

Article

Numerical Investigation of the Axial Load Capacity of Cold-Formed Steel Channel Sections: Effects of Eccentricity, Section Thickness, and Column Length

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Abstract: Cold-formed steel channel (CFSC) sections have gained widespread adoption in building construction due to their advantageous properties, including superior energy efficiency, expedited construction timelines, environmental sustainability, material efficiency, and ease of transportation. This study presents a numerical investigation into the axial compressive behavior of CFSC section columns. A rigorously developed finite element model for CFSC sections was validated against existing experimental data from the literature. Upon validation, the model was employed for an extensive parametric analysis encompassing a dataset of 208 CFSC members. Furthermore, the efficacy of the design methodologies outlined in the AISI Specification and AS/NZS Standard were evaluated by comparing the axial load capacities obtained from the numerically generated data with the results of four previously conducted experimental tests. The findings reveal that the codified design equations, based on nominal compressive resistances determined using the current direct strength method, exhibit a conservative bias. On average, these equations underestimate the actual load capacities of CFSC section columns by approximately 11.5%. Additionally, this investigation explores the influence of eccentricity, cross-sectional dimensions, and the point-of-load application on the axial load capacity of CFSC columns. The results demonstrate that a decrease in section thickness, an increase in column length, and a higher degree of eccentricity significantly reduce the axial capacity of CFSC columns.

Keywords: axial load capacity; AS/NZS; cold-formed steel (CFS) channel section; direct strength method (DSM); eccentricity; finite element model (FEM); section dimensions; AISI; thin-walled structures



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1. Introduction

Cold-formed steel sections have found widespread application in various fields, including solar photovoltaic brackets, lightweight portal frames, building structures, and storage shelves, due to their recyclability, quick construction speed, high degree of industrialization, high strength and stiffness, ease of industrial production, and light weight [1]. Thin-walled steel components are susceptible to three distinct buckling modes. Global Buckling (GB) refers to the overall instability of a structural member, characterized by deformation along its entire length. Local Buckling (LB) occurs when individual plate elements within the member buckle due to flexure, without any transverse deformation along the lines where adjacent plates connect. In contrast, Distortional Buckling (DB) involves not only local buckling but also a change in the cross-sectional shape of the member, leading to a more complex buckling behavior.

Many factors influence the buckling behavior of these members. These factors include the initial geometric defect mode, the slenderness ratio for local-distortion (LD) buckling,

the nominal yield strength of the stainless-steel components, and the improvement in material strength in the corner area. Notably, the increased material strength in the corner area of the lipped C-section column enhances the bearing capacity of LD interaction buckling [2].

Thin-walled components have the potential to suffer local buckling before experiencing complete failure [3]. The cross-section design and element length also affect the buckling modes of CFS built-up sections, with multiple buckling modes possible [4]. As the member's length increases, the single-channel section's buckling type transitions from local (L) to distortional (D) [5]. Furthermore, the cross section's and stiffeners' forms, sizes, and material composition significantly impact the section's final strength [6]. Stiffeners can help reduce local buckling in the web [7]. Increasing the web height and flange width of the lipped C-section stainless steel columns enhances their LD interaction buckling capability. Under the two boundary conditions, there is little variation in LD interaction buckling capability of sections with the same element [8]. On the other hand, LB is considerably simpler than DB [9].

Additionally, the load-bearing capacity [10–15], as well as the built-up columns' initial elastic stiffness [14], will be decreased as the eccentricity increases. The specimen's ultimate bearing capacity begins to drop when the eccentricity rises. Specimens subjected to positive eccentric compression have a higher ultimate bearing capacity than those exposed to negative eccentric compression when the eccentricity is lower (See Figure 8). Conversely, specimens exposed to negative eccentric compression have a higher final bearing capacity than those exposed to positive eccentric compression whenever the eccentricity is greater [16]. This investigation aims to quantify the individual and interactive effects of load eccentricity, section thickness, and section length on the axial load-carrying capability of CFSC columns.

2. The Design Criteria Are Based on the Direct Strength Method (DSM)

The design considerations for CFS sections without holes in compression are specified in the North American Specification for the Design of CFS Structural Elements (AISI S100) [17,18] and the Australian and New Zealand Standard (AS/NZS 4600) [19]. These guidelines provide the optimal method for calculating the maximum axial load that columns can withstand. Three different types of buckling may occur in CFS sections: distortional (D), global (G), and local (L) buckling (B) [5,17,18,20,21]. Global buckling is further divided into flexural-torsional (FT), torsional (T), and flexural (F) buckling [17,18], as illustrated in Figure 1. In terms of notation, the symbols N_{ol} , N_{od} , N_{oc} , N_{cl} , N_{cd} , N_{ce} , N_c , and N_y in AS/NZS 4600:2018 correspond to P_{crl} , P_{crd} , P_{cre} , P_{nl} , P_{nd} , P_{ne} , P_n , and P_y in AISI S100–16.

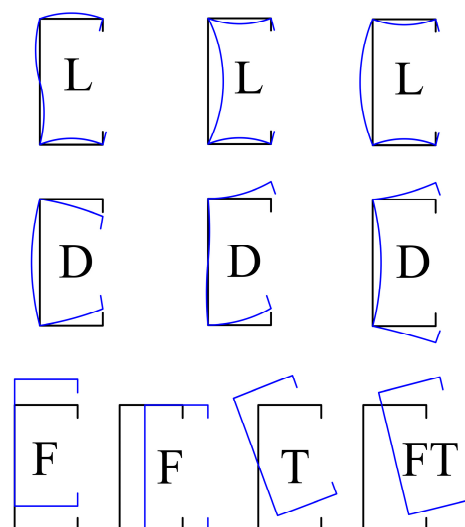


Figure 1. Types of Buckling in Thin-Walled Steel Columns.

The Effective Width Method (EWM) is a design approach that simplifies the analysis of local buckling in cold-formed steel members. By reducing the gross cross-sectional area to an effective cross-sectional area, the method approximates the nonlinear stress distribution with a linear one. In contrast, the Direct Strength Method (DSM) directly predicts member strength without employing effective width concepts, offering a more comprehensive analysis of the cross-section. The DSM in AISI [17,18,22] and the guideline [23] offer a more accurate method for determining strength than the effective width method (EWM) in the Eurocode 3, AISI, and AS/NZS [24].

A comparative analysis of the design strengths of EC9 and DSM revealed that DSM offers more precise predictions [25]. The adjusted slenderness ratio regulated the rise in slenderness, indicating a cautious DSM forecast [26]. The EWM offers cautious and dispersed approximations of the buckling capacity of C-section columns that have high strength and moderate to medium slenderness ratios. However, it produces very precise outcomes for columns with large slenderness ratios. On the other hand, the DSM provides quite precise forecasts for the resilience of high-strength C-section columns [27]. Furthermore, findings indicate that the average cross-section yield stress design parameter is suitable for use with the DSM to estimate the axial strengths of CFS elements experiencing GB or LB [28]. For most CFSC members with web holes and pinned–pinned boundary conditions, the DSM’s anticipated findings are not conservative. Current design standards for channel CFS elements with web holes are overly cautious and potentially dangerous, according to a comparison of test and finite element (FE) findings with AISI & AS/NZS data [29].

2.1. Global Buckling

The nominal yield capacity of the member is denoted by N_y . In terms of the buckling behavior, the minimum elastic element buckling stresses in F, T, and FT buckling modes are represented by N_{oc} . Additionally, the nominal section capacity for GB is indicated by N_{ce} . Furthermore, f_{oc} exhibits elastic behavior under flexural, torsional, and flexural-torsional buckling stresses (refer to Figure 1).

$$N_{oc} = A_g \times f_{oc} \quad (1)$$

$$N_y = A_g \times f_y \quad (2)$$

$$\lambda_c = \sqrt{N_y / N_{oc}} \quad (3)$$

$$\text{When } \lambda_c > 1.5 ; N_{ce} = 0.877 \times N_y \times \lambda_c^{-2} \quad (4)$$

$$\text{When } \lambda_c \leq 1.5 ; N_{ce} = N_y \times 0.658^{\lambda_c^2} \quad (5)$$

2.2. Local Buckling

The nominal capacity of an element with respect to local buckling is represented by N_{cl} . Additionally, the elastic LB load is denoted N_{ol} , and the LB stress of the members is symbolized by f_{ol} (refer to Figure 1).

$$\lambda_{\uparrow} = \sqrt{N_{ce} / N_{ol}} \quad (6)$$

$$\text{For } \lambda_{\uparrow} > 0.776 ; N_{cl} = \left[1 - 0.15 \left(\frac{N_{ol}}{N_{ce}} \right)^{0.4} \right] \left(\frac{N_{ol}}{N_{ce}} \right)^{0.4} N_{ce} \quad (7)$$

$$\text{For } \lambda_{\uparrow} \leq 0.776 ; N_{cl} = N_{ce} \quad (8)$$

2.3. Distortional Buckling

The nominal capability of a section for DB is represented by N_{cd} . Similarly, the elastic stress associated with DB in channels is indicated by F_{od} , while N_{od} represents the elastic

compressive buckling load resulting from D effects. In addition, N^* denotes the design's concentrated loads or reactions, and N_c stands for a member's nominal capacity. For a visual representation of these terms and their relationships, refer to Figure 1.

$$\lambda_d = \sqrt{N_y / N_{od}} \quad (9)$$

$$\text{For } \lambda_d > 0.561 ; N_{cd} = \left[1 - 0.25 \left(\frac{N_{od}}{N_y} \right)^{0.6} \right] \left(\frac{N_{od}}{N_y} \right)^{0.6} N_y \quad (10)$$

$$\text{For } \lambda_d \leq 0.561 ; N_{cd} = N_y \quad (11)$$

$$N_c \text{ represents the smallest value among } N_{cd}, N_{cl}, \text{ and } N_{ce} \quad (12)$$

$$N^* = \phi_C \times N_c = 0.85 \times N_c \quad (13)$$

3. Finite Element Modelling (FEM)

3.1. General

To create the FEMs of the tested specimens and forecast the behavior and compression capabilities of CFSC sections, the FE application Abaqus/CAE 2024 [30] was utilized. The FE models incorporated the exact section geometry (Figure 2), material characteristics, end boundary conditions, and initial geometric imperfections of the tested specimens. To ensure accuracy, the FE analyses included geometric and material nonlinear (second-order) inelastic analysis with imperfections (GMNIA). In the final step of the analysis, a dynamic-implicit step was employed, where a quasi-static approach [31] was chosen. Specifically, the initial, minimum, and maximum increment sizes were set to 1×10^{-5} , 1×10^{-55} , and 1×10^{-3} , respectively, with a maximum number of increments set at 1×10^6 . This methodology was applied consistently across all FE models.

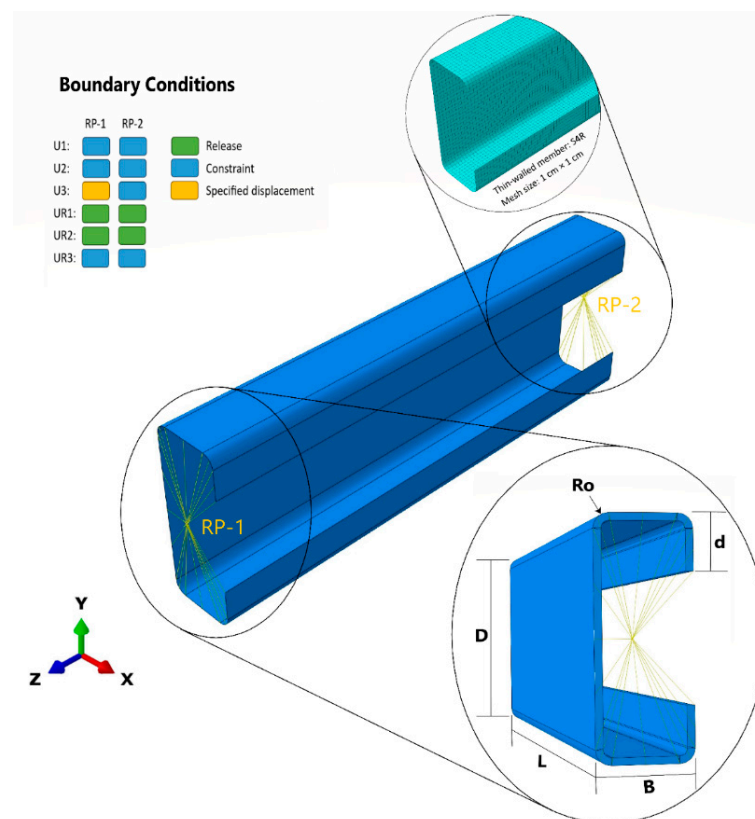


Figure 2. Finite element model parameters, boundary conditions, and cross-section geometry.

3.2. Mesh Size and Element Type

The components of each member were simulated using the S4R [32,33] element, a quadrilateral stress/displacement shell element featuring reduced integration and a large-strain formulation. With four nodes, each possessing six degrees of freedom, the S4R element is particularly adept for modeling thin-walled constructions and is highly efficient in terms of computational resources. To ensure accuracy, convergence studies were conducted, and a mesh size of 1 cm × 1 cm was determined as optimal, as illustrated in Figure 2.

3.3. Loading Method and Boundary Conditions

The FE method setup for this study involved implementing simply supported boundary conditions across all FEMs of the single channel members, as depicted in Figure 2. Initially, a coupling constraint was applied to each end cross-section, connecting it to a reference point (RP-1 or RP-2) positioned at its centroid. Subsequently, all movements (U1, U2, and U3) and rotations (UR3) of the lower reference point (RP-2) were constrained. Similarly, the initial reference point (RP-1) was fixed, except axial displacement (U3) was not restricted. Axial compression loading was applied through RP-1 using displacement control.

4. Validation of the Developed Finite Element Models

Based on the load-displacement curves, failure types, and ultimate loads, the derived FEMs were verified. As illustrated in Figure 5, the distorted forms of the test specimens closely match those predicted by the FE analyses. According to the linear buckling analysis conducted in accordance with AISI S100, the axial load capacity of the columns is governed by both local and global buckling. Moreover, the final loads predicted by FE analysis are consistent with those obtained from the tests, as depicted in Figure 4 and detailed in Table 2. The axial load versus axial displacement curve patterns from both the tests and FE studies are identical, further confirming the accuracy of the FEMs (Figure 4). Overall, the constructed FEMs are deemed reliable in predicting mode of failure and the compression capability of individual channel sections.

4.1. Specimen Design

To substantiate this research, four types of CFS with lipped channel members were selected, taking into account the width–thickness ratio (D/t) and the steel strength grade. Prior to conducting the tensile test, the initial geometric dimensions of each sample, including gauge length, thickness, and breadth, were measured. A comprehensive explanation of these dimensions is provided in Table 1, while Figure 2 illustrates the symbols representing the sizes of each section.

Table 1. Column specimen dimensions and material properties.

Columns	D	B	d	t	R _o	L	X	Y	A	F _y	E	N _y
	mm								mm ²	MPa	GPa	kN
Column 1	253.5	80.5	50.1	7.43	14.9	748	28.2	126.8	3461	297.9	202	1031
Column 2	255.2	80.7	50.3	7.81	15.1	749	28.3	127.6	3645	381.6	205	1391
Column 3	361.1	141.5	83.9	11.33	22.4	1053	53.7	180.6	8360	295.7	210	2472
Column 4	343.5	144.8	97.1	15.51	30.1	1052	59.1	171.8	11274	316.3	208	3566

4.2. Test Results Analysis

The yield strength of the material was determined by different methods depending on the presence of a yield platform. For the specimen with a yield platform, the yield strength was determined by the yield point, as described by Fu X et al. [34]. Conversely, for the specimen without a yield platform, the yield strength was determined by the stress

indicative of a 0.2% plastic strain. The dimensions, elastic modulus (E), yield strength (Fy), and center of gravity of all specimens are presented in Table 1.

4.3. Material Properties and Geometry

For the parametric study and test specimens, the entire geometry was meticulously modeled. To enhance accuracy, the FEM was modified to include material nonlinearity by specifying the “true” values of stresses and strains. All analyses and model validations were conducted using the ABAQUS classical metal plasticity model. In this model, the von Mises yield surface is implemented to define isotropic hardening behavior, associated plastic flow theory, and isotropic yielding. Furthermore, a simplified stress–strain curve for elastic–plastic that adheres to the von Mises yield criterion was employed for the parametric investigation. This approach ensured a robust representation of material behavior under various loading conditions.

$$\sigma_t = \sigma_E \times (\epsilon_E + 1) \quad (14)$$

$$\epsilon_t = \ln(\epsilon_E + 1) \quad (15)$$

$$\epsilon_t (\text{Plastic}) = \ln(\epsilon_E + 1) - \sigma_t/E \quad (16)$$

The variables E, σ_E , and ϵ_E represent the Young’s modulus, engineering stress, and strain, respectively. In contrast, σ_t and ϵ_t represent true stress and strain, respectively, in ABAQUS. The relationship between engineering and true curves is shown in Figure 3.

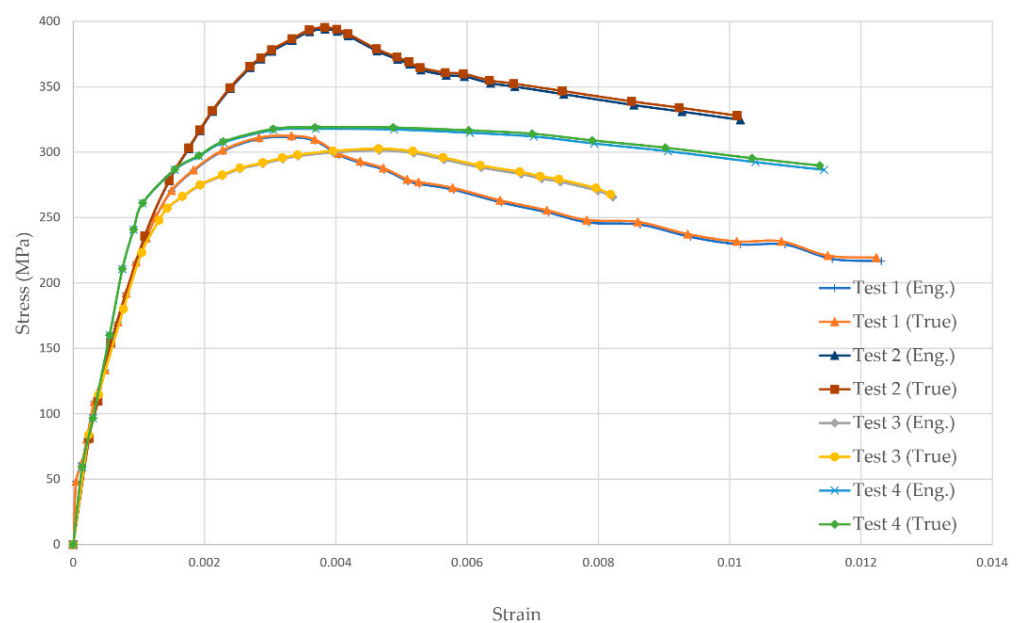


Figure 3. Engineering and True Stress–Strain Curves.

4.4. Comparison of Various Findings

In order to validate the precision of the FEM and the AISI & AS/NZS findings for the four chosen short columns, the calculated outcomes were compared to the results obtained from short column tests. The findings of the comparison are illustrated in Table 2.

Based on Table 2, the following conclusions can be inferred:

The mean value of the geometrically and materially nonlinear FEM results relative to TEST results is approximately 96.8%. This value, though slightly below 100%, indicates that the FEM results are conservative and deemed safe for assessing thin-walled steel short columns. Moreover, the proximity of the mean value to 100% suggests that the FEM results are accurate and reliable. In contrast, the sample standard deviation (S) of the FEM/TEST results is 0.0187, a small value indicating high precision in predicting the axial load capability of short columns using FEM.

Table 2. Comparison of Test Results [34] with the Finite Element Method, and AISI & AS/NZS Standards.

Columns	N_c [AISI & AS/NZS]	N_c [TEST]	N_c [FEM]	N_c [FEM]/ N_c [TEST]	N_c [AISI & AS/NZS]/ N_c [TEST]
	kN	kN	kN	%	%
Column 1	944	1142	1073	94	83
Column 2	1252	1488	1455	98	92
Column 3	2338	2609	2536	97	90
Column 4	3394	3787	3713	98	90
Average				96.8	88.5
S				0.0187	0.0401

However, the mean value of the elastic linear results obtained from AISI & AS/NZS relative to TEST results is about 88.5%. This value, below 100%, suggests that the AISI & AS/NZS results tend to be conservative and less economical for determining the axial load capacity of CFS short columns. Moreover, the sample standard deviation of the AISI & AS/NZS/TEST results is 0.0401, reflecting variability but still within acceptable limits for practical use. In summary, while the FEM approach proves accurate and reliable with minimal deviation, the AISI & AS/NZS method, while safe, may be overly conservative and less cost-effective for evaluating thin-walled steel short columns.

4.5. Verification of Finite Element Model

The load-axial shortening relationship, analyzed through both geometrically and materially nonlinear finite element method (FEM) simulations and physical testing (TEST) of all four columns, is presented in Figure 4. Initially, the relationship exhibited predominantly linear behavior, transitioning to nonlinear behavior until reaching the failure load. The validated finite element models were evaluated based on failure modes, load-displacement curves, and ultimate loads. Figure 4 illustrates a strong correlation between the deformed shapes observed in physical tests and those predicted by the finite element (FE) models.

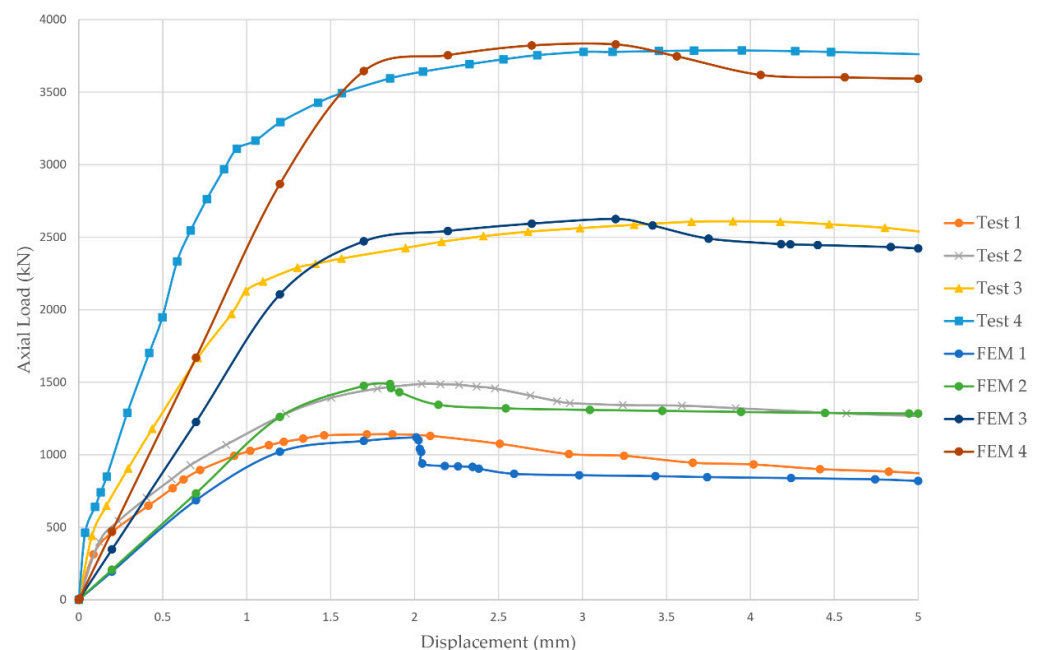


Figure 4. Comparison of load-displacement curves for fixed-ended columns between experimental tests [34] and FEM.

Figure 5 presents a comparison between the experimental test results and the FE results of short columns from column 1 to 4. Additionally, Figure 5 illustrates the failure scenarios for the aforementioned four columns. It is evident that the FE findings closely align with the experimental test results. Therefore, the experimental and finite element findings demonstrate a high level of concordance in terms of both the ultimate strength and the failure mechanism. Figures 4 and 5 provide evidence supporting the correctness of Table 2, which presents a comparison between the failure load obtained from experimental testing and the failure load predicted by finite element analysis.

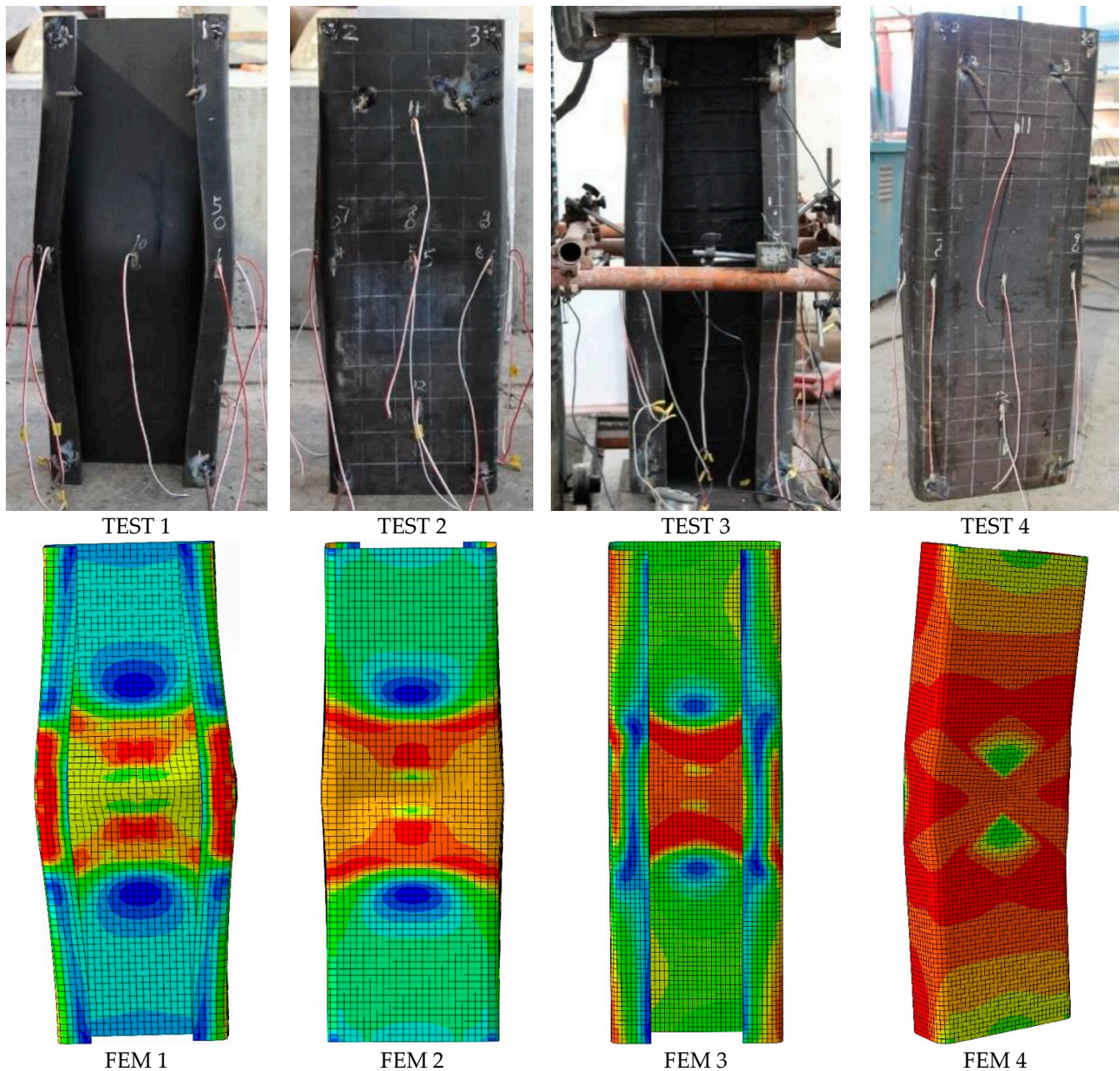


Figure 5. Comparison of column buckling shapes: experimental testing [34] vs. finite element analysis.

5. Parametric Study and Result

5.1. General

This study gives a thorough examination of the influence of key geometrical parameters on the compressive response and axial load capacity of CFS columns. To achieve this, validated nonlinear elastic-plastic FE models were employed. These models enabled a systematic parametric analysis encompassing 208 different configurations. Specifically, the analysis focused on the effects of element thickness, column length, and eccentricity from the centroid. The FE models were meticulously constructed based on the experimentally determined external dimensions of four CFSC sections. These sections comprised single channels subjected to axial compressive loading. The subsequent sections will elaborate on the specific challenges encountered during the modeling process, providing detailed insights into the methodologies and considerations involved.

5.2. The Effect of Member Thickness

This section explores the relationship between section thickness and the axial load capacity of validated finite element model (FEM) columns. To investigate this relationship, a systematic reduction in thickness was applied to each of the four columns, with reduction factors ranging from 1.0 to 6.0. Subsequently, the axial load capacities of the reduced-thickness columns were determined and documented. In order to facilitate comparison, the axial load capacity of each reduced-thickness column was normalized by dividing it by the capacity of the corresponding unreduced-thickness column. This normalization procedure was employed for all four validated FEM columns. By doing so, it allows for a clearer understanding of how section thickness impacts the axial load capacity, providing valuable insights for the design and analysis of FEM columns.

The present investigation reveals a substantial correlation between the thickness of CFS sections and the axial load capacity of columns. As evidenced in Table 3 and Figure 6, a direct linear relationship exists between these parameters. Specifically, a reduction in section thickness demonstrably translates to a proportional decrease in their capacity to resist axial loads. Notably, when the average thickness is reduced to one-half, one-third, one-quarter, one-fifth, and one-sixth of its original value, the corresponding reductions in average axial load capacity are approximately 56%, 77%, 86%, 91%, and 93%, respectively. This indicates that even slight decreases in section thickness can lead to significant reductions in the CFS columns' axial load capability. Consequently, careful consideration must be given to section thickness in the design and application of these structural elements.

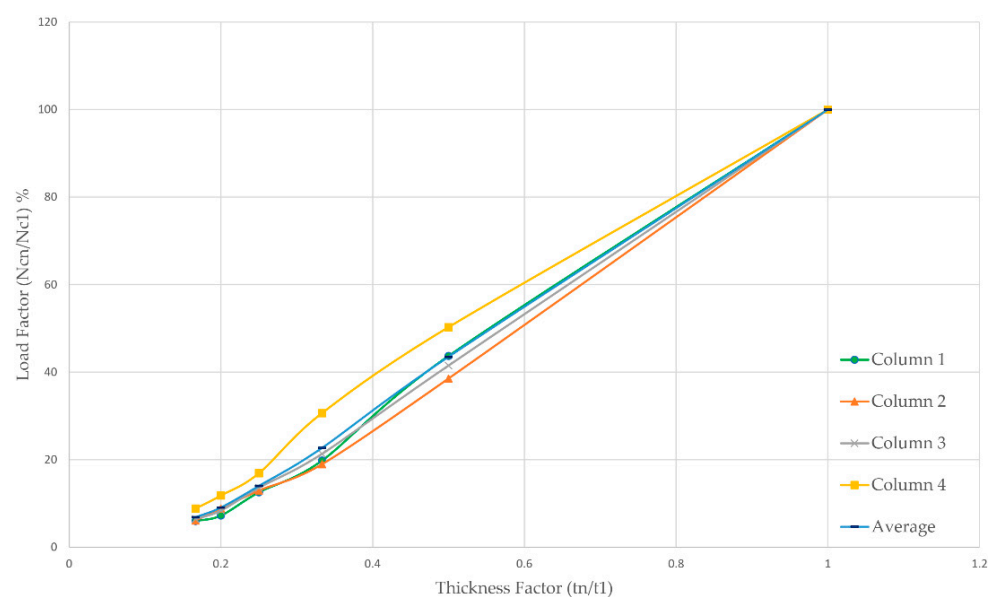


Figure 6. Correlation of axial load capacity with column thickness.

Table 3. Impact of thickness variation on the axial load capability of columns.

Columns	n = 1:6	$t_n = t_1/n$	X	N_{cn}	N_{cn}/N_{c1}
		mm	mm	kN	%
Column 1	1	7.43	28.23	1073	100
	2	3.72	27.92	469	44
	3	2.48	27.83	212	20
	4	1.86	27.78	135	13
	5	1.49	27.75	77	7
	6	1.24	27.73	64	6
Column 2	1	7.81	28.28	1455	100
	2	3.91	27.96	561	39
	3	2.60	27.85	276	19
	4	1.95	27.80	188	13
	5	1.56	27.77	129	9
	6	1.30	27.75	89	6
Column 3	1	11.33	53.70	2536	100
	2	5.67	53.38	1052	41
	3	3.78	53.27	541	21
	4	2.83	53.22	344	14
	5	2.27	53.19	212	8
	6	1.89	53.17	163	6
Column 4	1	15.51	59.12	3713	100
	2	7.76	58.78	1867	50
	3	5.17	58.67	1138	31
	4	3.88	58.62	627	17
	5	3.10	58.59	438	12
	6	2.59	58.57	328	9
Average	1				100%
	2				44%
	3				23%
	4				14%
	5				9%
	6				7%

5.3. The Effect of Member Length

This section investigates the influence of member length on the axial load capability of short and long columns. To achieve this, a systematic parametric analysis is conducted, utilizing four validated FEMs representative of both short- and long-column configurations. Initially, the length of these FEMs will be linearly varied to quantify the impact on their axial buckling behavior. This methodical approach will facilitate a comprehensive understanding of the interplay between section length and load-carrying capacity in compression members. By examining both short and long columns, the analysis aims to uncover the nuanced effects that section length has on the structural performance of these elements under axial loads.

This study examines the effect of column length on the CFS sections' axial load capability. As evidenced by Table 4 and Figure 7, minimal reductions in length have a

negligible impact on the axial capacity of short CFS columns. However, for longer members, a nonlinear inverse relationship between column length and axial capacity becomes evident. In particular, Columns 1 and 2 exhibit a significant decrease in axial load capacity with increasing length, with a critical point observed at approximately 5 m. Beyond this point, the influence of further length increments becomes less pronounced. Similarly, Columns 3 and 4 show a substantial reduction in capacity up to 11 m, followed by a more gradual decline for even longer columns.

Table 4. The effect of column length variation on axial load capacity.

Columns	n = 1:6	Short Columns			Long Columns		
		$L_n = L1/n$	N_{cn}	N_{cn}/N_{c1}	$L_n = L1 \times n$	N_{cn}	N_{cn}/N_{c1}
		mm	N	%	mm	N	%
Column 1	1	748	1073	100	748	1073	100
	2	374	1106	103	1496	973	91
	3	249	1100	102	2244	880	82
	4	187	1108	103	2992	653	61
	5	150	1125	105	3740	455	42
	6	125	1058	99	4488	320	30
Column 2	1	749	1455	100	749	1455	100
	2	375	1473	101	1498	1380	95
	3	250	1467	101	2247	1164	80
	4	187	1478	102	2996	744	51
	5	150	1500	103	3745	487	33
	6	125	1513	104	4494	341	23
Column 3	1	1053	2536	100	1053	2536	100
	2	527	2599	102	2106	2392	94
	3	351	2604	103	3159	2308	91
	4	263	2616	103	4212	2262	89
	5	211	2650	104	5265	1774	70
	6	176	2650	104	6318	1322	52
Column 4	1	1052	3713	100	1052	3713	100
	2	526	3775	102	2104	3548	96
	3	351	3793	102	3156	3335	90
	4	263	3840	103	4208	3080	83
	5	210	3823	103	5260	2417	65
	6	175	3844	104	6312	1841	50
Average	1			100%			100%
	2			102%			94%
	3			102%			86%
	4			103%			71%
	5			104%			53%
	6			103%			39%

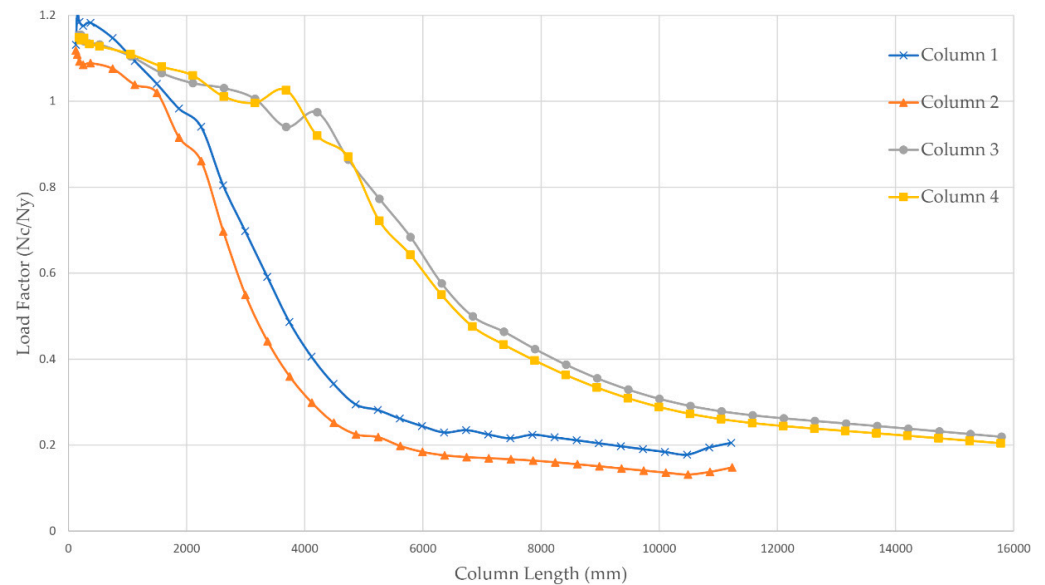


Figure 7. The correlation between column length and axial load capacity.

On average, a modest increase in axial load capacity, ranging from only 2% to 4%, was observed when the column length was reduced by factors of one-half, one-third, one-fourth, one-fifth, and one-sixth. Conversely, increasing the column length by factors of two, three, four, five, and six resulted in significant reductions in capacity, reaching approximately 6%, 14%, 29%, 47%, and 61%, respectively. These findings highlight the critical role of column length in CFS member design. Shorter columns demonstrably possess a superior capacity to bear axial loads compared to their longer counterparts. This knowledge can be strategically employed to optimize member selection and promote enhanced structural efficiency in CFS projects.

5.4. The Effect of Eccentricity from the Centroid

In prior studies and practical applications, boundary conditions, typically fixed or pinned supports, are conventionally applied at the centroid of member ends. This investigation specifically examines how the location of RP-1 and RP-2 relative to the centroid influences the axial load capacity of four validated columns. The findings uncover an intriguing “e-shaped” relationship, indicating that as RP-1 and RP-2 move away from the centroid, the axial load capacity of the columns demonstrates a significant variation. The performance of CFS beam-column elements is mainly affected by parameters such as the ratios of element and web slenderness, as well as the degree and direction of eccentricity [35]. When eccentricity is focused on the web or lipped side, the failure modes of the structural member are mainly determined by interactions between local–global or distortional–global buckling [36]. Specimens typically exhibit distortional or combined local–distortional buckling, contingent upon the eccentricity levels and cross-sectional thickness [35].

Analysis of the load-displacement curves reveals significant impacts based on the direction of eccentricities on beam-column member behavior. Members with low eccentricity values often experience a quick decrease in strength after reaching the maximum load, whereas higher eccentricity values lead to a more flexible behavior after buckling [37]. In summary, the interaction of element slenderness, eccentricity direction, and cross-sectional thickness play a critical role in determining the buckling behavior and post-buckling response of CFS beam-column elements.

The columns’ axial load capability is significantly influenced by the location of end reference points (RPs), as highlighted in Table 5 and Figures 8 and 9. These figures vividly illustrate how relocating RPs (RP-1 and RP-2) from the centroidal position to various points such as the top flange, bottom flange, web, or lip of the cross-section results in contrasting

effects. Notably, shifting both RPs from the centroid to the web of CFS sections leads to a 43% reduction in the average axial load capability. In contrast, a more substantial decrease of 67% in the average axial load capacity is observed when both RPs are moved towards the lip.

Table 5. The influence of eccentricity on the axial load capacity of columns with respect to their strong and weak axes.

Columns	N = 0:100%	Direction from Centroid to Web			Direction from Centroid to Lip			Direction from Centroid to Top Flange		
		X	N _{cn}	N _{cn} /N _{c0}	X	N _{cn}	N _{cn} /N _{c0}	Y	N _{cn}	N _{cn} /N _{c0}
		mm	kN	%	mm	kN	%	mm	kN	%
Column 1	0	28	1073	100	28	1073	100	0	1073	100
	10	25	1006	94	33	877	82	13	993	93
	20	23	945	88	39	742	69	25	890	83
	30	20	891	83	44	642	60	38	801	75
	40	17	842	79	49	567	53	51	727	68
	50	14	799	74	54	507	47	63	666	62
	60	11	760	71	60	458	43	76	614	57
	70	8	724	68	65	418	39	89	569	53
	80	6	692	64	70	384	36	101	530	49
	90	3	662	62	75	356	33	114	496	46
	100	0	635	59	81	331	31	127	466	43
Column 2	0	28	1455	100	28	1455	100	0	1455	100
	10	25	1357	93	34	1189	82	13	1321	91
	20	23	1268	87	39	992	68	26	1175	81
	30	20	1189	82	44	852	59	38	1054	72
	40	17	1120	77	49	746	51	51	955	66
	50	14	1058	73	54	664	46	64	872	60
	60	11	1002	69	60	597	41	77	802	55
	70	8	951	65	65	544	37	89	741	51
	80	6	909	62	70	498	34	102	690	47
	90	3	869	60	75	460	32	115	645	44
	100	0	827	57	81	427	29	128	606	42
Column 3	0	54	2536	100	54	2536	100	0	2536	100
	10	48	2371	93	62	2143	85	18	2354	93
	20	43	2225	88	71	1854	73	36	2122	84
	30	38	2091	82	80	1631	64	54	1920	76
	40	32	1974	78	89	1458	57	72	1754	69
	50	27	1868	74	98	1316	52	90	1612	64
	60	21	1772	70	106	1201	47	108	1487	59
	70	16	1687	67	115	1102	43	126	1385	55
	80	11	1608	63	124	1019	40	144	1294	51
	90	5	1538	61	133	948	37	162	1214	48
	100	0	1472	58	142	886	35	181	1143	45
Column 4	0	59	3713	100	59	3713	100	0	3713	100
	10	53	3427	92	68	3190	86	17	3424	92
	20	47	3176	86	76	2781	75	34	3087	83
	30	41	2956	80	85	2461	66	52	2792	75
	40	35	2766	74	93	2202	59	69	2546	69
	50	30	2602	70	102	2000	54	86	2337	63
	60	24	2453	66	111	1829	49	103	2162	58
	70	18	2324	63	119	1685	45	120	2007	54

Table 5. Cont.

Columns	N = 0:100%	Direction from Centroid to Web			Direction from Centroid to Lip			Direction from Centroid to Top Flange		
		X	N _{cn}	N _{cn} /N _{c0}	X	N _{cn}	N _{cn} /N _{c0}	Y	N _{cn}	N _{cn} /N _{c0}
		mm	kN	%	mm	kN	%	mm	kN	%
	80	12	2206	59	128	1563	42	137	1874	50
	90	6	2101	57	136	1457	39	155	1756	47
	100	0	2004	54	145	1363	37	172	1655	45
Average	0%			100%			100%			100%
	10%			93%			83%			92%
	20%			87%			71%			83%
	30%			82%			62%			74%
	40%			77%			55%			68%
	50%			73%			50%			62%
	60%			69%			45%			57%
	70%			65%			41%			53%
	80%			62%			38%			50%
	90%			60%			35%			46%
	100%			57%			33%			44%

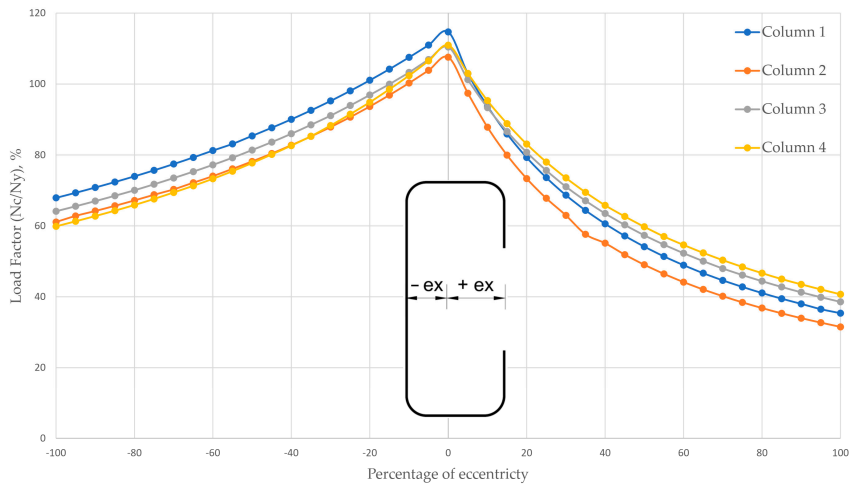


Figure 8. Effect of strong-axis eccentricity on column axial load capacity.

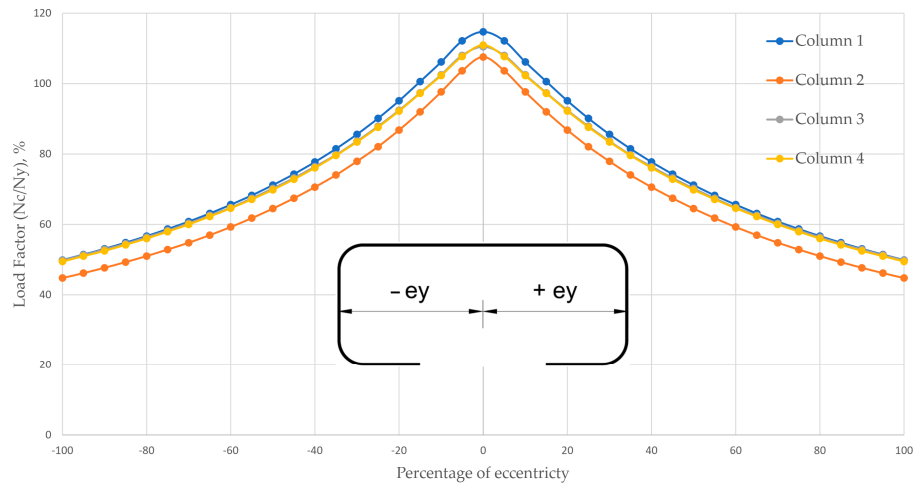


Figure 9. Effect of weak-axis eccentricity on column axial load capacity.

Conversely, when the reference points for axial load are shifted from the column's centroid to either the top or bottom flange, there is a significant impact on their load-bearing capacity. Figure 9 illustrates this effect, showing a substantial decrease of 56% in the columns' average axial load capacity. This observation underscores the critical importance of carefully considering the placement of reference points when assessing a column's ability to withstand axial loads.

6. Discussion

Eccentric loading in columns occurs when the load is not applied precisely along the centroidal axis of the cross-section, resulting in a significant reduction in the axial load-carrying capacity of the column. The effect of eccentricity on column strength is complex and difficult to quantify precisely. To investigate this, a series of 48 tests on long columns were conducted to assess the impact of loading eccentricity on the axial strength of cold-formed steel lipped channels. The findings reveal that even small eccentricities can markedly reduce axial strength. The AISI methodologies provide conservative estimates of strength, accounting for the effects of loading eccentricity. For instance, an eccentricity of 25 mm along the weak axis can reduce the failure loads by up to 30% compared to those expected under concentric loading [38].

The structural performance of beam-columns is influenced by several factors, including cross-sectional shape and size, the position of the eccentric load, column length, and the presence of bracing [17,23]. As eccentricity increases, the ultimate bearing capacity of the specimens consistently decreases. Notably, specimens under positive eccentric compression exhibit higher ultimate bearing capacities at lower eccentricities compared to those under negative eccentric compression. However, as eccentricity increases, this trend reverses, with positive eccentric compression resulting in lower bearing capacities than negative eccentric compression [16].

This research further shows that the mean value of geometrically and materially nonlinear FEM results is approximately 3% more conservative than experimental data. In contrast, the AISI and AS/NZS equations underestimate the actual concentric load capacities of CFSC section columns by an average of 11.5% relative to experimental data. The axial load-bearing capacity of CFSC section columns is significantly influenced by the location of reference points (RPs) for axial load application. Shifting RPs from the centroid to the web (negative eccentricity $(-ex)$) reduces axial load capacity by 43%, while moving them towards the lip (positive eccentricity $(+ex)$) results in a more substantial 67% reduction. Additionally, relocating RPs to the top or bottom flange $(+/-ey)$ leads to an average decrease in load-bearing capacity of 56%. For slotted perforated specimens under eccentric compression, eccentricity significantly impacts ultimate bearing capacities. Specimens subjected to positive eccentric compression exhibit an approximate 22% reduction in ultimate bearing capacity compared to those under pure axial compression, while those under negative eccentric compression show an approximate 16% reduction. The Steel Stud Manufacturers Association (SSMA) has identified that the ultimate bearing capacities for perforated CFS channels under eccentric compression, as specified in the AISI S100–16, present significant computational challenges [16].

7. Conclusions

This study presents a comprehensive investigation into the influence of key geometrical parameters on the compressive response and axial load capacity of CFS columns. Utilizing validated nonlinear elastic-plastic FEMs, a systematic parametric analysis was conducted encompassing 208 different configurations. The analysis specifically focused on the effects of section thickness, column length, and eccentricity from the centroid. The following conclusions can be drawn:

1. Validation of FE Models: The mean value of the geometrically and materially nonlinear FEM results divided by test results is approximately 96.8%. This value being less than 100% indicates that the FEM results are on the safe side. The mean value being

close to 100% demonstrates the accuracy of the FEM, making it a reliable tool for testing thin-walled steel short columns. Additionally, the sample standard deviation (S) of the FEM/TEST results is 0.0187, further proving the FEM's accuracy in determining the axial load capacity of short columns.

2. Evaluation of AISI & AS/NZS Design Methodologies: The mean value of the elastic linear results obtained by AISI & AS/NZS divided by test results is around 88.5%, with a sample standard deviation (S) of 0.0401. Although these results are on the safe side, the mean value being significantly less than 100% indicates that the AISI & AS/NZS methodologies are too conservative and not economical for determining the axial load capacity of thin-walled steel short columns.

3. Impact of Section Thickness on Axial Load Capacity: A direct linear relationship exists between the thickness of CFS sections and the axial load capacity of columns. A reduction in section thickness translates to a proportional decrease in their capacity to resist axial loads. Specifically, when the average thickness is reduced to one-half, one-third, one-quarter, one-fifth, and one-sixth of its original value, the corresponding reductions in average axial load capacity are approximately 56%, 77%, 86%, 91%, and 93%, respectively.

4. Effect of Column Length on Axial Load Capacity:

Short Columns: Reductions in length have a negligible impact on the axial capacity of short CFS columns. On average, a modest increase in axial load capacity, ranging from 2% to 4%, is observed when the column length is reduced by factors of one-half, one-third, one-fourth, one-fifth, and one-sixth.

Longer Members: For longer members, a nonlinear inverse relationship between column length and axial capacity becomes evident. Increasing the column length by factors of two, three, four, five, and six results in significant reductions in capacity, reaching approximately 6%, 14%, 29%, 47%, and 61%, respectively.

5. Influence of Eccentricity from Centroid on Axial Load Capacity:

Shifting Reference Points to Web or Lip: The axial load capacity of columns is significantly influenced by the location of end reference points (RPs). Shifting both RPs from the centroid to the web of CFS sections leads to a 43% reduction in average axial load capacity. A more substantial decrease of 67% is observed when both RPs are moved towards the lip.

Shifting Reference Points to Top or Bottom Flange: Shifting the reference points for axial load from the column's centroid to either the top or bottom flange results in a substantial decrease of 56% in the average axial load capacity of the columns. It is recommended to place the reference points at the centroid of members to maximize load-carrying capacity.

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Nomenclature

Symbols	Meaning
AS/NZS	Australian/New Zealand Standard
AISI	American Iron and Steel Institute
DSM	Direct Strength Method
EWM	Effective Width Method
FEM	Finite Element Model
CFS	Cold-Formed Steel
CFSC	Cold-Formed Steel Channel
GMNIA	Geometric and Material Nonlinear (Second-order) Inelastic Analysis with Imperfections
RP	Reference Point
S	Sample standard deviation
LB	Local Buckling
DB	Distortional Buckling
GB	Global Buckling
F	Flexural
T	Torsional
FT	Flexural-Torsional
LD	Local-Distortional
$\lambda_c; \lambda_l; \lambda_d$	non-dimensional slenderness
N_{ce}	The nominal member capacity of a member in compression for flexural, torsional or flexural-torsional buckling
N_{oc}	Least of the elastic compression member buckling load in flexural, torsional and flexural-torsional buckling
N_y	Nominal yield capacity of the member in compression
N_{cl}	The nominal member capacity of a member in compression for local buckling
N_{ol}	Elastic local buckling load
N_{cd}	The nominal member capacity of a member in compression for distortional buckling
N_{od}	Elastic distortional compression member buckling load
f_y	Compressive yield stress
A_g	Gross area of the cross-section

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