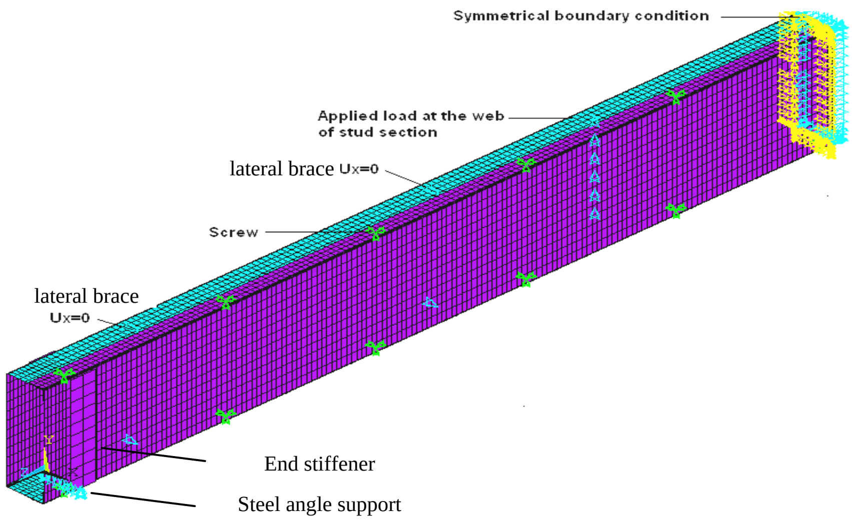


Figure 2: Finite Element Model

The Young's modulus of the steel is taken as 29435 ksi (20300MPa), and Poisson's ratio =0.3. The yield stresses of the steel are listed in Table 1. The effects of cold work forming and residual stress were not accounted for because the ultimate moment capacities obtained from the FE analysis were compared with the nominal moment calculated according to CSA-136 (2001) without considering the cold work of forming.

In the test carried out by Beshara and Lawson (2002), the CFS built-up box specimens were placed on top of an inverted structural steel angle at one end, and a roller on the other end to create the simply supported condition. There was no bearing plate at the support. Such support condition was first investigated in this study to validate the FE model.

The flexural behaviour of thin-walled structures is sensitive to initial geometrical imperfections, especially at the ultimate load level. No measurement was taken to identify the initial geometric imperfection of the CFS built-up box specimens tested by Beshara and Lawson (2002). In this study, first eigenvalue buckling analysis was performed on the model with no initial imperfections to establish the probable collapse mode using ANSYS. Initial imperfection was incorporated in the FE model by scaling the first eigenvalue buckling mode shape, and then including it in the FE model with perfect geometry so that the maximum imperfection does not exceed the thickness of the section, as proposed by Schafer and Pekoz (Schafer and Pekoz, 1998). Then, a nonlinear analysis of the structure containing the imperfection was carried out to determine the ultimate moment capacity.

In FE analysis the loading can be applied in either one of the two ways: apply the load directly on the model, or impose displacement on the model. In order to simulate the test results, loading was applied in both ways and a comparison was made between the results in terms of the ultimate moment capacity, load-deformation behaviour, failure modes, and stress conditions. In the FE analysis while loading was the applied force, a 650 lb (2890 N) load was applied vertically downward on each node at the locations of screws attaching the built-up box assembly and cross member. The load was applied incrementally by defining the initial load as 78 lb (347 N), with a maximum and minimum load increment of 195 lb (867 N) and 0.65 lb (2.9 N), respectively. When loading was applied as the controlled displacement, a 0.7 in. (17.78 mm) vertical downward displacement was applied incrementally by defining the initial displacement as 0.014 in. (0.35 mm), with a maximum and minimum displacement increment of 0.07 in. (1.7 mm) and 0.0000145 in. (0.0003 mm), respectively. After incorporating the initial geometric imperfections, a nonlinear static analysis was performed considering both material and geometric nonlinearities. In this study, the Newton-Raphson method was used.

Validation of Finite Element Model

The nonlinear analysis was conducted with the incremental load/displacement procedure using very small increments of applied force or controlled displacement. Based on the load deflection curves shown in Figure 3, and considering the difference of the ultimate moments obtained from FE analysis, and that the test are within 4%; it indicates that the FE model provides realistic simulation of the test up to the failure of the specimen. However, it was observed that the model could not predict the behaviour after reaching the ultimate load capacity; due to convergence problems even for very small increments of loading (applied force or controlled displacement). Convergence problems could not be overcome, even through the use of the Riks solution method and refining the mesh near the support. The same problem was also encountered in FE model for the built-up box section with a C-shape and a standard track section.

Upon investigating the stress and strain condition of the last converged solutions, it was found that both the von Mises stresses and strains in the C-shape at the location of the inverted steel angle support had reached the yield strength, and the percentage elongation that was reported in the material coupon test, respectively (Beshara and Lawson, 2002). To simulate the bearing support condition in practice, the inverted steel angle was replaced by a 5 mm wide steel bearing plate. The bearing plate was modeled as a 2D surface and then meshed with Shell181 elements. Bonded flexible-to-flexible contact was defined between the plate and the track section. As the C-shape comes in contact with the plate during application of load, standard flexible-to-flexible contact was also defined between the C-shape and the bearing plate at the support. The translational degree of freedom of all the nodes of the bearing plate at the support was restrained in the vertical direction.

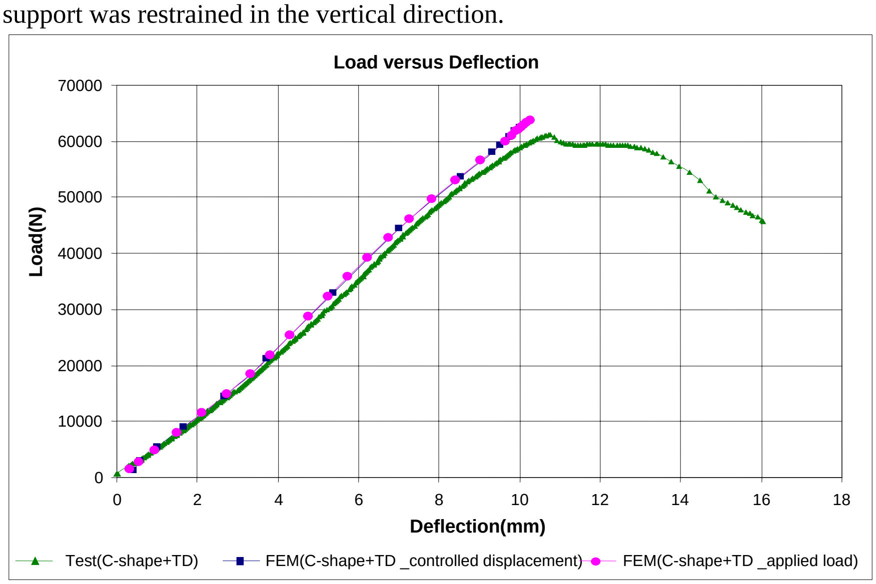


Figure 3: Load-deflection curves of built-up section (inverted angle support)

Figure 4 shows the load-deflection relationship of the built-up box section with a C-shape and TD track section supported by a bearing plate at one end. It can be seen from Figure 4 that the ultimate load capacity predicted by the FE analysis with the bearing plate support is higher than that of the test with the inverted steel angle support. This is due to the local failure of the C-shape at the inverted angle support, not occurring prior to the section reaching to its ultimate load capacity in the case with the bearing plate support. Also found in Figure 4 is that when the applied load is the controlled displacement, the FE analysis is able to simulate the post-ultimate load behaviour of the specimen. Similar observations have also been perceived in the built-up box specimen formed with a C-shape and a standard track section.

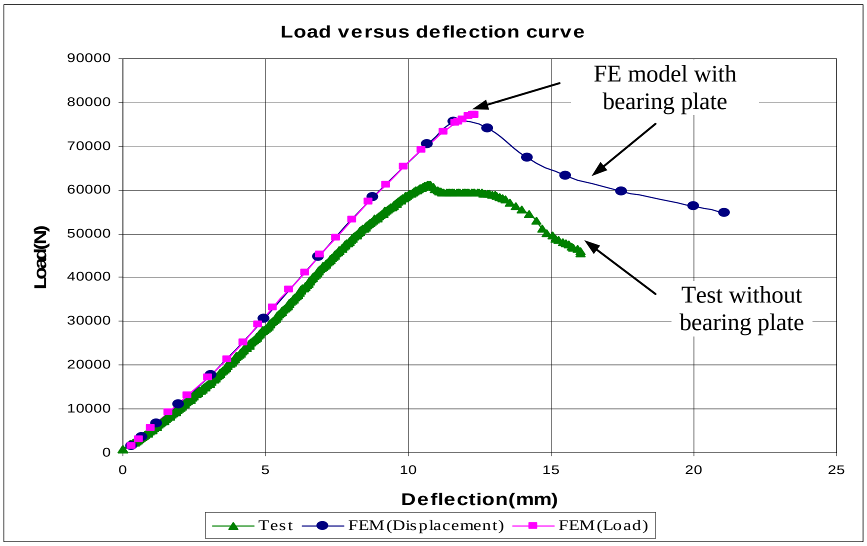


Figure 4: Load-deflection curves of built-up assemblies (bearing plate support)

Results Comparison

The results from the FE analysis were compared with the failure modes, load deflection curve, and ultimate moment capacity obtained from the tests (Beshara and Lawson, 2002). The failure modes shown in the FE analysis are consistent with that of the test. The top flange rippling of the track section was first observed prior to the flange buckling failure as shown in Figure 5, which was similar to that was described in the test as shown in Figure 6. The valleys of the ripples coincided with the locations of the fasteners. The load-deflection relationship obtained from the FE analysis is in good agreement as demonstrated in Figures 3 and 4. The ultimate load capacity of the built-up section is reached when buckling occurs at the top flanges of the C-shape and track section in the constant moment region. The distortion of the built-up box section due to the eccentric loading applied to the web of the C-shape was observed in both the test and the FE analysis, as shown in Figure 7.

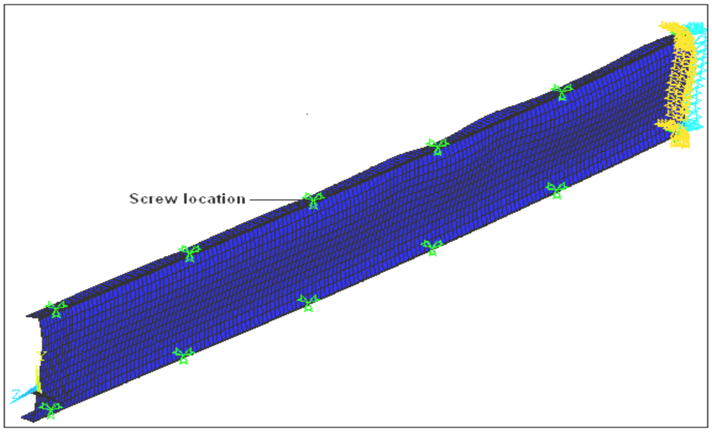


Figure 5: Rippled compressive flange of track section in FE analysis

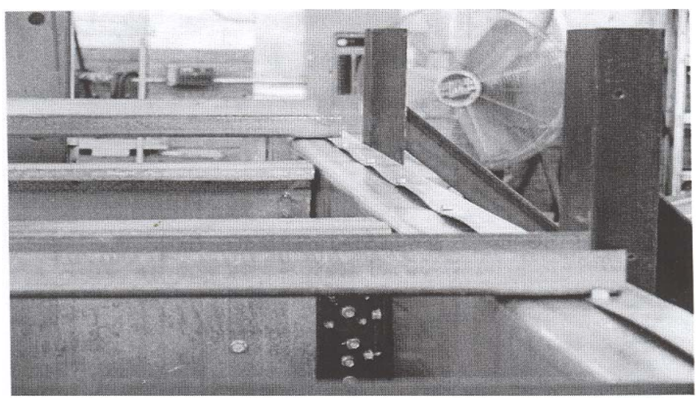
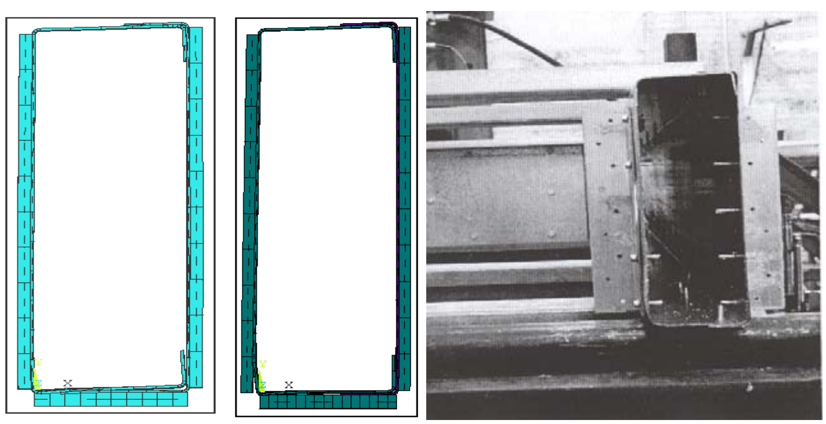


Figure 6: Rippled compressive flange of track section in test (Beshara and Lawson, 2002)

For each type of built-up box sections, the nonlinear FE analysis was pursued with the incremental applied force and controlled displacement procedures. The ultimate moment capacities obtained from the FE analysis (M_{FEM}) of both procedures are shown in a good agreement with each other, and with that of the tests (M_{test}), as illustrated in Table 2. Also presented in Table 2 are the FE analysis results of the built-up box sections supported by the bearing plate at one end of the specimens. It was found that the ultimate moment capacities of the built-up sections with the bearing plate support are 22% and 12% higher than that of the built-up sections supported by the angle support, made with TD and standard track sections, respectively. The nominal moment capacities of the built-up box sections (M_n) listed in Table 2 are the summation of the nominal moment capacities of the corresponding C-shape and track sections as suggested by CFS Framing Design Guide (AISC, 2002). The nominal moment capacities of the C-shape and track sections are calculated in accordance with the North American Specification for Design of CFS Structural Members (CSA, 2001) and 2004 Supplement (CSA, 2004). The ratios of M_{test}/M_n and M_{FEM}/M_n are also presented in Table 2. It is clear that no matter which support condition is applied, the ultimate moment capacities obtained from either of the tests or FE analysis are lower than the nominal moment capacities calculated based on CFS Framing Design Guide (AISC, 2002). Therefore, the procedure of evaluating the flexural moment capacity of the built-up box section recommended by CFS Framing Design Guide (AISC, 2002) may not be conservative.



(a) C-shape + TD track (b) C-shape + standard track (c) Test: C-shape + standard track

Figure 7: Distortion of built-up box sections

Parametric Study

The ultimate moment capacities of CFS flexural members are highly influenced by the yield strength of the material and the width-to-thickness ratio of the flat elements in compression; assuming the members are laterally restrained properly. For CFS built-up box sections, the capacities are affected by the effectiveness of the fasteners that connect the individual components to form the sections. Parametric studies were carried out in this study to investigate the effects of variations of web depth to thickness ratio, steel yield strength, and screw spacing, on the ultimate moment capacity of the CFS built-up box sections.

Table 2 Comparison of ultimate moment capacity of CFS built-up box girder

Built-up box section	Support condition	M_{test} (kN-m)[1]	M_n (kN-m)	$\frac{M_{test}}{M_n}$	Controlled displacement		Applied Force	
					M_{FEM} (kN-m)	$\frac{M_{FEM}}{M_n}$	M_{FEM} (kN-m)	$\frac{M_{FEM}}{M_n}$
C-shape and TD tack	steel angle	17.351	22.187	0.782	17.438	0.786	17.779	0.801
C-shape and standard track	steel angle	17.458	21.194	0.824	17.984	0.848	17.194	0.811
C-shape &TD tack	bearing plate		22.187		21.103	0.951	21.557	0.971
C-shape and standard track	bearing plate		21.194		19.888	0.938	19.466	0.918

[1] 1 kN-m = 0.738 kip-ft

The built-up box sections in the parametric studies were formed with a C-shape and a standard track section. The length of all built-up assemblies is 126 in. (3200 mm), and the assemblies are supported on bearing plates at one ends, and rollers at other ends. The depths of sections considered in the parametric studies were 8 in. (203 mm), 10 in. (254 mm), and 12 in. (305 mm). The section thicknesses for the C-shape and track section were taken to be 0.045 in. (1.14 mm), 0.057 in. (1.44 mm) and 0.071 in. (1.81 mm). The dimensions of the flange and lip of the C-shape is 3 in. (76 mm) and 1 in. (25 mm) respectively, while the flange width of the track sections is 1.25 in (32) mm.

Initial geometric imperfections, material nonlinearity, and geometric nonlinearity were considered in the same way as stated previously. In order to predict the post-ultimate load behaviour, the loading was applied as controlled displacement in the parametric study. Shown in Figure 8 is the load versus mid-span deflection relationship associated with variation of section depth.

It is observed from Figure 8 that the FE analysis has not only successfully simulated the nonlinear load-deflection behaviour of the built-up box specimens at both prior and post ultimate load stages, but also the bilinear behaviour at the initial loading stage. When the built-up section is initially loaded, only the bottom flange of the track section is in contact with the bearing plate at the support. As the load increases, the bottom flange of the C-shape comes into contact with the bearing plate which results in the change of the slope of the curves. The change of the slope of the load-deflection curve signifies the stiffness increase of the specimen, once the bottom flange of C-shape contacts the bearing plate.

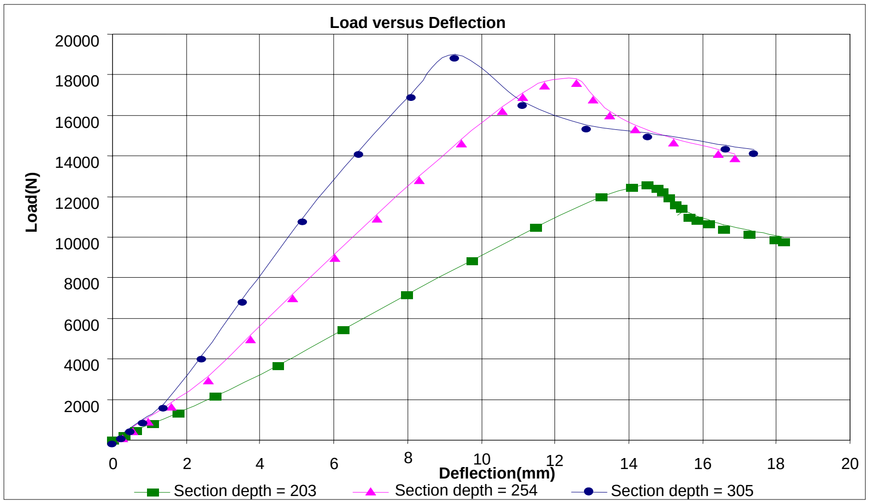


Figure 8: Load-deflection curves associated with variation of section depth

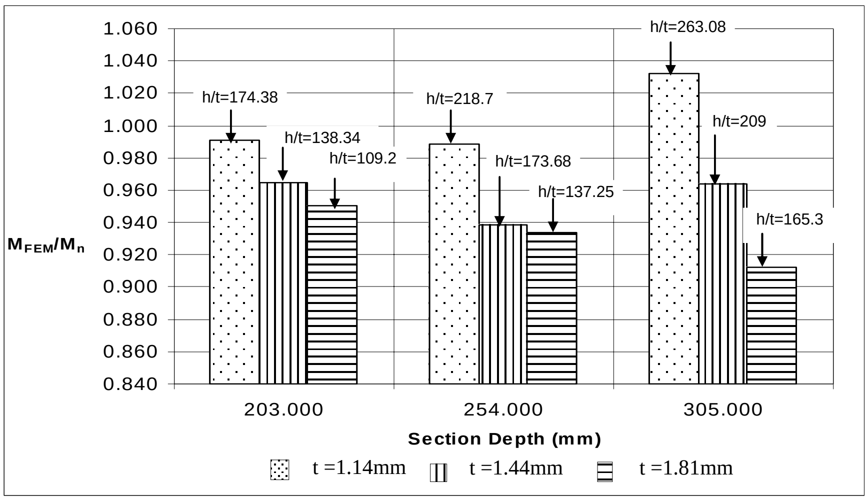


Figure 9: M_{FEM}/M_n ratio associated with variation of section thickness

The effect of section thickness on the M_{FEM} / M_n ratio is illustrated in Figure 9 for three section depths (203mm, 254mm and 305mm). It can be seen from the Figure that for a specific section depth, the M_{FEM} / M_n ratio decreases as the thickness increases, which indicates that the current practice provides better approximation of the ultimate moment capacity for the built-up sections with higher *h/t* ratios.

Two cases were investigated in the FE analysis on the effect of yield strength of material on the M_{FEM} / M_n ratio. In the first case, the yield strengths for the C-shape and the track section are as the same as that of the tests as 50.6 ksi(349 MPa) and 44.5 ksi (307 MPa), respectively. The yield strengths for both C-shape and the track sections are identical at 33 ksi (228 MPa) in the second case. The results for the three section depths which the section thickness t =0.057 in (1.44 mm) are presented in Figure 10, and it can be seen from the Figure that the results appears to be inconclusive.

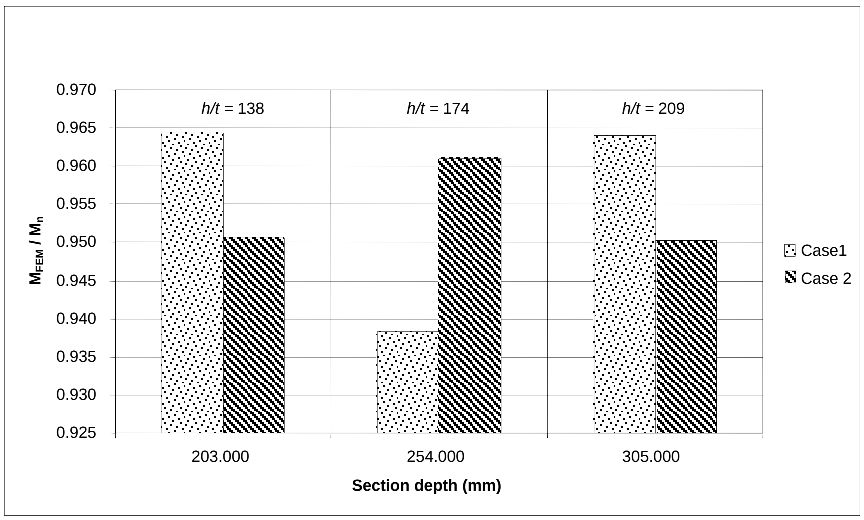


Figure 10: M_{FEM}/M_n ratio associated with variation of material yield stress

It is noted that the effect of screw spacing is not accounted for in the procedure of evaluation of the ultimate moment capacity of the built-up box sections suggested by the CFS Framing Design Guide (AISC, 2002). The influence of screw spacing on the ultimate moment capacity of the built-up box sections was investigated for screw spacing of 6 in. (150 mm), 12 in. (300 mm) and 24 in. (600 mm). The results of the FE analysis show that the ultimate moment capacity of the CFS built-up box sections is influenced by the screw spacing to some extent. As the screw spacing decreases, the predicted ultimate moment capacity of the built-up section increases. The increase of the ultimate moment capacity is generally less than 6% when the screw spacing is reduced to 12 in. (300 mm) from 24 in. (600 mm) or reduced 6 in. (150 mm) from 12 in. (300 mm). The M_{FEM}/M_n ratios are also affected by the variation of screw spacing as that indicated in Figure 11. It appears that the procedure recommended by the CFS Framing Design Guide provides better estimation of the ultimate moment capacity for the built-up sections with smaller screw spacing.

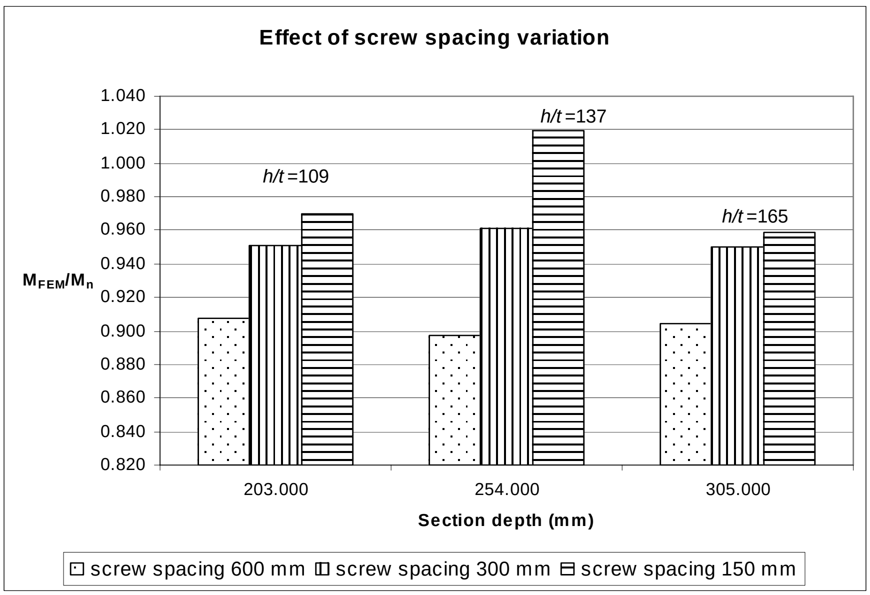


Figure 11: M_{FEM}/M_n ratio associated with variation of screw spacing

Conclusions

Numerical analysis was carried out with the aim of investigating the flexural behaviour of CFS built-up box sections subjected to eccentric loading, and of evaluating the appropriateness of the current design practice recommended by the CFS Framing Design Guide (AISI, 2002). A FE model was established to investigate the flexural capacity of CFS built-up box sections and validated with test results reported Beshara and Lawson (2002). Initial geometric imperfections, material nonlinearity, and geometric nonlinearity were considered in the FE analysis; and compared with the results obtained from the tests (Beshara and Lawson 2002). It was shown that the FE model could reliably predict the ultimate moment capacity as well as the prior and post ultimate load behviour of CFS built-up box sections. The FE analysis showed that by introducing a bearing plate at the support location, the local failure at that region can be minimized; and the ultimate moment capacity of the built-up box sections can be increased considerably compared to the inversed angle support.

A parametric study was carried out to investigate the influences of section depth, section thickness, screw spacing, and material yield stress on the ultimate moment capacities of CFS built-up box sections. From the results of the parametric studies, it was found that it is inappropriate to assume the moment capacities of CFS built-up box sections are the summation of the moment capacities of the individual components when subjected to eccentric loading. In fact, the ratio M_{FEM}/M_n was generally found to be less than one. Therefore, it is concluded that the current design practice may overestimate the moment capacities of CFS built-up box sections in the case of eccentric loading, and therefore, may not be conservative.

Acknowledgements

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References

AISI, “Specification for the Design of Cold-Formed Steel Structural Members”, American Iron and Steel Institute 2002.

AISI, "Cold-Formed Steel Framing Design Guide", American Iron and Steel Institute 2002.

Beshara, B., and Lawson, T.J., "Built-Up Girder Screw Connection Variation Flexural Tests", Internal Report, Dietrich Design Group, 2002.

CSSBI, "Low-Rise Residential Construction Details", Canadian Sheet Steel Building Institute, 1994.

CSA, "North American Specification for the Design of Cold-Formed Steel Structural Members", S136, Canadian Standard Association, 2001.

CSA, "Supplement 2004 to the North American Specification for the Design of Cold-Formed Steel Structural Members", Canadian Standard Association 2004.

Schafer, B.W., and Pekoz, T., "Computational modelling of cold-formed steel: Characterizing geometric imperfections and residual stresses", Journal of constructional steel research, Vol.47, 1998, pp.193-210.

Serrette, R.L., " Performance of Edge-Loaded Cold-formed Steel Built-up Box Beams", Practice Periodical on Structural Design and Construction, ASCE, Vol.9, No. 3, 2004, pp. 170-174.