JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Buckling Behavior of Built-up Cold-Formed Steel un-lipped rectangular Channel column Sections Under Axial compression

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering

By:

Adefris Alemu

JANUARY, 2022

JIMMA, ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

Buckling Behavior of Built-up Cold-Formed Steel un-lipped rectangular Channel column Sections Under Axial compression

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering

By:

Adefris Alemu

Main Advisor: Eng. Elmer C.Agon (Asso. Prof.)

Co-Advisor: Eng. Rihana Ibrahim (MSc.)

JANUARY, 2022

JIMMA, ETHIOPIA

DECLARATION

The entirety of the work contained herein is my own, original work and has not been submitted previously in its entirety or in part for obtaining any qualification at any other university or college. The thesis document contains no materials previously published except were due reference is made.

Candidate name:	Signature	Date
Adefris Alemu	…………………………	08/02/2022

This thesis has been submitted for examination with my approval as university supervisor.

Name	Signature	Date
Engr. Elmer C.Agon (Asso. Prof.) (Main Advisor)	…………………………	08/02/2022
Engr. Rihana Ibrahim (MSc.) (Co-Advisor)	…………………………	08/02/2022

APPROVAL SHEET

This thesis has been submitted for examination with my approval as university supervisor. The thesis titled **"Buckling Behavior of Built-up Cold-Formed Steel un-lipped rectangular Channel column Sections Under Axial compression"** by Adefris Alemu is approved for the degree of Master of Science in Structural Engineering.

Approved by Board of Examiners:

We, the undersigned, approve the Master's Thesis of Adefris Alemu

Name	Signature	Date
Engr. Elmer C.Agon (Asso. Prof.) (Advisor)	....	08/02/2022
Engr. Rihana Ibrahim (MSc.) (Co-Advisor)	....	08/02/2022
Engr. Diosdado john N.Corpuz (Asso. Prof) (Internal examiner)	....	09/02/2022
Dr. Binaya Patnaik (External examiner)	....	27/01/2022
Engr. Habtamu G/Medhin (MSc.) (Chair person)	....	08/02/2022
Engr. Abinet Alemseged (MSc.) (Structural Engineering chair)	....	....

ABSTRACT

Cold-formed steel structures can be used extensively in the building industry, either as complete structures in low to mid-rise construction, or in combinations with hot-rolled or fabricated steel framing. Cold formed steel (CFS) has been used as the primary structure for flexural and compression load-bearing members such as bridges, roof trusses, transmission towers, and other multi-storied buildings due to varieties of advantages such as high strength to weight ratio, high corrosion resistance, and ease of fabrication. Dual-channel sections, built-up of CFS into new member such as back to back without gap, back to back with gap, laced, battened or perforated plates with higher strength can be produced efficiently by attaching the CFS to each other at few places along the lengths using connectors. Built-up cold-formed steel sections commonly used as compression members to carry heavier loads and over longer spans when a single individual section is insufficient. These built-up columns improve the lateral stiffness when two individual channels shapes connected together thus preventing the structural member from wobbling during lifting and installation. Un-lipped rectangular CFS channel sections with constant size has been used to produce built-up, back to back (BTB), and face to face box-up (BU) with 6(0.6 to 3meter) varieties of length.

The double built-up back to back connected on the web and face to face sections connected at flanges were built-up from two identical un-lipped rectangular channel-sections attached with surface to surface tie constraint. The connector spacing is constant along the length of the column at 400 mm centre to centre with 5x5mm square surface area contact size. Concentrated centric unit load was applied at the centre of gravity in the reference point at the column top end. In total of 18 columns, 6 single, 6 back to back and 6 face to face built-up CFS un-lipped rectangular channel sections of size 200x80x4-section were modeled and analyzed on the axial capacity and mode of failures, and reported herein. The ultimate loads were compared to the experimental test results.

FEA simulation was conducted in ABAQUS 6.13-1/ABAQUS CEA software for different values of slenderness ratios covering from short to long columns on the axial capacity of single and built-up CFS channel sections. Initial imperfections were not considered for both single and built-up CFS sections. Load-axial capacity, failure modes, and deformed shapes at failure were discussed for all CFS columns. Because of the low thickness to width ratio, the members buckled at stresses that are lower than the yield stress when compressive forces are applied. Analysis results show that all short columns failed through local buckling. However, for long columns global buckling was observed. The structural response of the built-up CFS columns composed of two identical un- lipped channels in this study was significantly affected by the member's slenderness ratio and channels orientation. The orientation of the column also substantially impacts the ultimate load and buckling mode shapes of the dual built-up CFS columns.

Keywords: *ABAQUS, Cold formed steel (CFS), Built-up column, Back to back (BTB) column, face to face box-up (BU) column.*

ACKNOWLEDGEMENT

First of all, thanks to God for keeping me in a full health to stand as human being, and for each and every success in my life.

Grateful thanks to all peoples, to those who have contributed and helped me during this thesis development and documentation process. Special thank for the valuable support, guidance and suggestions to the development of this thesis paper from Eng. Elmer C.Agon (Associate Professor) and Eng. Rihana Ibrahim (MSc.) in Jimma University institute of technology at the division of Structural Engineering.

Thanks also extended to my friends and family for their support and help during this journey from the beginning to end.

At last but not least, my gratitude and appreciation goes to Jimma University institute of technology, department of civil and Enviromental Engineering (structural stream) and my second family, Jimma university community for all their support and contributions.

Table of Contents

LIST OF TABLES

LIST OF FIGURES

ABREVATIONS AND ACCRONYMS

A	Area of section (cm^2)
BTB	Back to back sections orientation
BU	Face to face, box-up sections orientation
C_y(CG)	Centre of gravity (cm)
CFS	Cold-formed steel
CEA	Complete ABAQUS Environment or Computer Aided Engineering
C	Single plain channel
EBCS	Ethiopian building code of standard
E	Young's modulus of elasticity (N/mm^2)
FE	Finite element
FEA	Finite element analysis
G	Shear modulus of elasticity (N/mm^2)
L	Column height or span between ends supports (mm)
L_{eff}	Effective buckling length (mm)
R_i	Root radius (mm)
X_o	Shear centre (cm)
i	Radius of gyration (mm)
v	Poisson's ratio in elastic stage
λ_c	Slenderness ratio (Leff / i),
p	Density (kg/m^3)

CHAPTER ONE

INTRODUCTION

1.1.Background of the Study

In steel construction, basically there are two types of structural shapes: hot-rolled and cold-formed steel shapes. Hot-rolled steel shapes are formed by rolling under significantly elevated temperatures (in essence above 900°c), while cold-formed steel shapes are formed at ambient or room temperature (Abhishek Dangi., 2017). Cold-formed steel shapes are manufactured by means of folding, cold-rolling (roll forming), which consists in a continuous bending operation of a long strip coiled steel metal into the required cross-section, and CFS shapes can also be obtained by press-braking of plates, given the fact that the maximum length of the member to be manufactured cannot be larger than the length of the press-brake. Sheet steel used in cold-formed shapes is typically (0.9 to 8 mm) thick. Cold working of the steel increases its yield strength but also lowers its ductility. For example, a 20% reduction in thickness can increase yield strength by 50% but reduces elongation to as little as 7%, which probably represents the limit of formability for simple shapes (L.H. Martin.,J.A. Purkiss., To EN 1993 and EN 1994). Cold-forming operations also increase the yield point and tensile strength by about 20% to 40%, depending on the type of manufacturing (Abhishek Dangi., 2017). The main benefits of using a cold-formed section are such as, lightness in weight, high strength to weight ratio and stiffness, ease of prefabrication and fast installation, ability to be not shrink and not to creep at ambient temperatures , capability to be recyclable, economy in transportation and handling, erection and construction of foundation, and due to ease of manufacturing at ambient temperature, cold-formed elements are available in a variety of complex structural shapes to fit the demands of optimized design.

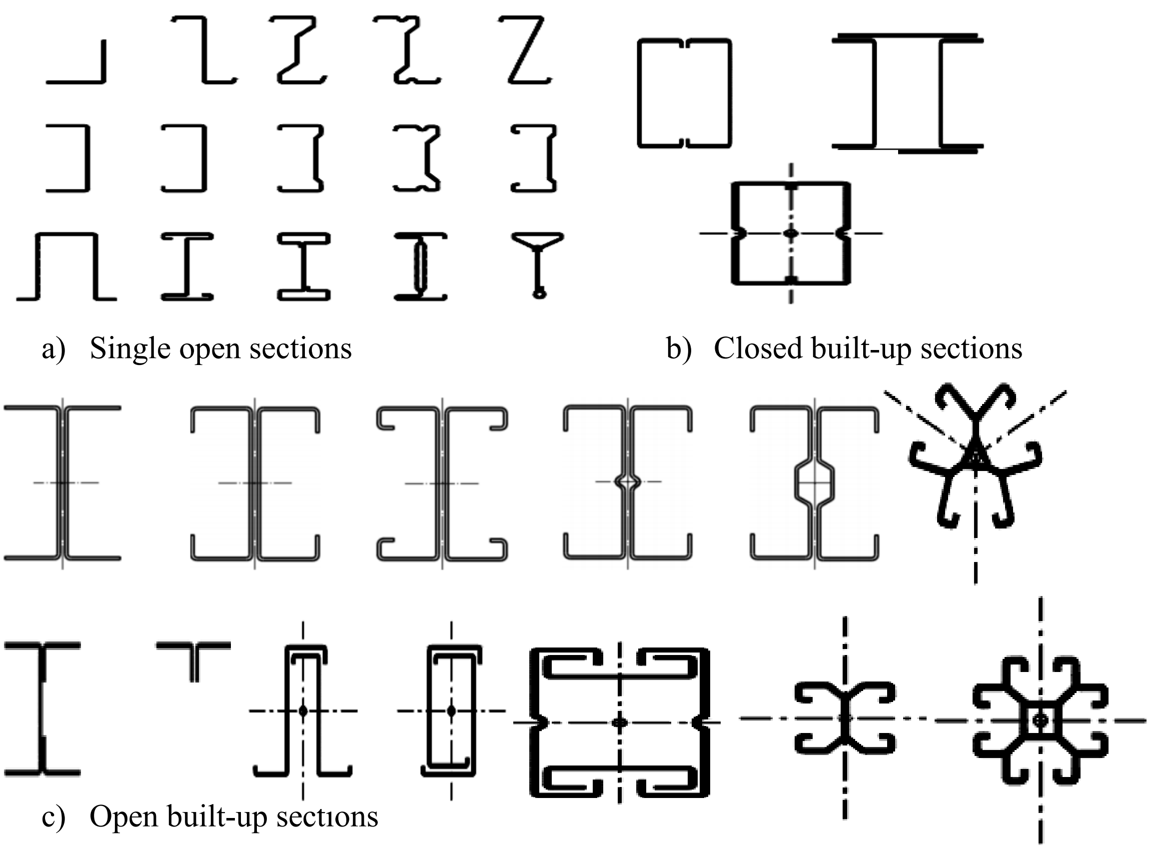
The prime difference between the behavior of cold-formed shapes and hot- rolled structural shapes is that cold-formed members involve thin plate elements which tend to buckle locally under compression. The concept of CFS elements is to use shape rather than the thickness to support the load. The secondary difference is that cold-formed members have low lateral stiffness and low torsional stiffness because of their open thin cross-sectional geometry, which gives great flexural rigidity about one axis at the expense of low torsional rigidity and low flexural rigidity about a perpendicular axis. Because of the open cross-section, channel shape components are susceptible to failure modes of local, distortional and global buckling under load. In the other hand, its high slenderness and low torsional stiffness brings a more complex analysis, due to the non-coincidence of the centre of gravity and the shear centre of the major part of these sections. One of the most common types of singly-symmetric section is channel (Ben young and Gregory J.Hancock., 2005).The mono symmetric sections have relatively small torsional rigidity that is weak in twisting compared to doubly symmetric sections. When distortional buckling occurs, the flange and lip will rotate around the intersection of the flange and the web, which will reduce the bearing capacity of the component, so it needs to be prevented. This leads to cold-formed members to distortional buckling and flexural–torsional buckling (L.H. Martin.,J.A. Purkiss., To EN 1993 and EN 1994).

When compared to a highly used structural steel member, hot-rolled steel for instance, CFS shapes prove to be a strong solution due to its accurate detailing, in which provides a considerable number of different types of cross-section that gives flexibility in design. CFS sections have been manufactured and used in construction for more than a century. The use of cold-formed steel sections in building construction began in 1980"s, where United State and Great Britain were the first countries to use this technology. However, in recent years, higher strength materials and development in structural applications have led to a significant growth of CFS in industrial, residential, agricultural and commercial applications relative to the conventional hot-rolled steel. It can be used as individual structure framing member in shape of Channel section, Z-section, I-section (Bharat C.Halagalimath.and Kiran Koraddi., 2019). Improved production of complex shapes,since modern rolling lines are computer controlled,the highly accurate complex sections can be made (Ranawaka Thanuja., 2006.).

Cold-formed steel structures are used extensively in the building industry, either as complete structures in low to mid-rise construction, or in combinations with hot-rolled or fabricated steel framing. Also used for load-bearing members such as bridges, roof trusses, transmission towers, and other multi-storied buildings. The cold- formed steel (CFS) is higher strength resulting in the reduction in dead weight. Due to the weight ratio and ease of construction, cold-formed light-gauge steel structural members have been widely used in steel-frame residential homes, low-rise office buildings, and industrial warehouses and in load-bearing walls. When a building is no longer needed it can be disassembled, stored or moved to another location and re-erected because only simple connections are used. Cold-formed steel structural members may lead to a more economic design than hot-rolled steel members as a result of their superior strength to weight ratio and ease of construction. By using cold- formed system economy is achieved with completion of project in minimized time. In industrial building the material & cost of the building is minimized in case of cold formed steel while in case of conventional hot-rolled steel industrial building higher both in two cases (Roshan S Satpute.and Valsson Varghese., 2012).

Dual-channel sections are frequently utilized as built-up columns, which are connected to each other at few places along the lengths by means of a connector such as back to back without and with gap, laced, battened or perforated plates. Cold-formed steel built-up sections are commonly used as compression members to carry heavier loads and over longer spans when a single individual section is insufficient. These built-up columns improve the lateral stiffness when two individual C-channels are connected together thus preventing the structural member from wobbling during lifting and installation. Built-up sections are now finding increasing use in Australia, North America and Europe, for example as primary structural members of mid-rise residential buildings and large-span portal frames. Built-up members are fast becoming the next generation of cold-formed steel structural elements and are producing innovative and efficient new solutions in mid-range and mid-span construction. Yet, current specifications offer very limited guidance for the design of built-up cold formed steel members (K. J.R. Rasmussen., etal. 2019). There are no specific guidelines and the only guidelines available are presented in most recent normative published for Europe was the European Standard Eurocode 3: Design of Steel Structures. Part 1-3: General Rules. Supplementary rules for cold formed thin gauges members and sheeting (EN 1993-1-3:, 2004).

Built-up CFS is usually a composition of different sizes and shapes of normal CFS - Channel, Z-section , hat or sigma -sections to produce a new section. In modern construction, the usage of CFS columns in frame structure has been applied in residential construction for building up to double story. Built-up section can simply gain higher stability and capacity due to double of the sections produce greater cross-section properties, the symmetry of built-up section can eliminate the eccentricity between the shear centre and centre gravity of single section consequently eliminate certain buckling effects in essence, distortional buckling and out of plain movement. Closed box sections allow spanning greater distances between supports and carrying heavier loads than single channel-sections (W. Reyes.and F. A. Guzman., 2010). Typical forms of cross-sections for cold-formed members includes channel, lipped channel, lipped channel with double edge fold stiffeners, lipped channel with intermediate web stiffener, sigma-section, z-section and double profiles from above sections (back-to-back and) as shown in Figure 1.1



a) Single open sections b) Closed built-up sections

c) Open built-up sections

Figure1.1.Typical forms of cross-sections for cold-formed members (EN 1993-1-3:, 2004).

In this research CFS- single plain channel (C), channels connected on the web known as back to back (BTB) and connected at the flange known as face to face box-up (BU) sections are considered. In such an arrangement, independent buckling of the individual channel-sections is prevented by the connectors. Such built-up box CFS sections are used in the construction due to the advantages of high load-carrying capacity, stability, and higher moment of inertia, when compared to the back-to-back built-up CFS channel sections (Krishanu R. et al, 2019).

However, the design of cold-formed members differs from that of conventional steel structures and therefore need special considerations. In most cases cold-formed members exhibit complex behavior governed by interacting local and global stability phenomena. In addition to the buckling modes present in hot-rolled steel members, the CFS members are usually subjected to other buckling modes such as, local, distortional, and flexural-torsional. Because of the low thickness to width ratio, the members will buckle at stresses that are lower than the yield stress when compressive forces are applied. One of the biggest difficulties with cold-formed steel design is the prevention of member buckling, which is unlike the behavior of hot-rolled steel where steel yielding is the leading design consideration. Yielding is mainly an issue for compact and short columns, which causes the failure of the entire column. For longer columns, it is likely that buckling will control rather than yielding of the member. There are multiple factors that can cause a compression member to buckle: slenderness ratio, which is the member length divided by the least radius of gyration, end condition of the member, length, eccentricity of the load, and imperfections within the material and geometry. Buckling can occur both elastically and in elastically as shown below (Thomas H.etal, 2013). Elastic buckling of cold-formed steel members under flexure or axial load may involve three distinct buckling modes: local, distortional, and overall or global (W. Schager Benjamin., 2002).

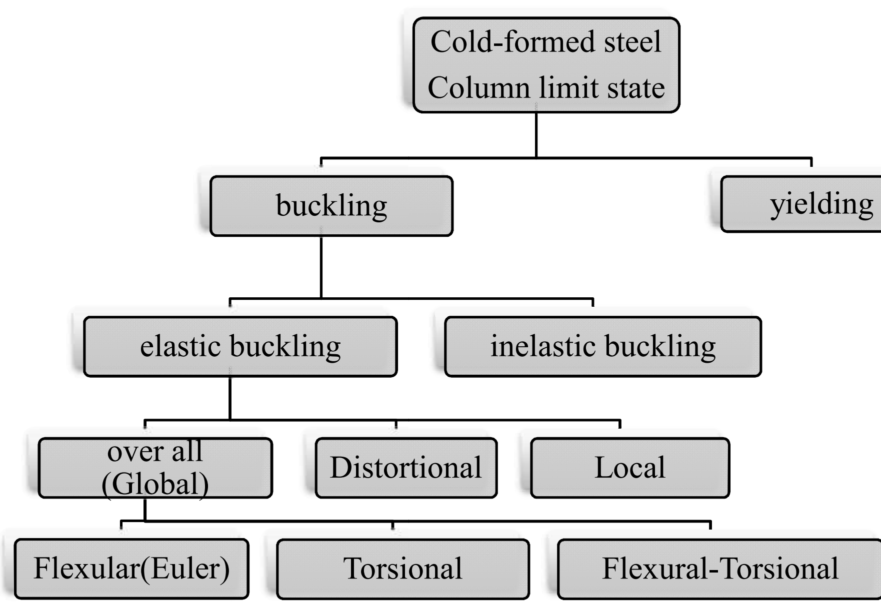


Figure1. 2.Cold-formed steel column limit states

Inelastic buckling often occurs in stocky and intermediate columns because these columns have slenderness ratios that are in the small to moderate range. Among various buckling modes, distortional buckling is a recently been investigated (Whittle, J. and Chris Ramseyer., 2009).It is a buckling of the compression flange acting as a group of plates rather than as individual plates. The distortional buckling was less understood than the other forms of instability, and was deemed more pertinent to thinner sections of high strength steel. The importance of distortional buckling should not be underestimated as it is just as likely to occur as local buckling. Local buckling is often seen as a rippling effect in the web, flange, or lip along the length of the member. The need for research of distortional buckling has increased as cold-formed design has shifted towards increasingly stiffened and slender sections that demand a more thorough evaluation of local and distortional buckling. The objective of this study is to determine the ultimate load of single and built-up CFS channel columns with constant connector spacing along lengths with constant size using FEA in ABAQUS validated with test results.

1.2.Statement of the Problem

The use of CFS -columns has been observed in the construction industry, this has instigated the researcher to determine the ultimate capacity of CFS channel column sections. In recent years, higher strength materials and development in structural applications have led to a significant growth of CFS in industrial, residential, agricultural and commercial applications relative to the conventional hot-rolled steel. Improved production of complex shapes,since modern rolling lines are computer controlled,the highly accurate complex sections can be made of cold-formed steel structural members may lead to a more economic design than hot-rolled steel members as a result of their superior strength to weight ratio and ease of construction. Dual channel cold-formed steel sections built-up in back to back (BTB) and face to face box-up (BU) orientations were commonly used as compression members to carry heavier loads and over longer spans when a single individual section is insufficient. Built-up section can simply gain higher stability and capacity due to double of the sections produce greater cross-section properties, the symmetry of built-up section can eliminate the eccentricity between the shear centre and centre gravity of single section consequently eliminate certain buckling effects in essence, distortional buckling and out of plain movement. Thus, it is important to analyze the effect of different orientation on the dual system to the performance of the column.

However, cold working of the steel increases its yield (yield point and tensile strength) but also lowers its ductility. The unique properties of thin-walled cold-formed C-section members originate from three factors: the fabrication process, the small thickness and high slenderness of the elements of the cross-section. Cold-formed sections are normally thin; consequently they have low torsional stiffness and the sections produced by cold-forming are mono-symmetric; their shear centre eccentric from their centre of gravity of sections. For columns axially loaded along their centroidal axis, the eccentricity of the load from the shear centre axis may cause buckling in the flexural-torsional mode at a lower load than the flexural buckling mode. Hence, checking for the flexural-torsional mode of buckling is necessary for such mono-symmetric columns. Buckling analysis has become a critical aspect, especially in the safety engineering design since, at the time of failure; the actual stress at the point of failure is significantly lower than the material capability to withstand the imposed loads. Because of cold-formed members involve thin plate elements were susceptible to failure modes of local buckling, distortional and global buckling under load. CFS open thin cross-sectional geometry gives great flexural rigidity about one axis at the expense of low torsional rigidity and low flexural rigidity about the perpendicular axes in which the flange will rotate around the intersection of the flange and web will reduce the bearing capacity of the component and leads to distortional and flexural-torsional buckling. In most cases cold-formed members exhibit complex behavior governed by interacting local and global stability phenomena. Due to the inherent presence of imperfection, buckling mode interaction always occurs in case of thin-walled members. It is a consequence of the increasing complexity of section shapes that local buckling calculation are becoming more complicated and that distortional buckling takes on increasing importance.

In this research CFS- single (C), and dual built-up connected on the web known as back to back (BTB) and connected at the flange known as face to face box-up (BU) rectangular un-lipped channel column sections were considered. Because of the low thickness to width ratio, the members will buckle at stresses lower than the yield stress when compressive force is applied. The concept of CFS elements is to use shape rather than the thickness to support the load. One of the biggest difficulties with cold-formed steel design is the prevention of member buckling, which is unlike the behavior of hot-rolled steel where steel yielding is the leading design consideration. In addition to the buckling modes present in hot-rolled steel members, CFS members can usually subjected to other buckling modes such as, local, distortional, and flexural-torsional. In the other hand, its high slenderness and low torsional stiffness brings a more complex analysis, due to the non-coincidence of the centre of gravity and the shear centre on the major part of these sections.

1.3. Research Question

i. What is the ultimate buckling capacity of single and dual built- up cold -formed steel channel column sections?
ii. What is the effect of channel orientation in ultimate buckling capacity and mode of failure on build-up cold-formed steel column sections?
iii. What is the improvement in ultimate buckling capacity and mode of failures after single channel(C) sections oriented to dual built-up back-to-back (BTB) and face-to-face box-up (BU) channel column sections?

1.4.Objectives of the Study

1.4.1. General Objective

❖ The main objective of this research is assessing the buckling behavior of single and dual built-up cold-formed steel (CFS) channel column Sections (ultimate buckling capacity and mode of failures).

1.4.2. Specific Objectives

i. To determine the ultimate buckling capacity of single and dual built-up cold-formed steel channel column Sections
ii. To examine the effect of channel orientations in ultimate buckling capacity and mode of failure on build-up cold-formed steel column sections
iii. To compare improvement in ultimate buckling capacity and mode of failures of single channel(C) with dual built- up back-to-back (BTB) and face-to-face, box-up (BU) channel column sections

1.5.Significance of the Study

Understanding the behavior and strength of dual built-up cold-formed steel channel column sections is important, as they used frequently in frames as higher load carrying capacity columns and shear wall chord studs, among other applications. The output of this research is expected to provide necessary data and information that motivate other researchers for further investigation on ultimate capacity and failure modes of cold-formed steel (CFS) channel column sections. It motivates other researchers to research on solutions for other more complex and innovative CFS shapes that will become a solution to complex behavior governed by interacting local and global stability phenomena of open sections. It will also increase the understanding of the significant impact of distortional buckling and its interaction with other mode of failures in CFS.

After completion and approval of the research, the result will be utilized in different structures such as industrial, residential, agricultural and commercial building applications, may lead to a more economic design than hot-rolled steel members as a result of their superior strength to weight ratio and ease of construction. On the other hand, this research can be a reference for academic researchers or students to those who wants to carry out their study on cold-formed steel (CFS) columns. The members with complex cross-section are potential solutions for structural problems where enhanced load-bearing capacity is needed, for instance in situations where no lateral support can be provided to the member; members with a double built-up sections arrangement can be used as an effective solution in engineering structures. The results on the complex arrangements provide insight, how detailing affects the structural behavior. Improved understanding about the distortional buckling behavior of CFS shapes. Enhancing knowledge and understanding on buckling behavior and ultimate capacity of un-lipped rectangular CFS channel sections subjected to concentric loading. This research also provides a solid framework allowing future researchers and practitioners to further investigate the behavior of built-up columns.

1.6.Scope of the Research

In this analytical investigation, ABAQUS 6.13-1(2013)/CEA simulation software is used in order to check the performance of the un-lipped rectangular CFS channel sections. Both single and built-up un-lipped rectangular channel columns were analyzed using the simulation software. In this software simulation single (C) and dual built-up CFS channels connected on the web known as back to back (BTB) and connected at the flange known as face to face box-up (BU) columns simulated with pin-end conditions. Un-lipped rectangular CFS channel of size 200 x 80 x4mm cross-section was used for the model creation of both single and build-up column. The lengths of the columns were 600mm to 3000mm. The center to centre fastener spacing was 400mm along the length, one fourth of the wed width (50mm) for back to back and half width of the flange (45mm) for face to face box-up. All dimensions and cross-sectional properties were taken from un-lipped rectangular channel sections catalogue (IS:811-1987, 2011).This research examines the parameters that are associated with the structure performance which includes the fastener spacing, cross-section of the column, material non-linearity and column length involved in finite element modeling. Furthermore, the research concentrates on the local, distortional and overall buckling. In comparing FEA software result with the experimental test results, improvement in ultimate load and mode of failures due to the channel sections orientation was considered. The models serve to evaluate the test results and observations from the experimental investigation. These models were then be used to generate more data on back-to-back and face-to-face box-up CFS channel columns without a gap.

1.7. **Limitation of the Research**

This thesis is limited to concentric loading character from short to long cold-formed steel (CFS) channel column sections. There is no experimental investigation (testing) of the section shapes and materials. Load eccentricities, maximum initial imperfections and residual stresses were not `taken in to consideration. In this paper, the safety factors were not included in calculating the buckling strength and mode of failures. This research was limited to the open un-lipped rectangular channel CFS-section compression members only. The end boundary condition at the bottom and top is considered to be pinned connection only. The column was modeled applying an assumption that its cross section is subjected to axial force only. The load was applied on top of the column axially at the centre of gravity without loading eccentricity.

CHAPTER TWO

REVIEW OF RELATED LITERATURE

2.1. Material properties of cold-formed steel

The introduction of structural steel, circa 1856, provided an additional building material to stone, brick, timber, wrought iron and cast iron (Bernard Godfrey., 2003). Steel is the most efficient material for slender columns due to its high strength, ductility, stiffness (modulus of elasticity), faster erection and resistance. In carrying out the analysis, the following material properties of structural steel coefficients were used.

Table2. 1.Input material property data for cold-formed steel (EBCS EN 1993-1-1:, 2013)

S.No	Name of property	Value
1	Modulus of Elasticity, E	210 000 N/mm^2
2	shear modulus, $G = \frac{E}{2(1+V)}$	81000 N/mm^2
3	Poisson's ratio in elastic stage, v	0.3
4	Density, p	7850 kg/m3

Table 2.2.Input values for plastic behavior of cold-formed steel (calculated in appendix)

True Stress(MPa)	Plastic strain
355.6	0
564.0	0.181

All steels used for cold-formed members and profiled sheets should be suitable for cold-forming and welding, if needed. Steels used for members and sheets to be galvanized should also be suitable for galvanizing. Because material properties play an important role in the performance of structural members, it is important to be familiar with the mechanical properties of the steel sheets, strip, plates, or flat bars generally used in cold formed steel construction before designing this type of steel structural member (Wei-wen Yu., 2000).

Mechanical properties of cold-formed steel sections differ from the commonly used hot- rolled steel sections, because of cold-formed steel sections are influenced by its manufacturing process as described earlier and need to be inspected accurately to study the behavior of cold-formed sections. The stress-strain curve is one of the important properties used to describe the behavior of hot- rolled and cold-formed steel. The stress-strain curves for hot-rolled and cold-formed steels are different, and the reason lies in the manufacturing process. .For hot-rolled steels, the yielding point is well defined by the stress-strain curve becomes horizontal; in the case of cold-formed steels the yielding point is not so clear, as the stress-strain curve is gradually growing reaching the yielding stage without a visible peak as indicated in Figure 2.1.below.

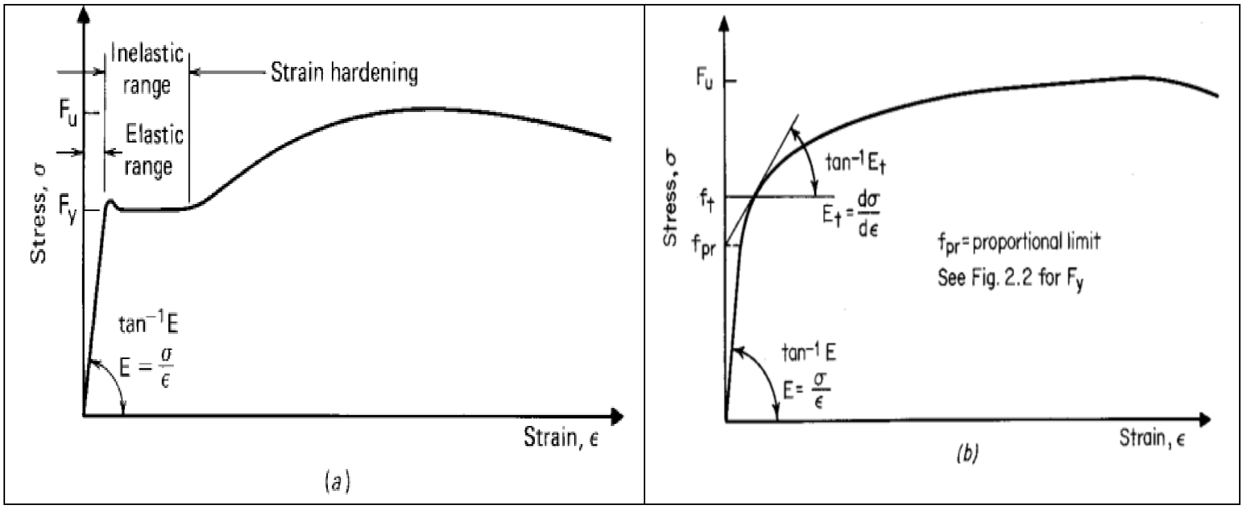


Figure 2.1.Stress-strain curves of a) Hot-rolled steels: Sharp-yielding and b) cold-formed steels: Gradual-yielding (Siti Nur Rahmah Anwar.etal.2020)

Stress-strain curves are usually presented as:

a) Engineering stress-strain curves, in which the original dimensions of the specimens are used in most calculations.
b) True stress-strain curves, where the instantaneous dimensions of the specimen at each point during the test are used in the calculations. This results in the "true" curves being above the engineering curves, notably in the higher strain portion of the curves.

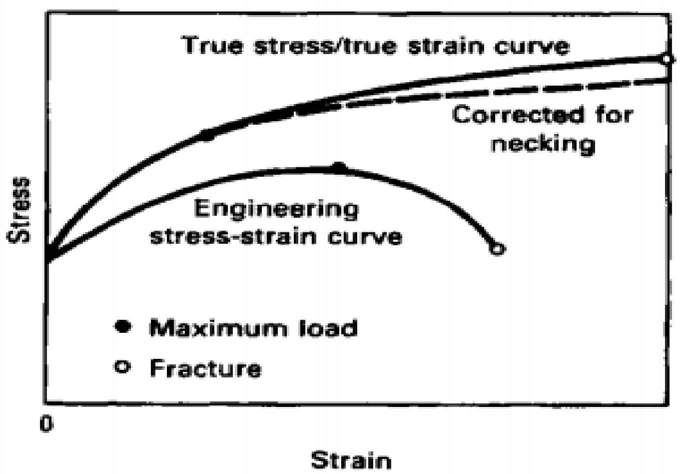


Figure 2.2.Comparison of engineering and true-stress/true-strain curves (Moosbrugger Charles., 2002)

2.2. Buckling behavior of cold-formed steel (CFS) compression members

The behavior of cold-formed steel structures is more complex than that of traditional hot-rolled steel structures. Research between 1940 and 1950 has highlighted that cold-formed steel members are subjected to various buckling modes including local, distortional and global modes, and their ultimate strength behavior is governed by these buckling modes. As an example, a short concentrically loaded C-section almost always fails because of a combination of local buckling of thin plate elements and distortional buckling of the edge stiffeners while the failure mode of longer columns is often governed by a combination of global buckling and local or distortional buckling. Although cold-formed steel members have been researched for a long time, the stability problem is not fully understood (Chernova Anna., 2019). Mainly, three types of buckling behavior can be observed in cold-formed steel compression members. They are called, local, distortional and flexural or flexural torsional buckling. In addition to these three main types their interactions can also occur as local and distortional, local and flexural and distortional and flexural (Ranawaka Thanuja., 2006.).

1. Local buckling: Cold-formed members are usually very thin, and thus the thin plate elements tend to buckle locally under compression. When an element buckles locally it does not necessarily mean that this element will collapse or loss its ability of carrying loads. In fact, a plate can be allowed to take a considerably increased load beyond initial buckling before any danger of collapse occurs. This is because the deflections due to buckling are accompanied by stretching of the middle surface of the plate. The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength, which depends on; the shape of the cross section, the slenderness ratio of the column, the type of governing overall column buckling (flexural buckling, torsional buckling, or torsional–flexural buckling), the type of steel used and its mechanical properties, influence of cold work, effect of imperfection and welding, effect of residual stress, interaction between plane components ,and effect of perforations. The yield point and tensile strength of the material vary from place to place in the cross section due to the cold forming and affects the local buckling behavior of cold-formed steel column (Wei-wen Yu., 2000).
2. Distortional buckling**:** Distortional buckling involves both rotation and translation at the corners of the cross-section (L.H. Martin.,J.A. Purkiss., To EN 1993 and EN 1994).Distortional buckling mode is a relatively new and less researched buckling mode compared to the other buckling modes. In cold-formed sections, it is characterized by relative movement of the fold-lines. It is a consequence of the increasing complexity of section shapes that local buckling calculation are becoming more complicated and that distortional buckling takes on increasing importance (EN 1993-1-3:, 2004). The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength.
3. Overall column buckling: For open thin-walled sections three modes of failure are considered in the analysis of overall instability (flexural buckling, torsional buckling, and torsional–flexural buckling). It is sometimes termed "rigid-body" buckling because any given cross section moves as a rigid body without any distortion of the cross section. Flexural column buckling (bending about a minor principal axis): A slender axially loaded column may fail by overall flexural buckling if the cross section of the column is a doubly symmetric shape (I-section), closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape (Z-shape or cruciform). For singly symmetric shapes, flexural buckling is one of the possible

failure modes. Torsional buckling (twisting about shear center): Usually, closed sections will not buckle by torsional because of their large torsional rigidity. This type of buckling may occur when the torsional rigidity of member significantly smaller than its flexural rigidity. The mono symmetric sections have relatively small torsional rigidity that is weak in twisting compared to doubly symmetric sections. Thin walled members with open cross sections are particularly susceptible to torsional buckling. Flexural-torsional buckling (bending and twisting simultaneously): is a mode of buckling in which compression members can bend and twist simultaneously without change in cross-sectional shape. This type of buckling mode is critical, in particular when the shear center of the section does not coincide with the centroidal axes for angles, channels, hat sections, T-sections, and I-sections with unequal flange singly symmetric shapes (Wei-wen Yu., 2000). In these cases, the shear center is outside the web and the applied load initiates rotation. In most cases cold-formed members exhibit complex behavior governed by interacting local and global stability phenomena. The possibility of occurrence of the complex buckling can be significantly reduced by providing sufficient end supports and intermediate lateral restraints (John L. Dawe.and Geoffrey L.Kulak., 1984). For members with point-symmetric open cross-sections (example Z-purlin with equal flanges), account should be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling. For members with mono-symmetric open cross-sections, account should be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling. For members with non-symmetric open cross-sections, account should be taken of the possibility that the resistance of the member to either torsional or torsional-flexural buckling might be less than its resistance to flexural buckling (EN 1993-1-3:, 2004).

2.3. Theoretical review/Conceptual Framework

Over the past few decades there was been major improvement in cold-formed steel members, and it's due to many factors, namely corrosion protection and improved technology of manufacture. As consequence to a better structural solution, its demand in the market of building construction increased leading on to the development of a great number of studies on this subject. The researchers have been focused on the behavior of cold-formed steel members, testing different cross-sections, different support conditions as well as different lengths. Following the experimental research numerical models have also been developed using the finite element method (FEM) in order to validate additional parametric studies. Some of them are briefly described within this section. Literature to date has investigated the structural behavior of the cold-formed steel (CFS)-sections and derived formulae of estimating local, distortional, flexural buckling and flexural-torsional buckling loads.

(Krishanu Roy.et al, 2019) conducted experimental investigation on the axial capacity of CFS built-up box sections fabricated by two identical lipped channel-sections connected at their flanges with self-drilling screws in which material properties and initial imperfections were measured for both single channel and CFS built-up box sections. Test results show that all short columns failed through local buckling. However, for slender columns global buckling was observed. The experimental test results were also compared against the designed strength calculated in accordance with the AISI and AS/NZS. From the comparison, it was found that the AISI and AS/NZS are conservative by around 15-19% on average, while predicting the axial capacity of such CFS built-up slender columns

The effect of screw spacing on axial strength of cold-formed steel sections-numerical investigation and parametric study carried on built-up box columns subdivided into short columns of 0.5 m height and slender columns of 1.5 m height with pin-ended boundary conditions were applied for all single channel and built-up columns .Both the finite element and laboratory test results were compared against the design strengths calculated in accordance with the American Iron and Steel Institute (AISI) and Australian and New Zealand Standards (AS/NZS). From the comparison, it was observed that the AISI and AS/NZS are conservative by around 17% while determining the axial capacity of such built-up CFS box columns (Krishanu Roy.et al, 2019).

The buckling behavior of back-to-back gapped built-up cold-formed steel channel sections under compression, because of the complex and non-uniform cross section of the back-to-back gapped built- up columns; it is difficult to calculate the strength of these sections accurately. Axial capacity of the columns, load-axial shortening, load-axial strain, failure modes and deformed shapes were observed and reported in this paper. The column strengths are compared against the design strengths calculated using the AISI and AS/NZS, direct strength method and modified direct strength method. From the experimental results, it concluded that the axial strength calculated in accordance with the current design guidelines for back-to-back gapped built-up CFS columns, can be conservative by as much as 53% when λc was used to calculate the design capacity. However, the design standards were conservative by only 5% on average to the experimental results, when λc, GAP was used. Hence it is recommended to use λc, GAP while calculating the axial strength of back-to-back gapped built-up CFS columns (Krishanu Roy.et al, 2019).

Also investigation on the buckling behavior of CFS built-up un-lipped channel section that joined back to back with the help of intermediate web fasteners under the effect of axial load and comparison of test results with FEA in ABAQUS shows good agreement. FE models can be used for predicting buckling behavior of the un-lipped channels that are built up. Material non-linearity and initial imperfections were included in the FEA model and strengths obtained were compared against the AISI and AS/NZS design strengths; obtained comparisons showed that the AISI and AS/NZS standards were un-conservative for stub and short columns which failed by local buckling whereas the standards were over-conservative for columns failed through overall buckling (Krishanu Roy.et al, 2019).

Investigation by (Thomas H.etal, 2013) on buckling behavior of built-up cold-formed steel channel column sections determined the buckling mode and maximum buckling capacity. The columns were tested in axial compression with pinned end connections and in two different orientations, one with the C-channels facing each other to produce a closed shape (rectangular), and the other orientation had the members facing away (referred to as an I-shaped hereafter). The closed rectangular-section provides exceptional torsional resistance, which could lead to an increased buckling capacity. One test for each member type was used to determine each of the buckling values. All members were created from two, lipped C-channels which were connected by 102 mm long welds at the top and bottom, and intermediate weld locations throughout the member which had a weld length of 25 mm. When the buckling load of the 1.8 m long rectangular columns is compared to that of the I-shaped columns, it can be seen that the I-shaped columns have a lower buckling capacity than that of the rectangular columns. The rectangular 1.8 m specimens normally failed in a form of global buckling. However, this was not always the case and the intermediate weld attachments did have a factor in the final failure pattern of the column. No flexural buckling occurred on the column; instead it was these distortional buckling modes that failed the column. I-shaped 1.8 m long columns were much more susceptible to distortional buckling of the flange and web than the rectangular columns. The majority of I-shaped columns failed in a form of distortional buckling in the flange and web, along with some failing by crushing at the end. The orientation of the column substantially impacts the maximum load of the column (as much as 20%).

(M. Anbarasu.etal.2014) studied the capacity of built-up battened cold-formed steel columns under axial compression on the theoretical and numerical investigation results of pin end. The battened columns were made from cold-formed steel lipped channel sections. The finite element model was developed using the finite element software ABAQUS. The comparison of finite element analysis results match with the test results available in the literature shows that the analysis model can simulate the buckling behavior and ultimate capacity of built-up cold-formed steel columns. Spacing between the chords is chosen such that the moment of inertia about major axis equals the moment of inertia about minor axis. After the verification of the finite element model, parametric study has been carried out by varying the slenderness ratio and number of battens. The FEM results are about 7% higher than the test results. The difference in the numerical model was more likely due to assumed imperfections and simplified modeling of fastener of the sections. For the slenderness ratio less than 70 all the specimens failed by combined local (L), distortional (D) and flexural (F) mode. For slenderness ratio greater than 70, the specimens failed by combined distortional (D) and flexural (F) mode. But the predominant mode is distortional buckling and it governs the strength. A total of 30 axially loaded built up battened columns were numerically and theoretically studied in the paper. The ultimate strength of the member decreased with the increase of overall slenderness ratio for irrespective of individual sections. For slenderness ratio between 70 and 120, the predominant failure mode was distortional buckling.

Research by (M. Rathipraba., etal .2020) shows the strength and behavior of the built-up cold-formed steel column sections comprising of two channels linked back to back with the batten plates. The sections were modeled in ABAQUS software. The material properties such as yield stress, modulus of elasticity and imperfection factors also were included in the developed sections. The boundary conditions were provided as pin-ended by allocating the appropriate displacement and rotation at both ends of the built-up column. The ultimate load-carrying capacity deformed shapes of the sections were obtained. The test results have been validated with numerical analysis using the ABAQUS software. Comparison of the mean and standard deviation of the finite element modeling and effective width method of built-up battened columns are quite conservative according to the given section. Available results show the design strength of the column is increased when the overall slenderness decreased. Finally, it has been concluded that the given models are fairly conservative for the built-up battened columns failed due to local buckling and it is unconservative for the built-up columns failed due to elastic flexural buckling.

(Bharat C.Halagalimath.and Kiran Koraddi., 2019) studied mainly the behavior of cold-formed steel sections under flexure by assuming three model sections C-10, C-20 and C-30 by increasing their lip depth such that specimen C- 10 has lip depth of 10mm, C-20 has 20mm and C-30 has 30mm also all the rest of dimensions and sectional properties are kept the same. Then the effect of variation in lip depth is studied, the nature of buckling and the different modes of buckling are studied using constrained and unconstrained finite strip method (CUFSM) software. The lips are observed to add up the overall stiffness of the sections. The effect of lip depth is seen on the moment carrying capacities as evaluated by IS801-1975 and AISI-S100-07. Mode of failure was due to local buckling in all the three sections as observed from CUFSM analysis and the lip depth of the sections doesn't affect the deflections much.

Experimental investigation Carried by (Maura Leece.and Kim J. R. Rasmussen., 2004) on distortional buckling behavior of cold formed stainless steel sections. The paper described the experimental procedures used for testing distortional buckling of simple lipped channels and lipped channels with intermediate stiffeners for austenitic 304, ferritic 430 and ferritic-like 3Cr12 (chromium weldable steel) stainless steel alloys. Material properties vary with each alloy and they depend on the direction of loading. The strength enhancement due to work hardening in the brake-pressed corners is particularly pronounced for the austenitic 304 alloy, with a proof stress 2.33 times larger than the flat sheet material. Non-linear stress strain behavior with a markedly low proportionality stress leads to a significant loss of stiffness at low loads and this is reflected in the test results.

Experimental investigation on the effect of screw fastener spacing on the local and distortional buckling behavior of built-up cold-formed steel columns by (David C. F. Etal. 2016) addressed specifically, deep lipped channel sections in a back-to-back, screw connected form and were chosen for their local and distortional slenderness to study the effect of fastener spacing and layout on local and distortional buckling and collapse behavior. The screw spacing is varied from

L to L/6, where L is the column length, with and without varying lengths of end fastener groups (EFG), which were a prescriptive layout of fasteners at the ends of built-up columns that is required by AISI S100-12 and is intended to insure end rigidity and increase composite action. Results yield two general types of deformation modes: compatible (where the connected webs conform to the same buckling shape) and isolated stud buckling. Buckling loads and deformation are shown to be affected by the tighter screw spacing. The test result showed that the stiffness and strength of two studied built-up CFS columns, with stiff end bearing conditions, that buckle and fail in either local and/or distortional modes are not highly dependent on the layout of fasteners that connect the two members. In particular, a costly end fastener grouping consisting of a large series of fasteners at the member ends is not shown to appreciably improve the local and distortional buckling behavior or capacity of the built-up CFS column.

(Anbarasu., 2019) Conducted experimental and numerical simulation on the behavior and strength of CFS built-up battened box column composed of lipped angles under axial compression. Tests on 10 built-up battened columns were undertaken in the scope of this investigation for two different types of built-up sections by varying the member slenderness, plate slenderness (width-to-thickness ratio of chord), chord slenderness and batten plate slenderness with measured initial geometric imperfections. It was found that the chord slenderness significantly affects the compressive strength of the built-up columns. When the chord slenderness is altered by increasing the depth of batten plate from 50 to 100 mm for the same column cross-section and overall member slenderness (length), the axial compression resistance was increased by 5.8%, 7.7% and 9.2% for nominal overall member slenderness of 20, 30 and 40, respectively. Increase in axial compression resistance by 5.5% and 6.7% when doubling the batten plate depth from 60 to 120 mm for the overall member slenderness of 20 and 30, respectively. The test strengths were compared with the design strengths predicted using the North American Specifications (AISI-S100:2016) and Eurocode (EN1993-1-3:2006). Observing the obtained results, the chord slenderness significantly influences the axial compression capacity of CFS built-up battened columns. The FE models of the built-up battened columns were developed by including the geometry, material nonlinearities, fastener modeling and geometric imperfections. The test results were compared with the design strength predictions calculated using the North American Standards and European Standards for CFS sections. By comparing the axial compression resistance of the built-up battened column which have the same nominal cross-section and overall slenderness but different in chord slenderness, it was found that the built-up section with low chord slenderness gives higher result than section with higher chord slenderness, which shows that the chord slenderness significantly affects the axial load compression resistance of the CFS built-up battened columns. The predicted axial compression resistances of the FE models were also in good agreement with the test results.

(Ziqi H.etal, 2020) studied the structural performance of cold-formed steel columns reinforced by channel sleeve under axial and eccentric compression due to the weak torsional stiffness of cold-formed thin-walled channel members, distortional buckling behavior maybe controlled the ultimate load-capacity under certain conditions. Therefore, a new section reinforced by channel sleeve is proposed in order to improve the structural capacity of channel columns in this paper and is performed on the axial and eccentric compression tests. The influence of channel sleeve spacing on the bearing capacity and failure mode is studied, and the beneficial effect of the channel sleeve on the bearing capacity is verified. In addition, the numerical model is used to analyze the parameters such as the slenderness ratio of the specimen, the magnitude and direction of eccentricity. The influence of these parameter changes on the structural performance of the members under the action of axial and beam-column compression members is obtained. ABAQUS finite element software was used to analyze the bearing capacity and buckling mode of the compression column with channel sleeves. The analysis results show that the setting of channel sleeves can increase the bearing capacity of the compression column with distortional buckling, and the improvement of the bearing capacity increases as the distance between the channel sleeves decrease. However, the effect of improving the bearing capacity of the column with local buckling and flexural buckling is not significant.

(Ahmed Shamel Fahmy.etal.2018) Estimated the Ultimate Load of Cold-Formed Steel Double Lipped Channel Columns Using a Simplified Method. Numerical investigations of buckling behavior are accomplished by using FE software ANSYS 16. The comparison between results obtained from the numerical method (FEM) and the proposed simplified method was achieved to validate the precision of the simple prediction method. An extensive parametric study is performed to analyze the performance of 215 lipped channel columns under axial pure compression. The specimens were classified into three zones depending on the values of their slenderness ratio. Each zone is divided into sub-zones based on their web to depth ratio. Ultimate loads obtained from the proposed method were compared to the ones obtained from FEM and they achieve a good match in all zones varying between ±15percent. Also the proposed method is applicable for columns that consist of 2-channels back to back. It achieved ultimate load values about 90% of those obtained from FEM. The positions of connections between the two channels have a great influence on the column’s ultimate load value.

(Busanaboyina Jagadish Chakravarti., Bhamidipati Sai Krishna., 2019) Conducted experimental study on buckling behavior of cold formed light gauge steel angle columns subjected to eccentric loading and validating the results in the finite element model by using the finite element software ABAQUS/CAE 6.14. Finite element nonlinear analysis of cold-formed steel angular columns were done by the static risk procedure available in the finite element package ABAQUS/CAE 6.14 which followed by eigen value buckling analysis done by linear perturbation step available in ABAQUS. From Eigen value buckling analysis buckling modes and loads are obtained. The ultimate load carrying capacity of the column decreases with increase in width to thickness ratio and, decrease in thickness of the column. The decrease in the load carrying ability with rise in width to thickness ratio is mainly due to plate buckling, predominantly due to local buckling of the member. Test results obtained from non-linear analysis in ABAQUS were compared with experimental results for lesser width to thickness ratio; the column ability predicted by

ABAQUS was in good concurrence with the experimental results. It can be seen that the local buckling starts in the sections with a larger flange width than the flange width lower width. The final load predicted by the ABAQUS / CAE 6.14 finite element analysis software is consistent with the experimental results. It should therefore be considered as an alternative method to the experiments.

Experimental Investigation by (David C.Fratamico.etal.2017) on buckling and collapse behavior of screw fastened, built-up cold-formed steel columns of varying cross-section size. Understanding the behavior and strength of screw-fastened built-up cold-formed steel (CFS) columns was important, as they were used with increasing frequency in CFS framing. The test series presented herein was developed to analyze a range of section types, fastened in a common back to-back built-up section; fastener layouts were studied and cross-compared with section types and steel plate thicknesses. A costly end fastener grouping consisting of a large series of fasteners at the member ends is shown to boost the capacity of columns only when buckling in minor-axis flexure, but even then, only a limited boost. As most columns in CFS structures were sheathed and/or braced, local and distortional buckling dominates the failure mode and column capacity, and therefore the EFGs (prescriptive end fastener groups) were not important. Ongoing work developed better design methods that incorporate more accurate estimations of column end conditions and explicit modeling of web fasteners. In doing so, the feasibility of partially-composite column curves can then be fully assessed.

(K.Thiyagu.etal.2019) investigated the behavior of Built -Up cold formed steel compression member. Cold formed built – up octagonal shaped closed box sections were tested under axial compression and the ends of columns were simulated as hinged ends with varying lengths and the test strengths were compared with strength values obtained from theoretical analysis and concluded that: Even though the failure modes of the columns involved local buckling, distortional buckling of the webs and flexural buckling, the significant mode of failure is controlled by distortional buckling. Ultimate load carrying capacity is inversely proportional to slenderness ratio (L/Ri ratio). When the slenderness ratio (L/Ri ratio) decreases the ultimate load carrying capacity of the specimens has been increased.

(Francisco Javier Meza Ortiz., 2018) carried out experimental and numerical investigation of built-up cold-formed steel stub column at the University of Sheffield. An experimental and numerical programmed of 20 built-up CFS stub columns with four different cross-sectional geometries and assembled using two different types of connectors is presented. All parameters that were thought to affect the buckling response of the built-up specimens, in particular the material properties, geometric imperfections and connector behavior, were measured and incorporated into detailed FE models. The stub column tests showed that reducing the connector spacing results in an increase in ultimate capacity which ranges from modes (up to 11%) to negligible for the range of geometries and connector spacing’s considered in the program. In some cases geometry a slight reduction in ultimate capacity was observed with reduced connector spacing. A good agreement was achieved between the predictions of the FE models and the test results, with differences in the ultimate load of less than 6 % for all geometries tested. Different approaches to modeling the connector behavior were investigated in the FE models. However, it was concluded that the modeling approach by extension the actual connector behavior does not have a significant effect on the ultimate capacity of built-up CFS stub columns.

(Yao Xingyou., 2021) Carried out experimental investigation and numerical analysis using direct strength method of built-up cold-formed steel (I-section) columns under axial compression force. Built-up I-section connected together with two lipped channel sections by screws through the webs is the most common built-up members used in CFS building. A total of 56 CFS built-up I-sectional columns were tested. Based on the experimental investigation, numerical simulation, and theoretical analysis, conclusions were drawn as follows. Interaction of local and distortional buckling and interaction of local, distortional, and overall buckling were observed in this test study. Interaction buckling had a significantly affect on the buckling behavior of CFS built-up I-sectional columns under axial compression. Built-up columns showed great composite action through the web screws. Spacing of screws and the end fastener group had a certain effect on ultimate strength of built-up columns, especially for columns failed with global buckling. Numerical simulations for buckling modes and ultimate capacities of built-up I-sectional members were reliable and accurate. Finite element parametric analysis indicated that the slenderness ratio had great influence on ultimate strength and stiffness for the built-up I-sectional members failed with local-distortional-overall buckling interaction and a few effect for the built-up I-sectional members failed with local distortional buckling interaction. Spacing of screws and the end fastener group had a certain influence on ultimate strength of CFS built-up I-sectional columns when the built-up members had large spacing of screws. Ultimate strength predicted using the proposed direct strength method in this paper can agree well with the test results and finite element results when the elastic distortional buckling and the elastic global buckling stress of the CFS built-up I-sectional columns were predicted by keeping the built-up cross section as a whole I-section, and a single C-section is used to determine the elastic local buckling stress. Comparison indicated that the proposed DSM design method is reliable for the common CFS built-up I-sectional columns in this paper. It needs to further verify for other CFS built-up I-sectional columns.

Analysis and design of hot-rolled and cold-formed steel industrial building was done by (Roshan S Satpute.and Valsson Varghese., 2012)for multi-storey building by considering various sectional properties. The total weight of cold- formed steel industrial building having area 15×50m and eave height 5 m, was found to be 15.92 Ton and cost of building is estimated 11.14 Lakh. The cost of cold formed steel is 70Rupees/Kg. The total weight of conventional industrial building having area of 15×50m and eave height 5m was, found to be 25.159 Ton and cost of building is estimated 15.09 Lakh. Cost of hot -rolled steel is 60 Rs/Kg. With the analysis and design of section, it has been observed that by using cold formed steel building instead of hot rolled steel building the material is saved was 9.239 T and cost saved to 5.54 lakh with spacing of centre to centre main frame is 7.14m. By using cold formed steel building instead of hot rolled steel building the material saved was 13.92 T and cost saved to 8.35 lakh with spacing of centre to centre main frame is 6.67m. In industrial building the material & cost of the building is minimized in case of cold formed steel while in case of conventional building it was be higher both in two cases. The saving in material and cost is about 25 %.

2.4. Critique of the existing literature relevant to the study

Cold-formed steel structural elements and its applications have extensively increased. Despite the advances in the development of cold-formed structures, the level of its use was still significantly lower than the hot-rolled steels. A considerable factor that caused this imbalance application was the absence of standards and norms. Much of the current understanding of built-up members is based on hot-rolled built-up sections. Only a handful of experiments of cold-formed built-up sections were conducted. Experimental, analytical and theoretical investigations carried out on cold-formed steel channel sections with the comparison of strengths from the experimental results with finite element analysis and design strengths compared with different national and international codes. From the available literature, it is observed that behavior of cold- formed steel channel sections under compression and eccentric compression with and without lips have been carried out by various authors. Tests on perforated cold-formed angles subjected to axial compressive loading show that the presence of perforations results in reduction of the ultimate capacity of the specimens. Although cold-formed steel members have been researched for a long time, the stability problem is not fully understood. Different countries national and international codes of standard did not have a means (equations) of calculating imperfections. In reality, basically the two types of imperfections: geometrical and material (mechanical) imperfections are always present.

2.5. Summary

A literature review of the previous researches, which broadly investigated the elastic buckling behavior, is conducted and it reveals that there is no enough information about the buckling behavior of cold-formed steel shapes with different slenderness range. Most of the previous researchers findings limited in experimental investigation of cold-formed double symmetric sections. Yet, current specifications offer very limited guidance for analysis of built-up cold formed steel members. Mechanical properties of cold-formed steel sections differ from the commonly used hot- rolled steel sections, because of cold-formed steel sections are influenced by its manufacturing process as described earlier and need to be inspected accurately to study the behavior of cold-formed sections. The geometric properties of the cold formed steel channels were taken from the table catalogue (IS:811-1987, 2011). For hot-rolled steels, the yielding point is well defined by the time as the stress-strain curve becomes horizontal; in the case of cold-formed steels the yielding point is not so clear, as the stress-strain curve is gradually growing reaching the yielding stage without a visible peak and not considered accurately by the limitation of standards and norms. Distortional buckling mode is a relatively new and less researched buckling mode compared to the other buckling modes and the reduction of the overall column strength due to interaction effect of the local and overall column buckling is less researched buckling behavior.

The demand of the construction industry call for innovation in the design of cold-formed steel structures and built-up sections is one of such innovation. There are insufficient guidelines in the current codes and design standards to better estimate the load bearing capacities of built-up sections. More design rules needed to provide guidelines for the design of built-up sections such as back-to-back without gap, back-to-back with gap, face to face without gap, face to face with gap, battened, and laced columns. The load bearing capacities of built-up sections is governed by the gap in between the chords, spacing and stiffness of connectors. Rules for modification of the design of cold-formed steel needs to be developed to better reflect the behavior of cold-formed built-up sections.

CHAPTER THREE

RESEARCH METHODOLOGY

3.1. Research Design

The research is exploratory type and it begins by taking samples and it follows the procedures including: taking a sample of CFS-channel section from specification for cold-formed light gauge structural steel sections, Soft ware modeling, analysis and interpretation. Finally, the FEA software result was compared with the test results taken from the available literature. The steel material properties adopted in FEA software for the structural CFS-sections was done based on the rules specified in (EBCS EN 1993-1-1:, 2013)code, which is the reference standard. This research design is carried out on the numerical buckling analysis of the sections with pin- end condition. The simulation and analysis was carried out by FEA software in ABAQUS 6.13-1 to obtain the maximum buckling load and failure mode of the CFS column sections. For this analysis an axial compressive load of 1Newton is applied over the top node of the CFS column sections.

3.2.Research process

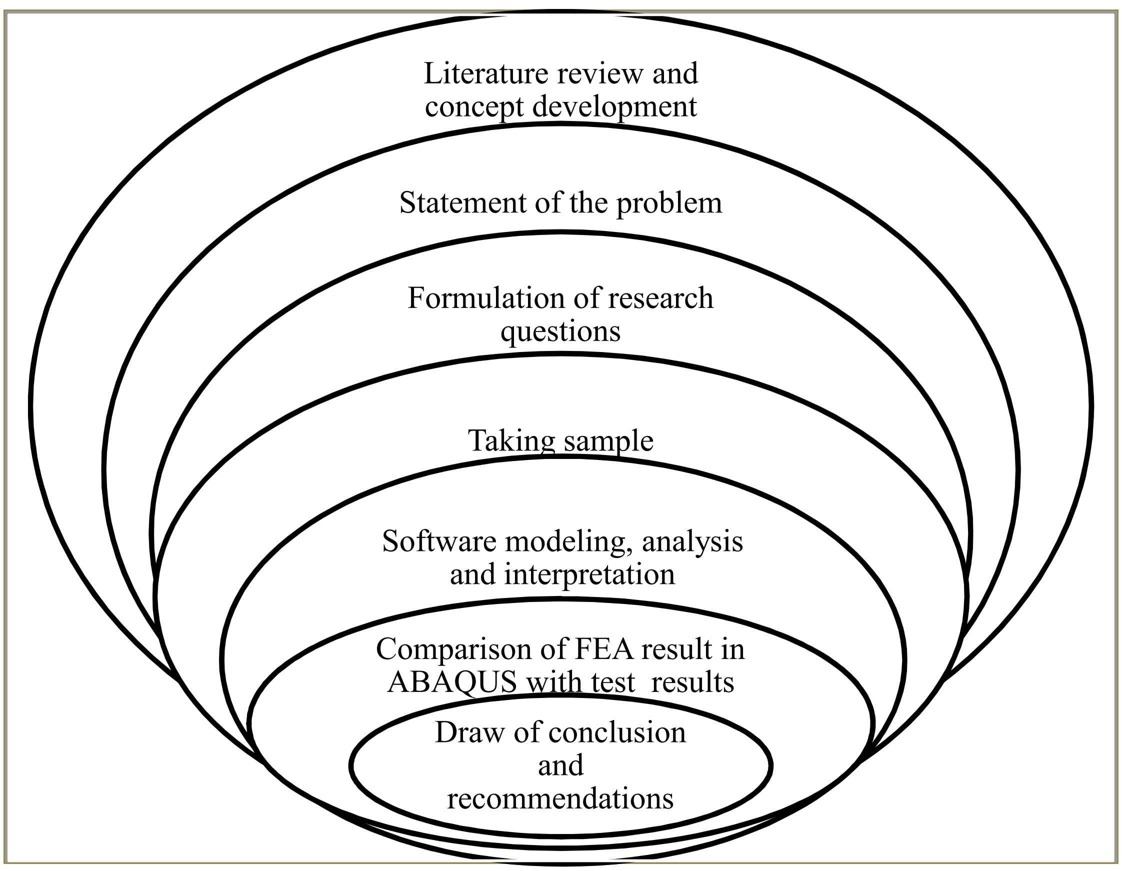


Figure 3.1.Research process

3.3. Study Variables

3.3.1. Dependent Variable

- ❖ Buckling behavior of built-up cold-formed steel un-lipped rectangular channel column sections (ultimate load and failure modes)

3.3.2. Independent Variables

- ❖ Cross-section geometry (shape, rectangular CFS-channel without lip and cross-section size of 200x80x4mm)
- ❖ Column length(short to long)
- ❖ The end conditions of the column (Pinned-end)
- ❖ Rectangular CFS-channel section orientations(C-channel,BU and BTB) without lips
- ❖ Slenderness ratio of the columns(λc)and
- ❖ Connectors spacing along the column length(constant 400mm center to centre)

3.4. Population and Sampling Method

3.4.1. Population

The populations under this research were single and dual- built-up rectangular cold-formed steel channels without lips of size 200x80x4mm column section at lengths of 0.6, 1, 1.4,2, 2.5, and 3 meters. The dimensions and section properties were taken from rectangular channel-section without lips catalogue for the results of the finding. These populations enable the researcher, to obtain the necessary data for this research.

3.4.2. Sample Size

200x80x4mm cross-section size single and dual- built-up rectangular cold-formed steel channel columns without lips at 6 different lengths and boundary conditions of pin- end were considered with a total of 18 modeling.

3.5. Sampling Procedure

The non-probability sampling method of purposive sampling technique was implemented in order to pick out the rectangular channel sections without lips from the specification for cold-formed light gauge structural steel sections for the requirement of the section to be in line with the section investigated experimentally to its validation, which is important variable considered for this particular research.

3.6. Sources of Data

To evaluate the buckling behavior of the columns in adequate and reliable manner secondary sources of data such as text books and relevant journals published by competent authorities was used. These literatures show the buckling behavior of different CFS steel column sections and their associated usage for practical implementations. Data can also found from code specifications and FEA of ABAQUS 6.13-1 software package.

3.7.Data Collection Procedure

The method engaged in this research involves qualitative and quantitative data. A literature search involved a thorough review of recently published journals and previous research in the area of buckling behavior of CFS-channel column sections and the steel materials mechanical behavior.

3.8.Data Presentation and Analysis

3.8.1. Data Analysis

In evaluating the buckling behavior of CFS-channel column sections, data collected through review of recently published journals and previous research in the area was used as an input for FEA in ABAQUS software and compared with the test results taken from the available literature for pin-end boundary conditions. The FEA in ABAQUS is validated with the test results.

3.8.2. Data Presentation

Finally, the analysis output of this thesis is presented using tables, graphs and charts. These tables, graphs and charts states in detail the maximum buckling load and mode of failure at different column heights with pinned-pinned end conditions were presented.

3.9.**Finite Element Modeling**

For this study ABAQUS 6.13-1(2013)/ ABAQUS CEA.link version which is suitable for analysis and visualization was used to develop a non-linear elasto-plastic FE models for both, single and dual built-up CFS rectangular channel cross-sections without lip. A solution sequence in ABAQUS consists of modeling, analysis and finally visualization. ABAQUS is built on modules (in essence part module, property module, assembly module, step module, interaction module, load module, mesh module, job module and visualization module) following a logical order of the modeling process. After modeling, the analysis was performed and the output database generated. Lastly, the visualization module allows reading the output database results. Rectangular channel and dual built-up cross-sections without lip was modeled by providing appropriate co-ordinates along with the length of the channel with the two pin ended condition. Deeper knowledge about the software has been received by reading the user manual SIMULIA (2013). The manual was essential for the contents of this chapter. CFS-channel section of 200x80x4mm size was investigated in this research. The detailed descriptions of the sections used in this FE (ABAQUS) analysis shown below.

Table 3.1.Cross section properties of rectangular CFS-channel section without lips used in FE analysis (IS:811-1987, 2011)

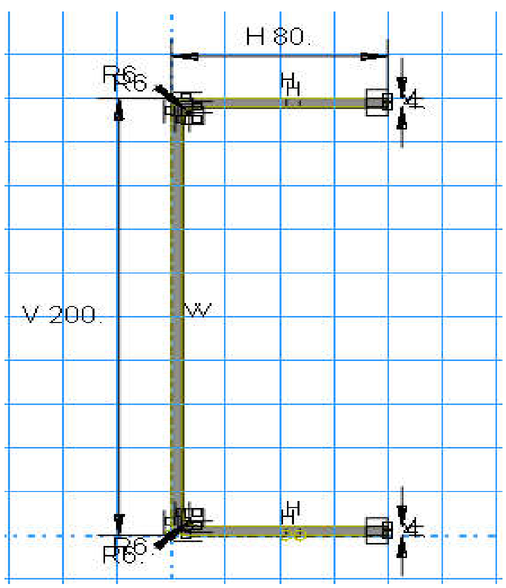
Designation (hxbxt), mm	Root radius $R_{i,}$ mm	Mass per unit meter Kg/m	Area of section A cm^2	Centre of gravity c_y cm	Shear Centre $x_{0,}$ cm
200×80×4	6	10.8	13.7	1.98	4.48

3.9.1. Modeling procedure

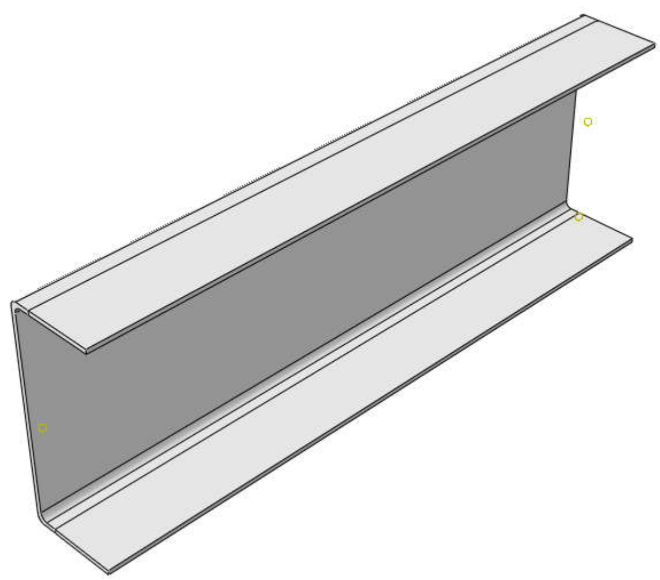
The model database in ABAQUS was a standard/explicit. Since ABAQUS is unit less, in order to be consistent to get the correct magnitude of the results, the models were chosen to follow SI units' in terms of N, mm and MPa.

3.9.2. Geometric modeling of CFS-sections

Each column was modeled in 3D- modeling space of deformable extruded solid base feature with approximate size of 1000. The 3D solids elements were with 6 degrees of freedom per node. The degrees of freedom were translation and rotational in the x, y, z directions. Figure3.2. were the section analyzed which is a rectangular single and dual built-up CFS-section with two different orientations, whose characteristics dimensions are: web depth of 200 mm, web width 80 mm, thickness 4mm and 6mm root radius at the four corners. The column is 600 mm to 3000 mm lengths. The following CFS-channel cross-section shapes were sketched using the create line connected option by entering x, y coordinate dimension option, entering the depth of the column that needed to extrude in edit base extrusion and the required sketch is shown below.



a) CFS- section sketch



b) CFS- Extruded 3D solid section

Figure 3.2.CFS- section and dual built-up columns

3.9.3. Material modeling

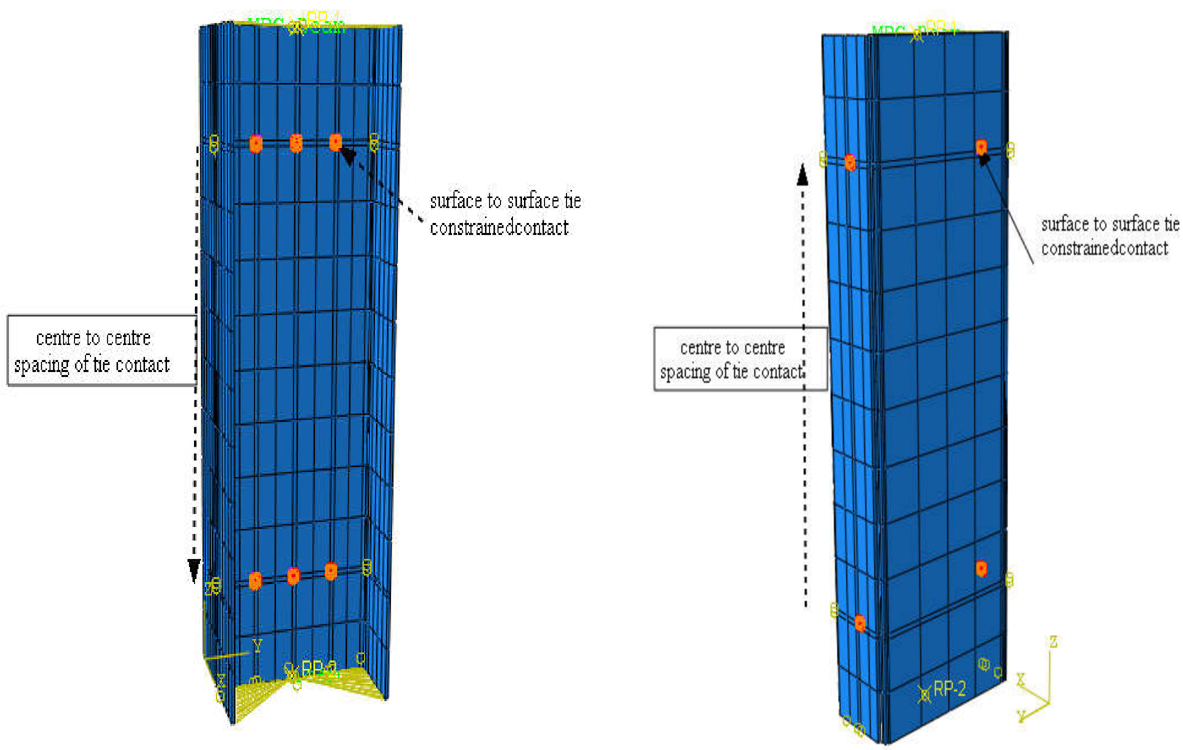
An elastic-plastic model was used for modeling the overall geometry of single and built-up channel sections. In order to define the isotropic yielding and plastic hardening of the steel, the material nonlinearity was included in the FE model by specifying the true values of stresses and strains were converted to true stress (σ_{true}) and plastic strain ($\in_{plastic}$) using the following equations as stated in the appendix; σ_{true} = fyb (1+$\in_{eng,\ el}$) and $\in_{true,\ pl} = \in_{total} - \sigma_{true}/E$ (Moosbrugger Charles., 2002). Steel mass density of 7850 kg/m^3 (general), Young's modulus of 210Gpa and Poisson's ratio was set to 0.3, to access solid homogeneous section.

3.9.4. Assembly and step modeling

Single rectangular CFS- channel section without lip instance were translated in linear pattern and rotated by the intended angle to produce the dual back to back and face to face built-up columns. All the instances were assembled as independent (mesh on instance). The type of FE analysis done to account for buckling of both single and built-up sections was linear elastic analysis performed using linear perturbation (buckle) procedure available in the ABAQUS library. Ten (10) numbers of eigenvalue have been provided and 500 maximum numbers of interactions was entered. The linear perturbation analysis step is created such that the response can only be linear.

3.9.5. Contact modeling (interaction)

Surface to surface tie contact was used for modeling the interaction between the webs of back-to back and flanges of face to face un-lipped rectangular CFS-channels. Master-slave surface contact pair option, available in ABAQUS library (2013) was used between the channels. The web or flange of one un-lipped channel was modeled as slave surface, while the web or flange of other un-lipped channel section was considered as the master surface respectively. Constraint conditions between un-lipped channels joined by the two orientations were modeled using surface to surface tie connector elements provided in ABAQUS library (2013). Constraint conditions between the webs of back-to back and flanges of facet-to-face channels were modeled adequately, so there is no slip between the channels to act as a rigid body. There was no penetration between the two contact surfaces.



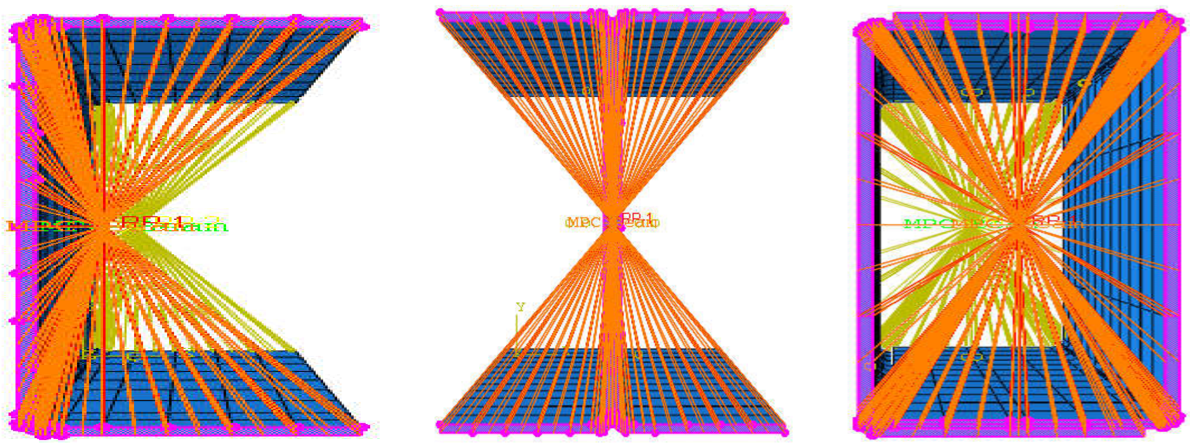
a) BTB, CFS- column b) BU, CFS- column

Figure 3.3.Surface to surface tie contact modeling

3.9.6. Modeling of boundaries and loading procedure

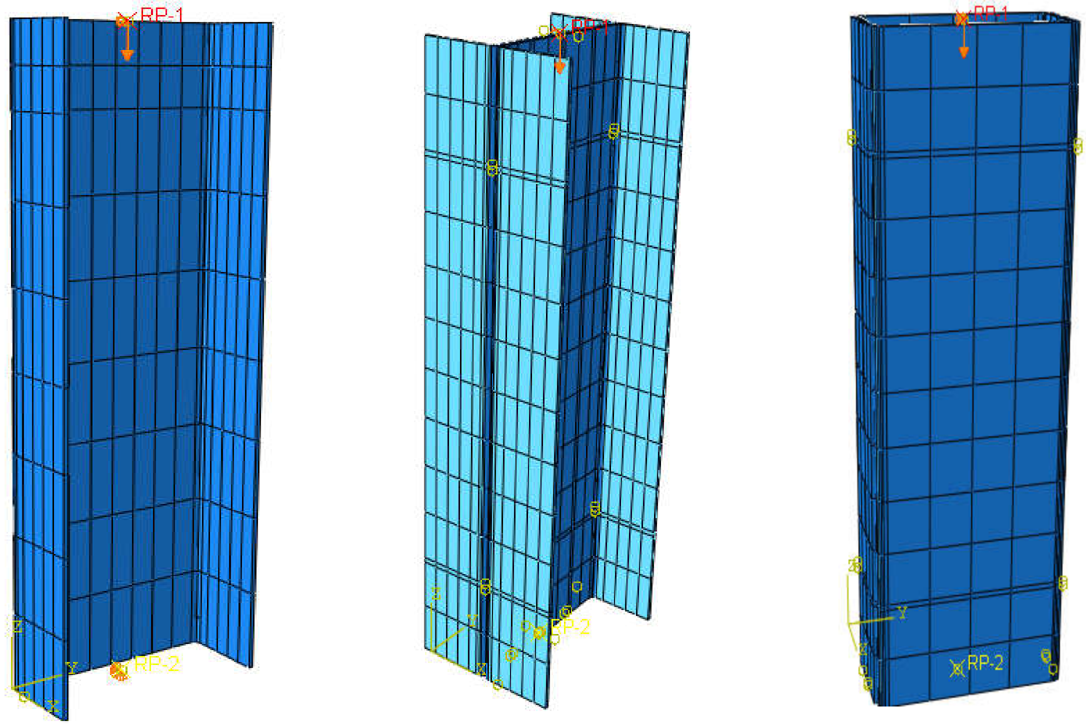
Pin-end boundaries were applied for all FE models of both single and dual built-up channel CFS columns. Pin-end boundary condition is modeled by applying translations and rotations free to top and bottom end of the CFS un-lipped channels at the reference points. The reference point was considered as center of gravity (CG) of the cross section of un-lipped single and built-up channel-sections. The Pin-end boundary conditions for both single and built-up CFS rectangular un-lipped channel sections were modeled by providing translations fixed in x, y, translation free in z and rotations free in all x, y, z axis's at the top loaded end, and bottom end translations fixed in all x, y, z and rotations free in all x, y, z axis's that modeled at the centroidal axes of the column section.

These boundary conditions were applied to the independent node of the rigid fixed MPC (Multi Point Constraint) located at the geometric centroidal axes of the section at top and bottom end of the model in the initial step. Dependent nodes were connected to the independent node using rigid beams and all six structural degrees of freedoms are rigidly attached to each other. This MPC acted as a rigid surface that was rigidly connected to the upper and lower end of the columns. The axial load is applied by specifying an axial compressive unit load of 1 Newton at the top end MPC node of the column, which is identical to the pin-ended column tests in analysis specific buckle step.



a)Single CFS- channel section b)BTB, CFS- channel section c) BU, CFS- channel section

Figure 3.4.MPC (multi point constraint) beam rigid region boundary conditions

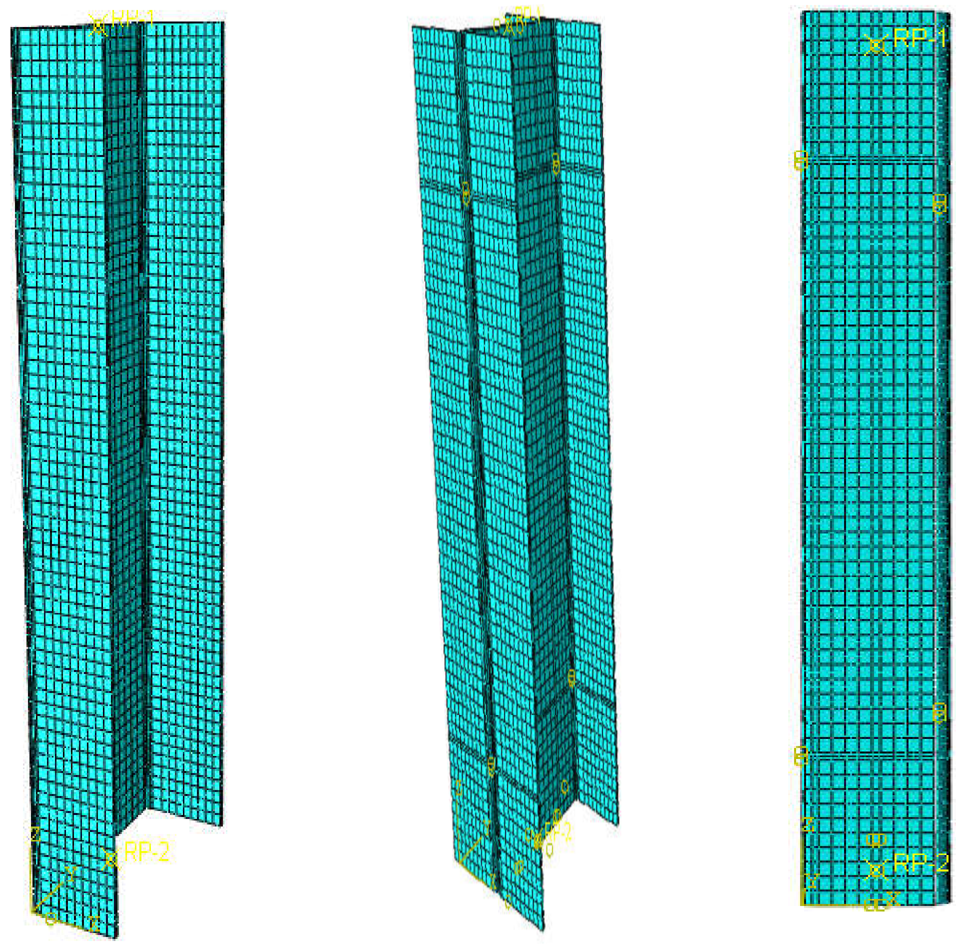


a)Single CFS- column b)BTB, CFS- column c) BU, CFS- column

Figure 3.5.Reference points and applied loading in the FE model for typical 600mm CFS-column length

3.9.7. **Element Type and Finite Element Mesh**

In the analysis presented in this document, the element type C3D20R in ABAQUS 6.13-1 element library was used to mesh a 20-node quadratic brick, reduced integration solid three-dimensional geometry using hexahedral element shape un-lipped rectangular CFS-channel columns. To decide on the appropriate mesh, various mesh configurations were examined in an attempt to settle on the appropriate mesh that grants the most accurate results and ensures utilization of the minimum computational time. A high mesh density increases the accuracy of the results obtained at the expense of computation time, while low mesh density can lead to unacceptable errors. 10 mm × 10 mm (length × width) mesh was used for convergence of model used in the present FE analysis since it gives an acceptable combination of accuracy.



a)Single CFS- column b)BTB, CFS- column c) BU, CFS- column

Figure 3.6.Finite element mesh

Load versus slenderness analysis was conducted on the cold formed single and built up sections under axial compression on un-lipped rectangular channel shapes at lengths of 0.6, 1, 1.4, 2, 2.5 and 3meters with pin-end support conditions. Elastic buckling analysis has been performed; results displayed with the deformed shape of the columns and results viewed by clicking the plot contours available on the ABAQUS library (2013). Output has been taken by using the ODB file output on viewport data which includes lateral deflections and deformed shapes have to be determined for predicting the ultimate load-carrying capacity of the column.

3.9.8. **Validation of the Model**

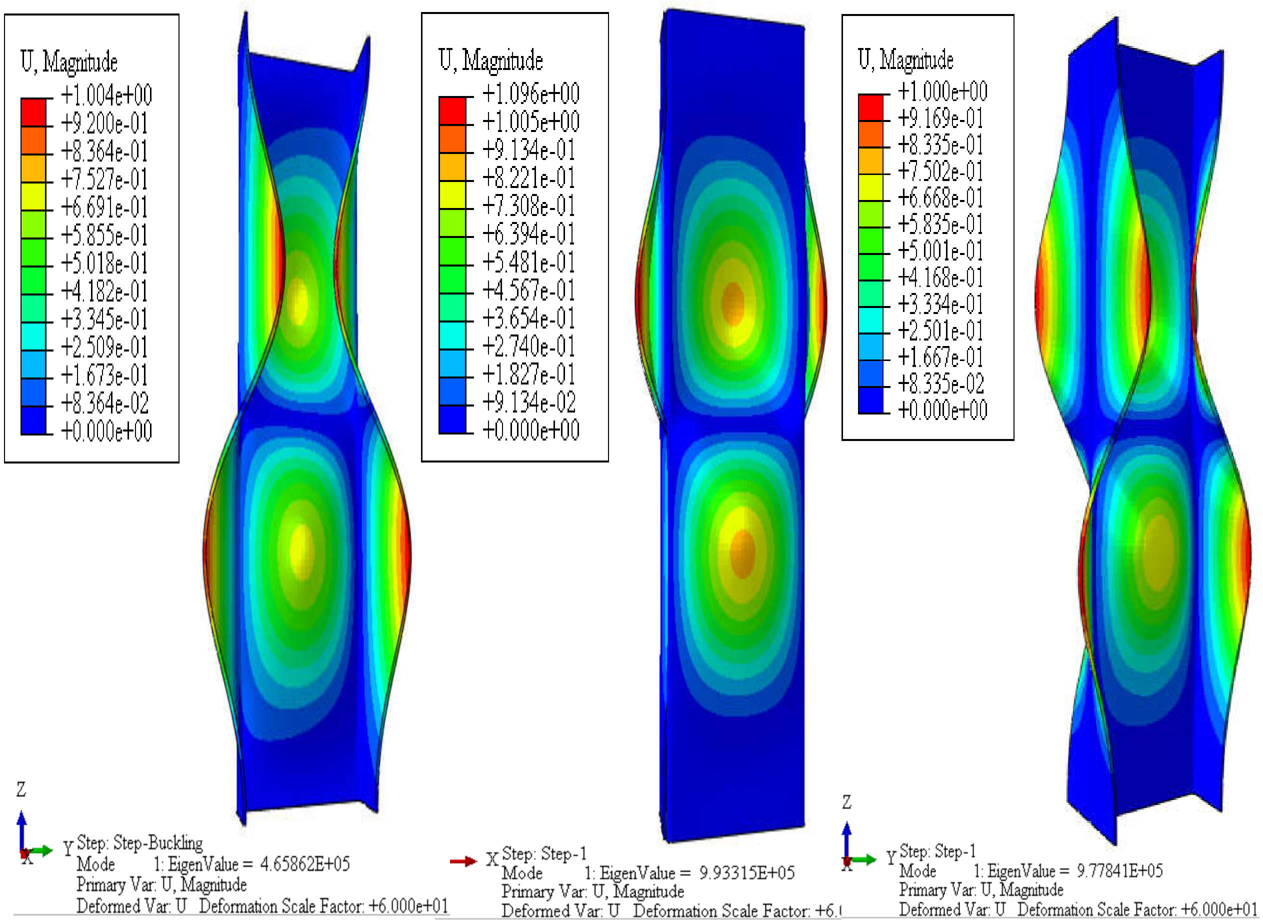
The analytical model was validated in comparison to an experimental research on ultimate load of lipped rectangular built-up cold formed steel channel column section known as KS20020C done by (Fadhluhartini Muftah.etal, 2014). From the experiment, it was found that the ratio of single to built-up column are greater than 2 were observed for all columns except for 2 m length of BTB and 2.5 m length of BU . The short column fails due to yielding of cross section; hence it can achieve full effective cross-sectional strength and effective to act as built-up column. Meanwhile, long column, fails due to varieties of buckling cause the built-up column difficult to attain its full cross-sectional strength. From the finite element model, it was found that the ratio of single to built-up column are greater than 2 were observed for all columns except for 2.5 and 3 meter length for both BTB and BU built-up columns with is greater than 3 were observed. The comparison results include ultimate capacity and mode of failures. In these comparisons, the length increments were selected such that column is from short to long assuming elastic buckling. In this study, eighteen numbers of channel sections were analyzed.

CHAPTER FOUR

RESULT AND DISCUSSION

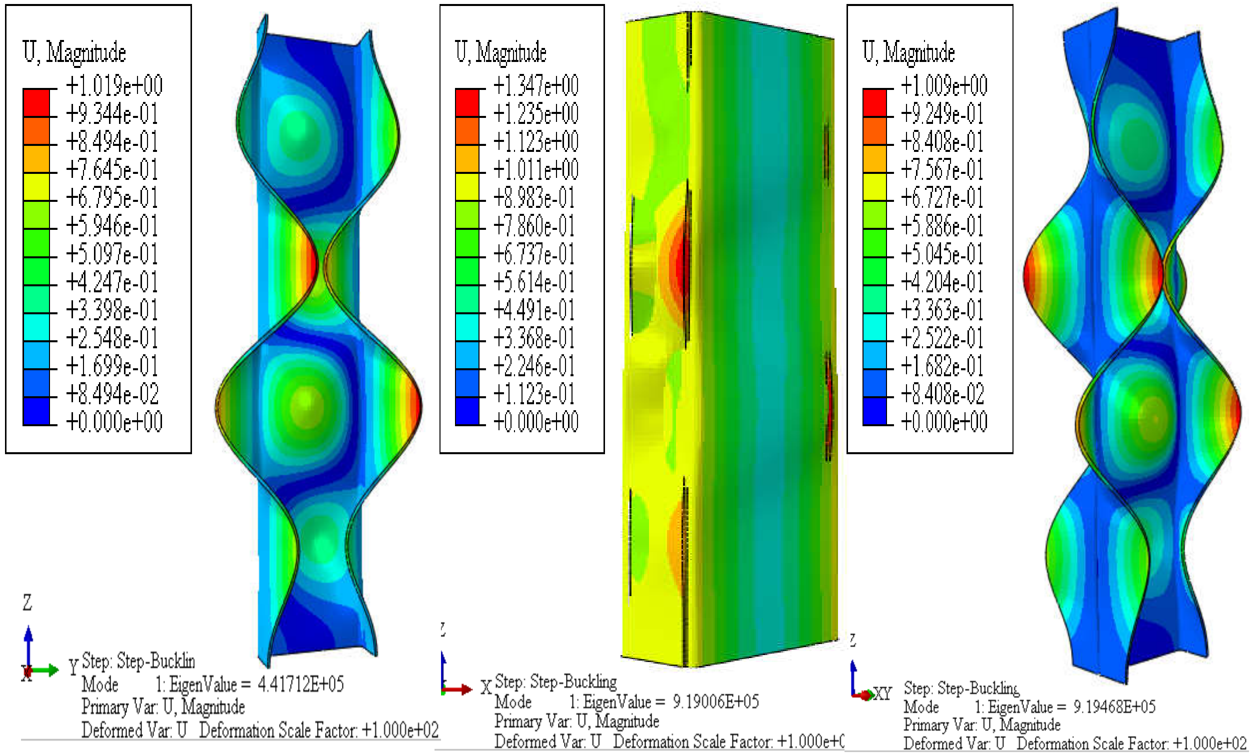
4.1. Results

The ultimate load determined when a concentrated compressive unit load of 1 Newton was applied at the column top end. The simulation was divided into three types, single, dual back-to-back (BTB) I-section, and dual face-to-face, box-up (BU) sections of un-lipped rectangular channel. The slenderness ratio accounts the geometric property of column according to its effective length and its cross-section properties. The aim of the FE analysis was to investigate the improvement in capacity of the built-up column sections due to the effect channel orientations. The various buckling strengths determined were; local, distortional and global buckling modes. The elastic buckling load was selected based on lowest values among 10 numbers of eigenvalue requested in ABAQUS FEA buckling results. Significant strength reduction occurred for both single and built-up CFS channel columns as the length increases. Various failure modes and associated failure shapes presented as below.



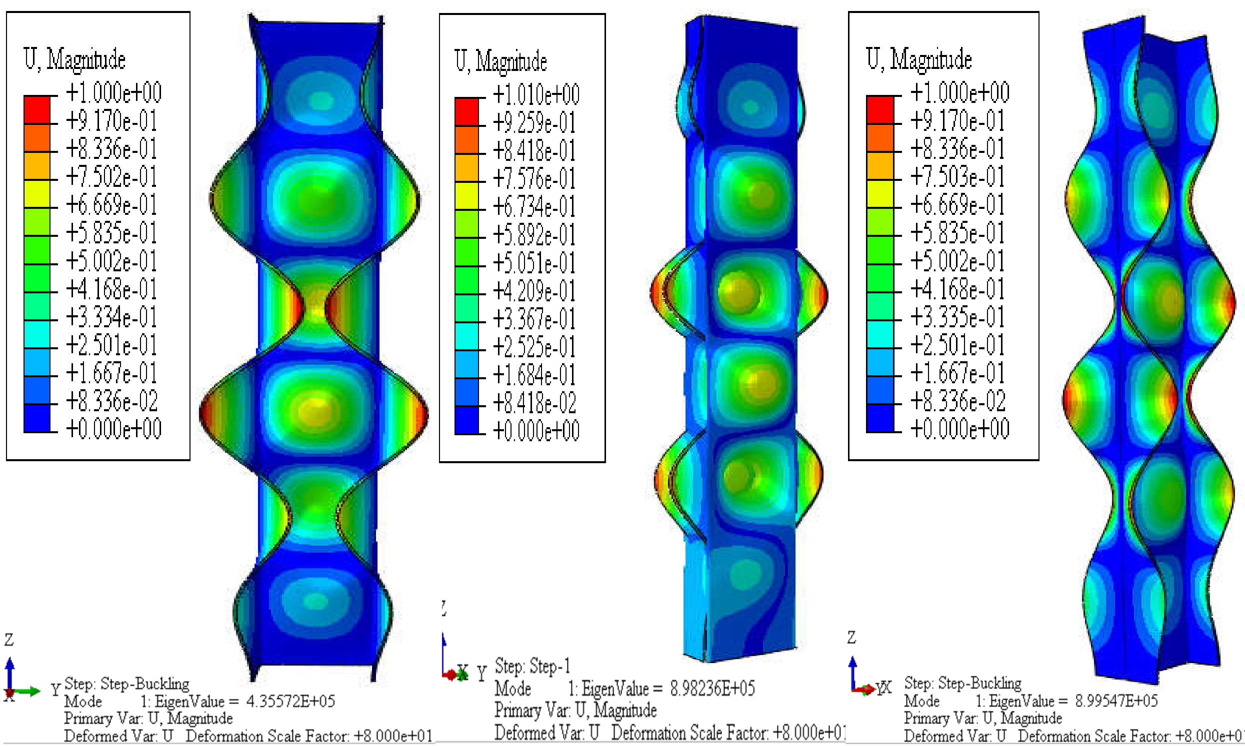
a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.1.Ultimate load and buckling modes of single and built-up 600mm CFS-columns



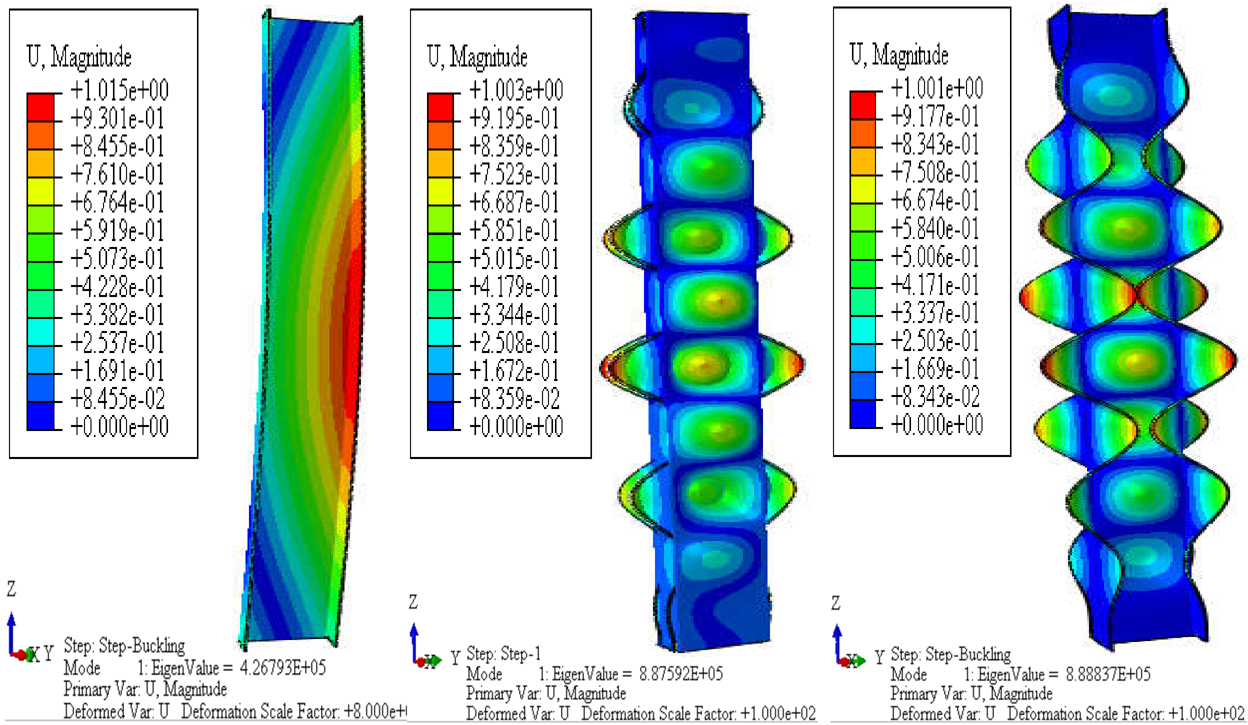
a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.2.Ultimate load and buckling modes of single and built-up 1000mm CFS-columns



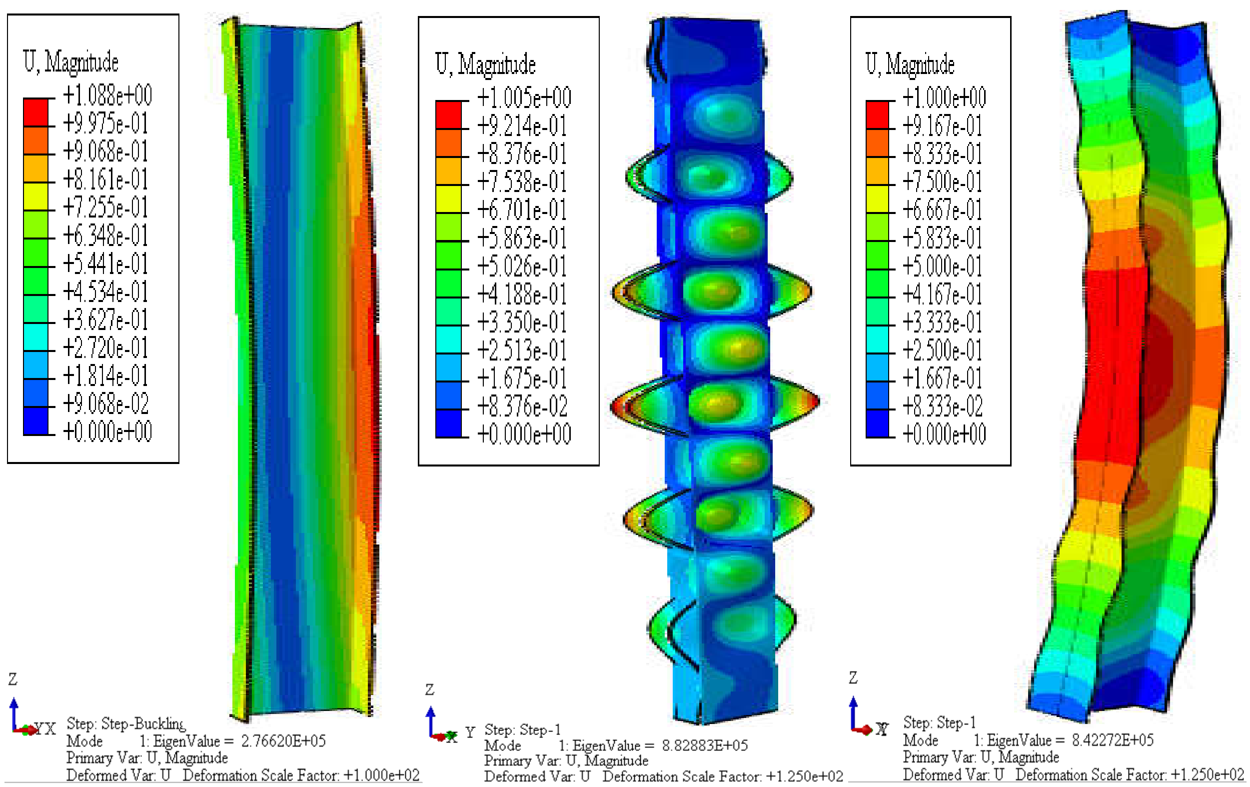
a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.3.Ultimate load and buckling modes of single and built-up 1400mm CFS-columns



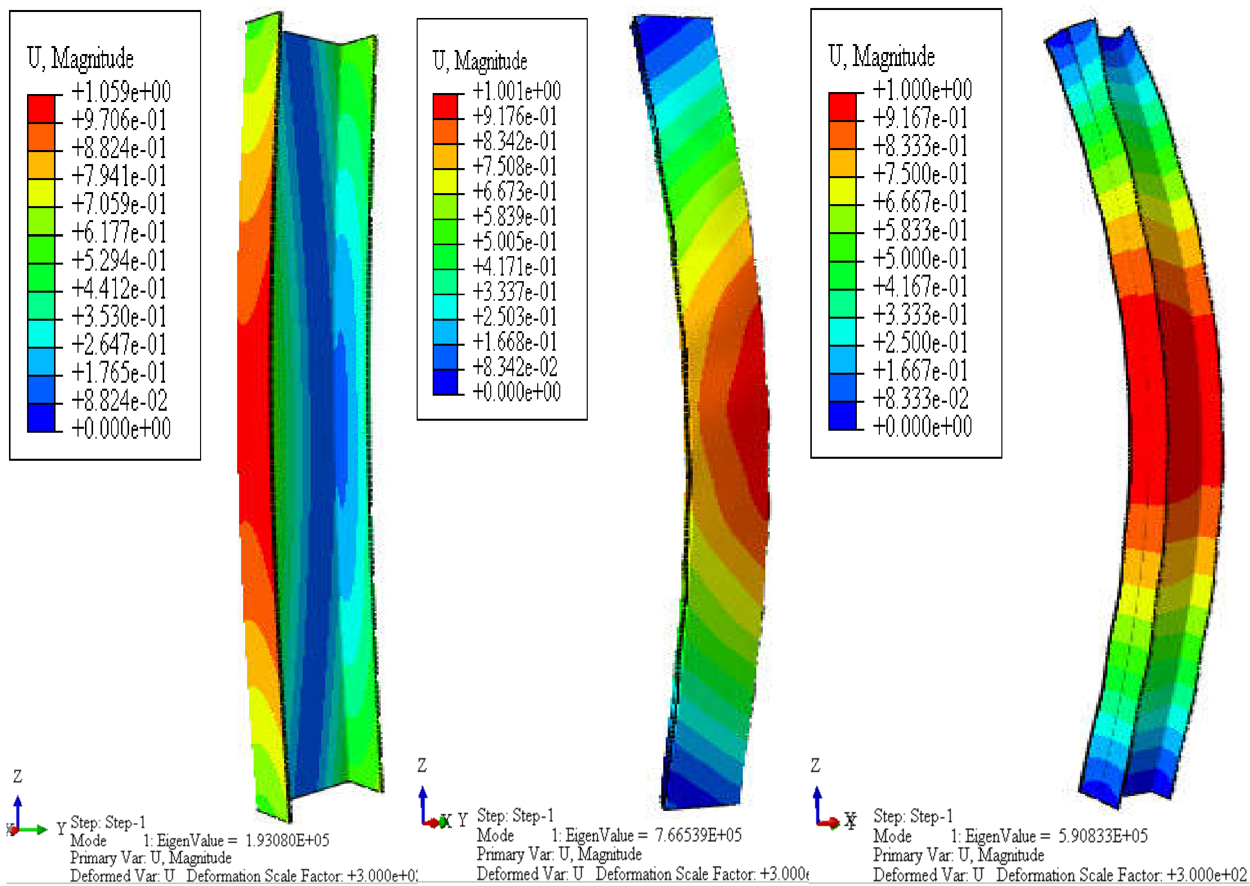
a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.4.Ultimate load and buckling modes of single and built-up 2000mm CFS-columns



a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.5.Ultimate load and buckling modes of single and built-up 2500mm CFS-columns



a)Single CFS- column b)BU CFS- column c) BTB CFS- column

Figure 4.6.Ultimate load and buckling modes of single and built-up 3000mm CFS-columns

4.2. Discussion

In this study two results have been discussed: the ultimate load and failure modes of single and built-up CFS channel columns. Calculation shows that the back-to-back I-section and face-to-face box-up sections yield higher value compared to double of single channel value in term of its buckling capacity. As the column lengths were increased the ultimate load was decreasing as expected. However the reality column did not fail in linear relation to its slenderness. Various factors could affect the column strength, such as imperfection in length and imperfection in cross-section of the CFS column. The short column fails due to yielding of cross section; hence it can achieve full effective cross-sectional strength and effective to act as built-up column. Two considerations are used in calculating the BTB and BU capacity. First, the CFS is analyzed as I-section of BTB and box section for BU. Second, both BTB and BU is analyzed as double of single channel capacity. The length of the column was determining the slenderness ratio of the column. The tables tabulated below shows the FEA results in terms of buckling strength for both single and dual built-up CFS columns predicted in ABAQUS

Table 4.1.Single channel FEA column load in ABAQUS

Designation (mm)	Length (mm)	Slenderness $\lambda(L_{eff}/i)$	maximum buckling load(KN)	
			Single channel (C)	2*Single channel (2C)
200X80X4	600	25	465.862	931.724
	1000	42	441.712	883.424
	1400	59	435.572	871.144
	2000	84	426.793	853.586
	2500	105	278.621	557.242
	3000	126	193.08	386.16

Table 4.2.Back-to-back (BTB) FEA column load in ABAQUS

Designation (mm)	Length (mm)	Slenderness $\lambda(L_{eff}/i)$	maximum buckling load(KN)		Ratio BTB/2C
			Back-to-back (BTB)	2*Single channel (2C)	
200X80X4	600	25	977.84	931.724	1.05
	1000	42	919.468	883.424	1.04
	1400	59	899.547	871.144	1.03
	2000	84	888.837	853.586	1.04
	2500	105	842.272	557.242	1.51
	3000	126	590.833	386.16	1.53

Table 4.3.Face-to-Face, Box-up (BU) FEA column load in ABAQUS

Designation (mm)	Length (mm)	Slenderness $\lambda(L_{eff}/i)$	maximum buckling load(KN)		Ratio BTB/2C
			Face to face box-up (BU)	2*Single channel (2C)	
200X80X4	600	25	993.315	931.724	1.07
	1000	42	919.006	883.424	1.04
	1400	59	898.236	871.144	1.03
	2000	84	887.592	853.586	1.04
	2500	105	882.883	557.242	1.58
	3000	126	766.539	386.16	1.99

Table 4.4.the ratio of single to built-up column (Experimental)

Length (mm)	Single C Ultimate load (kN)	BU Ultimate load (kN)	Ratio BTB/single	BTB Ultimate load (kN)	Ratio BU/single
600	127.5	300.1	2.40	305.6	2.39
1000	124.75	290.8	2.33	294.0	2.35
1400	100.0	209.3	2.093	239.1	2.39
2000	139.6	279.0	1.99	234.4	1.67
2500	109.5	281.1	2.56	149.5	1.36
3000	64.52	150.4	2.33	150.4	2.33

Table 4.5.the ratio of single to built-up column (FEA in ABAQUS)

Designation (mm)	Length (mm)	Slenderness $\lambda(L_{eff}/i)$	Single C ultimate load (KN)	BU ultimate load (KN)	ratio BU/C	BTB ultimate load (KN)	Ratio BTB/C
200X80X4	600	25	465.862	993.315	2.099	977.84	2.132
	1000	42	441.712	919.006	2.082	919.468	2.081
	1400	59	435.572	898.236	2.065	899.547	2.062
	2000	84	426.793	887.592	2.083	888.837	2.080
	2500	105	278.621	882.883	3.023	842.272	3.169
	3000	126	193.08	766.539	3.060	590.833	3.970

The short column capacity can be predicted as doubling of single capacity; however, it was slightly differed. Meanwhile, long columns fail due to varieties of buckling cause the built-up columns difficult to attain its full cross-sectional strength.

The higher differences were found at the slenderness ratios for length of 2500 mm and 3000 mm. 2500mm and 3000mm length of both BU and BTB results in higher ultimate load compared to buckling load when designated as dual built-up-section and considered unsafe. The BU and BTB columns observed were fails due to elastic local and distortional buckling of cross-section at a certain point along the column; the FEA load of BU and BTB column was higher as it has approached to its compression capacity. The differences in results may be due to element slenderness, the effect lips to add up the overall stiffness of the sections and the failure of column is not governed by this two buckling mode, there may be interacting buckling modes especially for intermediate and high slenderness ratio for all column types. The ratio of single to built-up columns greater than 2 were observed for all columns except for 2500mm and 3000 mm length of built-up BU and BTB CFS columns as tabulated in tables above.

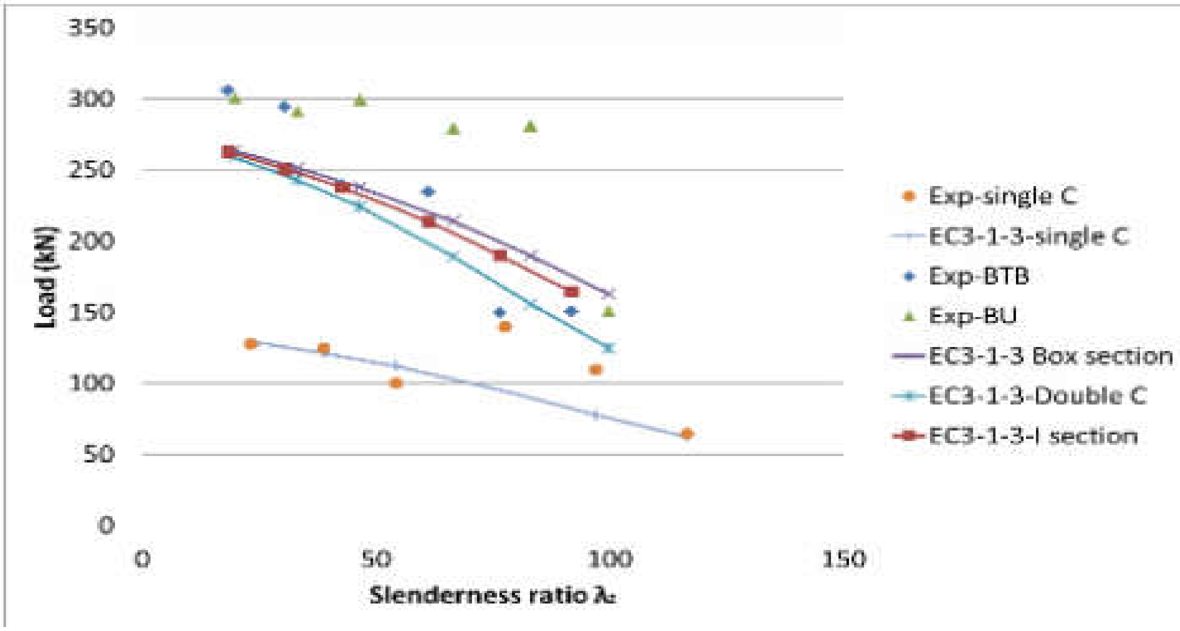


Figure 4.7.Ultimate capacity of single (C), face to face box-up (BU) and Back-to-back (BTB) cold formed steel (CFS) columns from test result (Experimental).

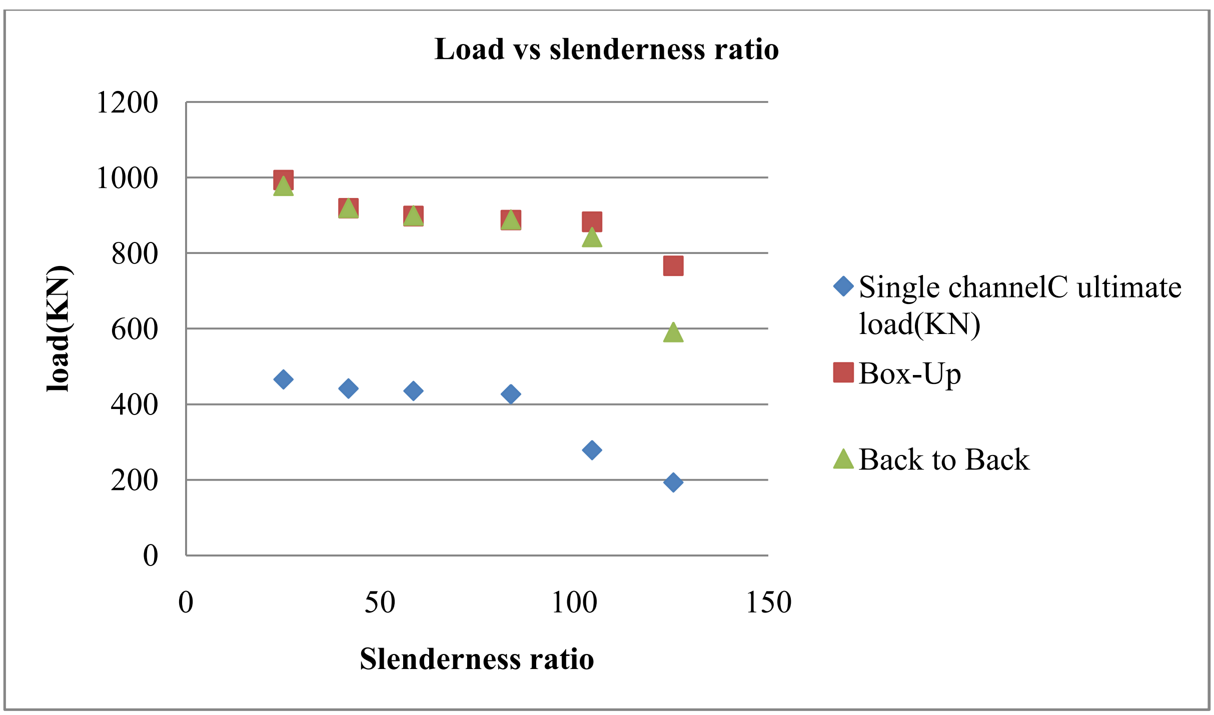


Figure 4.8.Ultimate capacity of single (C), face to face box-up (BU) and back-to-back (BTB) cold formed steel (CFS) columns from FEA in ABAQUS

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusions

Elastic buckling analysis of single and built-up cold-formed steel column was performed with numerical analysis. ABAQUS finite element software was used to analyze the ultimate capacity and buckling mode of single and built-up CFS un-lipped rectangular channel columns from short to long lengths and pin-end boundary conditions. The analysis results show that the dual-channel with different orientation can increase the ultimate capacity of the compression column. In each case, the elastic buckling loads was determined using the FEA in ABAQUS taking into account the actual column length, material non-linearity and end boundary conditions.

According to the results obtained from the findings of performed numerical, finite element analysis and literature review; the following conclusions were drawn:

- ☑ Ultimate buckling loads of both single and dual built-up CFS channel columns decrease as the length increases.
- ☑ Non-linear stress strain behavior with a markedly low proportionality stress leads to a significant loss of stiffness at low loads and this is reflected in the analysis results.
- ☑ Mainly three types of interaction buckling failures were identified for CFS: (i) Local flange and global, (ii) local web and global and (iii) local web/flange and global interaction buckling. The reduction in ultimate load capacities of CFS subject to interaction buckling was greater with local web and local flange buckling than with over all buckling.
- ☑ The dual built-up channel has significant influence on the columns failure mode. The column buckling was postponed effectively or even prevented from happening due the presence of built-up dual channel. The dual built-up channels had significant contribution on the columns ultimate load under the axial centric loading. The ultimate capacity of the column with dual built-up channel was greater by more than 51% of the column without dual built-up channel.
- ☑ The structural response of the built-up CFS columns composed of two un- lipped channels in this study was significantly affected by the local and distortional buckling than overall buckling behavior.
- ☑ For the intermediate column distortional buckling is the dominant failure. The failures pattern of local buckling was observed from 600 mm to 1400mm length of members. The failures pattern of local and distortional buckling was observed for 2000 mm and 2500 mm length of columns. Distortional and flexural buckling was observed when the length of column increases to 3000mm.
- ☑ The study and review helped better understanding of the buckling modes and the effect of channel orientations on the buckling behavior. The orientation of the column substantially impacts the ultimate load and buckling mode shapes of the built-up dual columns.

- ☑ Comparison of test results with FEA shows good agreement and that FE models can be used for predicting buckling behavior of the un-lipped channels that were built up.

5.2. Recommendation

The following recommendations should be considered for future researchers and practitioners to further investigate the behavior of built-up column sections considering different CFS shape and orientations.

- ☑ Study on the effect of varying plate slenderness ratios in terms of web-flange ratio and flange-thickness ratio on the behavior of back-to-back and face to face built-up CFS column shapes with and without a gap by more finite element modeling.
- ☑ Further in depth study on buckling behavior of slender built-up CFS columns.
- ☑ Additional work is needed to provide experimental data on different built-up cross-section types, orientation details and layouts, and continue to explore interacting buckling modes of built-up columns.
- ☑ Particular attention should be given to the interaction of global and local buckling which can result in excessive sensitivity to imperfections and in unstable behavior.
- ☑ Experiments on selected CFS sections subjected to local and global interaction buckling modes.
- ☑ The interaction of local and distortional buckling, especially for slender plain and lipped sections, should be investigated more precisely.
- ☑ The overall behavior of cold-formed steel dual assemblies under lateral and axial loads should also be studied.
- ☑ To ensure safety and prove the reliability of built-up CFS channel sections, it is necessary that member should be analyzed nonlinearly considering other sources of nonlinearities, cold-forming effect on material property, residual stresses, effect of buckling interaction and initial imperfections as literature states that these have an influence on the performance of CFS members, before the cross-section is adopted in the industry.
- ☑ Further investigations using finite element analysis is required to examine the influence of CFS steel material characteristics, imperfections, cross section slenderness and column length on the distortional buckling behavior.
- ☑ Evaluation of efficiency of single verses double channel with different other section shapes and orientations.
- ☑ The material properties and initial imperfections should measured for all CFS sections
- ☑ The lack of researches conducted on CFS members especially regarding its behavior when subjected to compression with different loading types is still an important issue to be addressed, namely in columns with various restraint condition.

REFERENCES

ABAQUS. (2013). Analysis User's Manual Version 6.13-1. SIMULIA,: CEA.

Abhishek Dangi. (2017). Nonlinear finite element bending analysis of Cold-Formed Steel of Z section beams. The Plymouth Student Scientist , p. 166-194.

Ahmed Shamel Fahmy.,Sherine Mostafa Swelem.,Rowida Saad Farrag. (2018). Estimating the Ultimate Load of Cold-Formed Steel Double Lipped Channel Columns Using a Simplified Method. Engineering Science. , 1-10.

Anbarasu., M. (2019). Behaviour of cold-formed steel built-up battened columns composed of four lipped angles: Tests and numerical validation. Advances in Structural Engineering. , 51–64.

André Filipe Ferreira Escudeiro. (2016). Experimental and numerical analysis of compressed cold-formed steel elements with sigma-plus cross-sections subjected to fire. 01-84.

Ben young and Gregory J.Hancock. (2005). Web crippling behavior of cold-formed steel sections with single web. Steel structures 5. , 79-85.

Bernard Godfrey. (2003). Steel Designers' Manual. 6thEd. UK:: The Steel Construction Institute.

Bharat C.Halagalimath.and Kiran Koraddi. (2019). Study on the Behavior of Cold-formed Channel Sections under Flexure. Bharat C.Halagalimath Journal of Engineering Research and Application. , pp 36-43.

Busanaboyina Jagadish Chakravarti., Bhamidipati Sai Krishna. (2019). Behaviour of Cold Formed Light Gauge Steel Angle Columns Subjected To Eccentric Loading. International Journal of Engineering and Advanced Technology (IJEAT). , 384 – 396.

Chernova Anna. (2019). Distortional buckling of cold-formed steel member. Saimaa University of Applied Sciences Technology. , 01-50.

David C. Fratamico., Shahabeddin Torabian., Kim J. R. Rasmussen., and Benjamin W. Schafer. (2017). Buckling and Collapse Behavior of Screw-Fastened, Built-Up Cold-Formed Steel Columns of Varying Cross-Section Size: Experimental Investigation. Proceedings of the Annual Stability Conference . (pp. 01-21). San Antonio, Texas.: Structural Stability Research Council.

David C. Fratamico.,Shahabeddin Torabian.,Kim J. R. Rasmussen.,Benjamin W. Schafer. (2016). Experimental Investigation of the Effect of Screw Fastener Spacing on the Local and Distortional Buckling Behavior of Built-Up Cold-Formed Steel Columns. Wei-Wen Yu International Specialty Conference on Cold-Formed Steel Structures. (pp. 555-570). Baltimore, Maryland, U.S.A.: Missouri University of Science and Technology.

EBCS EN 1993-1-1:. (2013). Euro code 3: Design of steel structures: Part 1-1: General Rules and Rules for Buildings. Ethiopia: Addis Ababa.

EN 1993-1-3:. (2004). Eurocode 3. Design of steel structures. Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting. Brussels.: European Committee for Standardization CEN.

Fadhluhartini Muftah.,Mohd Syahrul Hisyam Mohd Sani.,Shahrin Mohammad.and Mahmood Md. Tahir. (2014). ULTIMATE LOAD OF BUILT-UP COLD FORMED STEEL COLUMN. ARPN Journal of Engineering and Applied Sciences. , 2095-2101.

Francisco Javier Meza Ortiz. (2018). The Behaviour of Cold-Formed Steel Built-up Structural Members. The University of Sheffield.

IS:811-1987. (2011).Indian Standard.. specification for cold formed light gauge structural steel sections.

John L. Dawe.and Geoffrey L.Kulak. (1984). Plate Instability of W Shapes. Journal of Structural Engineering,ASCE. , 1278-1291.

K.Thiyagu, P.Kanaka, R.Elanchezhiyan. (2019). Behaviour of Cold Formed Built - Up Steel Compression Member. International Journal of Recent Technology and Engineering (IJRTE) , 2277-3878.

Kim J.R. Rasmussen.,Mani Khezri.,Benjamin W. Schafer.and Hao Zhang. (2019). THE MECHANICS OF BUILT-UP COLD-FORMED STEEL MEMBERS. School of Civil Engineering, Research Report R965. (pp. 01-160). Sydney.: School of Civil Engineering ,The University of Sydney.

Krishanu Roy.,Hieng Ho Lau.,Boshan Chen.,Tina Chui Huon Ting.,and James B.P. Lim. (2019.). Effect of screw spacing on axial strength of cold-formed steel builtup box sections-numerical investigation and parametric study. The 2019 World Congress on Advances in Structural Engineering and Mechanics (ASEM19)., (pp. 01-21.). Jeju Island, Korea.

Krishanu Roy.,Hieng Ho Lau.,Boshan Chen.,Tina Chui Huon Ting.and James B.P. Lim. (2019). Compression tests on cold-formed steel built-up box sections. The 2019 World Congress on Advances in Structural Engineering and Mechanics (ASEM19)., (pp. 01-16). Jeju Island, Korea.

Krishanu Roy.,Tina Chui Huon Ting.,Hieng Ho Lau.,James B.P. Lim. (2018). Experimental investigation into the behaviour of back-to-back gapped built-up cold-formed steel channel sections under compression. Wei-Wen Yu International Specialty Conference on Cold-Formed Steel Structures (pp. 283-297). St. Louis, Missouri, U.S.A.: Missouri University of Science and Technology.

Krishanu Roy.,Tina Chui Huon Ting.,Hieng Ho Lau.,Shashank Bhatnagar.,and James B.P. Lim. (2019 .). Finite Element Modeling of Back-to-Back Built-Up CFS Un-Lipped Channels under

Axial Compression. Materials Science Forum. (pp. pp 819-827.). Switzerland.: © 2019 Trans Tech Publications Ltd.

L.H. Martin.,J.A. Purkiss. (To EN 1993 and EN 1994). Structural Design of Steelwork . Burlington:: Butterworth-Heinemann Elsevier.

M. Anbarasu.,Bharath Kumar.,S. Sukumar. (2014). Study on the Capacity of Cold-formes Steel Built-up Battened Colums Under Axial Compression. latin american journal of solids and structures. , 2271-2283.

M. Rathipraba.,S. Vijayanand.,M. Vignesh. (2020). Numerical validation on strength and behaviour of cold-formed steel built-up battened column. International Journal of Advanced Science and Technology. , pp. 5263 - 5272.

Maura Leece.and Kim J. R. Rasmussen. (2004). Experimental Investigation of Distortional Buckling of Cold-Formed Stainless Steel Sections. Seventeenth International Specialty Conference on Cold-Formed Steel Structures. (pp. 383-398). Orlando, Florida, U.S.A.: Missouri University of Science and Technology.

Moosbrugger Charles. (2002). ASM International. Atlas of Stress-Strain Curves (pp. 01-816). Printed in the United States of America.

Ranawaka Thanuja. (2006.). Distortional Buckling Behaviour of Cold-Formed Steel Compression Members at Elevated Temperatures. School of Urban Developments Queensland University of Technology .

Roshan S Satpute.and Valsson Varghese. (2012). Building Design Using Cold Formed Steel Section. International Refereed Journal of Engineering and Science (IRJES). , PP.01-16.

Siti Nur Rahmah Anwar.,Buan Anshari.,A. Sjamsjar Rachman.,Suryawan Murtiadi.andNurchayati. (2020). Mechanical Properties of Cold -Formed Steel Under Tensile Testing. Proceedings International Conference on Science and Technology (ICST). (pp. pp: 261-270). e-ISSN: 2722-7375.

Thomas H.-K. Kang, Kenneth A. Biggs, and Chris Ramseyer. (2013). Buckling Modes of Cold-Formed Steel Columns. IACSIT International Journal of Engineering and Technology, Vol. 5, No. 4. , 447-451.

W. Reyes.and F. A. Guzman. (2010). Comparative Behavior of Built-up Cold-formed Box Sections under Rigid and Flexible End Support Conditions. Twentieth International Specialty Conference on Cold-Formed Steel Structures. (pp. 105-119). St. Louis, Missouri, U.S.A: Missouri University of Science and Technology.

W. Schager Benjamin. (2002). Progress on the Direct Strength Method. Sixteenth International Specialty Conference on Cold-Formed Steel Structures. (pp. 647-662). Orlando, Florida USA.: Missouri University of Science and Technology.

Wei-wen Yu. (2000). Cold-formed steel design 3rd ed. Printed in the United States of America,New York.: John Wiley and Sons, Inc.

Whittle, J. and Chris Ramseyer. (2009). Buckling capacities of axially loaded, cold-formed, built-up channels. Thin-walled Structures. , 190-201.

Yao Xingyou. (2021). Experimental Study and Direct Strength Method for Cold-Formed Steel Built-Up I-Sectional Columns under Axial Compression. Mathematical Problems in Engineering. , 01-22.

Yonas T.Y., Temesgen W., Senshaw F.W. (2018). Finite Element Analysis of Slender Composite Column Subjected to Eccentric Loading. International Journal of Applied Engineering Research ISSN, 11730-11737.

Ziqi He.,Qixiu Li.,Ben Schafer.,Xuhong Zhou. (2020). Structure Performance of Cold-formed Steel columns reinforced by Channel Sleeve under Axial and Eccentric compression. Proceedings of the Cold-Formed Steel Research Consortium Colloquium. , 1-8.

APPENDICESS

Non-linear material data for Steel grade S355, Basic yield strength (fyb =355 N/mm^2) and Ultimate tensile strength (fu=470 $N/mm^{2)}$ calculated as follow (Moosbrugger Charles., 2002).

1. Engineering elastic strain: elastic strain, $\in_{eng,el} = {}^{fyb}/_{E} = {}^{355Mpa}/_{210Gpa} = \underline{\underline{0.0016}}$
2. True elastic strain: to determine the true strain, the instantaneous change in length (dl) is divided by the length (l): $\in_{true,\ el} = \int_{lo}^{l} dl\ /l = \ln (l/l_o) = \ln (1+ \in_{eng,\ el}) = \ln (1+0.0016) \approx \underline{\underline{0}}$
3. Engineering plastic strain: maximum residual strain after failure taken to be $(\in_{eng,\ pl=}\ 0.2)$ (Yonas T.Y., etal. 2018).
4. True yield stress, σ_{true} is expressed in terms of engineering stress, by:

   $\sigma_{true} = fyb\ (1+ \in_{eng,\ el}) = 355Mpa\ (1+0.0016) = \underline{\underline{355.6\ MPa}}$
5. True ultimate stress, plastic:

   $\sigma_{true,\ pl} = fu\ (1+\varepsilon_{eng,\ pl}) = 470Mpa\ (1+0.2) = \underline{\underline{564\ MPa}}$
6. True plastic strain(Atlas, 2002).: $\in_{true,\ pl} = \in_{total} - \sigma_{true}/E = 0.2 - {}^{355.6Mpa}/_{210Gpa} = \underline{\underline{0.181}}$