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Studying The Behavior of Cold-Formed Steel Sections in Lightweight Buildings

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KEYWORDS:
cold-formed steel, distributed load capacity, finite element analysis, lightweight building

***Abstract*— Over the last decade, the use of cold-formed steel (CFS) sections has increased significantly increasing in lightweight buildings and many other industrial constructions worldwide. The present research paper aims to investigate the distributed load capacity of CFS sections in lightweight buildings. Three CFS built-up sections were investigated for column and other three for girder in a frame of building study case. Channels, channels with lip, sigma section, hollow section, and/or plates were used to obtain column and girder profiles in frame. Self-tapping screws were applied to assemble the elements of each section. The finite element (FE) model was used to study the distributed load capacity of frames using ABAQUS program. A total of nine frames were tested till failure, and their behavior was studied. The FE model was validated using data from prior tests. Different characteristics affecting the distributed load capacity of CFS built-up sections were studied using the FE model, including column profile, girder profile, steel grade, steel thickness, longitudinal spacing between screws (fasteners), and cross-sectional area. the distributed load capacities obtained from FE models were compared to develop the perfect section in each parameter.**

I. INTRODUCTION

Cold-formed steel sections (CFS) have been increasingly popular as key structural elements in building construction over the past few decades. Residential buildings with a low to medium rise and portal frames with a moderate span are examples, because of their high structural performance and durability. Purlins in roofs, medium-span joists in floors, studs in wall panels, storage racking in warehouses, and hoarding structures at building sites are all examples of using CFS as a secondary element.

Other features of the CFS section include ease of construction and fabrication flexibility, as well as a variety of cross-section shapes (Z-section, C-section, hat-section, and Σ-section) to serve various applications. The increased use of the CFS section can be attributed to improved rolling and forming technology, as well as enhanced fastening technology, such as blind rivets and self-drilling, and self-tapping screws [1, 2]. Many studies attempted to increase the corrosion resistance of CFS sections by utilizing galvanizing and other coating technologies [1]. Others began fabricating CFS sections out of stainless steel [3-7]. In 1994, there were roughly 75,000 residential and low-rise buildings made of CFS in the United States; this figure climbed by five times in 2002 [1]. Currently, several scientific studies are underway to employ CFS sections with heavier weights and larger spans [8]. For instance, Dundu conducted an analytical study on the use of CFS sections in a 12 m long portal frame with 4.5 m spacing and reported that as a result of the simplicity of the structure and erection process, valuable construction time could be reduced, resulting in a larger reduction in overall project costs. In addition, transport expenses would be reduced due to the low structural mass. [9].

One of the most powerful solutions to satisfy the present need for the cold-formed section is to use CFS built-up section. Many researchers investigated the behavior of CFS built-up sections made up of two elements, primarily two C-channels placed back-to-back or face-to-face with no space between them [2, 4, 10-12].

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II. LIGHTWEIGHT BUILDING

Nowadays, one of the structural engineers' main concerns is the use of lightweight buildings to provide more simplicity and construction speed at a lower cost of productivity, transportation and handling by reducing labor costs and worker fatigue. Numerous studies were conducted to analyze and design CFS sections as structural elements to achieve lightweight and economy. Furthermore, the choice of a material with a high strength-to-weight ratio is essential. Some researchers explored the benefits and drawbacks of various construction materials. For example, Qureshi et al. investigated the overall cost and time differences comparing reinforced concrete (RC) frames with light gauge steel construction. they found out that CFS construction is 40% less expensive than standard concrete construction for a one-story building with a total area of 81 m^2. They also reported that CFS is four times faster and significantly easier to manufacture than RC [13].

Sangave et al. published a study in 2015 that compared the building material costs consists of ground and six stories (G+6) and ground and ten stories (G+10) RC and steel bare frame construction. As per the findings, the bare steel frame for a G+6 building costs 31% more than the RC frame, while the bare steel frame for a G+10 structure costs 34% less than the RC frame. The authors concluded that in the case of mid-rise buildings, hot-rolled steel (HRS) frames are more expensive than RC frames [14]. Satpute and Varghese compared the cost and weight of the material of HRS with CFS in a one-story industrial structure of 750 m^2 area. Using CFS members instead of HRS resulted in a 35 percent material and expense savings [15].

However, a few studies explored on the frames formed of the CFS built-up section. For instance, Priyadarshini et al. did an experimental evaluation of CFS frames constructed from hollow sections capacity. Hollow section tests on cold-formed steel frames were performed on two different cases of single bayed-two-storied frames with a thickness of 3mm, both major and minor axes were involved [16]. the authors primarily focused on determining the impact of connecting axes on frame behavior and reported that the connecting axis has a greater impact on column behavior than on ultimate load capacity.

Doctolero and Batikha conducted a comparative study on a four-story office building (Appendix A) with a total area of 960 m^2. they used the CFS section, RC, and HRS section and estimated the building's weight, the cost of materials, the cost of construction, the overall cost (material cost + construction cost), and the duration of construction using the design outputs. In comparison to RC and HRS, the study revealed that employing CFS Sections in a mid-rise structure delivers considerable material, overall building cost savings, and significant construction time savings. CFS building construction time is 38 percent and 164 percent quicker than HRS and RC, respectively. CFS building was 61 percent cheaper than the RC building and 35 percent cheaper than the HRS building in terms of the total cost (material cost + construction cost) [17].

This present research paper aims to bridge a gab in the field of analyzing the behavior of CFS sections in lightweight structures, with a focus on the behavior of CFS sections in lightweight buildings made up of a multi-bay, multi-story frame. thus, the FE model was validated by experimental results obtained by Priyadarshini et al. [16]. Then, a numerical analysis was done on frames (connecting three innovative CFS built-up stub columns with the other three CFS built-up girders). The lightweight construction employed was found in a study by Doctolero and Batikha. The distributed load capacity of the frames was evaluated using numerical research. In this paper, distributed load capacity relates to the frame column and girder's ultimate load capacity, or the point at which the frame can no longer support any additional weight.

Priyadarshini et al. test results were as a guide to performing numerical analysis using the FE model [16]. Because there is a paucity of research that provides an appropriate FE model for CFS frames, which was critical to develop. ABAQUS (Abaqus/CAE 6.14 -2) was applied to create the FE model [18]. Different characteristics that impact ingenious CFS built-up section's load capacities in the frame were investigated using the validated FE model, including column profile, girder profile, steel thickness, steel grade, longitudinal screw spacing (fastener spacing), and cross-sectional area.

III. FINITE ELEMENT MODEL

It was essential to create a simple FE model that predicted the CFS frame's load capacity for the use of the parametric investigation. The following section shows the FE model created in this article.

A. FE Model Explanation

The FE model was created using the standard FE modeling application ABAQUS (Abaqus/CAE 6.14-2). All of the CFS section components are represented by the ABAQUS program library's particular four nodded shell element (S4R), with six degrees of freedom for each node (three translations and three rotations). The behavior of thin, thick, and doubly curved shells may all be simulated using S4R.

As shown in Fig. 1, the end through the section of the columns has a fixed end boundary constraint. Region placed at the column ends. Mesh was done with a maximum size of 10 mm and an optimum aspect ratio of 2. Static general analysis, including the nonlinear geometric effect was done by ABAQUS, as a method of analysis.

To simulate the interaction between the CFS elements representing the built-up section, the surface-to-surface contact in ABAQUS (Abaqus/CAE 6.14-2) was used. At the point of contact between the column and the girder contact, a tie constraint was applied. Figure 1 depicts the interactions and tie constraints.

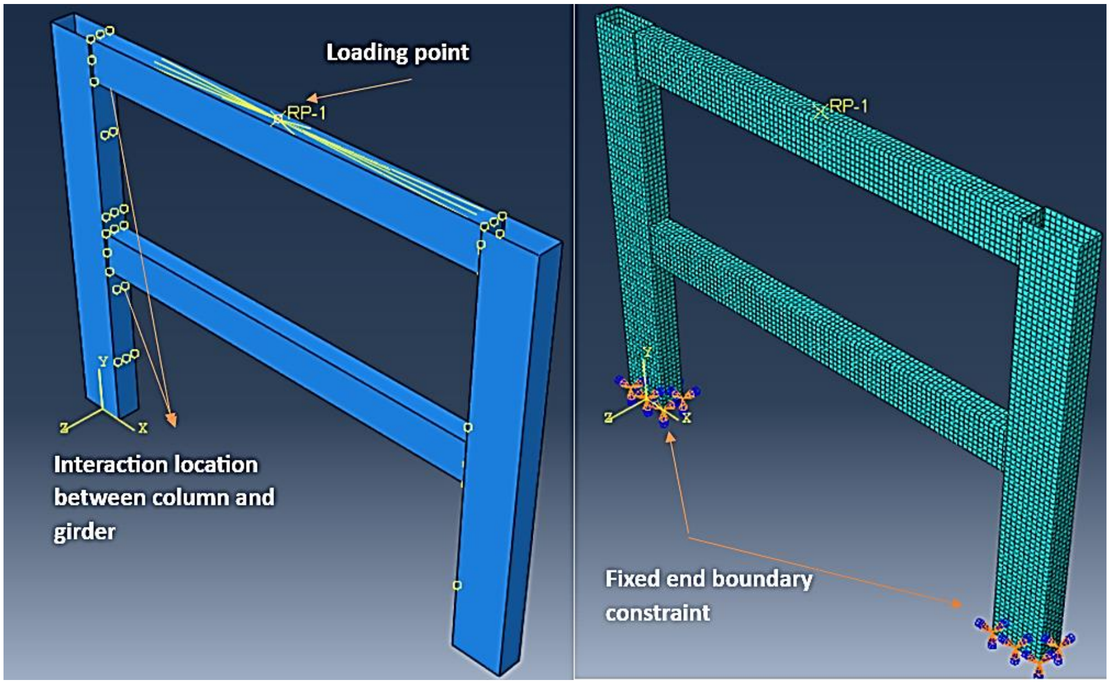


Fig. 1. FE model obtained in this paper

B. *Residual Stress and Corner Enhancement*

Residual stress has no major influence on the ultimate capacity on the columns, according to several investigations (e.g., Abdel-Rahman and Sivakumaran; Schafer and Peköz; Ellobody and Young) [19, 20, 5]. As long as the ultimate load capacity remains an issue, increased yield strength as a consequence of corner enhancement (due to the operation of cold forming) counteracts the impact of residual stress. According to Abdel-Rahman and Sivakumaran, Corner enhancement and residual stress have a noteworthy influence solely on column behavior, particularly post ultimate behavior, but not on the ultimate load capacity value [19].

Because the focus of this research is on the ultimate load capacity of the frame elements rather their post-ultimate load behavior, residual stress and corner enhancement in the FE model were ignored.

C. *The FE Model's Verification*

This section provides some background information on the test conducted by Priyadarshini et al. The test program examined how CFS single bay two-story frames behaved. One bay in the connection of the major and minor axes, two storied frames of hollow sections were examined. The profile of the frames is shown in Fig. 2, and the matrix of test specimens is presented in Table I [16]. It should be mentioned that Table I only displays specimens' specifications with no duplication, that is why only two frames are displayed in the table; these frames are used to verify the FE model. Table I shows the center-to-center vertical distance between beams (H) and the center-to-center horizontal distance between columns (L). The outcomes of the tests by Priyadarshini et al. were compared to the simplified FE model. The ultimate load imposed by Priyadarshini et al. test is P_{Test}, whereas the ultimate load from the FE model presented by the present search is P_{FE}.

The last column in Table I displays the proportion between the ultimate load test (P_{Test}) and the ultimate load derived from the FE model (P_{FE}). Figure 3 illustrates a comparison between both the load displacement curve of the test results and the FE model. There is a strong agreement between the experimental test results and the FE model, till the failure load.

TABLE I
DETAILS OF THE TEST SPECIMENS [16] AND COMPARISON OF RESULTS

Connecting axis	Column section	Beam section	L (mm)	H (mm)	P_{Test} (KN)	P_{FE} (KN)	P_{Test} / P_{FE}
major	100 x 50 x 3	80 x 40 x 3	900	380	36.1	42.6	0.85
minor	100 x 50 x 3	80 x 40 x 3	950	380	59.4	59.12	1.005

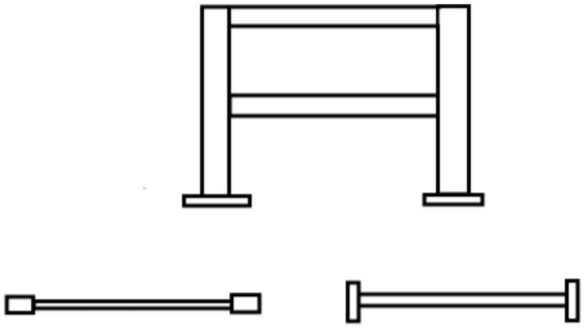
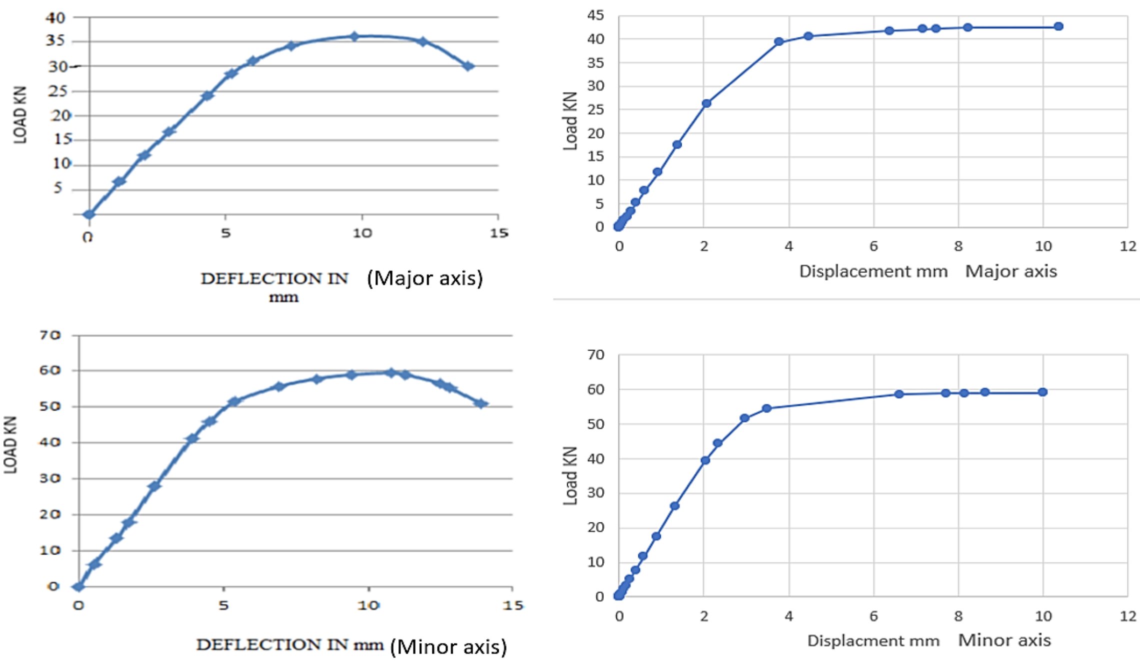


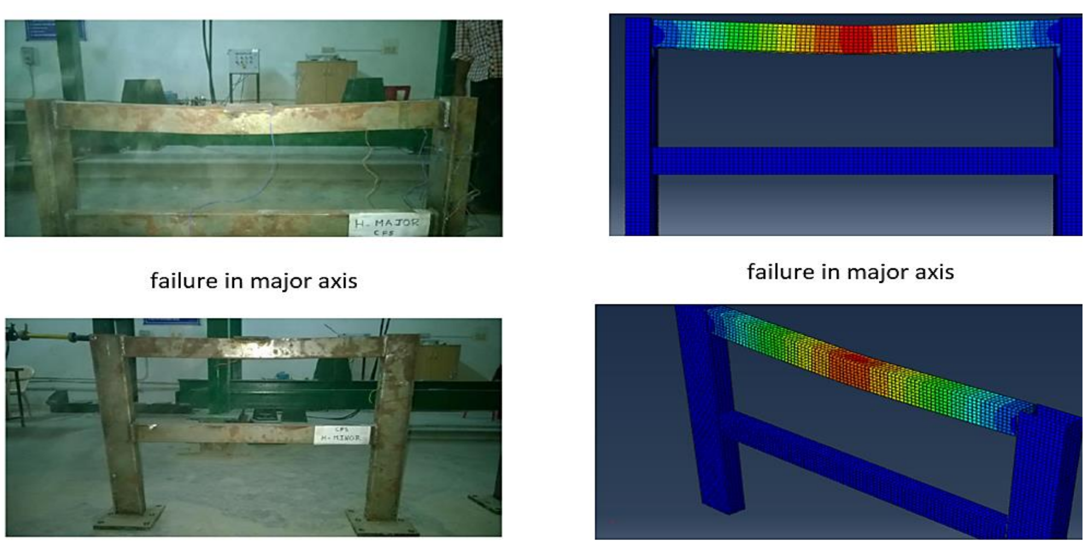
Fig. 2. Experimental test's frame profile [16]

Figure 4 illustrates a comparison of the failure modes detected in the test and the failure mechanism predicted by the FE model. The experimental findings and the FE model are in excellent accordance. This simplified FE model is in great agreement with the experimental test result because this research focused on the ultimate load capacity. Initial imperfection, residual stresses, and corner enhancement should all be seen in the FE model if the frame's entire behavior (pre and post ultimate stage) is of concern.



a- Test results b- FE model results

Fig. 3. The load-displacement curve of the experimental test results [16] and the FE model



failure in major axis failure in major axis

failure in minor axis failure in minor axis

a- Test results b- FE model results

Fig. 4. Failure modes done by Priyadarshini et al. [16] and the failure mode predicted by the FE model

IV. PARAMETRIC STUDIES ON CFS FRAME IN THE LIGHTWEIGHT BUILDING STUDY CASE

Parametric studies on distributed load capacities were undertaken on nine frames depending on lightweight building from Doctolero and Batikha [17] and using the verified model as described in the preceding section.

A. *Investigated Parameters in This Study*

Figure 5 illustrates the studied frame structure and each profile's schematic diagram in the present study. Nine frame profiles in Fig 6 consist of a three-column cross-section (C1, C2, and C3) and a three-girder cross-section each (B1, B2, and B3). They are investigated in different parameters, including steel yielding strength (Fy), CFS section thickness (t), stiffeners spacing (a), and cross sectional areas (A).

Each profile employed four cross-sections (S1, S2, S3, and S4), as illustrated in table II. In the column and girder profiles, each cross-section has the same area.

Table III indicates the many parameters used in this investigation. Regarding ECP-205 (2008), steel yielding strength (Fy) was used (240, 280 and 360 MPa). Four thickness values, i.e., 2, 2.5, 3, and 4 mm, were used.

In each profile, four distinct cross-sectional area values were chosen, as shown in table II and table III. It is worth noting that sections C1, C2, and C3 have the same cross-sectional area, but C3 has a different inertia moment value (I). There were five distinct spacing values employed between the fasteners joining the different elements of the CFS sections. In each profile, the position of the gap in the horizontal cross-section is indicated in Fig. 5. The spacing values for various profiles were set so that one fell within the limits specified by AISI-S100 (2007) in section D1.3, while the other four fell outside this boundary [21].

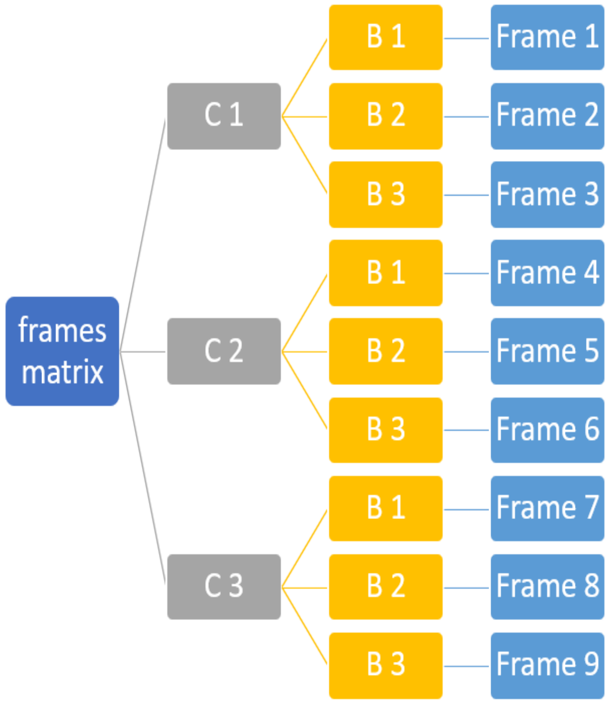


Fig. 6. Frames used in the present study

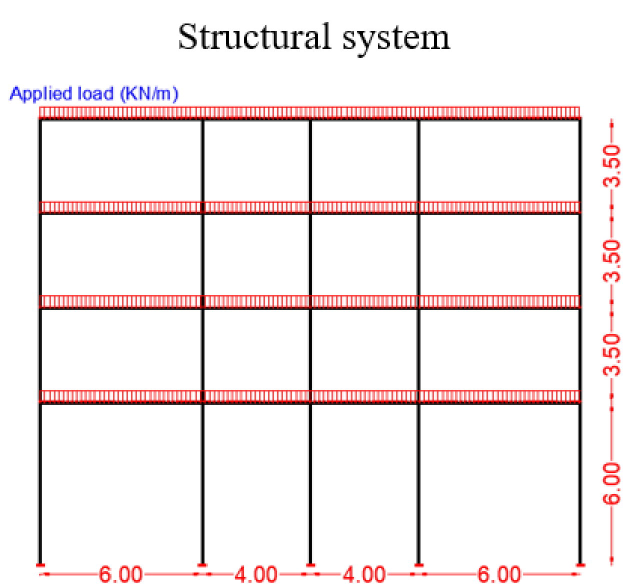


Fig. 5. Frame structure and a schematic diagram for each profile

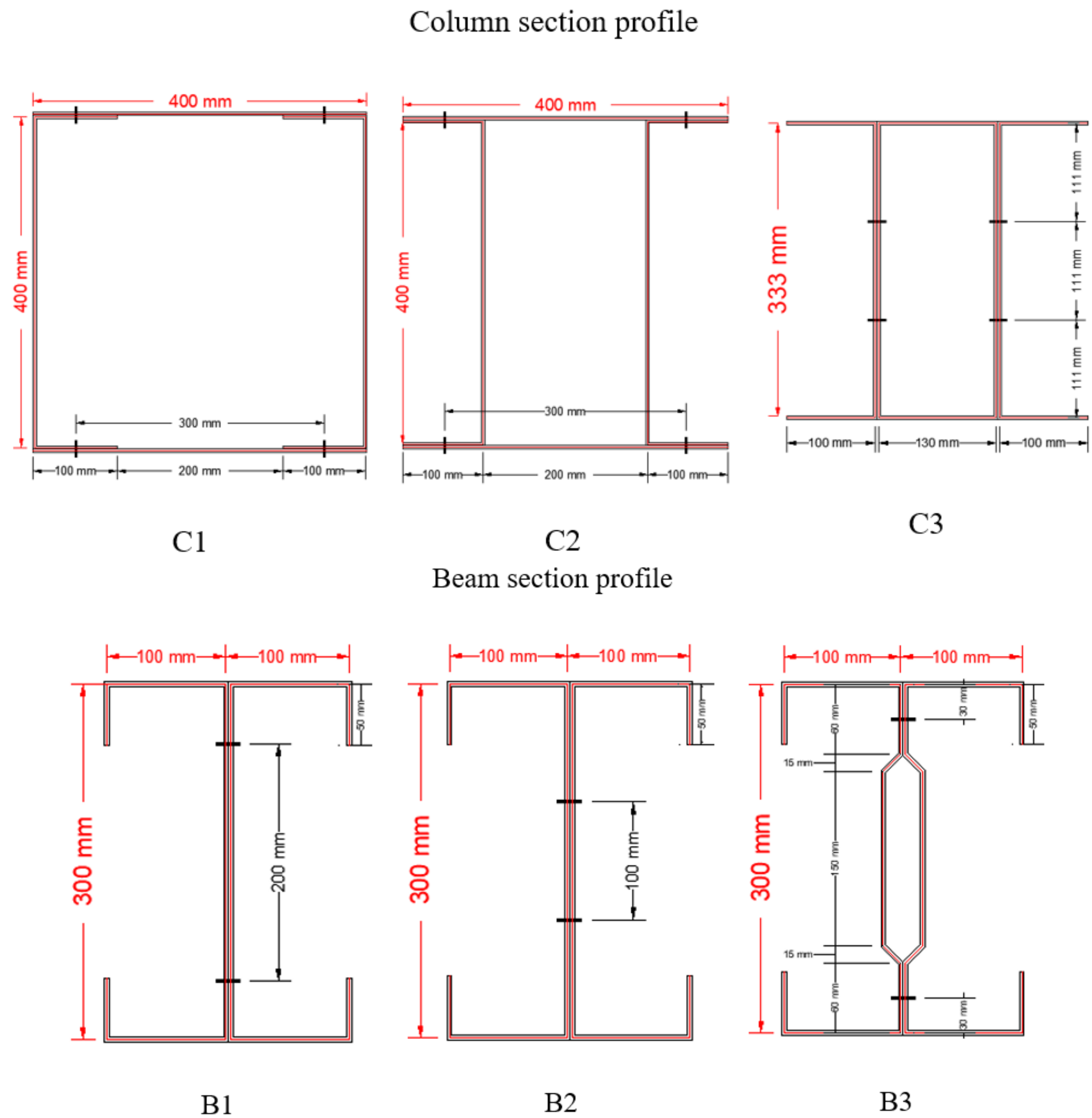


Fig. 5. Frame structure and a schematic diagram for each profile (continued)

TABLE II
DIFFERENT PROFILES IN SECTIONS USED IN THE PRESENT STUDY

COLUMN PROFILE

Cross section				Area (mm2)	Inertia x10^6 (mm^4)	Fastener location
C1	S 1	C 400 x 100 x 2	Pl 400 x 2	4000	0.121	In the middle of channel flanges
	S 2	C 400 x 100 x 2.5	Pl 400 x 2.5	5000	0.152	
	S 3	C 400 x 100 x 3	Pl 400 x 3	6000	0.195	
	S 4	C 400 x 100 x 4	Pl 400 x 4	8000	0.250	
C2	S 1	C 400 x 100 x 2	Pl 400 x 2	4000	0.121	In the middle of channel flanges
	S 2	C 400 x 100 x 2.5	Pl 400 x 2.5	5000	0.152	
	S 3	C 400 x 100 x 3	Pl 400 x 3	6000	0.195	
	S 4	C 400 x 100 x 4	Pl 400 x 4	8000	0.250	
C3	S 1	Rec 130 x 333 x 2	C 333 x 100 x 2	4000	0.0618	At one-third and two-thirds of channel web
	S 2	Rec 130 x 333 x 2.5	C 333 x 100 x 2.5	5000	0.0772	
	S 3	Rec 130 x 333 x 3	C 333 x 100 x 3	6000	0.093	
	S 4	Rec 130 x 333 x 4	C 333 x 100 x 4	8000	0.124	

Continue in the next page

TABLE II: Continued

GIRDER PROFILE

Cross section				Area (mm^2)	Inertia x10^5 (mm^4)	Fastener location
B1	S 1	C 300 x 100 x 2	C 300 x 100 x 2	2400	0.334	In the half a third of channel web from the edge
	S 2	C 300 x 100 x 2.5	C 300 x 100 x 2.5	3000	0.417	
	S 3	C 300 x 100 x 3	C 300 x 100 x 3	3600	0.546	
	S 4	C 300 x 100 x 4	C 300 x 100 x 4	4800	0.669	
B2	S 1	C 300 x 100 x 2	C 300 x 100 x 2	2400	0.334	At one-third and two-thirds of channel web
	S 2	C 300 x 100 x 2.5	C 300 x 100 x 2.5	3000	0.417	
	S 3	C 300 x 100 x 3	C 300 x 100 x 3	3600	0.546	
	S 4	C 300 x 100 x 4	C 300 x 100 x 4	4800	0.669	
B3	S 1	Σ 300 x 100 x 2	Σ 300 x 100 x 2	2400	0.334	At middle of a groped vertical part of sigma web
	S 2	Σ 300 x 100 x 2.5	Σ C 300 x 100 x 2.5	3000	0.417	
	S 3	Σ 300 x 100 x 3	Σ C 300 x 100 x 3	3600	0.546	
	S 4	Σ 300 x 100 x 4	Σ 300 x 100 x 4	4800	0.669	

TABLE III
DIFFERENT PARAMETERS IN THE PRESENT STUDY

Parameters used for CFS frame sections

Steel yielding strength (Fy) (Mpa)	240, 280 and 360	
CFS thickness (t) (mm)	2, 2.5, 3 and 4	
Fasteners spacing (a) (mm)	2 mm	40, 100, 200, 400, 600
	2.5 mm	50, 100, 200, 400, 600
	3 mm	60, 100, 200, 400, 600
	4 mm	80, 100, 200, 400, 600
cross sectional area (A) (mm^2)	Column	4000, 5000 ،6000 and 8000
	Girder	2400, 3000, 3600 and 4800

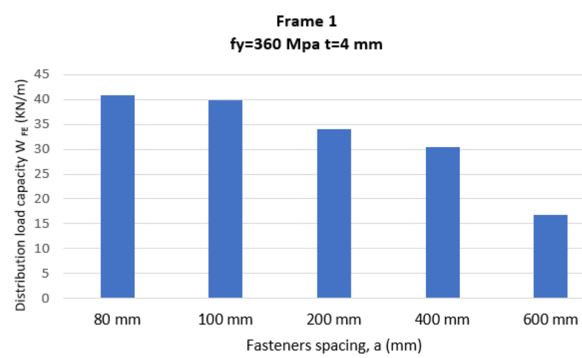
B. Finite Element Analysis Results

The distributed load capacity (W_{FE}) at CFS frames, which was calculated by the FE model, was affected by different parameters, as shown in Fig 7. As expected, steel yielding strength (Fy) directly affected the distributed load capacity. the same happens to CFS thickness (t), fasteners spacing (a), and cross-sectional area (A).

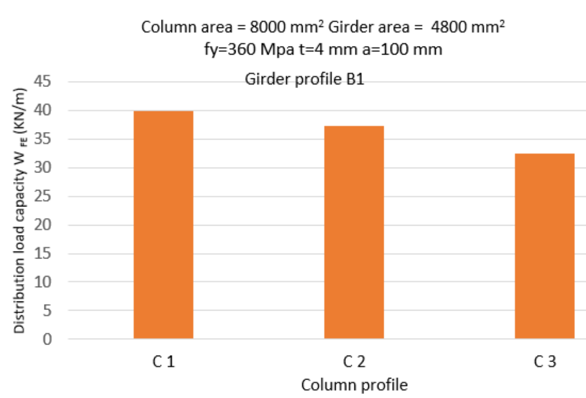
Comparing the distributed load capacity of frames with different column profiles (C1, C2, and C3) using the same girder and parameters showed that C3, whose area is the same, but the moment of inertia is different, its capacity is slightly less from C1 and C2, which give similar results, indicating that the moment of inertia of the column is influential in frame capacity, as expressed in Fig. 7 (b).

For load capacity, the total moment of inertia of the column section is ineffective, as shown in fig. 7 (d) while fig. 7 (e) indicates that the total moment of inertia of the girder section is effective on the load capacity.

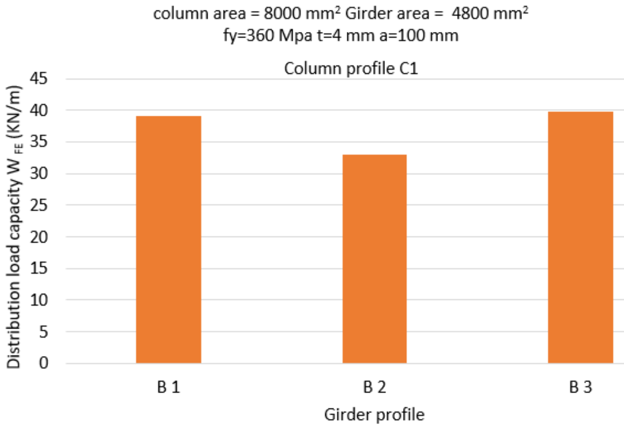
Figure 7 (f) shows that increased fastener spacing has a more noticeable effect in frames 7 and 8 than in frames 1 and 2. It is clear that frame 7 provides the uppermost load capacity, followed by frames 4 and 1. This finding results from using girder profile B1, which is fastened in the middle of the third of channel web from the edges.



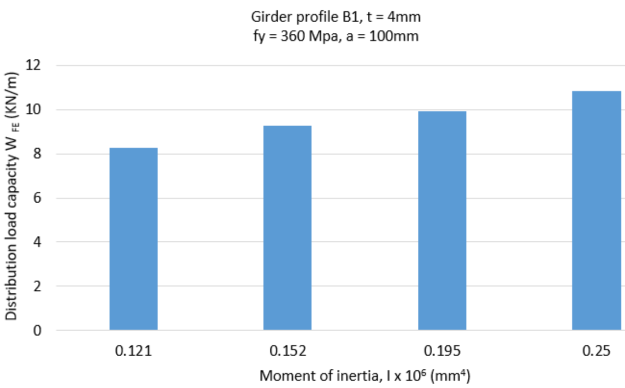
a) Fastener spacing, a



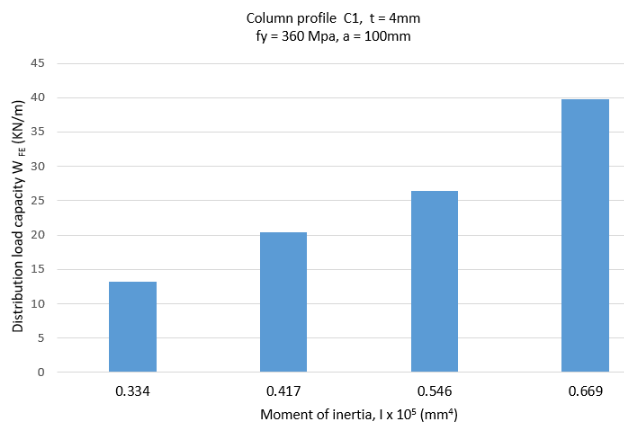
b) Column profile



c) Girder profile

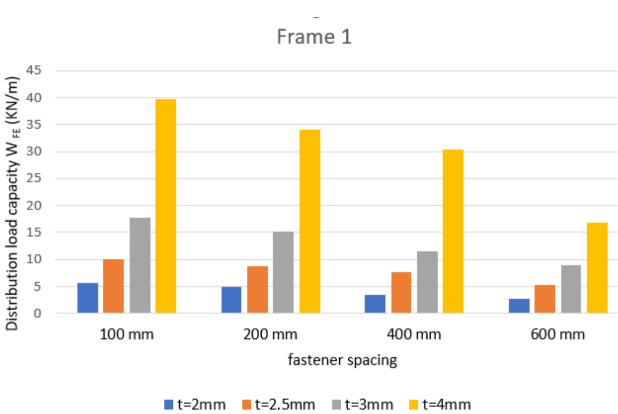


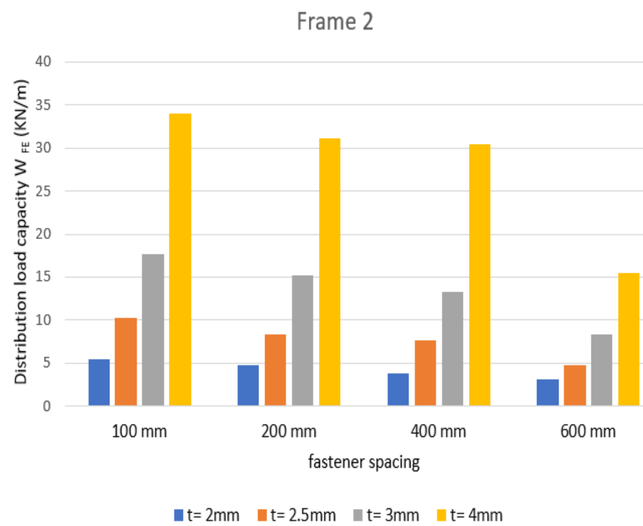
d) Moment of inertia of column, I

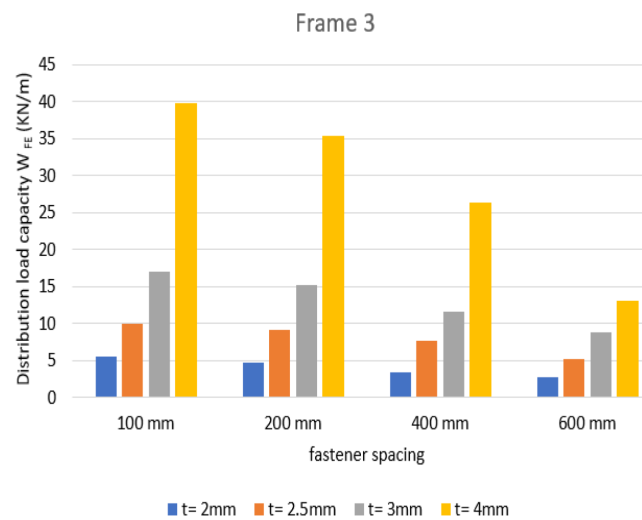


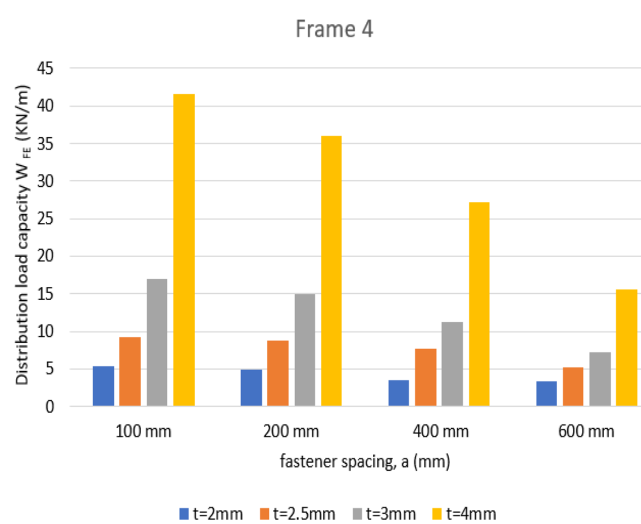
e) Moment of inertia of girder, I

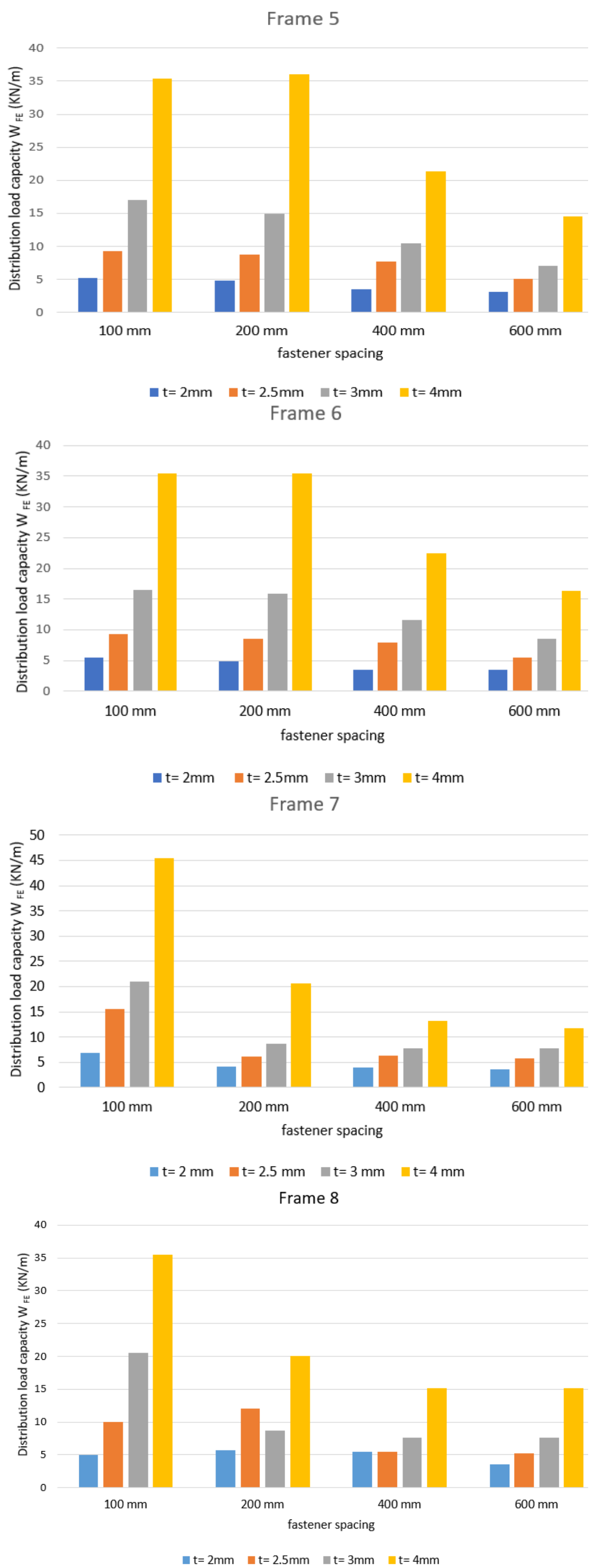
Fig. 7. Effect of different parameters on CFS frames

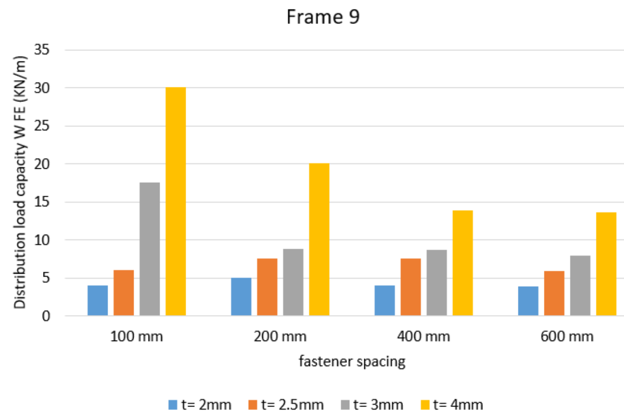












f) Fastener spacing and steel thickness for each frame

Fig. 7. Effect of different parameters on CFS frames

The load capacity of frames 3, 6, and 9 is similar. This finding is because girder profile B3 comprises Σ sections with a higher stiffness than the C channel section because the section's web is strengthened, while the channel's web is not.

Figure 7 (d) illustrates that the distributed load capacity of frames decreases while spacing between fasteners increases. However, increasing steel thickness influences the load capacity of frames in the opposite direction. No effect is noted on the load capacity as a result of using the spacing within AISI-S100 (2007) limits [21].

V. CONCLUSION

In this paper, nine frames with three several innovative profiles of CFS built-up columns and girders were described and investigated using a validated finite element model.

Regarding the distributed load capacity of each frame, a detailed parametric study was conducted on the frames. Depending on the outcomes of the parametric analysis, conclusions are made and stated below:

1- Distributed load capacity is directly related to steel yielding (Fy), and CFS thickness (t), and cross-sectional area (A), which are inversely proportional to fastener spacing (a).
2- From the profiles studied in this search, C1 provides the highest value of distributed load capacity as channels with plates present box section.
3- B1 and B3 give the same distributed load capacity, and B2 gives a minimal capacity.
4- Using fastener spacing in AISI-S100 (2007) limits is the perfect case as while the spacing between fasteners increases, the distributed load capacity of frames decreases.
5- Frame 7 provides maximum distributed load capacity as containing C3 column profile and B1 girder profile

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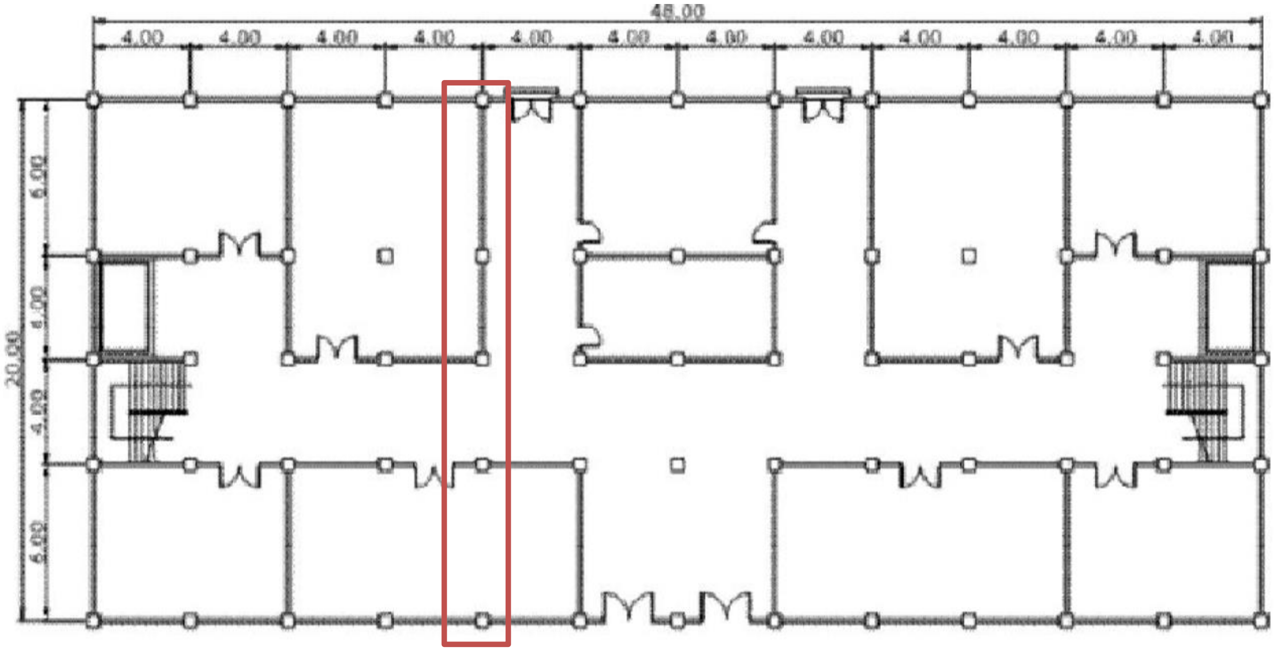
APPENDIX



Building plan provided by Doctolero and Batikha [17]

AUTHORS CONTRIBUTION

N.S. Mahmoud and F.A. Salem conceived the research strategies.

Aml Badr assembled input data, invalided the FE program, parametric studies, and numerical analysis.

N.S. Mahmoud and F.A. Salem supervised the program and conducted the observations.

N.S. Mahmoud, F.A. Salem, and Aml Badr analyzed the output of the program.

All authors discussed and validated the results and commented on the research results at all stages.

Aml Badr wrote the paper.

N.S. Mahmoud and F.A. Salem supervised and edited the paper.

The work reported in this publication was equally contributed by all authors.

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"There are no potential conflicts of interest concerning the research, authorship or publication of his article".

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Title Arabic

دراسة سلوك القطاعات المعدنية المشكَلة علي البارد في المنشآت الخفيفة.

Arabic Abstract

تستعرض هذه الدراسة سلوك القطاعات المشكَلة علي البارد في المنشآت الخفيفة، حيث تم دراسة ثلاثة قطاعات للعمود وثلاثة قطاعات أخري للكمرات المكوّنة للإطار المستخدم في البحث. تم استخدام أشكال مختلفة لتشكيل القطاعات ومسامير الربط الذاتي للتجميع بين أجزاء القطاع. طبقت طريقة العناصر المحدودة لحساب أقصي حمل موزع يتحمله الأطار تحت تأثير عدة عوامل متغيرة،هي: تخانة القطاعات، و درجة الحديد المستخدم، و قطاعات الأعمدة، و قطاعات الكمرات و المسافات بين المسامير.