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LOAD CAPACITY OF AXIALLY LOADED COLUMN OF COLD FORMED STEEL STRUCTURE

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BY

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Assist. Prof. Dr. M. Tolga GÖĞÜŞ

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LOAD CAPACITY OF AXIALLY LOADED COLUMN OF COLD FORMED STEEL STRUCTURE

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Khalilullah MOJADDIDY

ABSTRACT

LOAD CAPACITY OF AXIALLY LOADED COLUMN OF COLD FORMED STEEL STRUCTURE

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The axial load capacities of the compression members are often controlled by local and distortional buckling, buckling modes can be to a substantial reduction in the load bearing capacity of these members. The current tendency towards deeper, more slender sections makes this stability problem even more critical. The current methods for calculating the axial load capacity of Cold-formed steel C-channel section columns is based upon experimental studies conducted different web and flange dimensions and 300 mm long. The results of full-scale testing collapse loads were compared to theoretical capacities of the current AISI and EC3.

This study is based on theory and experimental data which has been carefully analyzed and complied with this study. The results obtained from the experiment are compared with the analysis. In this study, the un-lipped and lipped columns analyzed using the EC3 and the AISI standards investigated which standard is more conservative. Finally, a suitable conclusion has been made and utilized in this thesis.

ÖZET

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Basınç elemanlarının eksenel yük kapasiteleri genellikle yerel ve çarpıtılmış burkulma tarafından kontrol edilir. Bu burkulma modları, bu elemanların yük taşıma kapasitelerinde önemli bir azalmaya sebep olabilmektedir. Günümüzde eğilim derin ve daha narin kesitlere doğru stabilite sorununu daha da kritik hale getirmektedir. Farklı flanş ve gövde ölçülerindeki boyu 300mm olan eksenel yüklü soğuk şekillendirilmiş C kanal kesitli kolonlar mevcut şartnamelerde kullanılan yöntemlerle hesaplanmıştır. Bu hesaplamalar yapılan testlerle birlikte incelenmiştir. Tam ölçekli çökme test yüklerinin sonuçları mevcut AISI ve EC3 ile hesaplanan teorik kapasiteler ile karşılaştırılmıştır.

Bu çalışma, teorik ve deneysel veriler dikkatle analiz edilmiş ve bir araya getirilmiştir. Deneylerden elde edilen sonuçlar analizlerle karşılaştırılmıştır. Bu çalışmada, soğuk şekillendirilmiş C ve kulaklı C kesitler EC3 ve AISI şartnamelerine göre analiz edilmiş ve şartnamelerin daha konservatif olduğu incelenmiştir. Bu tezde, son olarak, elde edilen sonuçlar değerlendirilmiştir.

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It is my pleasure to express thanks to my family and friends for their support, encouragement and contribution they have made for my research to be successful.
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CHAPTER 1

INTRODUCTION

1.1 Introduction

In recent years a vital propensity of Civil Engineering industry is the application of progressively slender elements and structures. A characteristic appearance of this propensity is the largest application of Cold-formed Steel (CFS) members, which is supported not only by the development of production technology but also by the improvement in designs (standards, methods) and generally, computation techniques. The use of CFS structures is presently increasing in nowadays because of important progresses to manufacturing technologies and development of thin, high strength steels. Cold-formed channels are used extensively in a large area of light-weight construction, including steel frame houses and low-rise office building.

To support this, a thin-walled CFS structural member have been widely utilized in several applications including purlins and girts, steel framed housing, steel storage racks, lighting towers, sheeting and decking, and storage silos. Cold-formed sections have complicated deformation failure modes, initial imperfections and residual stresses produced during the Cold-formed process which changes from those of hot-rolled sections. Increased usage has led to further interest in the buckling and post-buckling behaviour and ultimate limit state of thin-walled Cold-formed sections. Consequently design specifications for CFS have been amended and continue to be amended. In this understanding, the compression of the column is displacement controlled and moments can arise at the supports. The support conditions found in practice to belong on the connection details and will usually be closer to fixed than pinned, as in the case of stud wall panels. Hence, it is important of the tests on Cold-formed lipped and plain channel columns compressed between fixed and pinned ends are presented.

Analysis were performed at different column lengths, which involved distortional buckling and pure local-buckling additionally overall flexural-torsional buckling and flexural buckling. The experimental and theoretical local buckling loads were also determined. The material properties, geometric imperfections and residual stresses were measured prior to testing. The aim of this article is firstly to present and compare the test strengths of fixed-ended Cold-formed plain channel columns with design strengths predict by three different specifications are the American Iron and Steel Institute Specification (AISI) for the Design of CFS Structural Members (1996), and Euro code 3 (EC3) Design of Steel Structures Part 1.3: Cold-Formed Thin Gauge Members and Sheeting (1996).

Nowadays, the field of CFS structures is among the busiest research area at the department of structural engineering. The major reason for the common use of the CFS members in the structural engineering field is because they are easy and cheap to fabricate, need minimal or less maintenance due to the zinc coating, no special tools nor heavy cranes are needed for them to be erected in many cases even lack of experience with the erection of steel structures may not be a problem. However, the use of CFS members in the construction industry is motivated by (i) the high structural efficiency of these profiles, namely by having a high strength/weight ratio, (ii) the wide range of shapes (cross-section shapes), (iii) the low-cost of production and transportation. The most commonly used shapes of CFS member are lipped channel, Z and C shapes. The figure below illustrates the most common cross-section shapes available for the construction industry.

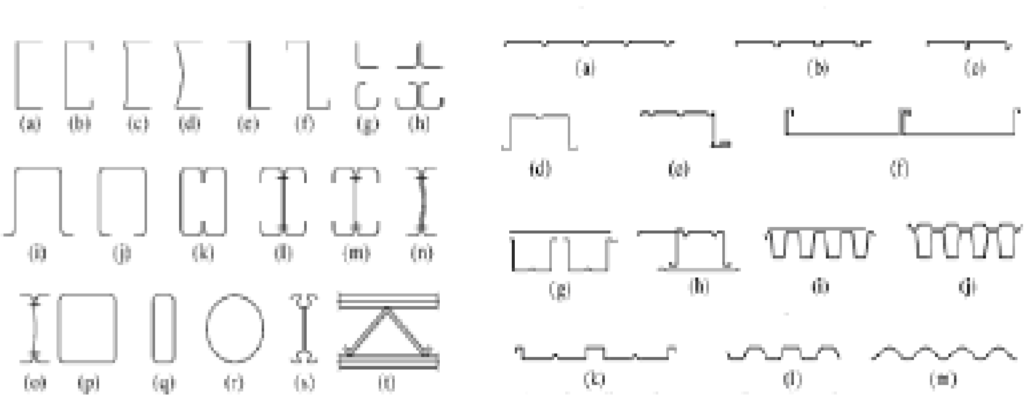


Figure 1.1 Most common Cold-formed cross-section configurations.

1.2 Development of CFS

From the late 1970s till now, CFS has brought about the diverse and important development in Europe, and there have been several researches and the improvement of new design codes. It started with the publication of a new Swedish design specification in 1982 followed by European approvals at different stages.

In relation to design methods, some specifications are based on Allowable Stress Design (ASD) approach while others are based on limit states design. For examples:

- The AISI are made up of both (ASD) and Load and Resistance Factor Design (LRFD).
- EC3: Design of Steel Structures Part 1.3 General Rules Supplementary Rule for Cold-formed Thin Gauge Members and Sheeting was published as a European pre standard in 1996 and is having substantial and increasing effect on CFS design throughout Europe.

However, having a thin and light-weight, CFS performs very differently compared to conventional hot-rolled steel. The characteristics of CFS make it high in strength but low in stiffness and because of this it can be easily deformed. The construction industry did not have enough confidence to largely manage CFS members until the 1940s when more understanding was developed. Since then, CFS has become famous and was used at a large scale. Although, individual columns were no longer sufficient to fulfil construction needs. In order to utilize it as load bearing members, designers came up with many methods to strengthen the individual column. This individual column is now one of the most commonly used sections in the field of CFS structures.

1.3 Application of CFS Structure

There has been a vast application of several variations of the trapezoidal profiles and shapes which calls for a wide usage in commercial and industrial buildings, restaurants, hotels, sports arenas, and other types of building constructions. These building materials encompass columns, beams, joist, studs, built-up sections and other components. The CFS is manufactured under room temperature using rolling or pressing and its elements strength is usually controlled by buckling. The CFS construction materials have proved to be more useful since it’s firstly introduction in 1946. Though, there have been many challenges to improve strength, deflection, lateral stability and construction of structures when dealing with both industrial and commercial projects, diverse progresses have been achieved in the area of profiled sheeting in recent times.

The composite steel, concrete flooring is in some other areas in which there have been fast developments in the past few years. Below are examples that illustrate the application of CFS structures that is in the pure line arena, framing, residential use building and roof trussing respectively.

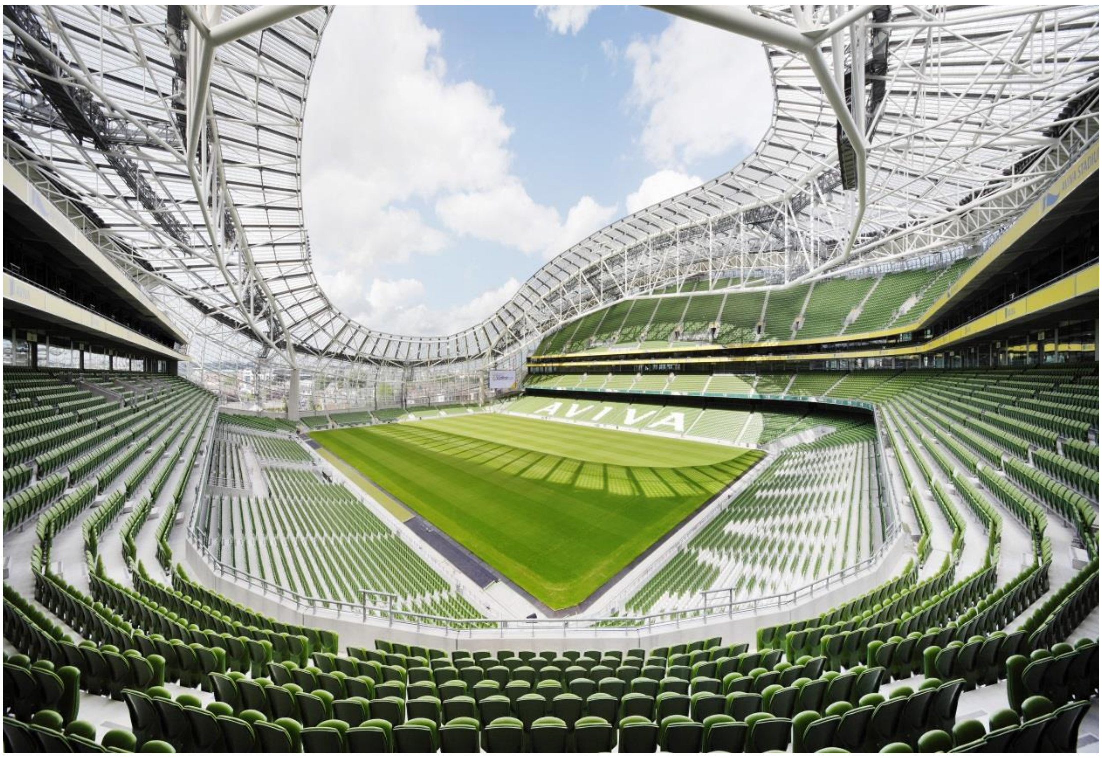


Figure 1.2 Pure Line Arena

Figure1.3 CFS Framing

Figure 1.4 CFS in Building

Figure 1.5 CFS Trussing

1.4 Design of CFS Structures

The first CFS standard was developed by the AISI in 1946. This design study was largely based on the study done by Professor George Winter. Currently, there are many international standards for the design of CFS structures such as EC3 Part 1.3, British Standards (BS) 5950-Part 5, AISI specifications, other the Standard Australia AS4600. These design standards use the Direct Strength Method (DSM) and Effective Width Method (EWM).

The EWM is a common method for determining the sectional strength of slender sections. This method considers each of the cross sections individually in its calculation. The effective area of the section consists of the effected areas of the flange, web and stiffener calculated separately.

The EWM column design covers four types of elements, namely:

a) Uniformly compressed stiffened elements,

b) Uniformly compressed stiffened elements with an edge stiffener,

c) Uniformly compressed unstiffened elements,

d) Uniformly compressed elements with multiple intermediate stiffeners.

Therefore, the type of elements can affect the calculation of effective width.

The effective area is calculated by multiplying the effective width of each element by its thickness *(A_e=b_{eff} t)*. The design of CFS is then obtained by the product of the effective area *(A_e)*, and nominal compressive stress of *(F_{ne})* of the CFS.

1.5 Advantages of CFS Structure

CFS is recommended over hot-rolled steel sections due to the following:

- The high strength to weight ratio is attained.
- High resistance to corrosion at ambient temperature.
- Uniform quality, easy and fast erection and installation.
- More precise detailing with high strength and stiffness in yield strength.
- High resistance to both insects and fungal infections.
- Large scale production, easy in making and non-combustibility.
- They are light for easy transportation and easy removal of delayed in case of the weather.

All these advantages can bring about reduction of cost in building /constructions.

1.6 Disadvantages of CFS Structure

- For most structures, the use of steel column is very economical because of their high strength to weight ratios. However, as the length and slenderness, its danger for buckling increases.
- Poor fireproofing

1.7 Objectives

The purpose of this research is to investigate the behaviour of C-channel columns based on experimental work and theoretical analysis. The objectives are outlined:

- Predict the design strength of CFS columns using the AISI and EC3.
- Study experimentally the behaviour of axially loaded CFS columns and examine design recommendation.
- The steady state test method is mainly used to investigate the structural behaviour of the column to obtain the load capacity of the column.

1.8 Thesis Outline

The following chapters focus experimental and theoretical analysis, the load capacity of steel column. Chapter 1 Brings out the motivation for the present study, and gives a clear picture of the objective to be pursued in this Thesis. Chapter 2 Contains a review of the relevant literature that covers previous studies conducted on CFS structure. Chapter 3 describes the types of buckling, explains the concept of EWM and presents the column end fixity and effective length, the buckling stress coefficients and also its factors. Chapter 4 discusses AISI and EC3 standard design codes. Chapter 5 investigates the experimental work and compares it with the standard design codes. Chapter 6 summarizes the main important conclusions achieved in this thesis with findings, recommendations and future works.

CHAPTER 2

REVIEW OF LITRATURE

2.1 Introduction

This chapter presents a review of available literature in the area of CFS structure design, research, test and design methods used in structural engineering and behaviour of structures under compression condition.

Also, some background knowledge on the theoretical and practical development of the design of CFS structures. Although the main purpose of the research is to investigate the behaviour of unlipped CFS columns single and component open section subjected to compression load, current design practice codes and experimental investigations which are carried out the last decades presented in this chapter. Literature relating to analytical and numerical investigations is presented in the first part, literature relating to experimental investigations is presented in the second part and studies on theoretical and numerical investigations are presented in the third part.

2.2 Analytical and Numerical Investigations

The Finite Element Analysis (FEA) model, the post buckling behaviour of CFS members subjected to axial compression was developed by Nabil et al. [1]. Finite analysis model was composed of a total Lagrangian non-linear 9-node ‘’assumed strain’’ shell finite element. Experimentally deduced residual stress difference and initial geometric imperfections were included. A special loading technique and a displacement solution algorithm were used to gain a uniform displacement condition at the edge loading edges. Twenty non-performed and performed tests on CFS stub columns were carried out. The test results with the results of finite element study were compared. Behaviour of axial and lateral displacement, buckling loads, ultimate loads and axial stress distributions found a common ground.

The strength and the behaviour of Cold-formed lipped channel columns using FEA studied by Young et al. [2]. In this study, a Finite Element Model (FEM) was developed to look into the behaviour and strengths of cold-formed plain and lipped channel columns compressed between fixed-ends and pinned-ends using ABAQUS. The ultimate loads and failure modes obtained were conducted against the column tests introduced by Rasmussen et al. [3].

Yan et al. [4] conducted the studies on channel columns with inclined edge stiffeners. The edge stiffeners were composed of simple lipped that were inclined at diverse angles both interiors and exteriors. A nonlinear FEM was developed and testified by using fixed-ended column tests. Geometric and material nonlinearity were added in the FEM. The results of experimental ultimate loads of the CFS channel columns with inclined edge stiffeners were closely predicted by using the FEM. The column strengths predicted by FEA and design column strengths calculated by using the specifications of New Zealand, America and Australia for the CFS structures were being compared. These design rules for designing an edge stiffened element were purposely based on tests of channels with simple edge stiffeners perpendicular to the flanges. Although, these specifications allowed the design of inclined edge stiffeners that were completely intuitive. This proved that the design column strengths calculated is based on the specifications with inclined edge stiffeners.

Ting [5] presented that the DSM was initially extended to the idea that the initial buckling strength for any cross section is easily accessible by numerical means. This approach was initially used in the analysis of simple cross sections such as the C-channel, where the ultimate strength was empirically linked to the critical load using a simple power law based on test results. Additionally, they introduced that the resolution was only approximate for sections other than simple cross sections, which the method was developed for. The DSM design guide proposed two potential models to analyse the critical elastic buckling strengths of built-up columns using the CUFSM. They are:

I. Modelling the built-up section as a rigidly connected I-section, and

II. Modelling the built-up section as an individual C-channel and taking the requisite values as twice of the C-channel.

From these models, model (i) gives the upper boundary of the calculated elastic buckling strengths whereas model (ii) gives the lower bound of the calculated elastic buckling strengths. Therefore, there is still room for improvement on modelling built-up columns using the CUFSM software.

2.3 Experimental Investigations

A research was carried by Seah [6] on the collapse behaviour of some uniformly compressed edge stiffened thin-walled sections. It was carried on fifty specimens under uniform compression. Among them 26 were discovered as inwardly-turned lipped channel sections while the others were outwardly-turned lipped top-hat sections. Due to their differences of the draft, EC3 design procedures were implemented to estimate the ultimate strength of the same edge stiffened thin-walled sections. When the width of the simple edge stiffener becomes very large, it makes the simple edge stiffened element of the same area smaller than the ultimate load of a component edge stiffened element of the same area. The research shows that have is shifting of the stiffener effective natural axis due to various rates of reduction in the effectiveness of the stiffener and plate that causes a vast reduction in the load carrying capacity. The estimated outcome was compared to which the reason of the approach is confirmed by an additional comparison of other source of experimental outcomes.

The results of experimental investigations carried out to estimate the mechanical properties and the built in residual stresses of CFS sections were offered by Nabil et al. [7]. The research was carried on channel shaped CFS sections made using cold roll-forming technique and the mechanical properties of the section was evaluated by the Tinsel coupon test [8]. The magnitudes and distributions of residual stresses were introduced by use of electrical strain gauges. Analytical models for the stress-strain relationship, residual stresses and the differences in the yield strength in CFS channel section have been introduced upon the experimental outcome. The accuracy and adequacy of the analysed model and characteristics have been observed by analysing both the experimental outcome of CFS sections subjected to axial compression loads with FEA results.

Dieter et al. [9] on the tests and design approach for plain channels in local and coupled local-flexural buckling based on EC3 they investigated experimental and theoretical programs on local and coupled local-flexural buckling and the interaction of both instability types. As a result the bending direction of the single symmetric plain channels essentially influences the local buckling failure, which again affects the flexural buckling failure.

Additional research was carried out by Young et al. [10] on the both EWM and DSM. They tested, fixed-ended channel columns with complex stiffeners in length ranging from 500mm to 3500mm. The experimental program includes four series with different thicknesses and flange widths. The four series are 1.5mm and 1.9mm thickness each with 80mm and 120mm flange widths. They concluded that the design strengths predicted by the EWM of AISI specification are commonly not conservative for the channel columns with complex stiffeners. The failure modes predicted are generally in accordance with the failure modes studied in the test for long columns, but not for short and intermediate columns.

Ting [5] recalculated the design calculated results using the DSM. Same as the EWM, the failure modes predicted by the DSM was accurate in the tests for long columns, but not for the short and intermediate columns. The comparison also shows that the DSM provides a good understanding with the column strength obtained from the test. The DSM is generally more accurate compared to the EWM because the EWM is not sensitive to buckling interaction, which is a common buckling mode in long columns. Unlike the EWM, the DSM covers the failure modes resulted from the interaction between local and overall buckling, interaction between distortional and overall buckling, also distortional buckling alone.

The distortional buckling of light-gauge lipped channel short columns was investigated by Morino [11]. Compression tests were conducted with plate thickness of 1mm and the fundamental behaviour of thin-walled compression members failing in the local and distortional buckling modes were classified. 18 specimens with three different web depths and three different web-flange width ratios were tested. The test results were compared with the numerical analysis based on the yield line theory. Stiffness, strength and failure modes were also investigated. The local and distortional buckling types were two types of the failure modes observed. The modes shown in the tests coincided very well with failure modes determined from the analysis. It was concluded that yield patterns assumed in the analysis, were suitable to estimate the ultimate strength and failure mode accurately.

The distortional buckling behaviour of a series of innovative CFS columns was studied by Narayanan et al. [12]. More than 15 laboratory experiments were conducted in these innovative steel columns of intermediate length under axial compression. All these columns failed by distortional buckling with very small post-buckling strength. The buckling properties of the columns and the section were specified using the Finite Strip Analysis Program which is called thin-walled.

The nonlinear ultimate strength behaviour of the columns and distortional buckling were examined in detail using FEA program ABAQUS. The FEA was composed of related geometric imperfections and residual stresses. The ultimate design load capacities were calculated using the provisions of Australian CFS structures standard AS/NZS 4600-1996, and was compared with FEA with the results from related experiments. Several parametric studies were also performed by modifying the yield strength, thickness and column length. It was found that residual stresses had an insignificant effect on the ultimate load. The results of ultimate strength experiments, FEA and the design standards were compared, and suitable recommendations were prepared. Additional study required to enhance the accuracy of design, code predictions for distortional buckling strengths of innovative CFS columns made of thin, high strength steels was shown to be necessary by this investigation.

A series of experiments on lipped channel section column made up from cold-reduced high strength steel of thickness 0.42 mm with nominal yield stress 550 MPa was conducted by Young [13]. It was proven from the experiments that interaction of local buckling and the distortional buckling had an important effect on the strength of the sections formed from such thin high strength steel. Two simple design methods were suggested for intermediate length lipped channel sections which gave a lower limit of the experiment outcome. Because of the fact that distortional bucking was ignored, it was found that prediction by the NAS (AISI 2001) gave an un-conservative result. Following that it ignores the interaction of local and distortional buckling AS/NZS 4600 predicts an un-conservative result for intermediate length.

The lipped C-sections were observed with local carrying capacities of CFS cut stub columns were discussed by Lam et al. [14]. Initial geometric imperfections might be as a result of cutting of roll-formed steel C-sections by the extent of cross sections distribution along the lengths. 10 stub columns were cut from two separate sections and tested under axial compression. This was carried out in an attempt to find out about the effect of the geometric imperfections on the ultimate strength and modes of failure of the cut stub columns. Distortional mode of failure was observed while the sign of local buckling was observed with the webs. Upon BS 5950: parts, the ultimate compressive strengths which were derived from the test outcomes were 75 to 77 % of the strength estimated. The significant reduction of the ultimate strength of the stub columns is indicated by geometric imperfections caused by cutting.

Kalavagunta et al. [15] testable study on Carbon Fiber Reinforced Polymer (CFRP) strengthened Cold-formed channel column was investigated several lengths were used and tested, the column ultimate strength obtained from the test results compared with the strengths estimated using the AISI for CFS sections. Comparison of the analytical and experimental study indicates that the flexural capacity of CFRP strengthened steel channel columns can be predicted within a preferable accuracy.

The effective shift of effective centroid was investigated by Ting [5]. They studied in the design CFS plain channels, flat-ended and pin-ended columns. Established along the experimental studies and finite element investigations by many researchers, their investigation evaluated the design procedures on:

- Minor axis bending with stiffened element in tension
- Minor axis bending with stiffened element in compression
- Flat-ended columns
- Pin-ended columns
- Beam-columns

They illustrated that, the design methods should differ relating to the end support conditions. They suggested using the equation to design fix-ended columns and beam-columns equations. The eccentricity of the load in the basis of the location of the load and the average deflections of the beam column was defined instead of the maximum deflections as in Young et al. [16]. A modified plate buckling co-efficient was suggested, due to the shift effective centroid, the beam-columns are better designed with column equation while pin-ended channels column is also better designed with beam column equations. Nevertheless, the shift of effective centroid is not of concern for doubly symmetrical built-up columns.

2.4 Theoretical and Numerical Investigations

The buckling capacity of irregularly shaped thin-walled structural members under any general load and boundary conditions expected to be predicted by the finite element method was introduced by AL-Bermani et al. [17]. At 30 degrees rectangular thin plate element was chosen. The symbolic manipulation was used to obtain precisely the linear and geometric stiffness matrices of these elements. For sections models were predicted by the use of lower order plate elements in combination with general beam examples column element to form a super element. The demonstration of the certainty, versatility and adequacy of the method was the reason the numerical examples of thin-walled structural elements, including flexural-torsional, local and distortional buckling modes were introduced.

Megnounif et al. [18] presented a design procedure for predicting the ultimate strength of CFS built-up columns based on the EWM from the EC3 and the DSM. In their investigation, they suggested many design approaches. Their proposal for the EWM included:

- EWM with buckling factor 4.0 and 0.43, and
- EWM with the buckling factor calculated from buckling stresses obtained from a come up with combination of spline Finite Strip Method (FSM).

For the DSM, design option contains:

- DSM with the local buckling strength obtained from the classical element method,
- DSM with the local buckling strength calculated by hand,
- DSM with a proposed equation for distortional buckling strength, and
- DSM with the buckling strength calculated from buckling stresses obtained from a presented combination of spline FSM.

They expressed that whether performed as a classic hand method or spline FSM, the DSM provides remarkably various predictions as compared to their test data.
Thus, the accuracy of the DSM requires more investigation.

The certainty of the available codes of practice for the design of CFS sections was inspected by Chou [19]. BSI (BS: 5950 Part 5-1987), AISI (1996), and EC of practice European Coal and Steel Community-1987 (ECCS-1987) were reviewed. The ultimate load capacities of the columns predicted by codes of practice were compared to the chosen experimental results. Plain sections, lipped sections, hat sections and compound lipped hat sections were considered for the study. It was discovered that the accuracy of the codes of practice varied with several types of cross-sectional shapes. It was also examined that all the codes of practices were not precisely sufficient. From the results, though they produced similar results sometimes, it was observed that each code of practice predicted the behaviour differently.

Gao [20]. Presented the effects of cross section elements slenderness on the behaviour and ultimate strength of a built-up stub column. They found out that, the ultimate strength of the columns with rectangular cross-sections (large web-flange ratio) is normally lesser than that of the columns with the square cross-section (smaller web-flange ratio). As the web-flange ratio becomes larger, the failure mode changes from material strength failure to buckling failure. Although the investigation was not obvious for columns with small width-thickness ratio, their investigation shows that, the slenderness of a plate element effect the overall strength and behaviour and also effects the effectiveness of the cross-section of the columns. The overall slenderness ratio of the columns has little effect on the design strength of the cross-section when designing stub columns. They also found that, the AISI provisions results when compared with the actual test results for stub columns, the results were exceedingly conservative.

The ultimate load carrying capacity and post buckling characteristics of thin-walled Cold-formed and welded stub columns under a constant eccentric load was studied by Guo et al. [21]. With regards to the material nonlinear finite strip and geometric analysis, theoretical investigation was done to the vast deflection elastic-plastic range. The result of the geometry of the section on the ultimate load was examined based on the numerical results derived by the FSM. There was an existence in the optimum aspect ratio of sections where the stub column displayed the most optimal design profit, and there was an optimal load eccentricity in Cold-formed channel stub columns in which the stub column could achieve maximum ultimate percentage incomplete section yield load.

The buckling and post-buckling behaviour of thin-walled stiffened elements under uniform compression with geometric and material uncertainties which were modelled by a convex model was examined by Pantelides [22]. The results showed that with regards to elastic buckling, the reduction in the buckling load because of geometric uncertainties could be valuable. The effective width uncertainty was observed to differ within the range of testable results that is in case of post-buckling.

Schafer et al. [23] studied the behaviour and design of CFS sections with single and multiple intermediate stiffeners in compression, available testable data were applied to calculate critically the AISI specifications and EC3. AISI specification was observed to be un-conservative for bending strength prediction of sections with multiple intermediate stiffeners and EC3 yielded conservative results. A FEM was developed and calibrated using existing testable data. A vast parametric research was carried out using the FEM. The outcomes of this research were used to achieve a comprehensive understanding of the behaviour of the elements and to help calculate alternatives to existing design procedures. The recommendations were made to improve strength predication procedures, based on testable data and finite element study.

A new analytical technique called Erosion Critical Bifurcation Load (ECBL) approach to the stability problem of thin-walled Cold-formed members was developed by Dubina et al. [24]. A very convenient theoretical conception of the interactive buckling phenomena and an accurate design procedure were also presented. The variety of the cross-section shapes in addition to the concentrated number of testable outcomes in two range , have brought about the concept of changing the test results can be found controlled with numerical outcomes. In this reason, a wide curvature elastic-plastic FEM was created based on test results. This model considers geometrical imperfections and inelastic behaviour of members. Just a few numbers of experiments were needed to verify the FEM. This example could be further utilized to obtain numerical results for the specimens through the coupling limit and taken into account for the computation of the corrosion point of critical bifurcation load. Valuable numerical outcomes and comparison with experiments were presented. The ECBL method offered a more suitable theoretical realizing of the interactive buckling case and made it possible to create. Beginning from a hard scientific basis, in order to prove the way to incorporate the design approaches used in stability analysis was an accurate design procedure.

Srinivasa Rao et al. [25] studied the torsional-flexural buckling of singly symmetric CFS beam columns. A review was made in the various methods presented in the international standards and other literature was made of tor the evaluation of strength of singly symmetric elements subjected to eccentric compression in the plate of symmetry. It was seen that, the process needed by the AISI and EC3 for considering torsional-flexural buckling stress, disregarded the effect of local buckling and the strength was calculated multiplying the effective area by the stress. Additional, the effective width of the element should be taken into account for the member stress gradient of the element when considering the effective section properties.

The outcome of shift of effective centroid of channel columns experimentally based on fixed-ended test results was experimentally studied by Young et al. [26]. The effective width rules of the American and Australian standards did not accurately predicted for lipped channels with slender flanges but shows the direction and magnitude of the movement of effective centroid for plain channels. The study suggested simple adjustment to the current effective width rules to enable understanding between the measured and predicted shifts of the effective centroid for lipped channels. The changes were inducted to producing more precise design strengths for lipped channel columns.

The applicability of EC3 to the prediction of the compression capacity of short fixed-ended columns with various cross-sections was assessed by Kesti et al. [27]. The compression capacity was specified by combining the effective width of plane elements due to local buckling and the effective stiffener thickness due to distortional buckling. Numerical calculations were done in order to compare alternative methods for determining the minimum elastic distortional buckling stress in compression. It was concluded that the end boundary conditions had an important influence on the distortional buckling strength, and also on the compression capacity of short columns. The capabilities of EC3 to the prediction of the compression capacity of short fixed-ended columns with different cross-sections were presented. Selected experiments from compression tests on C-channels, hat and rack upright-sections were compared with results of EC3. The method was improving via defining distortional buckling stress Generalized Beam Theory (GBT). This goes on to better compromise between the theoretical predictions and the experimental results.

Chou et al. [28] presented that hat sections stub columns and FEA on post buckling characteristics of thin-walled Cold-formed lipped channels. The specification was as like the well-controlled stub column test carried out by the Zaras et al. [29]. A nonlinear FEA was used to derive the ultimate load carrying capacity and the predictions on the load versus end shortening behaviour of the sections. The effects of the input parameters (prescribed initial imperfections, sign of the element mesh, on the convergence of the solution) were studied. The exact linear buckling wave form was used to confirm the post buckling analysis of the initial imperfection. The specifications were developed by using FEM for post buckling analysis of thin-walled stub columns. The results derived from the design procedure correlated well with the BS: 5950 (1998) predictions and experimental results.

The performance of stiffened and unstiffened Cold-formed channel members in axial compression was studied by E1-Sheikh et al. [30]. A parametric study was done on channel sections with diverse aspect ratios, sizes and slenderness ratios. It was observed that increasing the web stiffener beyond certain limits was counterproductive. Load eccentricity in the weak direction led to significant loss in the buckling strength. The effect of web stiffeners in reducing the torsional buckling strength, lower than the flexural buckling strength gave overall strength losses. On the other hand, it was found that providing flange stiffeners in the channel section resulted in a consistent increment in the member's buckling strength.

The local, distortional and Euler buckling behaviour of thin-walled columns was examined by Schafer et al. [31]. A closed form prediction of the buckling stress in the local mode and distortional mode agreed well with numerical results. The formulation included the interaction of the connected elements and the elastic and geometric stiffness on the web and flange juncture. Experiments and Numerical analysis demonstrated that post buckling capacity in the distortional mode was lower as compared to the local buckling. A new design method was offered that expressed incorporate local, distortional and Euler buckling. A sensitivity analysis for the prediction of elastic buckling stress was introduced for thin-walled columns.

Kad Vasanti Badasaheb et al. [32] compared the strengths of channel, hat, box and I sections subjected to compression, using LRFD by the AISI and BS by varying the yield strength and slenderness ratio was compared. In each shape one section was considered as having an average area of the sections listed in Indian Standard: 811-1987. The slenderness ratios were chosen from 25 to 175. BS and AISI codal previsions were reviewed with reference to the compression load capacity of the sections. For most of the shapes irrespective of the yield strength and slenderness ratio, prediction by BS gave higher compressive strength as compared to AISI provisions. It was observed that there was a marginal difference between the predictions by BS and AISI for smaller slenderness ratios of I-section where lipped were not included.

The basic principles of the finite element method for thin-walled members were presented by Bakker et al. [33]. The study focused on the possible sources of error in linear and nonlinear finite element solutions and offers were given to control and prevent these errors. Errors in idealization, discretisation, geometry, element formulation, solution and convergence were examined.

The problem related to sensitivity analysis of thin-walled members with open mono-symmetric cross-section warping which was taken into account was reviewed by Szymczak [34]. The first variants of state variable due to a modification in the design variable were examined. Arbitrary displacement, internal force or reaction of the member subjected to static load, critical buckling load, frequency and mode of torsional vibration were assumed to be the condition variables. Dimensions of cross-section, material constants, restraint stiffness, and their locations positions of the member ends were selected as the design variables. Accuracy of the approximate changes of the test variables achieved by sensitivity analysis was also examined.

The Lau et al. [35] which were introduced for beam columns to axially load columns exposed to biaxial and axial compression was extended by Teng, [36]. While combining the axial compression with uniaxial bending in the plate of symmetry a detailed observer was made. A comparison of numerical results from the current closed-form solution and those from the FSM was made, for which the two shows a close agreement between them. It was stated that, by including the effects of shear and flange distortion through a simple modification of the expression for the rotational stiffness offered by the web, the accuracy of the closed-form solution can be enhanced effected. It was observed that, the mutual relationship between the bending moment and the axial load was likely to being linear.

However, the load eccentricity had little effect on the buckling half-wavelength, as this was proved.

The study by Camotim [37] on lipped channels undergoing global buckling-local buckling modes hence showed significant findings regarding the behaviour of pin-ended columns. They introduced a collection of 12 column tests with different cross section. Their investigation shows that intermediate length columns are sensitive to buckling mode interaction. They concluded that local buckling deformations have no effect on the post-buckling strength of lipped channels. Therefore the failure mode of their columns involves a composition of symmetric distortional and global deformations where the global deformation seems well known.

The establishment of a common formula for calculating the unstiffened element with stress gradient with the moment capacity of a section was presented by Bambach et al. [38]. The method used the formulae for effective widths of unstiffened element with stress gradient derived from the plate test results. The investigation presented formulae for calculating the capacity, where inelastic reserve capacity can be considered for sections having a fully effective unstiffened element with stress gradient. The formulae were also shown as a good agreement with the experimental results of plain channels and I-sections in minor axis bending. Special attention was given to the effect of the two elastics considerations on the bending capacity .Specific designs were also presented in the form of alteration of the existing Australian and American standard for CFS structures for sections in bending and elements.

Recent developments and usages of an ECBL Method for the interactive buckling were presented by Ungurean et al. [39]. Two different types of problems were analysed. The occurrence of local plastic mechanisms in CFS sections in compression and implementation in the ultimate limit state analysis of the members was analysed. The study suggested the use of interactive local overall buckling analysis instead of traditional effective sections. The ECBL method was used to implement the proposed interactive buckling model. Results were compared with those of DSM and Plastic EWM. Comparison with European and American design codes were also presented. It was concluded that the ECBL elastic-plastic interactive approaches, based on the erosion theory of coupled bifurcation was much more rigorous and understandable than the semi-empirical methods used in existing design codes and for the buckling curves.

The implementation of GBT was built on examining the buckling properties of thin-walled members with irregularly branched open cross-section was validated and illustrated by Dinis et al. [40]. The derived formula was used to examine the global, distortional, and the local-plate buckling property of fixed and asymmetric E-section columns. They dealt with the matters related to the choice of suitable basic warping functions to determine the initial flexural shape functions. The derived formula was then occupied to study the various buckling behaviour of simply supported, fixed asymmetric and beams with unequal stiffened flanges. For confirmation targets, a series of GBT-based results were compared with precise values acquired from Finite Strip Analysis and a perfect coincidence was found in all cases.

The three different lengths of 30 pin-ended Cold-formed channel columns were studied by Wang et al. [41]. The three sections had defined post buckling strength, and the failure mode of most of the specimens comprised distortional buckling. It was seen that, the capacity of specimens with 90 or 135 inclinations at the same negative eccentricity is lower than the type of specimens with 45 inclined angles of bearing compression, but obviously the other two of the same positive eccentricity. In addition, tests were simulated by FEA. Results from the tests were in good assent. All the positively loaded the columns failed in combining with flexural and distortional buckling mode while the concentrically loaded columns with 45 sloping lipped stiffeners also failed in combining distortional and flexural buckling modes and the negatively concentrically loaded columns failed with flexural buckling modes combined with the local.

The recent improvement in the DSM which formerly served as an optimal design procedure in the NAS for design of CFS structural members, also in AS/NZS for CFS design was reviewed by Schafer [42]. The advantage of the methods which integrated calculative stability analysis into the design process, such as DSM was emphasized. The DSM employed gross sectional characters, which required a precise examination of elastic buckling property members. FSM of GBT, where the appropriate requirement for the stability estimations. It was calculated that, the efficiency of the DSM equalled or better than the traditional EWM for a long database of test of beams and columns. There were provisions of the development of the DSM for columns and beams, including the reliability of the method.

Jian-Kang Chen et al. [43] studied in lipped channel elements under compression and bending perpendicular to the web and an analytical method. It was used in calculating the critical stress of distortional buckling of compression flange. Finite element method was used to confirm the closed formed expressions. Several methods had been carrying out extensively to demonstrate the current model for which the prediction of the distortional buckling stresses of channel section and for specifying the minimum elastic distortional buckling stress have been good acceptable.

The capability of DSM for estimating the design strength of C-channel, I and rectangular sections were studied by Anil Kumar et al. [44]. Comprehensive experimental data produced using currently accepted EWM based on better calculation of local buckling stress calculated expose to discussion element Interaction was utilized therein work. The applicability of this method for the stability of C-channel which was investigated. It was seen the DSM could use to estimate the strength of compression members not including either lipped or intermediate. The comparative results test and EWM effects had proved that DSM which is estimated the design strength of compression members were acceptable accuracy, for feasible aims.

2.5 Literature Review Conclusions

Extensive literature reviews as described in the earlier sections have enabled the accumulations of the require knowledge in the following.

Several researchers carried out several researches by using analytical and numerical, theoretical and numerical, experimental work and comparison of codes on CFS such as un-lipped, lipped and I sections…etc.

They predicted the load capacity of various cross sections by using AISI, EC3, and AS/NZS, experimental works and FEA.

Generally, DSM and EWM are used to calculate the load capacity of CFS columns.

The use of FEA is very effective to investigate the structural behaviour of CFS members. However, the effects of some factors such as true material behaviour, residual stress and the strain-hardening which may influence the column behaviour are unknown and those were simply ignored or too simplified in the past research.

CHAPTER 3

CONCEPT OF STEEL COLUMN

3.1 Introduction

Several kinds of compression members, for which the column is the well-known. Bracing members, top of the trusses, rolled beams and compression flanges of built up beams are examples of compression factors. Columns are generally assumed as straight vertical members whose lengths are greatly greater than cross-sectional dimensions. To start with straight column or strut, compressed by gradually struts is termed short and long belonging to their proneness to buckling. CFS columns are also columns that are becoming well-known within the construction industry because of their higher strength to weight ratio. They are usually subjected to axial compression loads in a series of applications. The design of Cold-formed members in compression is mostly complicated than hot-rolled steel members.

One of the biggest difficulties with CFS design is the prevention of member buckling. Because of the low thickness to weight ratio, it is likely that the numbers will buckle at stresses that are lower than the yield stress when compressive, bearing and shear bending forces are applied.

Commonly CFS columns open sections fail in three separate modes of buckling such as, local, distortional and flexural or flexural-torsional buckling.

However, Current North American design specifications for CFS columns ignore local buckling interactions and do not provide an explicit check for distortional buckling.

3.2 Buckling Modes

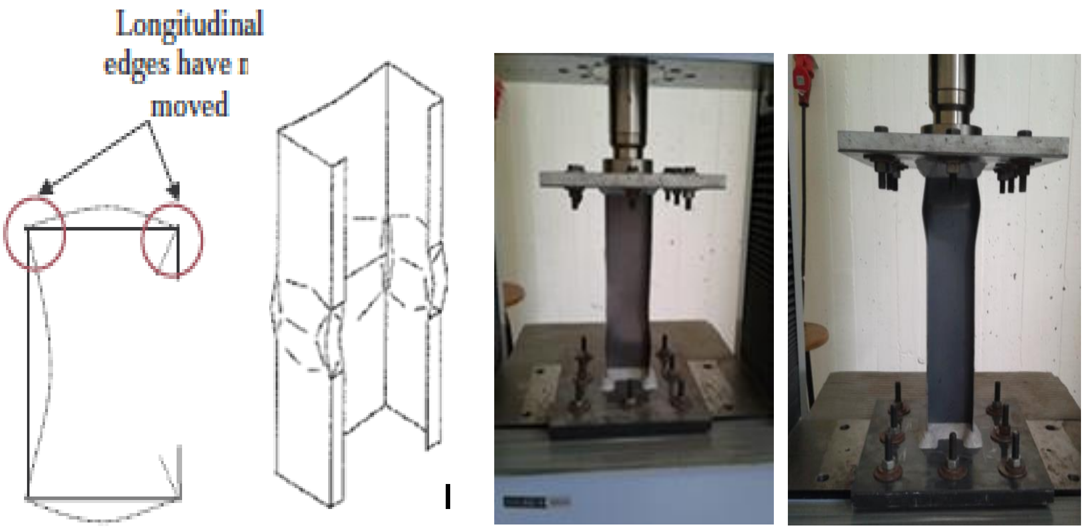
Mostly, the CFS compression members can be categorized in three forms. These are:

- Local buckling
- Distortional buckling
- Flexural or flexural torsional buckling

Furthermore, relatively these three forms can also interact with each other, such as local and distortional, local and flexural, distortional and flexural and local distortional and flexural.

3.2.1 Local Buckling

Local buckling occurs in the slender plate elements without changing the position of longitudinal edges of compression members. On the other hand, it occurs due to the buckling of individual plate elements. Local buckling is a common buckling failure in compression members, which is made of slender plate elements. The cross section and the typical failure mode are shown in Figure 3.1.



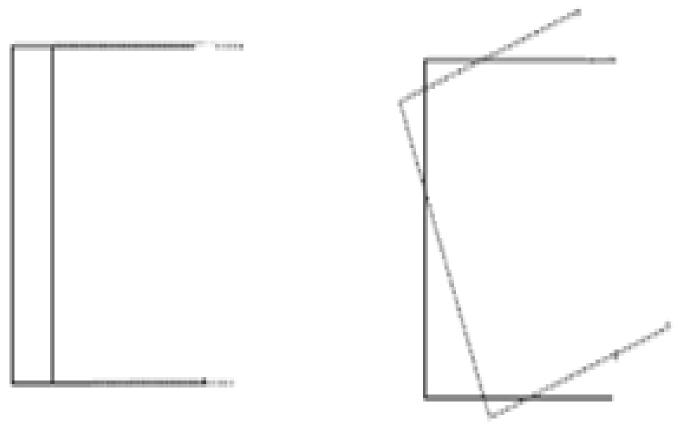
a) Analytical result b) Experimental results

Figure 3.1 Local buckling modes of compression members

The half-wave-length of the local buckling mode is the shortest one among the other failure modes. Since local buckling has a higher post buckling range, it is not considered as failure of the whole column when columns buckle locally. However, if the column is pin-ended it might have an additional moment after local buckling due to the shift in the line of axial force [45].

3.2.2 Flexural Buckling

Generally, this mode is known as global buckling. Since the behaviour is similar to the behaviour of beams, this is known as flexural torsional buckling. Since the behaviour is similar to the behaviour of beams this is known as flexural or flexural torsional buckling. In flexural buckling, the cross sectional shape remains unchanged and it has only lateral or lateral torsional movements. This buckling mode is sometimes called rigid-body buckling since the cross section remains the same at any given section after global buckling occurs. The lateral deflection of the flexural mode is larger than the local and distortional mode. The half-wave-length of the flexural mode is the largest among the buckling modes. The half-wave-length is equal to full column length if it is pin-ended and half of the full length if it is fixed ended [46].



a) Flexure buckling　　b) Flexural torsional buckling　　c) Flexural of failure mode of column

Figure 3.2 Flexural Buckling [Ranawake (2006)].

3.2.3 Distortional Buckling

Distortional buckling is a characterized by the rotation of flange at the flange web junction for the members with edge stiffeners figure and displacement of the intermediate stiffeners normal to the plane of the element for the intermediately stiffened members. However, the buckling of lipped channels with narrow flanges show lateral bending of the whole cross-section while the flange web junction of the buckling of lipped channels with wide flanges remain nearly straight. Distortional buckling exists at an intermediate half-wave-length between local buckling and flexural or flexural torsional buckling. However, the distortional buckling can be safely ignored if members are designed to achieve significantly lower local buckling stress than the distortional buckling stress. Most probably, members with narrow flanges fail by local buckling mode since, the web is much slender and buckles locally first, while members with wide flanges buckle distortionally. However, by introducing the stiffeners to the web, narrow flange members can buckle distortionally [46].

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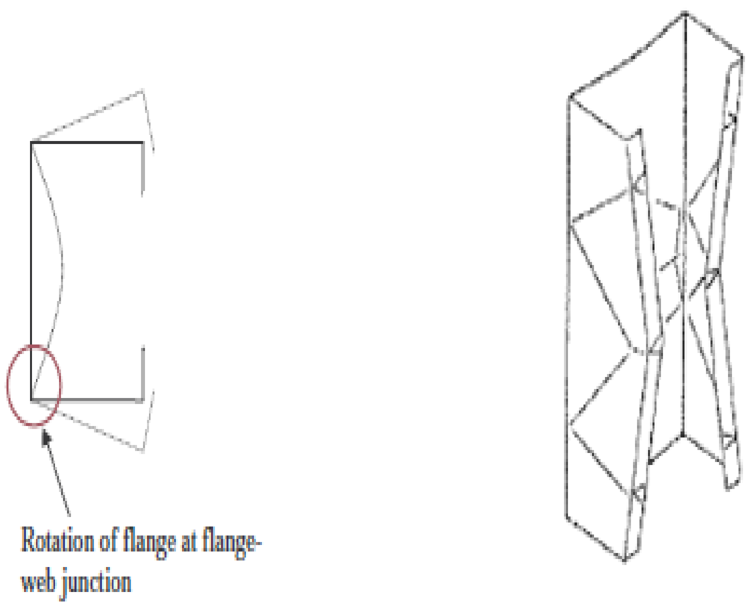


Figure 3.3 Distortional buckling [Ranawake (2006)].

3.3 Concept of Effective Width and Effective Width Equation

The concept of effective width and effective width equation accounts for the common design procedure to specify the sectional strength of slender section. It was first nominated by Von Karman (1932) et al. [47].

The introduction of this concept was due to the fact that local buckling of plate elements causes a concentration of the longitudinal stress near the supporting edges of a plate element.

However, the EWM does not have adequate methods for predicting the distortional buckling failure. When sections more complex and optimized, with intermediate stiffeners and /or extra edges, the computation of the effective widths become extremely complicated and time consuming. In addition, the effective area of the section is composed of the effected areas of the flange, web and stiffener calculated separately.

The following formula gives the effective width of a plate element associated to the AISI. This formula is called Winter formula [48].

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{b}{t}\right)\sqrt{\frac{f}{E}} \tag{3.1}$$

3.4 Column End Fixity and Effective Length

The effective, *KL,* is calculated from the actual length *(L)* of the member, investigating the translational boundary conditions at the final stages and relative rotational.

The actual length should be read at length from center to center of its interactions with the sustaining members in the plane of the buckling deformation, or in the instance of a member with a free end, the free standing length from the center of interacting member at the end support end.

However, its restraint at the ends of a column has an important influence on the buckling strength and the effective length. Due to the importance of the effective length, the behaviour of how a column buckles under load will depend on how the column is built. If one end a slender column totally free of any restraint and the other end fixed at its base, the column will be excessively prone to lateral buckling under load. In constant, a column which is fixed at both ends will be much more stable.

EC3 gives no direct guidance on calculating the buckling length; therefore it is acceptable to use those given in BS 5950 Table (13). Some typical effective lengths are given below.

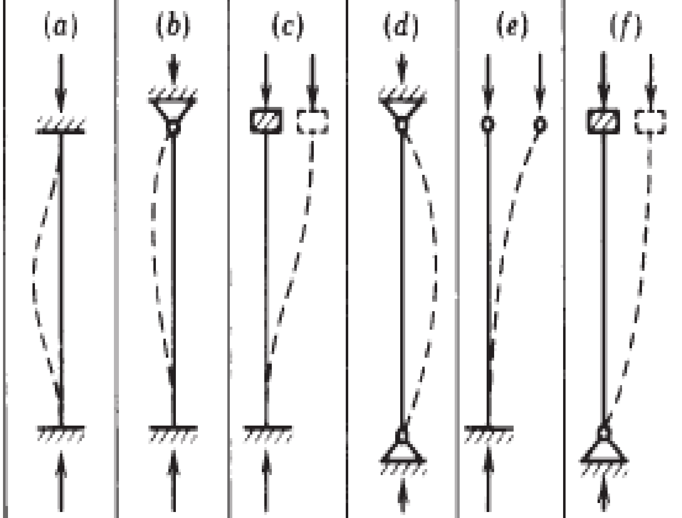
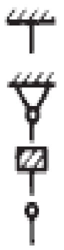
	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended K value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed, Translation fixed Rotation free, Translation fixed Rotation fixed, Translation free Rotation free, Translation free				

Figure 3.4 Theoretical and recommended *K*-factors for isolated columns with idealized end conditions.

3.5 Buckling Stress Coefficients

The buckling coefficient *K* for uniformly compressed stiffened element which has an adequate lipped is taken as 4.0 in all standards and specifications. For the uniformly compressed unstiffened elements, the *K* value is taken as 0.5 in AS1538 and 0.43 in ECCS, and EC3 and the AISI specifications. AISI and ECCS have provisions for determining the *K* value for elements which are partially stiffened. Since AS1538 does not have explicit provisions, a rational elastic buckling analysis, such as a semi analytical finite strip analyses of the type described in Lau et al [49]. Or formulae of the type described which is permitted to calculate *K* values for inadequately elements [50, 51].

3.6 Summary

This chapter deals with the introduction and concepts of steel columns with the effective width method. Generally, failures of axially loaded C-channel columns are governed by three buckling modes, the behaviour of CFS columns affected by these buckling modes under axially loading.

CHAPTER 4

DESIGN CODES FOR CALCULATION OF AXIALLY LOADED MEMBERS

4.1 Introduction

Many of the CFS structure design codes currently in use are based on the effective width concept. The EWM approximates the process and considers the effects of local buckling of the member's wall in the resistance of the member. This method is composed of reducing the area of each of the member wall, taking into account boundary conditions. The reduced cross-section calculates the member's resistance. In 1932 Von Karman [47]. Originally proposed the indirect way of getting the post buckling resistance of a member and in 1968 G. Winter [52]. Subsequently adjusted this experimentally.

4.2 AISI Specification (2007)

AISI specification deals with uses the effective width concept of the stiffened and unstiffened elements of a section to find its resistance to local and global buckling due to uniform and non-uniform stress gradient. The effective area method considers the elements forming a cross-section in isolation, thus minimizing the interaction between the elements.

Calculating the capacity of a Cold-formed member subjected to an axial load is described in Chapter 4 of the AISI specification (2007). Individual member capacities are determined as a function of the member's resistance to local buckling, global buckling and yielding. The capacity of a concentrically loaded compression member is given by:

$$P_n = A_e F_n \qquad 4.1$$

A_e= effective area calculated at stress Fn.

F_n = combined resistance to buckling yielding calculated as follows:

For $\lambda_c \leq 1.5$: $F_n = (0.658^{\lambda_c{}^2})F_y$ (Inelastic buckling) 4.2

For $\lambda_c \geq 1.5$: $F_n = \left[\frac{0.877}{\lambda_c{}^2}\right] F_y$ (Elastic buckling) 4.3

Where $\lambda_c = \sqrt{\frac{F_y}{F_e}}$ (Slenderness factor) 4.4

F_e= the least of the elastic flexural, torsional and torsional-flexural buckling stress defined conforming to sections C4.1.1 through C4.1.5 of the above expressions.

F_y= the specified minimum yield point of the type of steel used to determine according to section *F3, A7.2* and *A2.3.2*.

The effective width for a member without intermediate stiffener is calculated as:

$$\rho = \left(\frac{1-0.22}{\lambda}\right)\lambda \quad 4.5$$

$$\lambda\text{=slenderness factor=}\sqrt{\frac{f}{F_{cr}}} \quad 4.6$$

$$F_{cr} = \frac{K\pi^2 E}{12(1-\nu^{2)}}(\frac{t}{w})^2 \quad 4.7$$

Section *F1* of the AISI specification (2007) gives the criteria for accepting experimental capacities of CFS members or systems. It also provides equations for determining an appropriate factor of safety (Ω) for or resistance factor (ϕ) for the LRFD. The factors Ω *and* ϕ are calculated using.

$$C_\phi = (M_m F_m P_m)e^{-\beta 0\sqrt{V_M{}^2+V_F{}^2+C_p C_p{}^2+V_Q{}^2}} \quad 4.8$$

4.2.1 Design Procedure

EWM:

Figure shows the summary of the design processes in the determination of the design strength of a C-channel column according to the EWM.

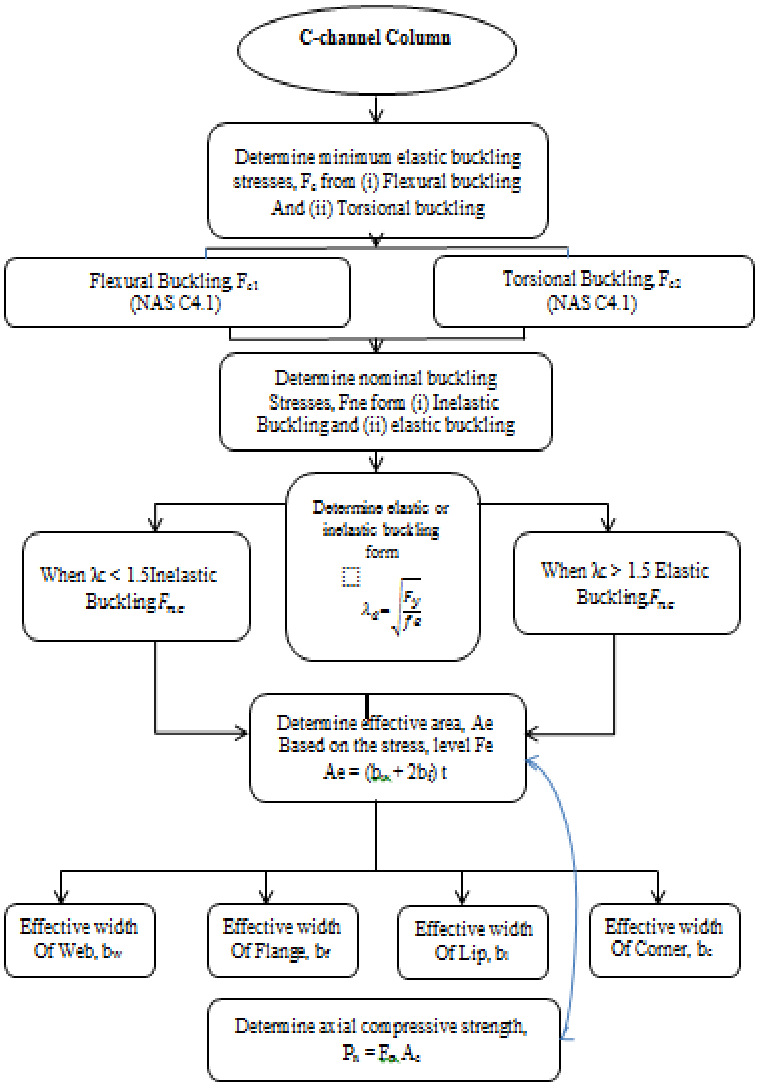


Figure 4.1 Design procedure of the C-channel column using EWM.

4.3 EC3 Design Methods

EC3 shows the extra rules for Cold-formed members and also emphasizes on the two ways to directly specify the design resistance of members, these are the application rules and test-based design method.

The supplementary rules include: Effect of the cold-forming process in yield stress of the steel, Influence of rounded corners and determination of the section's properties considering the rounded corners, a series of limitation to the cross-section's geometry and Methods to determine distortional buckling. However, the following passage defines and gives an account on the application rules and test-based design methods.

Applications rules are design methods supplied by the EC3 to calculate the design resistance for different failure modes by using closed formulae enhancing fast and gradually easy calculation. Application rules regulate every component of the calculation, and describe when alternative advanced methods may be used to update the accuracy of the effect. Although, they may be used directly only to cases that are principally the same as one handled by a given regulation, in any other case approximations from the safe side are to be used.

Test based design allows deriving design resistances of methods from load bearing capacities measured in laboratory tests. The extended design method may consist of new formulae, but as possible failure modes are covered by EC3 on application rules for the related failure modes may be measured load bearing capacities but still provide the safety of the design method.

In the case of failure modes involving stability the application rules provide formula to calculate the relative slenderness and derive reduction factors for various forms of global buckling. The basic formula for checking is the in all cases, the χ reduction factors are to be calculated based on a relative slenderness calculated according to failure mode [51].

According to the application rules of EC 3-3-1:1996.

For classes 1, 2, 3

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \qquad 4.9$$

For class 4

$$N_{b,Rd} = \frac{\chi A_{eff}\, fy}{\gamma_{M1}} \qquad 4.10$$

$\gamma_{M1} = 1.0$

4.3.1 Buckling Curves

The European buckling curve which is introduces the behaviour of columns with initial imperfections. The reduction factor χ is given as a function of column non-dimensional slenderness λ for different types of cross-sections with different values of imperfection factors.

Reduction Factor, $\chi \le 1$

$$x = \frac{1}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}} \qquad 4.11$$

Where

$$\phi = 0.5(1+\alpha\ (\lambda-0.2) + \lambda^2 \qquad 4.12$$

For class 1, 2 and 3

$$\lambda = \sqrt{\frac{A f_y}{N_{cr}}} \qquad 4.13$$

For class 4 $$\lambda = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \qquad 4.14$$

$$N_{cr} = \frac{\pi^2 EI}{L^2} \qquad 4.15$$

N_{cr} is the elastic critical buckling load for the related buckling mode based on the gross properties of the cross-section.

4.3.2 Imperfection Factor (α):

EC3 uses an imperfection coefficient *(α)* to differentiate between various column strength curves. The geometry and material properties of the cross-section and the axis of buckling decides the choice of which buckling curve could adopt. For flexural buckling, five cases are termed in EC3 (Table 6.1). EC3 tabulates the rules for selecting the proper column strength curve.

4.3.3 Reduction Factor

Where χ the reduction factor, alternatively, may be read from (Figure 6.4) in the EC3 by using λ and the required buckling curve.

4.3.4 Slenderness for Flexural Buckling

The non-dimensional slenderness λ is given by:

For class 4 sections: $$\lambda = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \qquad 4.16$$

Or

$$\lambda = \frac{L_{cr}}{i}\frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda 1} \qquad 4.17$$

Where

L_{cr} =buckling length about the relevant axis,

i=radius of gyration about relevant axis, calculated from the properties of cross-section.

Where

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon \qquad 4.18$$

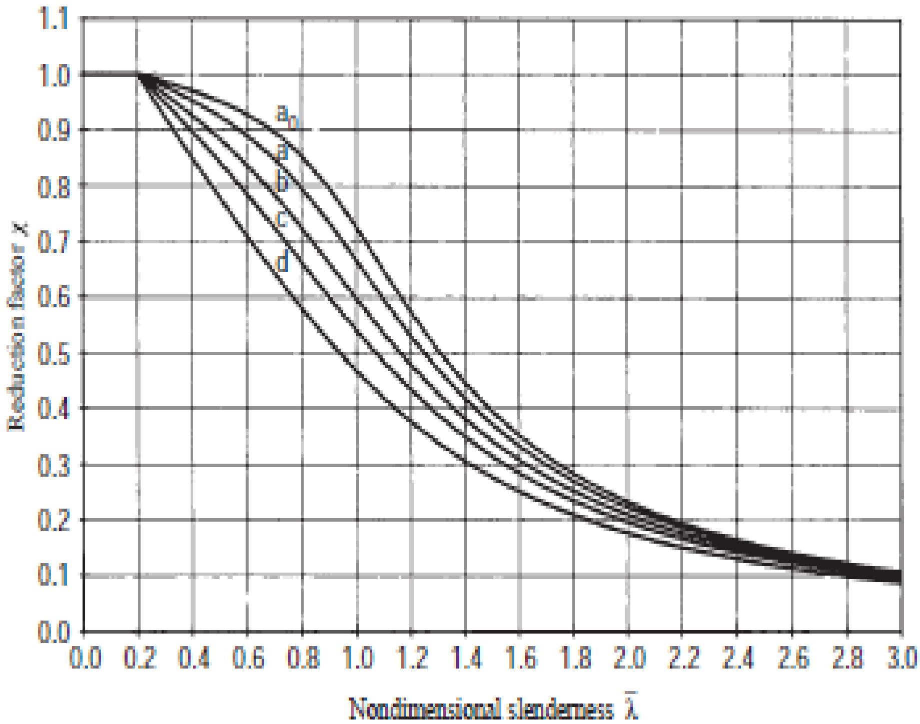


Figure 4.2 Buckling curves (EC)

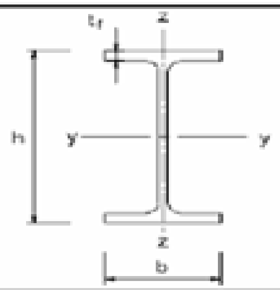
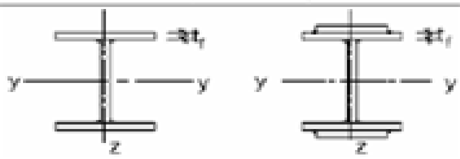
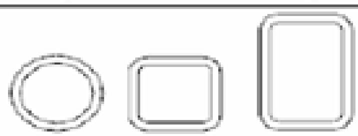
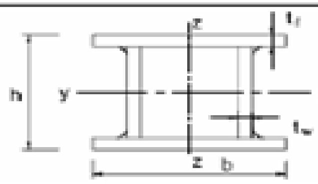
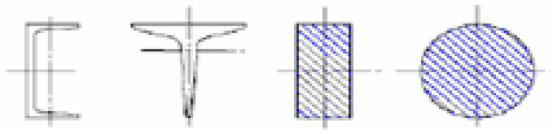
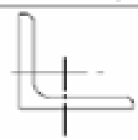
Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		h/b > 1,2	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \le 100$	y – y z – z	b c	a a
		h/b ≤ 1,2	$t_f \le 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \le 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0,5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

Table 4.1 Selection of buckling curve for a cross-section (EC3)

4.3.5 Design Procedure

Figure shows the summary of the design processes in the determination of the design strength of a C-channel column according to the EC3.

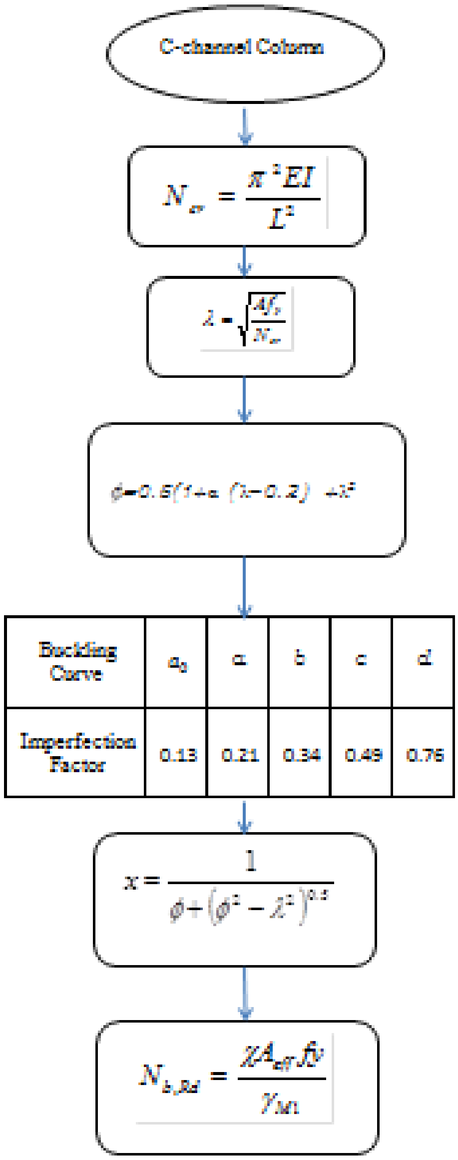


Figure 4.3 Design procedure of the C-channel column using EWM.

4.4 Summary

This chapter deals with the formula of calculating the load capacity of CFS column. It has different procedures for calculation of load capacity, more specifically, it addresses the applicability of the provisions of EC3 and AISI specification, and both developed for CFS members, to estimate their load capacity. It is worth nothing that EC3 and AISI adopt different approaches to perform this task, while the former is based on the effective width concept.

Different methodology to extend the application of the EC3 and AISI design provisions of CFS columns. Hence, the process of getting the reduced (effective) area of the cross-section is sometimes difficult to understand and has little correspondence with the reality of the problem.

In order to account for the local buckling effects, the EWM requires the determination of an effective (reduced) area of the cross-section.

CHAPTER 5

EXPERIMENTAL INVESTIGATION

5.1 Introduction

This chapter presents the experimental investigation on Cold-formed C-channel columns. This experimental investigation covers testing on unlipped columns fabricated from CFS C-channels.

However, the codes of practice (AISI and EC3) of CFS sections have been existing for some time now; this chapter is given to examine this comparison. Only columns loaded axially are considered.

Selected experiments results (ultimate load capacity) on columns with several cross-sections were used to compare with calculations by the codes of practice. Results compared that the accuracy of all the codes of practice examined changes from section to section.

This chapter included, first part of this chapter comparison experimental and numerical comparison with AISI and EC3 the second part experimental investigation on unlipped C-channel columns and compared with AISI and EC3.

Each specimen was cut to specified length 305 mm, at the end of each column were milled flat by an electronic machine to an accuracy of 0.01 mm to ensure full contact between the specimen and end bearing. Five specimen unlipped C-channel columns with different flanges and webs were tested.

The test specimens were brake-pressed from structural steel sheets of 1.977mm thickness. The channel sections had a nominal depth of the webs ranging from 82.69

to 123.17 mm; nominal flange widths ranging from 31.74 to 71.59 mm. The test specimens were formed from structural steel sheet and had a nominal yield stress of 288.3MPa.

5.2 Verification Results from Literature Review

The C-channel columns are compared with design strengths predicted using EC3 and AISI specification.

The ultimate loads for the cases under consideration were compared to failure loads observed from test and calculated by the following codes and all the C-channel columns (lipped and Un-lipped) that can be assumed to be fully fixed at the ends.

Example 1:

The dimensions and properties presented in Table 5.1 and geometrical configuration are shown in Figure 5.1. The test results are originating from reference.

Table 5.1 Section dimensions

Specimen	b_f (mm)	b_w(mm)	t (mm)	L (mm)	R (mm)	F_y(MPa)
UC 80-30	33.07	77.95	0.9	600	3	235
UC 80-50	53.35	79.52	0.9	600	3	235

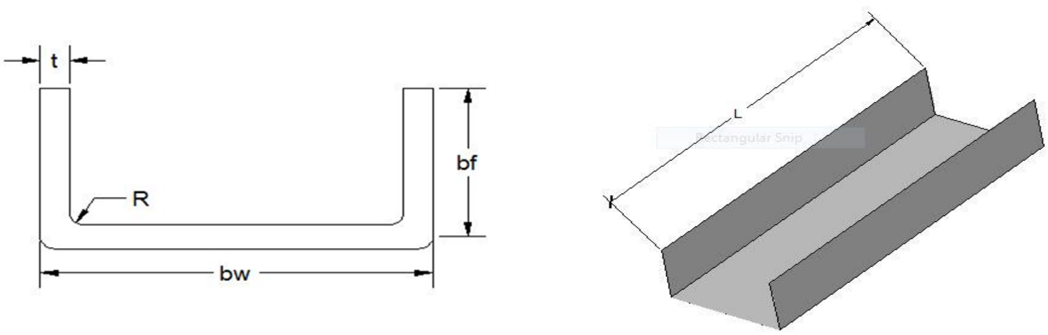


Figure 5.1 Geometrical configuration

Table 5.2 shows the comparison of test results and design calculated results for the un-lipped C-channel column test specimens of UC 80-30 and UC 80-50 test series.

The ratios of the test result of the design calculated results of the AISI and EC3 are also included in Table 5.2.

Discussion of results: The results obtained from the experiment compared with FEA by GÖĞÜŞ et al. [53] in Table 5.2. Close agreement between results can be observed.

It is shown that the EC3 and AISI conservatively predict the design strength of UC 80-30 and UC 80-50 unlipped C-channel column specimens.

Among the design methods, AISI is the most conservative design method for the unlipped C-channel column.

Table 5.2 Comparison of test & design results for C-channel columns (kN)

Specimen	GÖĞÜŞ et al [53]	Present Study		Test/ Design	
	Test	EC3	AISI	EC3	AISI
UC80-30	15.66	13.46	15.11	1.16	0.98
UC80-50	18.12	14.84	16.77	1.22	1.08

Example 2:

The details of geometrical configurations of test specimens are given in Table 5.3. Corresponding conventions are shown in Figure 5.2. Only single case is presented here.

Table 5.3 Section dimension

Specimen	b_f (mm)	b_w (mm)	d (mm)	t (mm)	L (mm)	F_y(Mpa)
LC 100-50	50	100	10	1	600	250

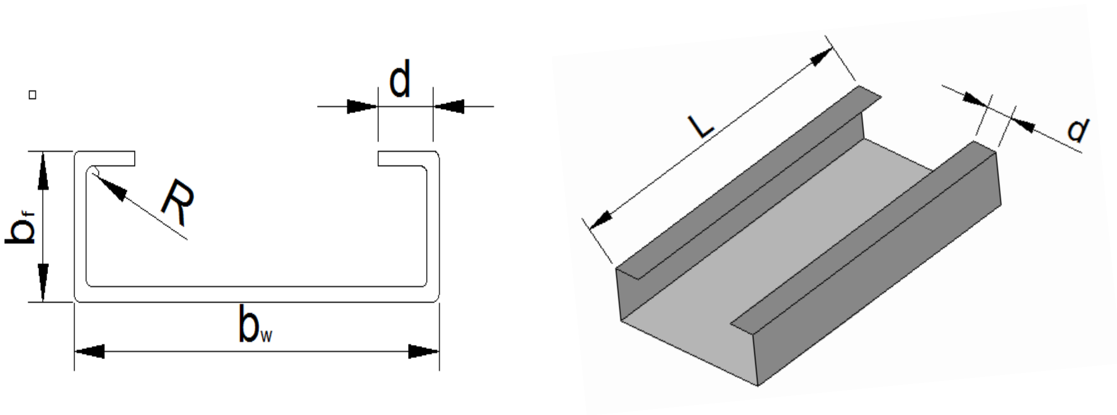


Figure 5.2 Geometrical configuration

Discussion of results: Table 5.4 shows a comparison of FEA results and design calculated results for the lipped C-channel column. The results obtained from the FEA and compared with direct strength method by Silvestre et al. [54] compared with standards for the lipped C-channel column, predictions by AISI and EC3 are almost similar and DSM result is slightly lower.

Table 5.4 Comparison of ultimate loads FEA with design standards (kN)

Specimen	Present Study		Silvestre et al (54)	
	EC3	AISI	DSM	FEM
LC 100-50	35.11	34.55	34.3	35.7

Example 3:

The dimensions presented in Table 5.5 are used. Just one case is presented here. More cases to all the examples can be found in the reference.

Table 5.5 Section dimension

Specimen	b_f(mm)	b_w(mm)	d (mm)	t (mm)	L (mm)	F_y(MPa)
LC60-30	30	60	9	0.95	190	550

The Table 5.6 shows the comparison of test results and design strength calculated results for the lipped C-channel column test specimens of the LC 60-30 test. The ratios of the test results to the design calculated result of AISI and EC3 are also included in the Table 5.6.

Discussion of results: The results obtained from the test and compared with FEA and DSM by Gunlan et al. [55] results show the prediction by the AISI is most conservative. That the EC3 relatively lower estimate for the most results.

Table 5.6 Comparison of ultimate loads from tests, FEA and design standards (kN)

Specimen	Present Study		Gunlan (55)				Test/Design	
	EC3	AISI	Test	DSM	FEM	BS5950	EC3	AISI
LC60-30	50.09	55.23	53.9	49.04	55.05	52.89	1.07	0.97

5.3 Tensile test for sheet metals (Coupon Test)

The material properties of the specimens were specified by tensile coupon tests. The coupons were wire cut from the centre of the web plate form specimens of the same batches as the column the column test specimens. This is to ensure the results illustrated the material properties of the column test specimens.

The mechanical testing has a very important role in estimating properties of engineering materials and controlling the quality of materials for use in design and construction.

A very common type of test used to measure the mechanical properties of the material is the tension test. This test it mostly used to produce primary design information on the strength of materials and is the confirmatory test for the standard of materials.

The important parameters that set up the stress-strain curve taken during the test are yield point, tensile strength elastic modulus, percent elongation and the reduction area. Poisson's ratio, toughness, resilience, can also be realized by using this test**.**

The purpose of this tension test to obtain the mechanical properties such as yield strength and ultimate stresses of used sheet metals. Unfortunately the extensometer there is not available for measuring the other parameters. Results from tensile test, yield stress 288.3 were used for design calculations. According to ASTM E8M standardization the specimen dimensions were shown in Figure 5.3.

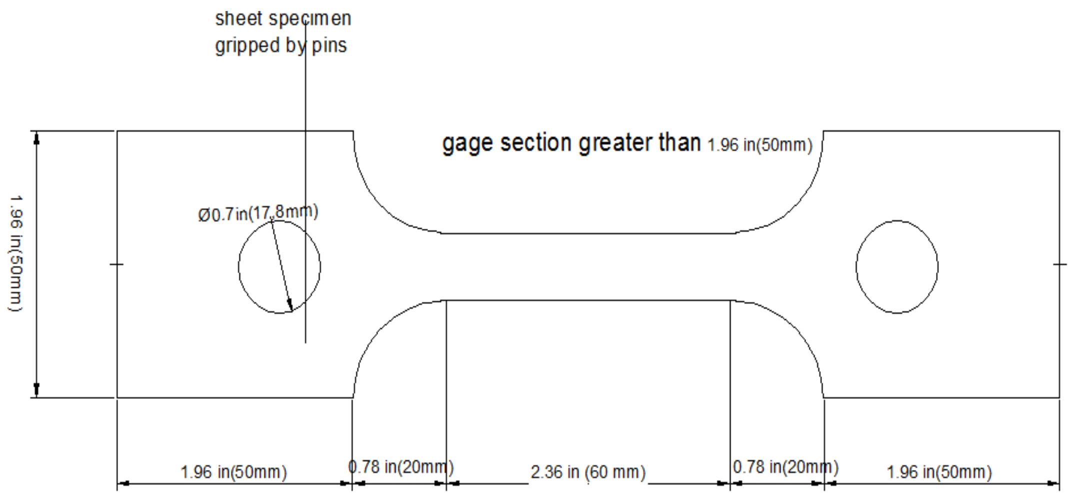


Figure 5.3 The geometric configuration according to ASTM E8M Standardization.

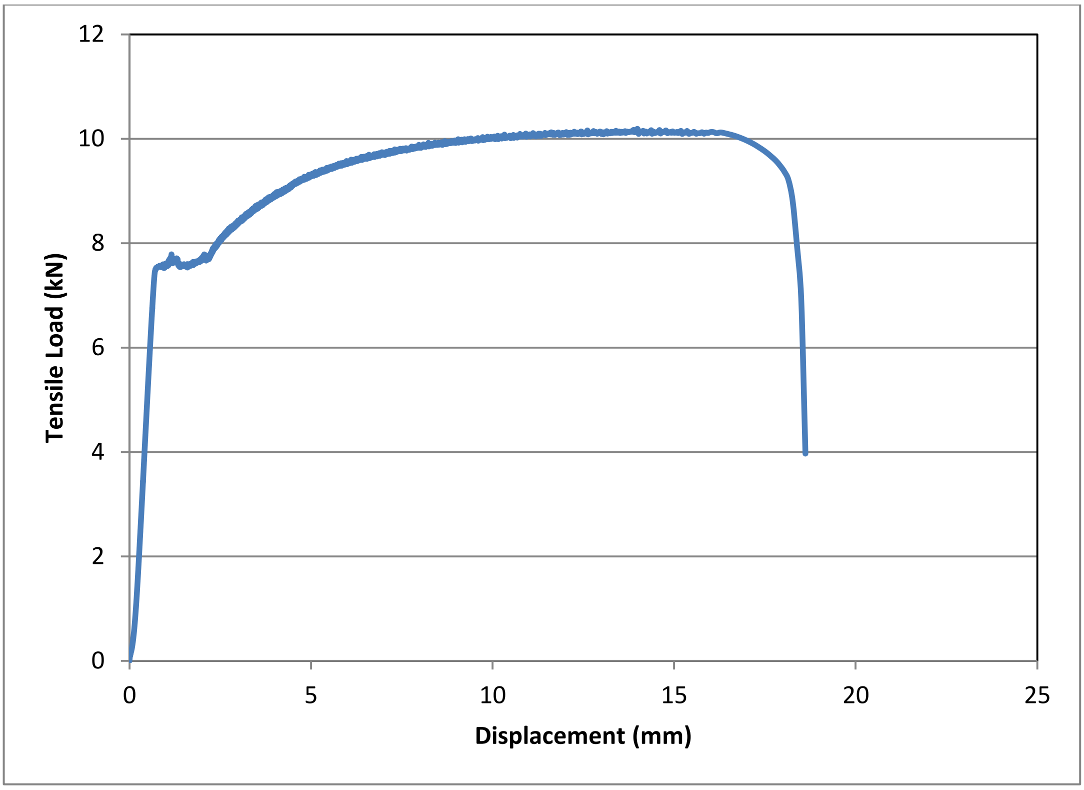


Figure 5.4 Load vs. displacement curve for N1

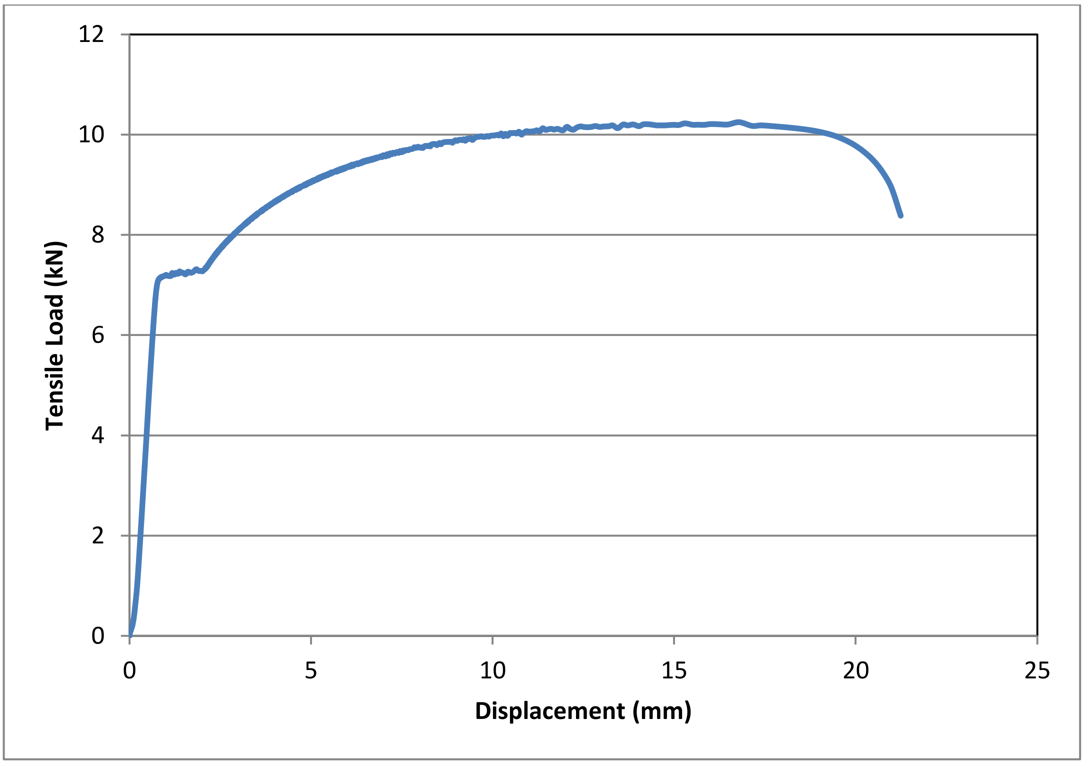


Figure 5.5 Load vs. displacement curve for N2

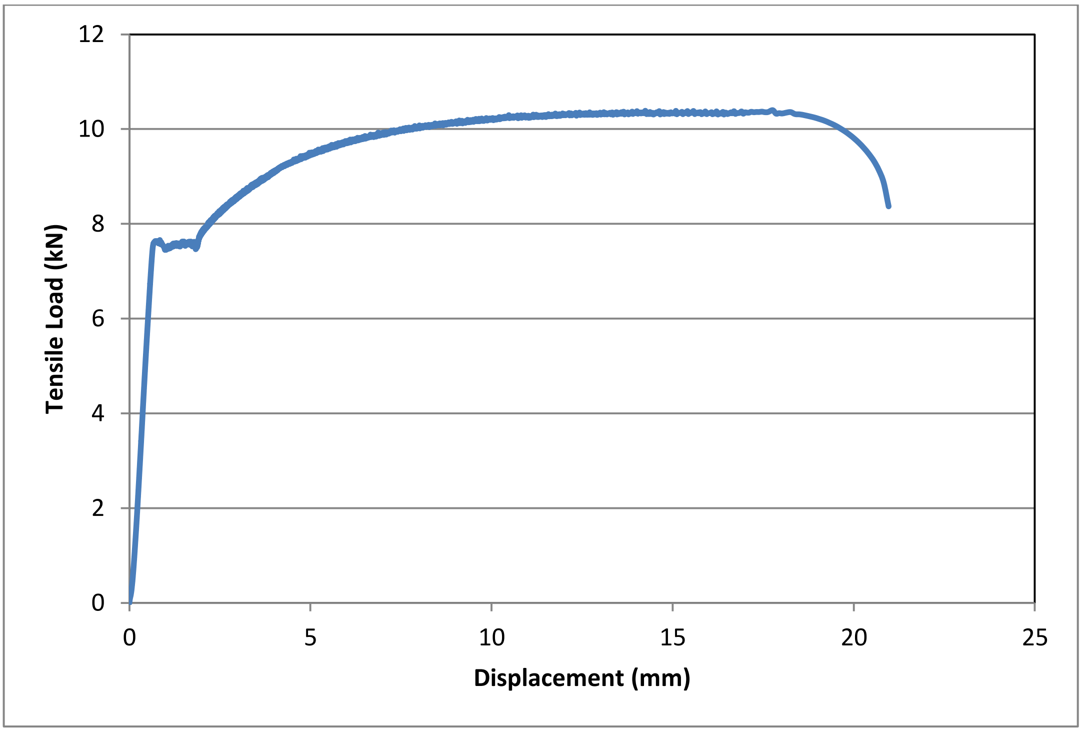


Figure 5.6 Load vs. displacement curve for N3

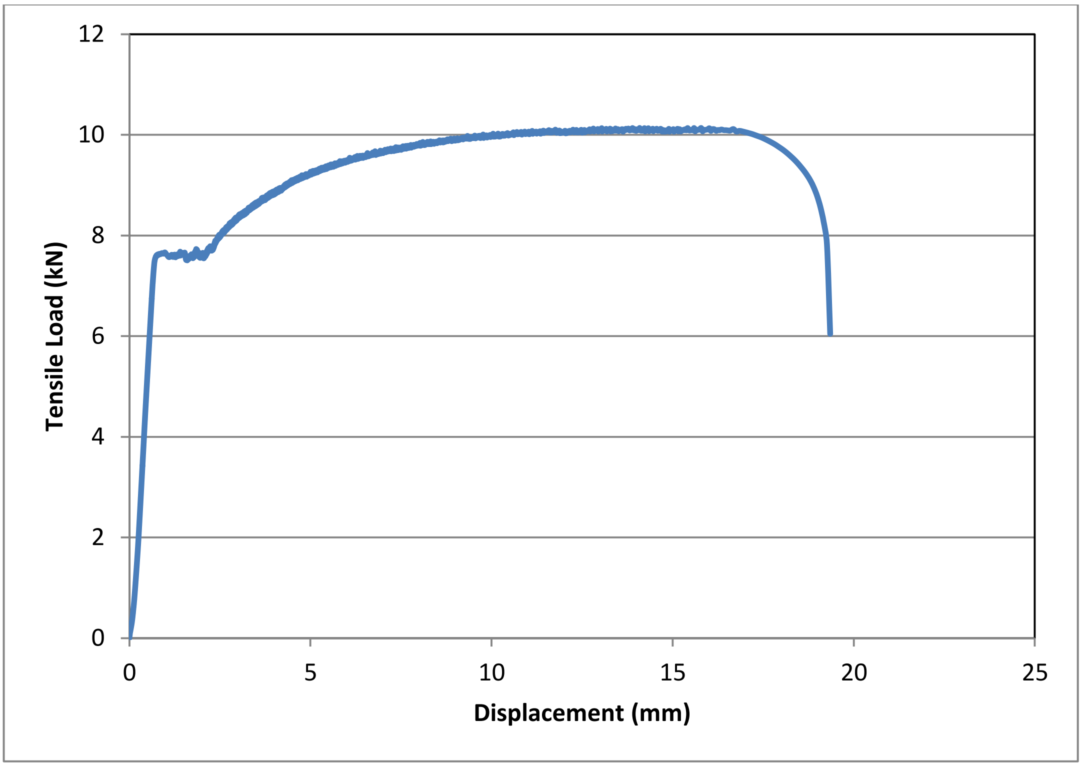


Figure 5.7 Load vs. displacement curve for N4

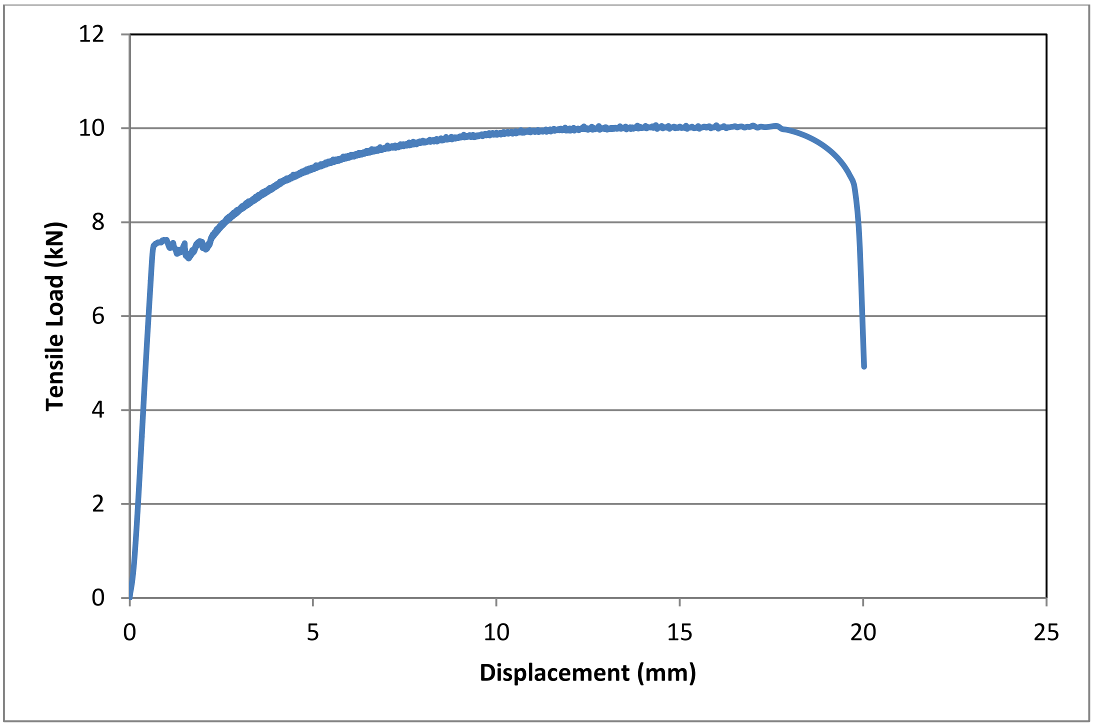


Figure 5.8 Load vs. displacement curve for N5

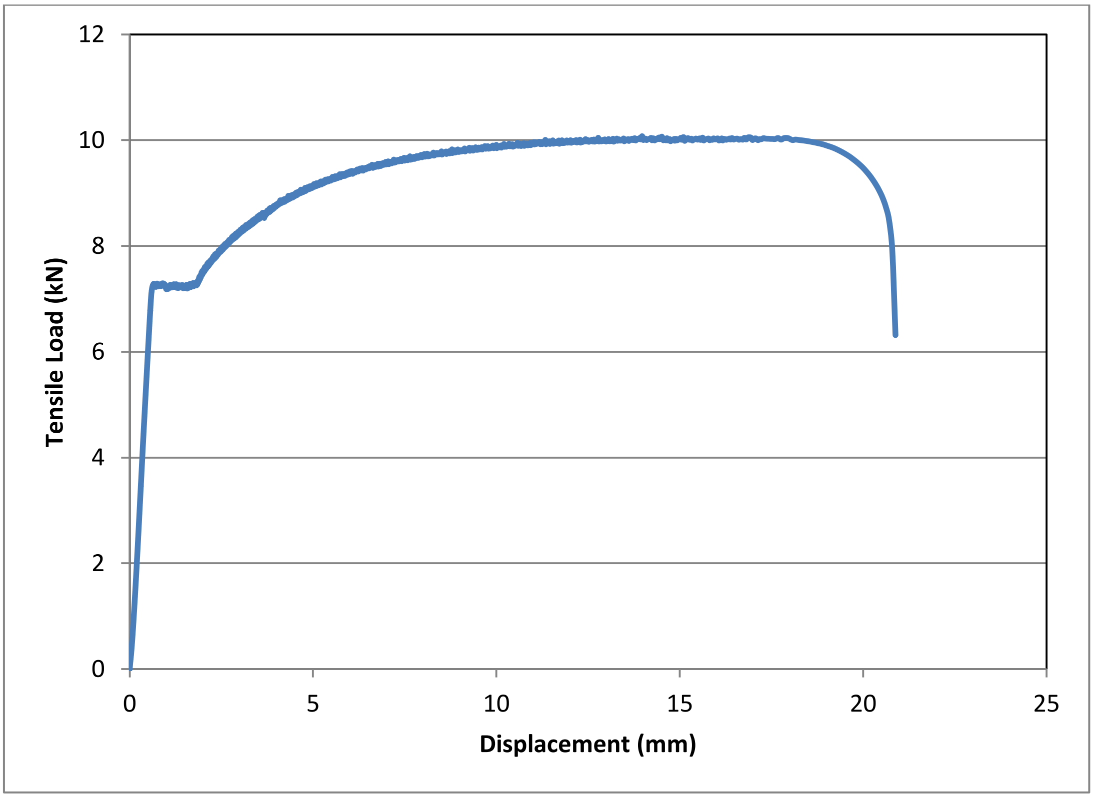


Figure 5.9 Load vs. displacement curve for N6

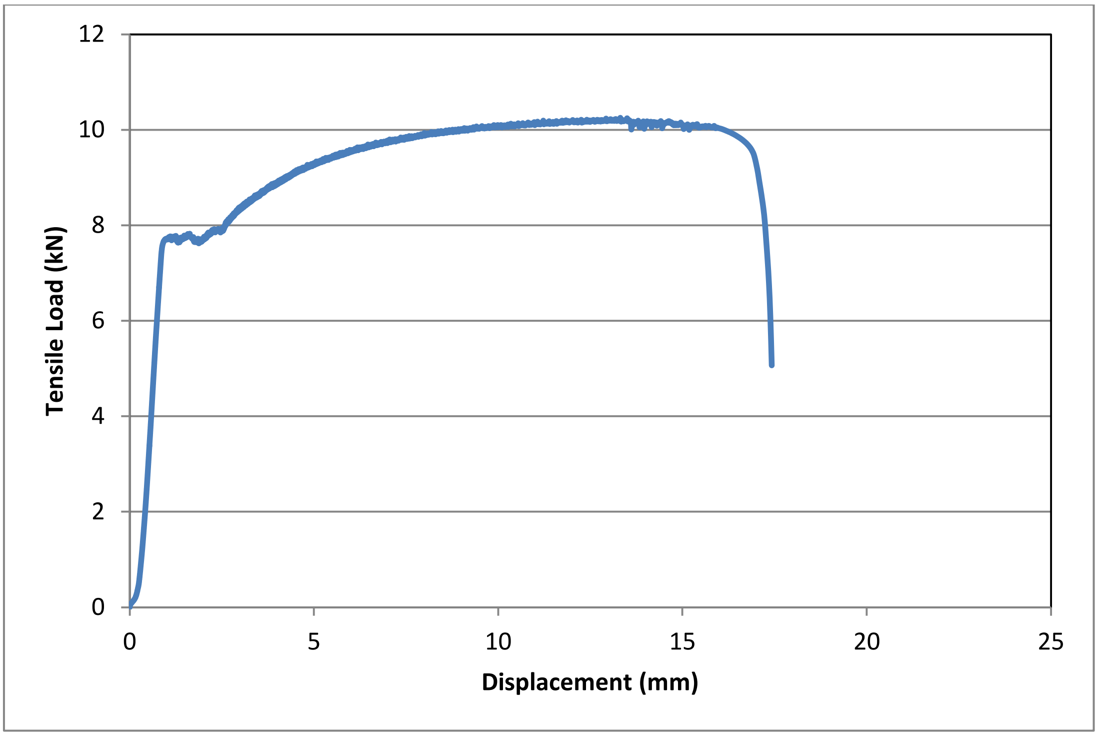


Figure 5.10 Load vs. displacement curve for N7

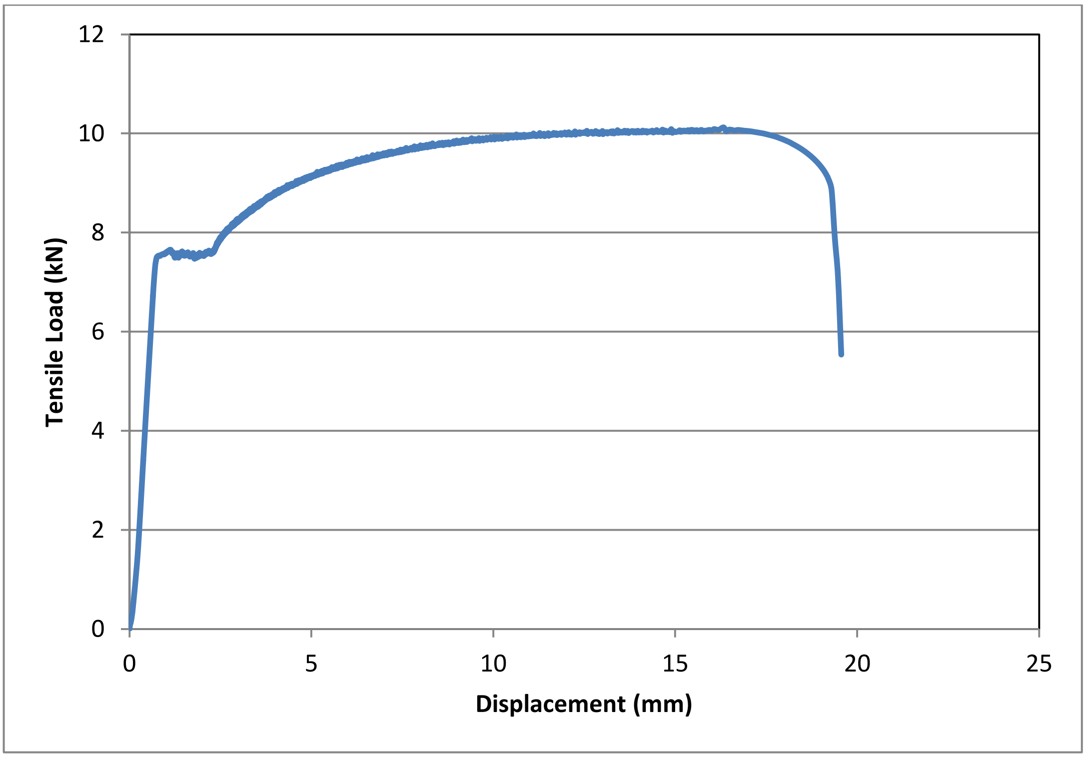


Figure 5.11 Load vs. displacement curve for N8

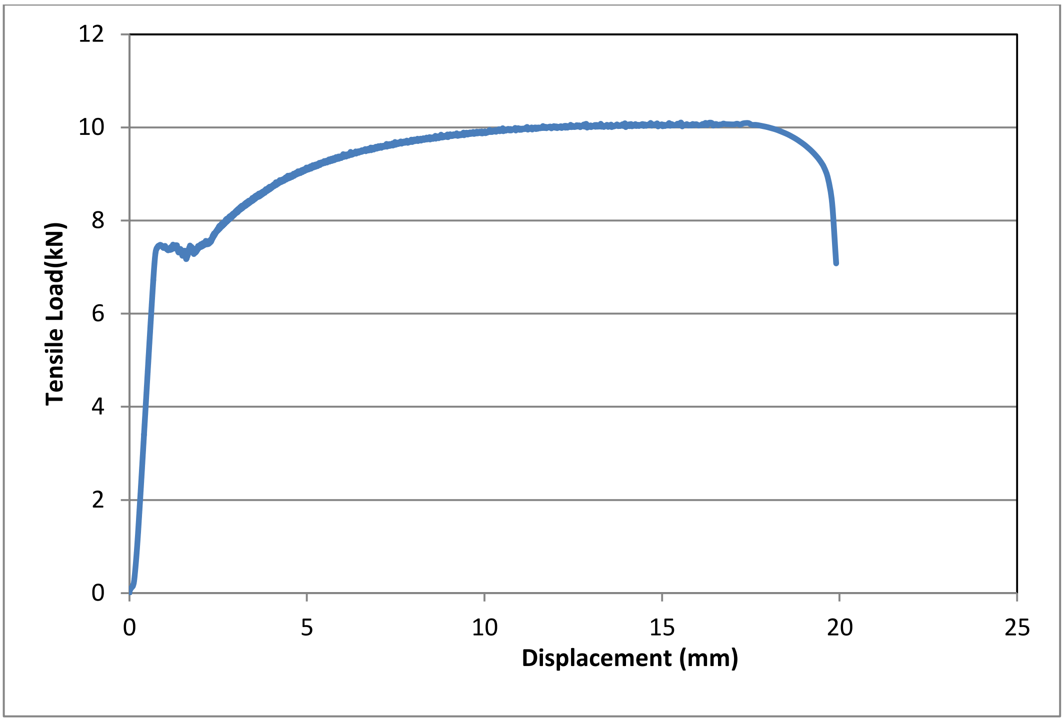


Figure 5.12 Load vs. displacement curve for N9

Table 5.7 Tensile test results

	Ultimate Stress (MPa)	Yield Stress (MPa)
N1	407.20	291.16
N2	409.60	274.24
N3	415.96	294.84
N4	405.40	293.84
N5	402.56	288.32
N6	402.96	278.64
N7	410.00	297.96
N8	404.88	288.40
N9	404.08	287.36
Average	406.96	288.30

5.4 Test Specimen

The nominal cross-sectional dimensions of the test specimens in this experimental investigation are presented in the Figure 5.13.

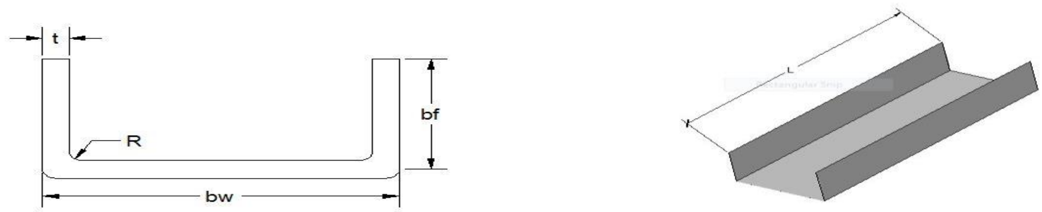


Figure 5.13 Geometrical configuration unlipped column

The test specimen dimensions, using Digimatic Caliper defined in Figure 5.13 where b_w is the overall depth of web, b_f is the overall width of flange, t is the thickness and R is the inside corner radius of the channel sections.

The characteristics of the test specimen configurations are introduced by the following figures, schematic drawings and short descriptions.

A total of five unlipped columns were tested. The fix ended unlipped columns were cast to end plates using high strength epoxy and are tested on fixed ends.
The Dimensions of unlipped column specimens are tabulated in Table 5.8.

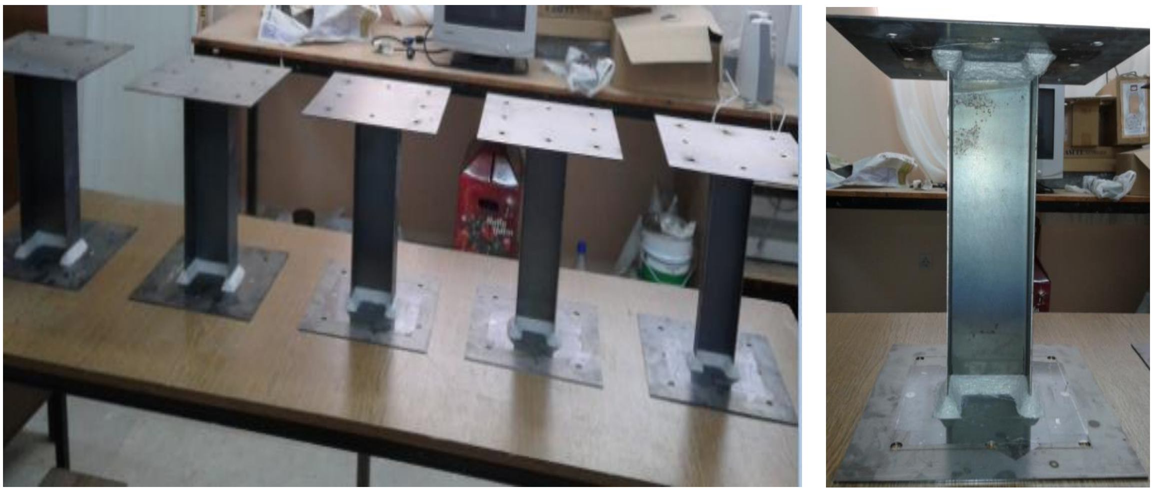


Figure 5.14 Unlipped column specimens

Table 5.8 Unlipped column specimen dimensions

NO	Specimen	b_f(mm)	b_w (mm)	L(mm)	t(mm)	R (mm)	Fy (N/mm^2)
1	UC 82-31	31.74	82.69	305	1.977	1.77	288.3
2	UC 93-41	41.95	93.13	305	1.977	1.77	288.3
3	UC 102-52	52.09	102.87	305	1.977	1.77	288.3
4	UC 113-62	62.58	113.19	305	1.977	1.77	288.3
5	UC 123-71	71.59	123.17	305	1.977	1.77	288.3

5.4.1 Test Setup

The laboratory tests set up for the testing of column specimens is illustrated in Figure 5.15.

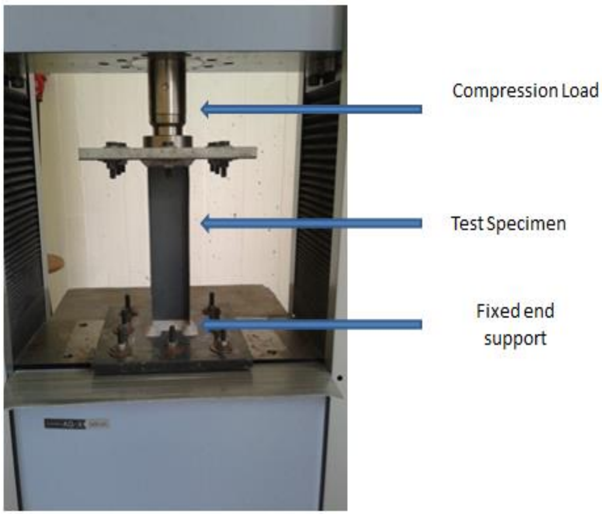


Figure 5.15 Test Set-up

Fixed-end support conditions were considered adequate for the unlipped column tests Additional lengths of 20 mm were added at the top and bottom ends so that any restraint at the fixed ends did not cause reductions between to the buckling half-wave lengths and stresses. The test specimens were located between the large cross heads of a universal testing machine and applied uniform axial compression load.

5.5 Ultimate Load

The ultimate strength of the test specimens is defined as the maximum load achieved. These ultimate strengths are calculated according to AISI and EC3 using Ms Excel program. The test results are later compared with AISI and EC3 results in this chapter.

5.6 Imperfection

Fabrication is the process used to manufacture steel work components. A column may have imperfections induced during manufacturing or erection.

There are different kinds of imperfections it is realized that the imperfections depend on the aspect ratio of the web, flange width as well as other factors such as material properties and brake process.

The imperfection is caused by some difference in the flanges and webs which may have a direct effect of the laboratory results.

The manufacturing industries making up a large number of columns without imperfection are not easy work. For this reason, good and quality materials must be considered for the columns. In addition, adequate time, resources and appropriate machines must be put in place to get an accurate result to overcome the imperfections thereby making the results acceptable when tested during laboratory work. Inspected imperfection are summarizes for each specimen;

Column- UC 82-31

The imperfection of this column, the flanges are almost equal, the back side (web) is bigger than the front side but at the last quarter if the web at the front side is greater than the back side. It has wave (inclined) on the web and not accurately perpendicular to the flanges.

Column- UC 93-41

In this column, there are equal imperfections in right and left flanges. The flanges are perpendicular to web and it is perfectly flat.

Column- UC 102-52

In this column, the differences of the flanges are somehow (almost flat) the same. But the web, the first half the web is greater than the front and the last half the front is also greater than the back side which make it not perfectly flat and it is not accurately perpendicular to flanges.

Column- UC 113-62

In another perfection in this column, the flanges are almost the same and also the web is flat they are almost the same. The web is not perpendicular to the flanges which mean the angle between flanges and web more than 90 degree.

Column- UC 123-71

In this column imperfection, some deference between flanges. At the last quarter of the web it’s smaller than front side which is making less than 90 degree between flanges and web.

With regards to the columns, though, there are some minor imperfection in geometrical aspects of the columns and these imperfection effects on the test results.

5.7 Test Results

For the UC-A test specimen with 31.74mm flange width, the permanent effect of local buckling was visible near quarter, half-length as well as UC-B and UC-C as shown in the Figures 5.16, 5.18 and 5.20.

At the elastic stage, the unlipped columns experienced, local buckling at web as the applied load increased. The effect of local buckling at mid-length increased when the column reached ultimate strength. Deflection in web and flanges continued to increase, accompanied by a sudden drop in load carrying capacity when the specimen failed. The deformation occurred in the web and flanges unlipped column specimen as shown in the Figures 5. 22 and 5. 24.

However, Load displacement curves of specimens were recorded during the tests using equipment of the testing machine and presented in the Figures 5.17,5.19,5.21,5.23and 5.25.

The experimental ultimate loads were compared with theoretical results as shown in the Table 5.9. The proposed design method results are good agreement with experimental test results.

A) UC 82-31

Figure 5.16 Failure modes of test specimens UC 82-31

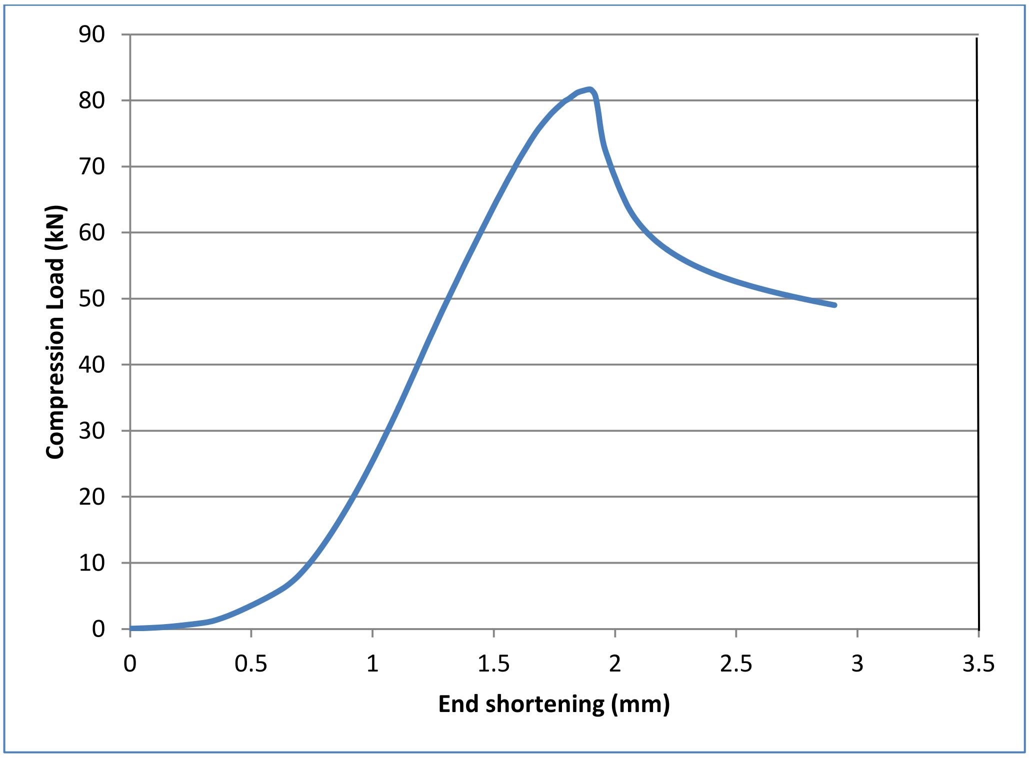


Figure 5.17 Load vs. end shortening curve for UC 82-31

B) UC 93-41

Figure 5.18 Failure modes of test specimens UC 93-41

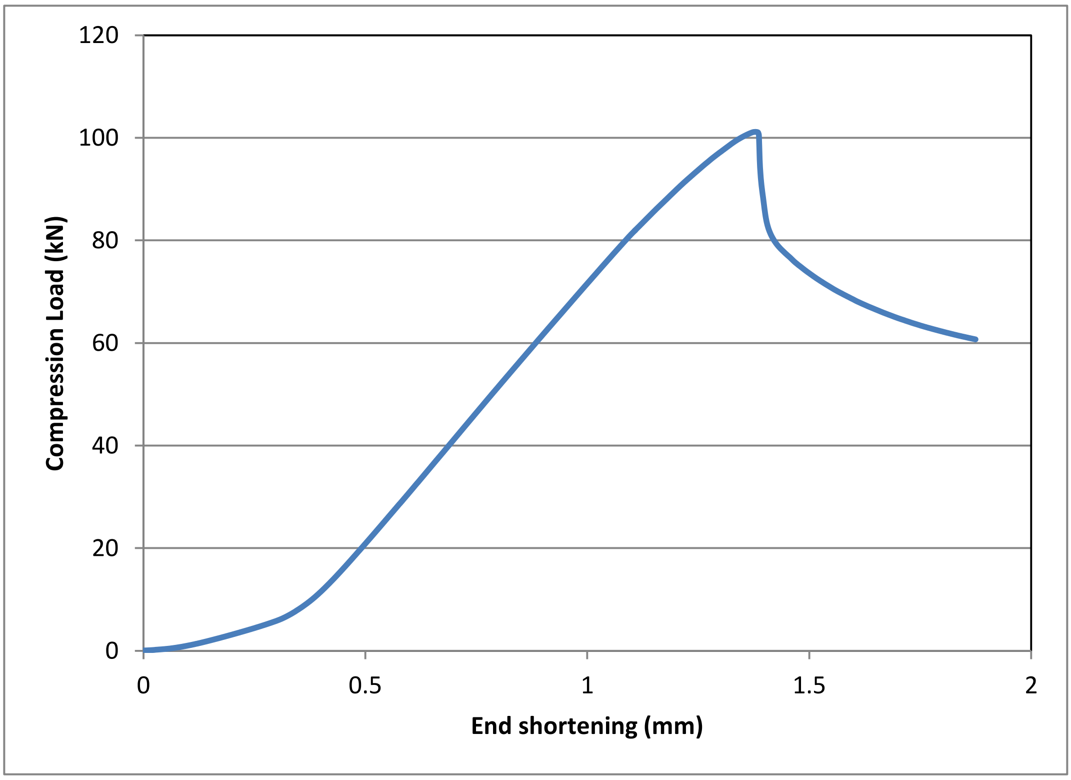


Figure 5.19 Load vs. end shortening curve for UC 93-41

C) UC 102-52

Figure 5.20 Failure modes of test specimens UC 102-52

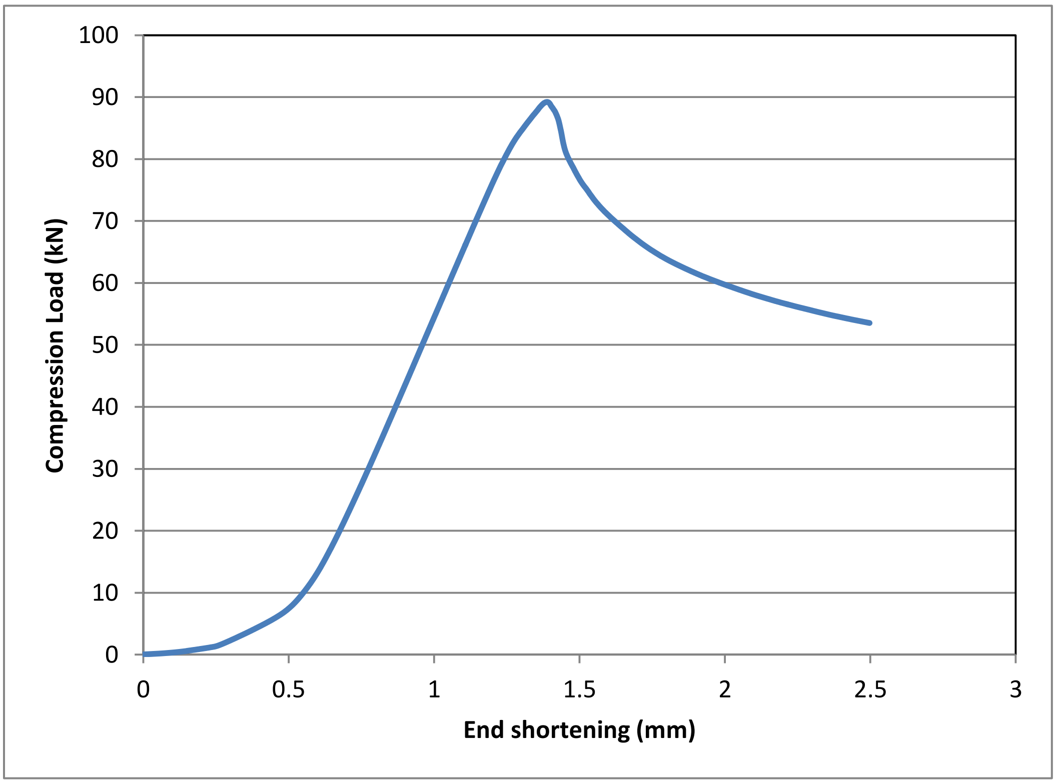


Figure 5.21 Load vs. end shortening curve for UC 102-52

D) UC 113-62

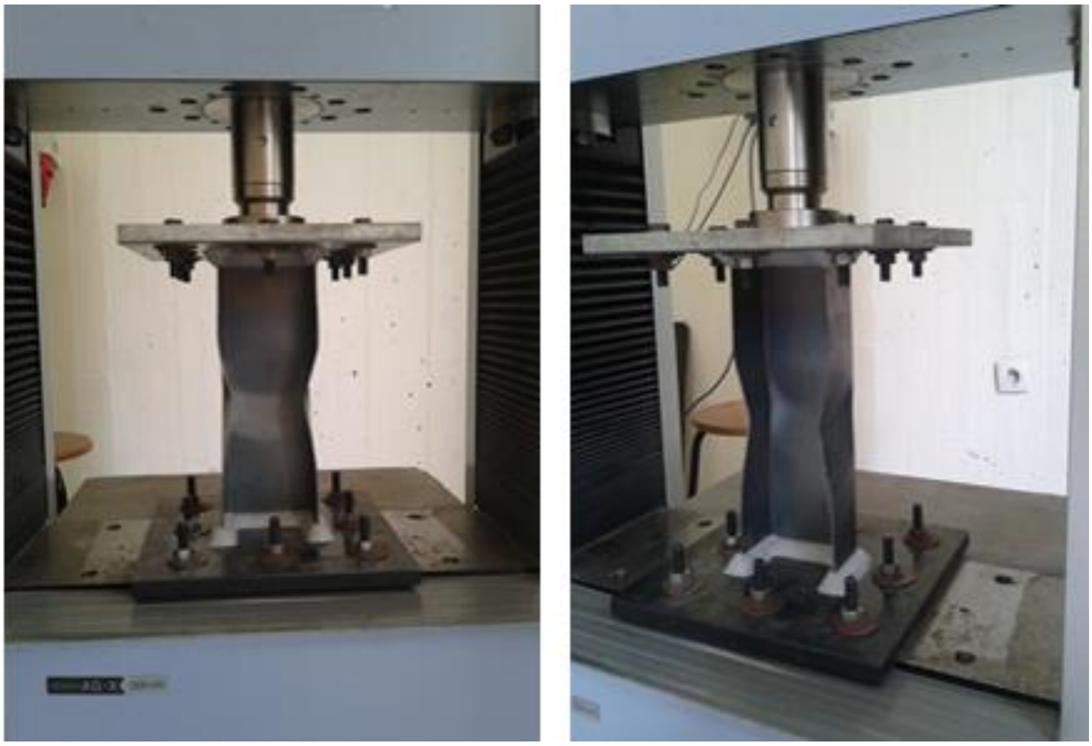


Figure 5.22 Failure modes of test specimens UC 113-62

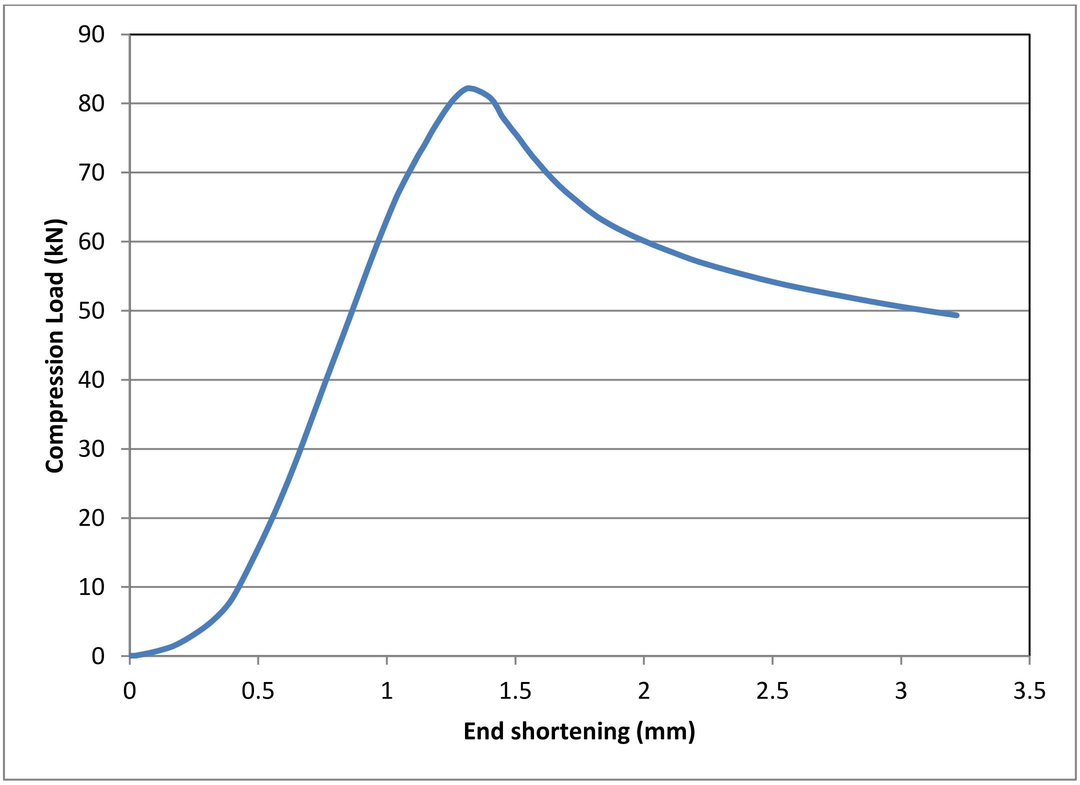


Figure 5.23 Load vs. end shortening curve for UC 113-62

E) UC 123-71

Figure 5.24 Failure modes of test specimens UC 123-71

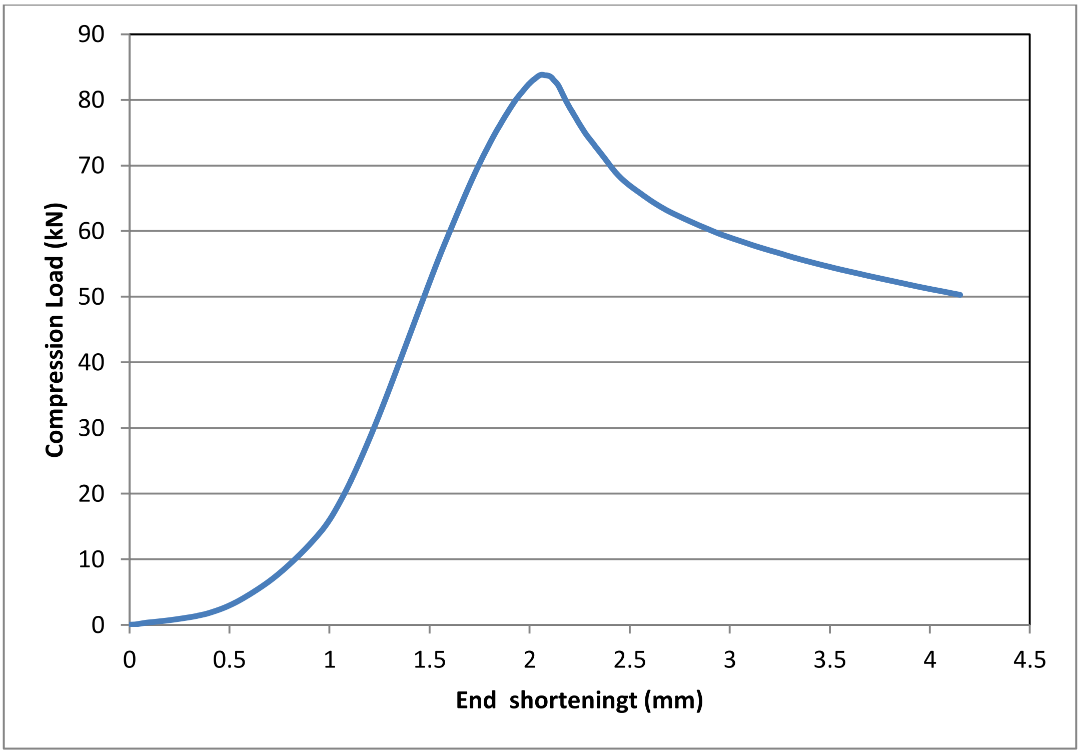


Figure 5.25 Load vs. end shortening curve for UC 123-71

After the test

(A)

A) UC 82-31 B) UC 93-41 C) UC 102-52

D) UC 113-62 E) UC 123-71

(B)

Figure 5.26 Failure modes of tested unlipped columns

5.8 Comparison of Test and Code Predictions

In this part the design strength of each cross-section is calculated by using codes and results compared with corresponding experimental result and end shortening curve based on test design strength.

The design strength calculated by using codes and experimental results shown in Table 5.9. The ratios of the test result of the design calculated results of the AISI and EC3 are also included Table 5.9.

Discussion of results:

The comparison of the results clearly indicates that due to the in some cases, some differences between the values in the checking formulae the offering of the axial action to the collected utilization is different, but as the results are in general in good agreement, both codes of the contain a consistent method to estimate cross-sectional properties and design strength over a vast range of parameters; although, the cross-sectional properties and formulae of the two standards may not be mixed.

Table 5.9 Comparison of test with ultimate loads (kN)

Specimen	AISI	EC3	Test	Test/Design	
				AISI	EC3
UC 82-31	70.73	68.37	81.67	1.15	1.19
UC 93-41	76.10	74.34	101.14	1.32	1.36
UC 102-52	79.46	77.09	89.21	1.12	1.15
UC 113-62	81.97	79.22	82.21	1.002	1.03774
UC 123-71	83.75	80.76	83.81	1.000	1.03776

5.9 Summary

In this chapter, the verification results were carried out under the finite element method, experimental work and other methods by above mentioned researches then a program was made by the present study using two international codes.

Several comparisons were made from verification results such as GÖĞÜŞ et al [53]. Silvestre et al [54)]… etc.

There are some discrepancies in the ultimate strength results between experiments and standards. This may be due to the imperfection such as, differences in flanges and web… etc.

This research described an experimental investigation on the mechanical properties of CFS undertaken to determine such properties and to develop suitable guidelines to predict the yield strength of steel.

The load capacity of the CFS columns was analysed, experimented and compared by using two standards. It gives a good result for which can be recommended by researchers in practical aspects of construction. But careful measures must be put in place to avoid any mistake in constructions.

CHAPTER 6

CONCLUSIONS AND FUTURE WORKS

6.1 Conclusions

The design of CFS columns in this study is based on the currently available standards for the CFS structures. This thesis has outlined two current approaches to the design of unlipped and lipped columns using the EWM to the AISI and EC3. This study on application of EC3 (Part 1.3) and AISI specification provisions to estimate the ultimate loads of CFS structure. The following points show the distinctive conclusion from the results.

- The test results give a vast basis for analysing the behaviour of the tested members, checking the design standards and design method for developing failure modes.
- By performing more tests and evaluating the results statistically can be very conservative and can increase the base design.
- The variety of C-sections or CFS in general makes direct comparison very difficult due to abundance of tested results.
- The design approaches provide design resistance which may be used directly in the design, but have main disadvantages of being time consuming and expensive.
- Two methods are involved in obtaining design strength, which are laboratory test to study and analysis the process of failure based on observation, the principle of calculation can be developed by standards.
- Comparison of two methods shows that, the prediction is generally conservative of C-channel. Among the design methods, AISI is the most appropriate design method for this unlipped C-channel column.

6.2 Future Works

Below are possibilities that exist for extending the different aspects of the present work.

- The behaviour of structures should be investigated continuously with different member length to make is save to improve on the design procedures.
- The failed C-section columns can be applied to make it possible for analysing the post failure behaviour of them.
- The laboratory test results should save as basis of the studies for some detailed analysis.
- Studies on local, distortional and global failure obtained in the test con also set down the foundation of such system.

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