ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

A Computer Program for the Analysis and Design of Composite Timber-Steel Floor Joists Considering Local Buckling

A Thesis in Structural Engineering

By Samrawit Amsalu

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the **Degree of Master of Science**

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UNDERTAKING

I certify that research work titled "**A Computer Program for the Analysis and Design of Composite Timber-Steel Floor Joists Considering Local Buckling Effects**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

This paper reports the local buckling effect on CFS section and an analysis and design program for composite timber CFS floor joist slab was developed. Cold formed steel structural members have enjoyed an increasing popularity over recent years. These sections offer various advantages over hot rolled steel sections such as, high strength-to – weight ratio, a straight forward and versatile manufacturing process. The local buckling effect on CFS sections was studied using numerical analysis on a finite element simulation software ABAQUS. Five tests were conducted to study the local buckling effect on channel CFS sections. Results show that all specimens but one had a failure stress less than the yield strength. Indicating that the specimens with web height ranging from 64 mm to 203 mm are class 4 and one of the specimens with web height of 40 mm is different from class 4 however, Eurocode 3 Part 1-3 treats all CFS sections as class 4 which leads to conservative and uneconomical design. The classification stated on Eurocode 3 Part 1-1 classifies the sections used as class 1 for web height ranging from 40 mm to 102 mm and the one with web height of 203 mm as class 3. This contradicts with the results obtained from ABAQUS. From this it can be concluded that this classification cannot be applied for CFS sections and since all CFS sections are not class 4 there should be a classification for CFS sections alone. Furthermore, the designing program gives out sufficient sections for the given slab size and the intended purpose of the slab. Results include sufficient timber section, CFS floor joist section from database with its spacing or checks if the user inserted section of timber and CFS are sufficient or not. If transversal beam is said to be needed by the user it gives out the sufficient IPE section. Last but not least the sufficient IPE section for main beam is also provided with their corresponding weights for economical and efficient design. Based on this study cold formed steel sections tend to buckle locally as the web height increases, so while using slender sections it is better to use stiffeners or other infill materials to prevent the section form buckling locally and the classification of sections in to classes should be given attention for efficient use of the sections.

Keywords: Cold formed steel (CFS), joists, oriented standard board (OSB), local buckling

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LIST OF ACRONYMS

CFS- Cold-formed steel

OSB- Oriented Standard Board

FEM- Finite element model

LIST OF SYMBOLS

Latin upper case letters

A	Area
As	Effective area of the edge stiffener
B	Flange width of CFS section
C	Lip length
E	Young’s modulus of elasticity
F_v	Shear force
$I_{eff,y}$	Second moment of area
I_s	Effective second moment of area of stiffener
I_t	Torsional moment of inertia
I_w	Warping constant
I_y	Moment of Inertia in the Y-axis
I_z	Moment of Inertia in the Z-axis
J_s	Joist spacing
J_{sd}	Joist spacing based on deflection and the load applied on the slab
J_{sm}	Joist spacing based on the moment capacity and the load applied on the slab
J_{sv}	Joist spacing based on the shear capacity and the load applied on the slab
K	Spring stiffness per unit length
$M_{b,Rd}$	Buckling moment resistance
M_{cap}	Moment capacity of the timber panel
M_{cr}	Critical moment
$M_{cR,d}$	Design bending resistance.

$R_{w,Rd}$	The local transversal resistance of the section
$V_{b,Rd}$	Shear buckling resistance-
V_{cap}	Shear capacity of the timber panel
$W_{eff,y,c}$	Effective section modulus with regard to the flange in compression
$W_{eff,y,t}$	Effective section modulus with regard to the flange in tension
$W_{eff,y}$	Effective section modulus
Z_c	Position of the neutral axis with regard to the flange in compression
Z_t	Position of the neutral axis with regard to the flange in tension:

Latin lower case letters

b_1	Total width of flange in compression
b_2	Total width of flange in tension
b_{eff}	The effective width
b_{p1}	Width of flange in compression
b_{p2}	Width of flange in compression
c	Total width of edge fold
c_{eff}	Effective width
c_p	Width of edge fold
f_{bv}	Shear strength
f_u	Ultimate strength
f_y	Yield strength
f_{yb}	Basic yield strength
h	Total height

h_c	The position of the neutral axis with regard to the flange in compression
h_{eff}	The effective width of the web in compression
h_p	Web height
iy	Radii of gyration
iz	Radii of gyration
K_σ	Relative buckling factor
k_2	Wet exposure condition
k_3	Load duration
k_5	Shear at notched ends
k_6	Form factor

Greek lower case letters

α	Coefficient for applied load based on support condition
ρ	The width reduction factor
ϕ	Capacity reduction factor
λ	Relative slenderness of web
σ	Bending stress
τ	Shear stress
ψ	Stress ratio
$\sigma_{m,adm,ll}$	Permissible bending stress
$\sigma_{m,g,ll}$	Grade bending stress parallel to grain
$\sigma_{com,Ed,i}$	Compressive stress
$\sigma_{cr,s}$	The elastic critical buckling stress for the edge stiffener
χ_d	Thickness reduction factor for the edge stiffener

γ_G	Factor of safety for dead load
ϕ_{LT}	Ultimate strength
α_{LT}	Imperfection factor
λ_{LT}	Non dimensional slenderness
χ_{LT}	Reduction factor due to lateral torsional buckling
Δ_m	Deflection due to bending and shear
$\tau_{m,adm,\mathrm{ll}}$	Permissible shearing stress
$\tau_{m,g,ll}$	Grade shear stress parallel to grain
γ_{m1}	Safety factor
γ_{mo}	Safety factor
$\lambda_{p,b}$	The relative slenderness for flange
$\lambda_{p,c}$	The relative slenderness for lip
$\lambda_{p,h}$	The relative slenderness for web
γ_Q	Factor of safety for live load
Δ_s	Deflection due to shear

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Chapter 1 INTRODUCTION

1.1 Background

Two types of steel are used in the construction industry, cold formed steel and hot rolled steel. Hot rolled steel is much familiar of the two and is formed by shaping the steel under high temperature whereas cold formed steel is formed at room temperature using mechanical action.

Usually cold-formed steel elements are used as secondary load bearing members such as cladding and roof purlin, but now a days cold-formed steel elements are being used as a primary load bearing members. Examples are low-rise to mid-rise buildings constructed entirely from cold-formed steel [11].

On the other hand a developing country like Ethiopia can benefit from using cold-formed steel structures, which provides faster, durable and cost effective alternative. Furthermore it fills the gap between demand and supply requirements of the construction industry. The continually developing technology of CFS construction appears to bring a viable solution to the demand of better quantity and quality construction. Cold formed steel is one of the most innovative construction technologies; it is one of the fastest ways to manufacture and erect a building.

While a whole building can be constructed of Cold formed steel, it is also possible to combine CFS structural members with other type of structural systems. The local buckling effect on a cold formed steel floor joists is studied in this study so that the advantages that can be gained from using this type of floor system can be implemented in the construction industry, and furthermore a computer program for a slab system of a composite timber cold formed steel floor joist is developed.

1.2 Statement of the Problem

In a developing country like Ethiopia it is challenging to convince the society to adapt a new technology. CFS construction is somehow growing in our country, but yet the acceptance is low. Since the advantage of using this type of construction methods are high there should be a knowhow on how it performs, constructability and other parameters must be studied, but not much researches has been done in Ethiopia.

Eurocode 3 Part 1-3 does not specify depth to thickness ratio limit for classification of sections considering all CFS sections as class 4. Is this always the case?

Furthermore, according to the new Ethiopian building code earthquake has become critical in some regions of Ethiopia. Using CFS as a construction material could be a way to steer clear of damages that could be caused by earthquake, for the proposed case since the slab is light weight it is a way of reducing earthquake load on the structure and also the load transferred to the beam and column will be low relative to the conventional construction material.

The ease of construction is the other reason that makes a CFS structure preferable. Reduction of formwork and labor cost makes it more advantageous. Even though it needs an accurate construction technique there is no worry about the grade of the steel since it is produced in a factory with a precision unlike the conventional construction method.

Time value of money is another critical point on the construction world. This construction method needs relatively less time to be constructed and give service for the intended purpose. However, empirical evidences seem scanty and this research is envisaged to fill the knowledge gap.

1.3 Objectives

The aim of this thesis is to study the potential use of cold formed steel structural system in Ethiopia.

1.3.1 Specific Objectives

- To study the local buckling effect on cold-formed steel sections.
- Detailed assessment and design of CFS sections.
- To develop a computer program for composite timber CFS floor joist floor system.

1.4 **Methodology**

Extensive reading and research is done to understand and restate the properties and advantage of cold-formed steel structures.

Tests are conducted numerically using simulation on ABAQUS 6.13 to study the behavior of a C section floor joist. The computer program is developed using Microsoft visual studio C# to design any slab size using the proposed slab system.

1.5 **Scope**

This study addresses the local buckling effect on cold formed steel sections. Furthermore, this study is limited to the following:

- Plane C section are considered.
- Furthermore a computer program on analysis and design of composite timber CFS floor joist floor system is developed.
- The composite action effect and connection design is not considered here in.

1.6 **Content and Organization**

The first chapter deals with an introduction part followed by the next chapter where the relevant literature reviews are presented to successfully conduct this study and previous researches regarding the flexural behavior of CFS joists are described. A suitable finite element model of CFS floor joists to simulate the buckling strength behavioral characteristics of CFS floor joists is presented under chapter three.

Chapter four presents the analysis and designing procedure for the proposed slab and the algorithm used for the designing program including the graphical interface of the overall program is presented under chapter five. The final chapter presents a summary of the study and recommendations made for further study.

Chapter 2 LITERATURE REVIEW

Cold-formed steel structural members have enjoyed an increasingly popularity over recent years and offers various advantages over hot rolled sections , such as high strength-to –weight ratio and a straight forward and versatile manufacturing process. Its lightness is also another advantage of cold formed steel over hot rolled sections which can save costs on transport, erection and construction of foundation. Furthermore, profiles can be produced in a wide range of variety, which results in more cost effective designs.

Cold-formed sections can be used for floor joists, roof members, steel framing, wall partitions, large panels for housing, lintels, modular frames for commercial buildings, trusses, space frames, curtain walling, prefabricated buildings, storage racking, lighting and transmission towers, motorway crash barriers, etc.(Figure 2-1, 2-2 & 2-3)

Figure 2-1 Wall framing

Figure 2-2 Floor joist

Figure 2-3 Modular frame

Even though the use of cold formed steel structures could be considered to be a new concept by many, it has been used in North America for over 100 years. In the 1850, cold formed steel was used for construction in US and England. However, the use was highly experimental and was limited to the construction of basic structures [2].

In 1849, a roofer from New York called Peter Naylor, who is to be named among the first people to engineer the construction of cold formed steel structure, advertised that he could build 20’ x 15’ (4.6m x 6m) in less than a day which are cheaper than wood, more resistant to fire and comfortable [11].

Cold-formed steel structural members are shapes commonly manufactured from steel plate, sheet or strip material. The manufacturing process involves forming the material by either press braking or roll forming. Press braking is commonly used for forming open sections such as angles and channels [7].

Press brake operation consists of a moving top beam and a stationary bottom bed on which the dies applicable to the particular required product are mounted. Simple sections are formed with no more than two operations and more complicated sections take several operations [11].

The machine used in cold roll forming consists of pairs of rolls which gradually form strips in to the shape desired. A simple section may be formed by as few as six set of rolls. But when it comes to a complex section the pair of rolls will be as high as 15 sets of rolls. At the end of the production the section is cut at the desired length without stopping the machine. Maximum cut length of the machine is 6-12m [9]. Figure 2-4 a and b shows the method of cold forming.

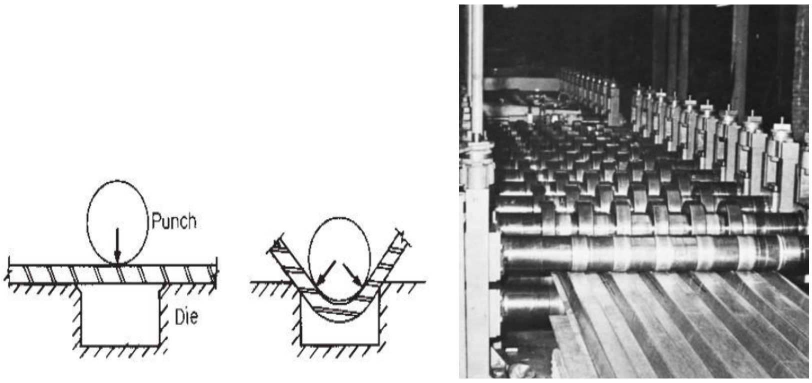

Figure 2-4: (a) Press braking[1]   (b) Cold Roll forming machine [2]

2.1 Cold Formed Steel Design Standards

Cold formed sections are thinner than hot rolled sections and are characterized by local instabilities while local buckling is not a concern on hot rolled sections. It is a major concern in the case of cold formed steel members due to high depth to thickness ratio, with a buckling stresses well below than the yield point. However it’s observed that the member continues to take more load developing stresses that are even higher than the stress that was reached when local buckling first occurred.

Due to thickness, imperfections, residual stresses, stress-strain relationships, different type of failures and behaviors is considered in characterizing cold formed steel members and has a different design specification as of hot rolled steel members.

Professor G. Winter initiated a research on the structural behavior of cold formed steel members at Cornell University in 1939, which was the first research regarding cold formed steel. American Iron and Steel Institute (AISI) prepared a design clause for cold formed steel members in 1946. In 1991, AISI introduced the Limit States method, which is based on load and resistance factors applied to the loads and design strengths respectively such that the factored design loads should not exceed the factored strength. The British Steel Standard was modified to include the design of cold formed steel members based on the work done by Professor A.H Chilver in 1961. EN 1993-1-3 also specifies guidance for the design of cold formed steel work.

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[1]Wei-Wen Yu and Roger A.Laboube, Cold Formed Steel Design, page 16
[2] Wei-Wen Yu and Roger A.Laboube, Cold Formed Steel Design, page 13

Since local buckling is critical while designing cold formed sections this effect is taken in to account by using effective cross sectional properties, calculated based on widths of those elements that are prone to local buckling. This method is named as effective width method adopted on EN 1993-1-3. This method was first introduced by Von Karmen after various experiments he concluded the ultimate load is independent of the width and length of a plate and assumed the buckled portions of a plate do not carry any load but the portion that is not affected by bulking carries load up to yield point. The effective width method assumes a uniform stress equal to edge stress is distributed along a portion of the width instead of non-uniform stress along the full width. While, direct design method is adopted by AISI, 2004. This method is the latest method for the design of cold formed steel members.

2.2 Mechanical Property

Material property plays an important role on the performance of structural members. The most important properties of steel from structural point of view are yield stress, tensile strength, stress-strain characteristics, modulus of elasticity, tangent and shear modulus, ductility, weld-ability, fatigue strength and toughness. In addition formability and durability are also important properties for thin walled cold formed steel structural members [11].

Tensile strength and ductility as mentioned above are essential factors for cold formed steel material property due to their relation to form-ability and local deformation demands of bolted and other types of connections. Tensile strength must be considered due to the high stress concentration caused by connections or bolts. [11]

2.2.1 Yield Stress F_y and Stress-Strain Curve

The strength of cold formed steel structural members depends on the yield strength of steel. As specified in the Northern American specification the yield stress ranges from 165-552MPa [11].

There are two general types of stress-strain curves sharp yielding and gradual yielding. Hot rolled steels are usually sharp yielding, while cold formed steels show gradual yielding [11].

2.2.2 Tensile Strength

Cold formed steel sections, sheets or strips tensile strength has little direct relationship to the design. The load-carrying capacities of cold-formed steel flexural and compression members are usually limited by yield stress or buckling stresses that are less than the yield stress of steel, particularly for those compression members having relatively large slenderness ratios [11].

2.2.3 Modulus of Elasticity, Tangent and Shear Modulus

The strength of members that fail by buckling depends on modulus of elasticity and tangent modulus in addition to yield stress. And the shear modulus is used to compute the torsional buckling stress which is important for the design of beams, columns and wall studs [11].

2.3 Influence of Cold Work on Mechanical Properties of Steel

The mechanical properties of cold formed steel sections are sometimes substantially different from the virgin material. This is due to the forming method used to form the sections which results in the increase in yield stress and tensile strength and at the same time decrease in the ductility. Depending on the type of cold forming and thickness the strength might increase up to 50% [11].

The study conducted by Winter and Uribe indicates that for the steels commonly used in thin-walled cold- formed steel construction considering the effects of cold work only in the corners of the formed sections, the moment capacities can be increased by 4–22% compared with those obtained when neglecting cold work. If the effects of cold work is considered on both flat and corner portions of the section, the increase in bending strength ranges from 17 to 41% above the virgin value [11].

For example, a 20% reduction in thickness can increase yield strength by 50% but reduces elongation to as little as 7%, which probably represents the limit of formability for simple shapes [9].

Since the corner portion of the section is cold worked to a considerably higher degree than the flat portion of the section, the mechanical properties are different in various parts of the cross section. Due to the lower yield stress of the material around the flat portions buckling or yielding begins on those portions. Any additional load applied to the section will spread to the corners [11].

Strength of cold formed steel mainly depends on the extent at which it has been formed. It increases as the shape gets complicated. Other than this, the influence of cold working on the mechanical properties of the corner portions also depends upon:

1. Type of steel
2. Type of stress (compression or tension)
3. The direction of stress with respect to the direction of cold work
4. Fu/Fy ratio
5. R/t ratio

Results of investigations conducted by Winter, Karren, Chajes, Britvec, and Uribe on the influence of cold work indicates that the mechanical properties that are changed due to cold work are caused mainly by strain hardening and strain aging [11].

On the figure 2.5 below the influence of cold working is shown using different curves with their relative values. Curve A represents the stress–strain curve of the virgin material. Curve B is due to unloading in the strain-hardening range, curve C represents immediate reloading, and curve D is the stress–strain curve of reloading after strain aging. It is interesting to note that the yield stresses of both curves C and D are higher than the yield stress of the virgin material and that the ductility decrease after strain hardening and strain aging. In addition to strain hardening and strain aging, the changes in mechanical properties produced by cold work are also caused by the direct and inverse Bauschinger effect [11].

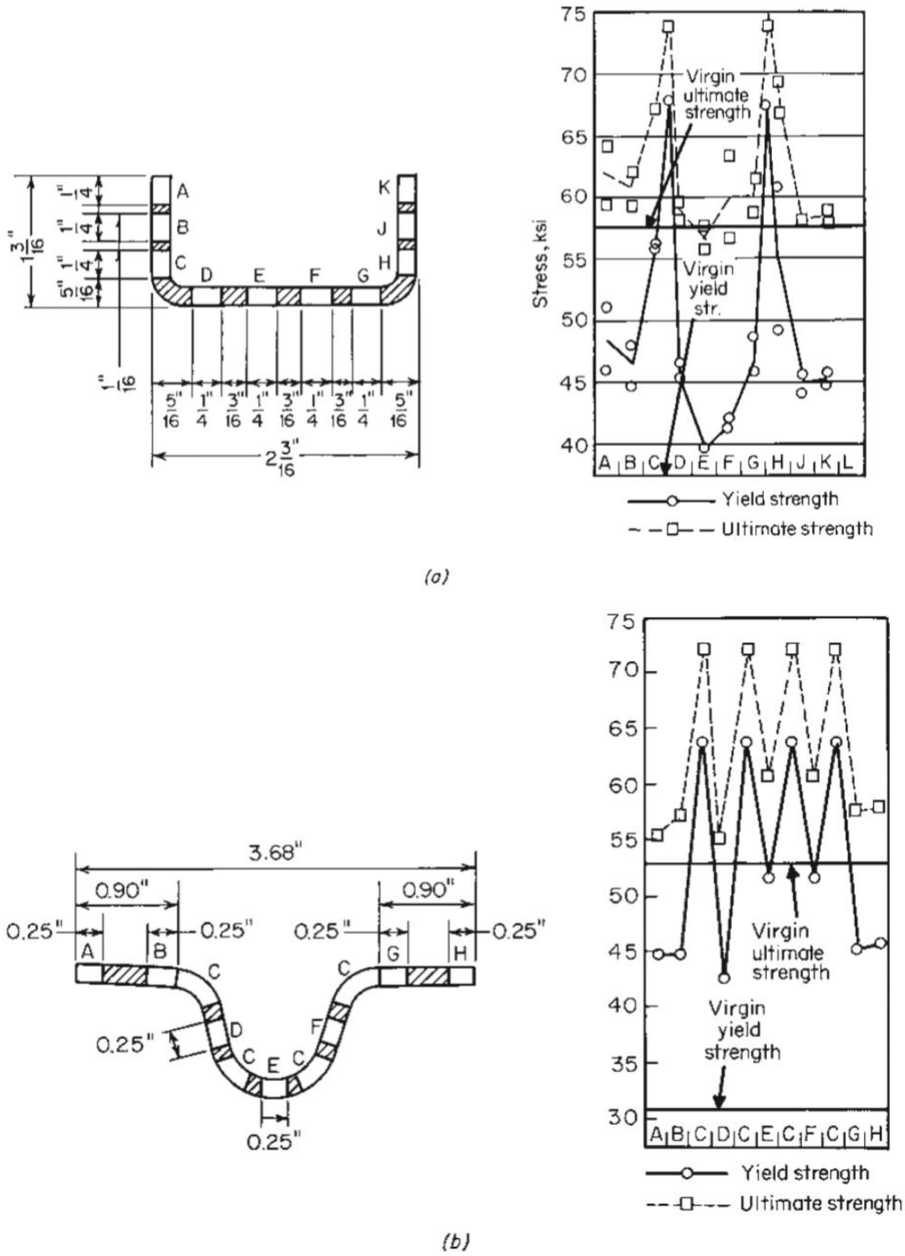


Figure 2-5 Effect of cold work on mechanical properties in cold formed steel sections[3]

The other impact of cold forming on the mechanical property of the cold formed steel is the formation of residual stress which varies across the sheet thickness, the outer fiber tends to elongate, while the center tends to remain unchanged, but yet there must be some deformation between the surface and the center through the thickness. The internal

[3] Wei-Wen Yu and Roger A.Laboube, Cold Formed Steel Design, page 38

fibers resist the elongation of the external fibers while the external fiber tries to stretch the internal once, which results in residual stress distribution with compression on the surface and tension within the thickness.

2.4 Flooring

Cold formed steel floor system comprises series of equally spaced CFS joists (usually c or z shapes are used) sheathed with subfloors and reinforced with transverse elements. The sub floor could be plywood, oriented standard board (OSB), cementations board or corrugated steel deck with lightweight gypsum based underlayment [8].

This floor system can span about 9 m between supports. Because of their light weight, CFS members can be handled and installed easily and quickly. Connections of cold-formed members are usually accomplished by welding or by the use of self-drilling screws [7].

Load-carrying panels and decks not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist force in their own planes if they are adequately interconnected to each other and to supporting members [11].

Oriented standard board (OSB) is a structural panel widely used for construction and industrial applications. It is a mat formed panel made of strands sliced from small diameter, fast growing round wood logs and bonded with an exterior type binder under heat and pressure. The bending strength of OSB comes from uninterrupted wood fiber, interweaving of the long strands and orientation of strands in the surface layers. The surface layers are aligned in the long direction of the panel for superior bending strength and stiffness in this direction and the core layer is either randomly oriented or perpendicular to the face layers. The water proof and boil proof resin binders combined with the stands provide internal strength rigidity and moisture resistance [3].

The OSB sheathing serves the primary function of providing lateral restriction for cold formed steel joists and transmitting loads into the structures [10].

Other timber panel as plywood can be used as a sheathing panel, but OSB panel is popularly used with cold formed steel joists. Figure 2-6 shows the layer of OSB and Figure 2-7 shows the proposed slab system.

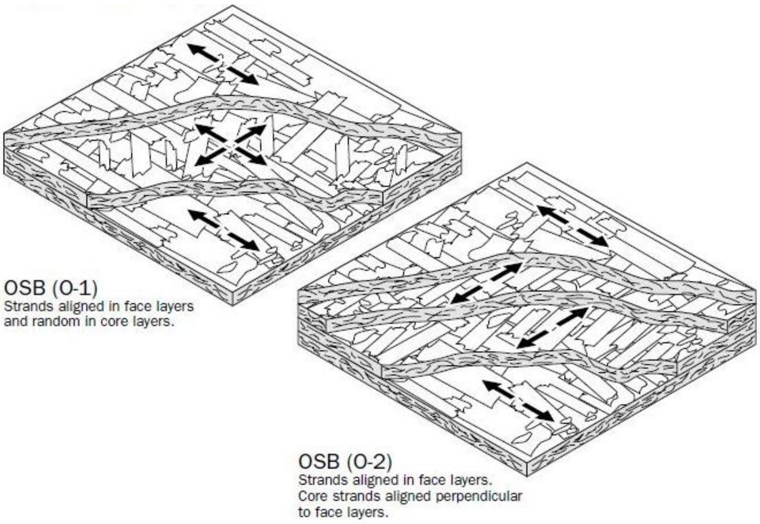


Figure 2-6 Oriented Standard Board [4]

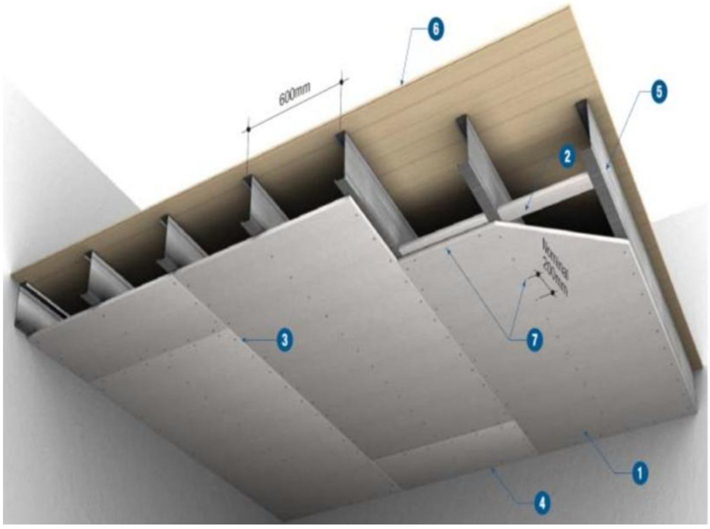


Figure 2-7 Joist floor system

[4] Construction sheathing and design rated oriented strand board structural board association, OSB design manual, page 2

2.5 Structural behavior of CFS floor Joists

According to tests conducted on composite CFS and OSB flooring the system has small deformation and high bearing capacity furthermore, the screw spacing significantly affected the load carrying capacity [10].

The main type of failure that occurs on CFS flooring system is flexural and torsional buckling of the floor joists and interactive buckling of compression flange web [10].

The wave length of buckling equaled to the adjacent screw spacing. From FEM analysis results reasonable screw spacing was suggested, 150mm center spacing at OSB edge and 150-300mm at intermediate supports [10].

2.6 Related researches

Alex. J investigated the flexural behavior of cold formed steel section, Experimental investigation were carried out to make a comparative study on the flexural behavior of built up cold formed steel section without lip and cold formed built up steel section with lip and from the experiments it was found that the ultimate load carrying capacity of cold formed built up steel section without lip was 16% higher than that of cold-formed built-up steel with lip and the ultimate deflection of the cold-formed built-up steel section without lip was 3.66 mm and for the one with lip was 4.18mm [1].

Analytical study on flexural behavior of cold formed hollow Flanged Z-sections was conducted by P. Mangala Gowri. From this study it was concluded that the rectangular hollow flanged section shows 9% increase in performance to sustain maximum load than the triangular hollow flanged section.

The other research conducted was on the flexural behavior of cold formed steel beams filled with concrete by Lenin Muthu Olivu.M and it was concluded that failure occurred due to buckling of the in filled beams where concrete in filled was easily separated from the steel section. It was also noticed that local buckling occurred in the compression zone.

Nader R.(1999) investigated the horizontal diaphragm experiments of cold formed steel joists-OSB composite floor in order to test its stiffness and ultimate shear strength.

Chapter 3 FINITE ELEMENT MODELING OF COLD-FORMED STEEL FLOOR JOISTS

To use CFS sections no depth to thickness ratio limit has been specified on Eurocode 3 Part 1-3 [5]. Suggesting that all CFS sections are class four. Numerical simulations are conducted to check if this is valid.

The finite element modeling of cold formed steel floor joists is conducted on ABAQUS commercial software. The test specimens are of a C section joists with different geometric property.

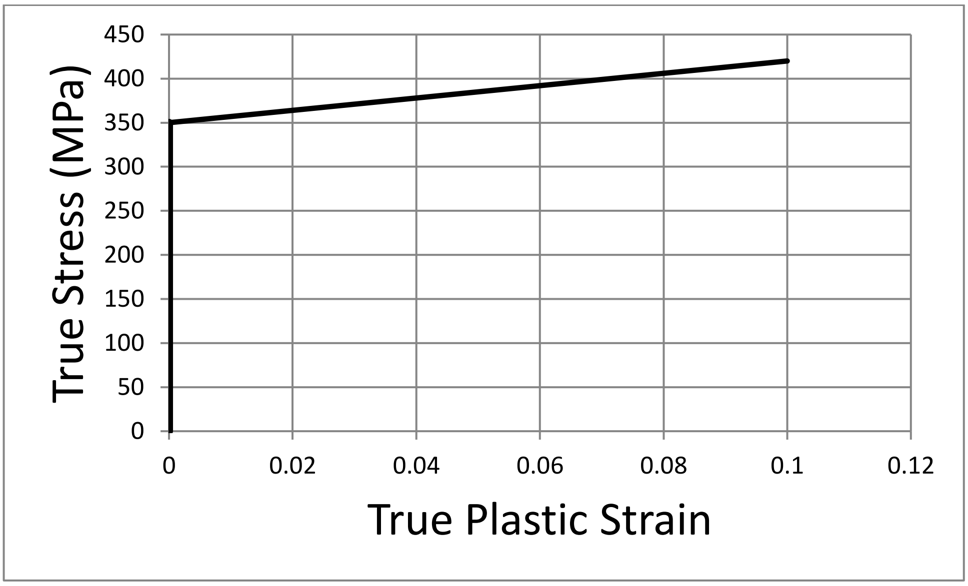


Figure 3-1 Constitutive Model

3.1 Validation of Experimental Finite Element Models

The validity of finite element models must be established before undertaking detailed parametric studies, which ensures that the developed finite element model can be used to simulate the desired behavior of CFS joists.

A nonlinear analysis was conducted in this research to determine the load and deformations. The CFS joists were considered to be fully restrained against lateral torsional buckling by the OSB/plywood and plaster board at the bottom and top flange, meaning the members capacity is controlled by material yielding, distortional and local buckling. To validate the model the mid span flexural deflection calculated based on elastic theory for a simply supported beam at a different loading step is compared with the deflection results from the FEA.

A plane channel CFS section with a web height of 76mm flange width of 44 mm, a lip length of 11mm and thickness of 1 mm was modeled on ABAQUS. As shown on Figure 3-2 below the result from the FE analysis agrees well with the results calculated based on elastic theory which validates the model and can be used for further study.

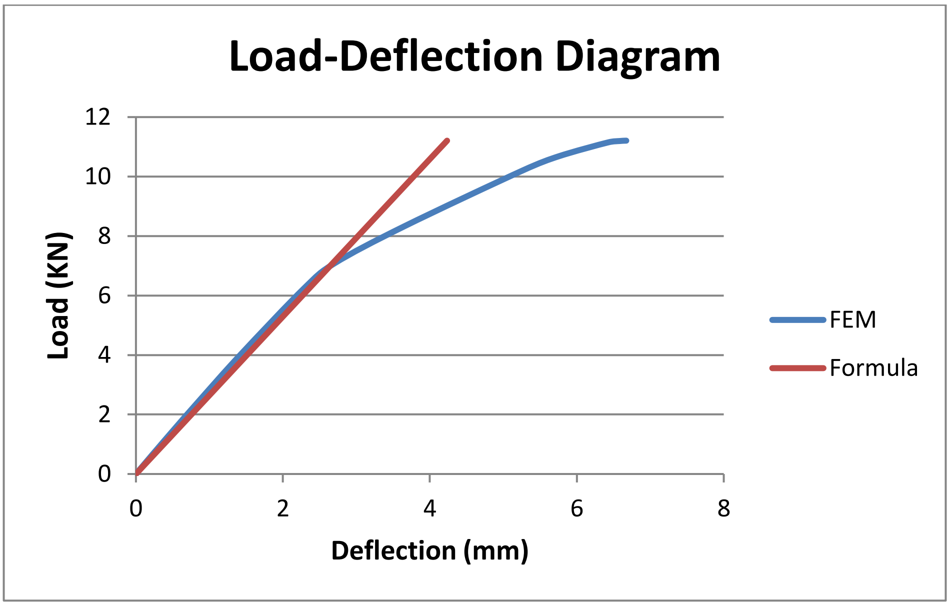


Figure 3-2 Load deflection diagram

3.2 Finite Element Modeling

ABAQUS 6.13-1 is used to simulate a cold formed channel floor joist and study the local buckling behavior of the section.

3.2.1 Element type and meshing

The thickness of cold formed steel ranges between 1-8mm, due to the thinness of the section thin shell theory is more suitable for CFS floor joists. Based on researches conducted there are some suitable shell elements available on ABAQUS. Four nodded shell elements S4, S4R, S4R5, eight nodded shell elements and nine nodded shell elements are amongst the types of element.

Based on a study conducted by Balachandren. B, comparison was made on the element types available on ABAQUS and some consumed enormous memory and analysis time. The study concluded that S4R is the most cost-efficient and which can explicitly model the behavior of CFS floor joists.

The cold formed steel joist is modeled using S4R shell and a mesh size of 10mm is used as shown of Figure 3-1 below.

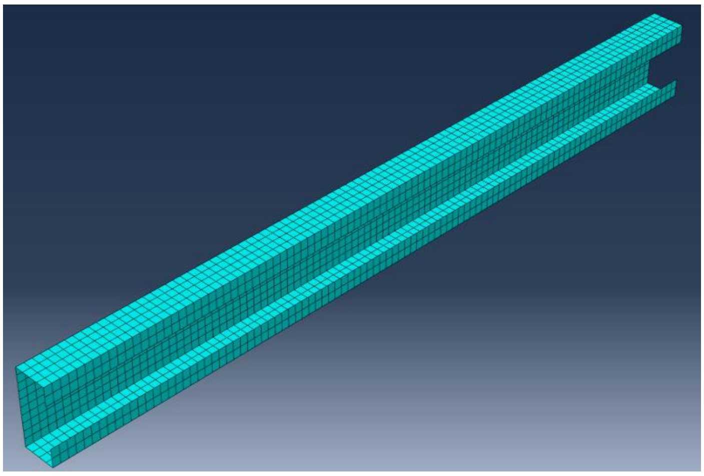


Figure 3-3 FE model mesh

3.2.2 Material modeling

Mechanical properties required for elastic and non-linear analyses are Young's modulus of elasticity, yield strength and poison's ratio.

S350GD+Z $\quad\quad$ v=0.3

E=210000N/mm^2

3.2.3 Boundary condition

To simulate the pin support at one end single point constraint of "1234" is applied on the web element and "234" degrees of freedom of all the other nodes are restrained. The roller support is modeled by restraining "234" degrees of freedom of all the nodes.

To simulate the restraint against lateral displacement provided by the OSB/Plywood, restraint against lateral displacement along the Z-axis and the rotation about the X-axis is considered at several regions of the top and bottom flanges depending on screw spacing required for the tests conducted to study the local buckling behavior of CFS floor joists as show on Figure 3-4 below.

3.2.4 Loading

The joists are assumed to be equally spaced and uniformly loaded 5 N/mm over the upper flange as show on Figure 3-4 below.

The joist is loaded at an increment size of 0.01, minimum increment size of 1E-15 and a typical maximum number of load increments of 100.

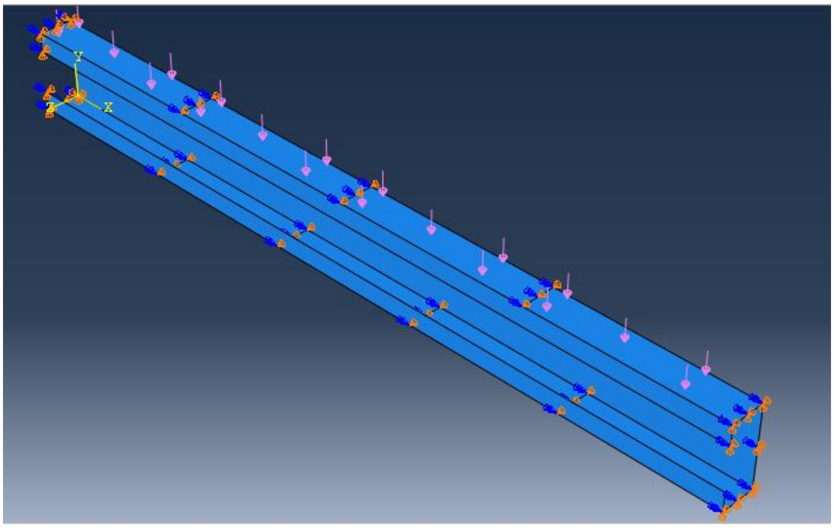


Figure 3-4 FE model boundary condition and loading

3.3 Description

The local buckling behavior of channel cold formed steel joists is studied by considering various web length while the span, thickness, flange width and yield strength are kept constant.

3.4 Test specimens

3.4.1 Simulation of test set 1 (H64, H76, H102, H203)

The FE models were created by following the steps in section 3.2 to study the local buckling effect of CFS channel sections on a flexural member. Steel grade of S350GD+Z was used for the numerical model and the geometric properties of the tests are shown on Table 3-1 below.

Table 3-1 Test sets

Span	Flange width(mm)	Web height (mm)
1m	44	64
		76
		102
		203

3.4.2 Simulation of Test 2

A CFS section with a web height of 40 mm, flange width of 20 mm, a lip length of 10 mm and thickness of 3mm were created by following the steps in section 3.2 and section 3.4.1.

3.5 Result and Discussion

According to the numerical analysis, the local buckling effect on the flexural capacity of CFS floor joists can be concluded as follows, as the web height increases the failure stress decreases and it's more prone to local buckling than a section with a shorter web. The detailed results are shown of Table 3-2 below.

Table 3-2 Test result output

Web height (mm)	40	64	76	102	203
fy (N/mm^2)	350.407	248.06	223.62	220.552	201.033
Δ (mm)	11.6656	7.34	6.9893	6.86119	1.63443

Furthermore, it is obtained that the sections fails due to local buckling before it gets to the yield point for this case $350N/mm^2$ was used for yield strength. But one of the tests with a web height of 40mm and thickness of 3mm do not fail due to local buckling, the section yields before failure as shown Figure 3-5 below.

Eurocode 3 part 1-3 has no section classification for CFS as of hot rolled sections, assuming all CFS sections are class four which underestimates the capacity of sections and leads to uneconomical design. Eurocode 3 Part 1.1 [4] classifies hot rolled sections into class 1, 2, 3 and 4 based on the depth to thickness ratio. From Table 3-3 below the classification shows that all the sections are not class 4 but from the simulation it's shown that the sections fail due to local buckling meaning the sections are class 4 except the one with web height of 40mm.

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/350} = 0.819$$

$$d/tw \le 124\varepsilon (= 101.606)$$

Table 3-3 Classification of web according to Eurocode 3 Part 1.1

Web height (mm)	40	64	76	102	203
d/tw	13.33	25.6	30.4	40.8	81.2
Class	1	1	1	1	3

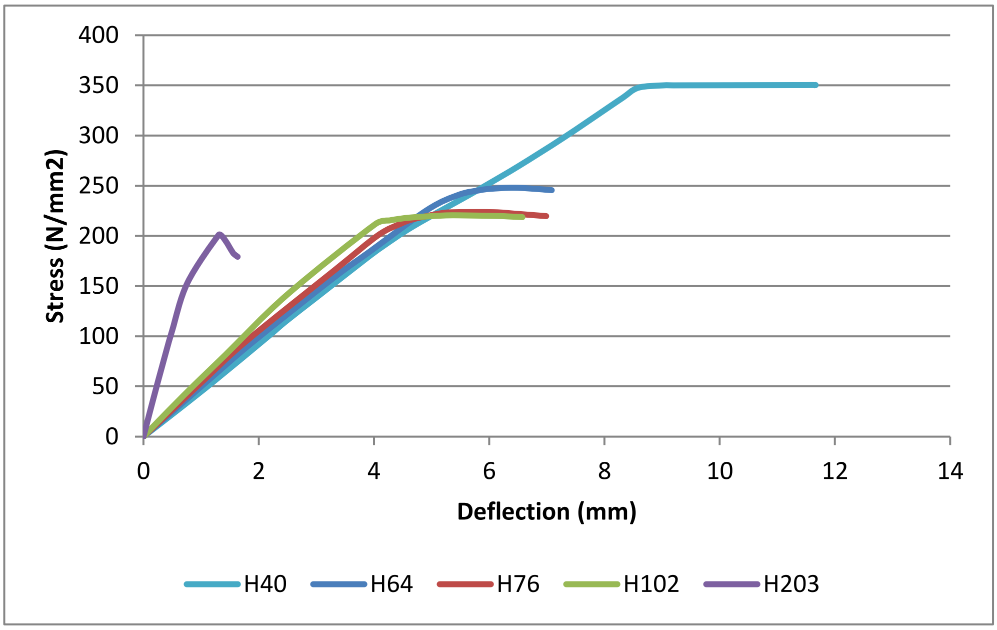


Figure 3-5 Stress deflection diagram

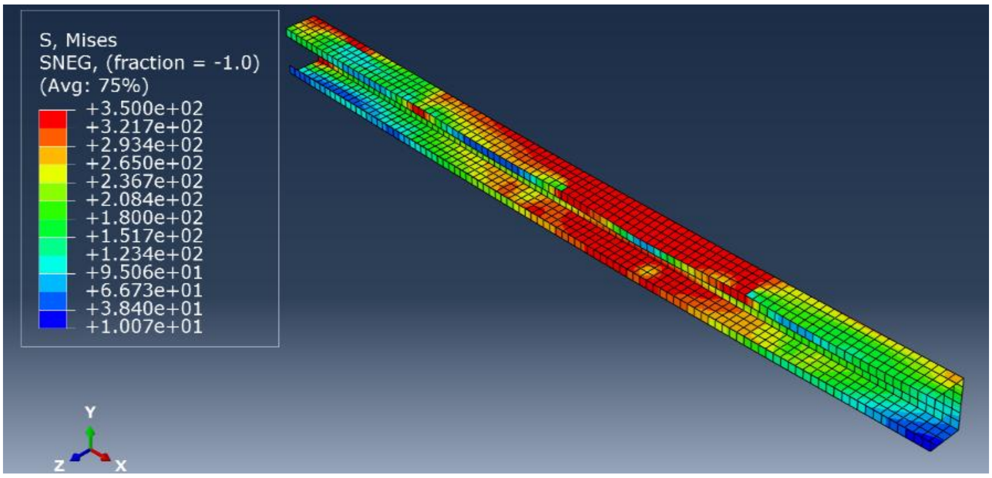


Figure 3-6 H64 Von-Mises Stress

As a result, the classification for hot rolled sections cannot be used for CFS sections, thus the limitation for classification of classes should be reviewed for CFS sections on the code for more efficient classification and economical design. As shown from the numerical analysis CFS sections are prone to local buckling therefore, the computer program is fed to check local buckling according to Eurocode 3 Part 1-3.

Chapter 4 ANALYSIS AND DESIGN OF COMPOSITE TIMBER-COLD FORMED STEEL JOIST SLAB

4.1 General

4.1.1 Loading

The loads that are considered on a slab for the purpose of computing the maximum effect are the following;

1. Permanent loads: Dead loads, superimposed loads
2. Transient Load: Live loads

Dynamic load (earthquake loads) are not considered here in.

4.1.2 Material properties

i. Oriented Standard Board(OSB)/ plywood

Oriented standard boards are usually used as a sheathing plate for a light weight floor system, providing lateral restrictions for CFS joists. Here in thicknesses ranging from 11mm to 28.5mm are used. A detailed section properties of OSB is attached on Annex B.

ii. Cold formed steel sections

Cold formed steel sections can be formed in variety of shapes and size which makes the sections versatile and convenient for economical design. Here in only plane channel sections are considered. Thickness of sections inputted for the program ranges from 1mm to 3mm with different web and flange sizes as shown on Table 4-2 below. The virgin material steel grade is attached on ANNEX B.

Table 4-1 Cold formed steel section properties

Section name	H	B	C	t
	mm			
LL06425	64	38	13	2.5
LL07610	76	44	11	1
LL07625	76	44	16	2.5
LL10225	102	51	18	2.5
LL10230	102	51	19	3
LL12725	127	51	18	2.5
LL12730	127	51	19	3
LL15230	152	64	21	3
LL20330	203	76	24	3

iii. Transversal and main beam

IPE sections are used for transversal and main beams on this program. The standards used are from IPE 80 to IPE 600, with a steel grade ranging from S235 to S350.

4.1.3 Assumptions

The following assumptions are made in developing the program.

1. The composite effect of the OSB/plywood with the cold formed joist is not considered, it is assumed as if they are acting independently.
2. All joists carry the same amount of load from the applied load and equal joist spacing is used for the design.

4.2 A Step By Step Analysis and Design

4.2.1 Design loads

- Self-weight of the OSB/Plywood panel, cold-formed joists, the transversal and main beam and floor finish.
- Live load for the intended purpose of the slab as specified on EN 1994-1-1[4]
- The load combination is;

$$q_d = \gamma_G q_G + \gamma_Q q_Q \tag{4-1}$$

Where: $\gamma_G - 1.35$
$\gamma_Q - 1.5$

4.2.2 **Design procedure**

I. OSB/Plywood

The main design considerations for flexural members based on BS 5268: Part 2: 2002 are:

- bending stress and prevention of lateral buckling
- deflection
- shear stress

Step 1-**Bending stress and prevention of lateral buckling**

$$\sigma = \frac{M * C}{I} \tag{4-2}$$

The permissible bending stress $\sigma_{m,adm,ll}$ is calculated as the product of grade bending stress parallel to grain $\sigma_{m,g,ll}$ and any relevant modification factors (K-factors).

$$\sigma_{m,adm,ll} = \sigma_{m,g,ll} * K_2 * K_3 * K_6 * K_7 * K_8 \tag{4-3}$$

Where: k_2 - Wet exposure condition. Values are attached on Table B-2 on Annex B

k_3-Load duration Values are attached on Table B-3 on Annex B

K_6 - Form factor (1 for rectangular sections)

k_7 -Depth factor for:

For :

$t \leq 72$	$k_7 = 1.17$
$72 < t < 300$	$k_7 = \left(\frac{300}{t}\right)^{0.11}$
$t > 300$	$k_7 = 0.81 * \frac{t^2 + 92300}{t^2 + 56800}$

k_8-Load sharing system

Grade stresses and moduli of elasticity for various strength classes are given on BS 5268: Part 2: 2002.

The moment capacity of the timber panel is calculated as:

$$M_{cap} = \frac{\sigma_{m,g,ll} * k_2 * k_3 * k_6 * k_7 * K_8 * I}{C} \tag{4-4}$$

Lateral stability

BS 5268: Part 2: 2002 recommendation on the depth to breadth ratio of solid and laminated rectangular sections is attached on annex Table B-1

Step 2- **Shear stress**

$$\tau = \frac{1.5 * F_v}{A} \tag{4-5}$$

The permissible shearing stress $\tau_{m,adm,ll}$ is calculated as the product of grade shear stress parallel to grain $\tau_{m,g,ll}$ and any relevant modification factors (K-factors).

$$\tau_{m,adm,ll} = \tau_{m,g,ll} * k_2 * k_3 * k_5 * k_8 \tag{4-6}$$

Where- k_5 - Shear at notched ends (notches are not considered herein therefore, $k_5 = 1$

The shear capacity of the timber panel is calculated as:

$$V_{cap} = \frac{\tau_{m,g,ll} * k_2 * k_3 * k_5 * k_8 * A}{1.5} \tag{4-7}$$

Step-3 **Deflection limits**

In most cases, the combined deflection due to bending Δ_m and shear Δ_s should not exceed 0.003 of the span to satisfy this recommendation.

$$\Delta_m \leq 0.003 J_s$$

Step-4 **Joist spacing**

The joist spacing of the cold formed steel joist is calculated based on the moment capacity, shear capacity, deflection and the load applied on the slab.

$$J_{sm} \leq \sqrt{\frac{\alpha * M_{cap}}{q_d}} \tag{4-8}$$

$$J_{sm} \leq \sqrt{\frac{\alpha * V_{cap}}{q_d}} \tag{4-9}$$

$$J_{sd} \leq \sqrt[3]{\frac{0.003 * \alpha * E * I}{q_d}} \tag{4-10}$$

Where: α - Coefficient based on support condition

$J_s = Min[J_{sm}, J_{sv}, J_{sd}]$ but, $250 \leq J_s \leq 600$

II. Cold formed steel joist

For cold formed steel sections local buckling is critical due to high width to thickness ratio. This effect can be accounted for by using effective cross-sectional properties, calculated on the basis of the effective widths of those elements that are prone to local buckling.

In determining resistance to local buckling the yield strength f_y should be taken as f_{yb}.

Step 1- **geometric proportion check**

$$b/t \leq 60$$
$$c/t \leq 50$$
$$h/t \leq 500$$
$$0.2 \leq c/b \leq 0.6$$
$$r/t \leq 5$$
$$r/b1 \leq 0.1$$

Step 2- **Effective section properties of the flange and lip in compression**

Effective width for a doubly supported compression elements is calculated based on stress ratio. K_σ is read from the table B-5 based on the stress ratio

$$\varepsilon = \sqrt{235 / f_{yb}} \quad \textbf{(4-11)}$$

The relative slenderness:

$$\lambda_{p,b} = \frac{bp1 / t}{28.4 * \varepsilon * \sqrt{K_\sigma}} \quad \textbf{(4-12)}$$

The width reduction factor is:

$$\rho = \frac{\lambda_{p,b} - 0.055(3+\psi)}{\lambda_{p,b}{}^2} \quad \textbf{(4-13)}$$

The effective width is:

$$b_{eff} = \rho * bp1 \quad \textbf{(4-14)}$$

Factors for b_{e1} and b_{e2} is read from the table B-5 or B-6 based on the stress ratio attached on Annex B.

Effective width of the edge fold

If, $c_p / bp1 \leq 0.35$ then $K_\sigma = 0.5$

If $0.35 < c_p / bp1 \leq 0.6$

$$K_\sigma = 0.5 - 0.83 * \sqrt[3]{\left(c_p / bp1 - 0.35 \right)^2} \quad \textbf{(4-15)}$$

$$\lambda_{p,c} = \frac{c_p / t}{28.4 * \varepsilon * \sqrt{K_\sigma}} \quad \textbf{(4-16)}$$

$$\rho = \frac{\lambda_{c,p} - 0.188}{\lambda_{c,b}{}^2} \quad \textbf{(4-17)}$$

The effective width is:

$$c_{eff} = \rho * c_p \quad \textbf{(4-18)}$$

Effective area of the edge stiffener

$$A_s = t(b_{e2} + c_{eff}) \quad \textbf{(4-19)}$$

The elastic critical buckling stress for the edge stiffener

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} \quad \textbf{(4-20)}$$

$$b_1 = b_{p1} - \frac{b_{e2}{}^2 * t/2}{(b_{e2} + c_{eff})t} \quad \textbf{(4-21)}$$

$$K = \frac{Et^3}{4(1-v^2)} * \frac{1}{b_1{}^2 * h_p + b_1{}^3} \quad \textbf{(4-22)}$$

$$I_s = \frac{b_{e2} * t^3}{12} + \frac{c_{eff}{}^3 * t}{12} + b_{e2} * t\left[\frac{c_{eff}{}^2}{2(b_{e2} + c_{eff})}\right]^2 + c_{eff} * t\left[\frac{c_{eff}{}^2}{2} - \frac{c_{eff}{}^2}{2(b_{e2} + c_{eff})}\right]^2 \quad \textbf{(4-23)}$$

$$\lambda_d = \sqrt{f_{yb} / \sigma_{cr,s}} \quad \textbf{(4-24)}$$

Thickness reduction factor χ_d for the edge stiffener

For $\lambda_d \leq 0.65$

$$\chi_d = 1$$

For $0.65 < \lambda_d < 1.38$

$$\chi_d = 1.47 - 0.723\lambda_d$$

For $\lambda_d \geq 1.38$

$$\chi_d = \frac{0.66}{\lambda_d}$$

Iteration is done to refine the value by modifying ρ

$$\sigma_{com,Ed,i} = \chi_d * f_{yb} / \gamma_{mo} \quad \textbf{(4-25)}$$

Effective section properties of the web

The position of the neutral axis with regard to the flange in compression

$$h_c = \frac{c_p(h_p - c_p/2) + b_{p2} * h_p + h_p^2/2 + c_{eff}^2 * \chi_d/2}{c_p + b_{p2} + h_p + b_{e1} + (b_{e2} + c_{eff})\chi_d} \quad \textbf{(4-26)}$$

The stress ratio:

$$\psi = \frac{h_c - h_p}{h_c} \quad \textbf{(4-27)}$$

K_σ Is read from the table B- 5 based on the stress ratio attached on annex.

The relative slenderness:

$$\lambda_{p,h} = \frac{h_p/t}{28.4 * \sqrt{235/f_{yb}} * \sqrt{K_\sigma}} \quad \textbf{(4-28)}$$

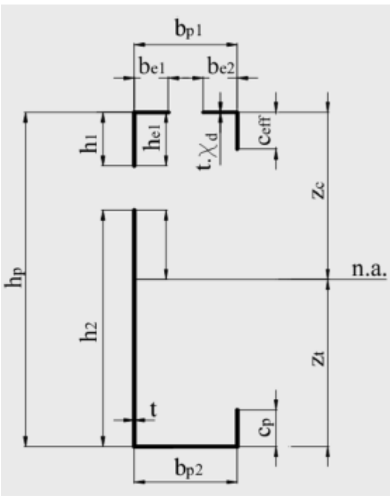
$$\rho = \frac{\lambda_{p,b} - 0.055(3+\psi)}{\lambda_{p,b}^2} \quad \textbf{(4-29)}$$

The effective width of the zone in compression of the web is:

$$h_{eff} = \rho * h_c \quad \textbf{(4-30)}$$

Factors for h_{e1} and h_{e2} are read from table B-5 or B-6 based on the stress ratio attached on Annex B.

Effective section property



Effective section area

$$A_{eff} = t\left[c_p + b_{p2} + h_1 + h_2 + b_{e1} + (b_{e2} + c_{eff})\chi_d\right] \tag{4-31}$$

Position of the neutral axis with regard to the flange in compression:

$$Z_c = \frac{t\left[c_p(h_p - {c_p}/{2}) + b_{p2} * h_p + {h_1^{\,2}}/{2} + h_2(h_p - {h_2}/{2}) + ({c_{eff}}^2 \chi_d/2)\right]}{A_{eff}} \tag{4-32}$$

Position of the neutral axis with regard to the flange in tension:

$$Z_t = h_p - Z_c \tag{4-33}$$

Second moment of area:

$$\begin{aligned} I_{eff,y} = {} & \frac{h_1^{\,3}t}{12} + \frac{h_2^{\,3}t}{12} + \frac{b_{p2}t^3}{12} + \frac{c_p^{\,3}t}{12} + \frac{b_{e2}t^3}{12} + \frac{b_{e2}(\chi_d t)^3}{12} + \frac{c_{eff}^{\,3}(\chi_d t)}{12} + \\ & c_p t(Z_t - {c_p}/{2})^2 + b_{p2}tZ_t^{\,2} + h_2 t(Z_t - h_2/2)^2 + h_1 t(Z_c - h_1/2)^2 + b_{e1}tZ_c^{\,2} + \\ & b_{e2}(\chi_d t)Z_c^{\,2} + c_{eff}(\chi_d t)(Z_c - {c_{eff}}/{2})^2 \end{aligned} \tag{4-34}$$

Effective section modulus:

- With regard to the flange in compression

$$W_{eff,y,c} = \frac{I_{eff,y}}{Z_c} \tag{4-35}$$

- With regard to the flange in tension

$$W_{eff,y,t} = \frac{I_{eff,y}}{Z_t} \tag{4-36}$$

$$W_{eff,y} = \min(W_{eff,y,c}, W_{eff,y,t}) \tag{4-37}$$

Check bending resistance at ULS

$$M_{c,Rd} = {W_{eff,y} f_{yb}}/{\gamma_{mo}} \tag{4-38}$$

Check of shear resistance at ULS

Design shear buckling resistance

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi} t f_{bv}}{r_{mo}} \quad \textbf{(4-39)}$$

$$\lambda_w = \frac{0.346 * hp}{t}\sqrt{\frac{f_{yb}}{E}} \quad \textbf{(4-40)}$$

Based on λ_w, f_{bv} is read from table B-7 attached on Annex B

Check of local transverse resistance at ULS

The following criteria should be satisfied

$$h_w/t \le 200$$
$$r/t \le 6$$
$$45° \le \phi \le 90°$$

The local transverse resistance of the web

For unstiffened single web, which is the case considered on this paper the local transversal resistance of the section can be determined by:

For $S_s / t \le 60$

$$R_{w,Rd} = \frac{k_1 k_2 k_3 [5.92 - \frac{h_w / t}{132}][1 + 0.01 S_s / t] t^2 f_{yb}}{\gamma_{m1}} \quad \textbf{(4-41)}$$

For $S_s / t > 60$

$$R_{w,Rd} = \frac{k_1 k_2 k_3 [5.92 - \frac{h_w / t}{132}][0.71 + 0.015 S_s / t] t^2 f_{yb}}{\gamma_{m1}} \quad \textbf{(4-42)}$$

Where: $$\begin{aligned} k &= f_{yb}/228 \\ k_1 &= 1.33 - 0.33k \\ k_2 &= 1.15 - 0.15r/t \\ k_3 &= 0.7 + 0.3(\phi/90)^2 \end{aligned}$$

Buckling strength

Lateral torsional buckling

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{2} - \lambda_{LT}^{2}}} \text{ , but } \chi_{LT} \leq 1 \qquad \textbf{(4-43)}$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^{2}] \qquad \textbf{(4-44)}$$

$\alpha_{LT} = 0.34$ -buckling curve b

$$\lambda_{LT} = \sqrt{\frac{W_{eff,Y} * f_{yb}}{M_{cr}}} \qquad \textbf{(4-45)}$$

$$M_{cr} = \frac{1.127 * \pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_Z} + \frac{L^2 GI_t}{\pi^2 EI_z}} \qquad \textbf{(4-46)}$$

$$M_{b,Rd} = \chi_{LT} W_{eff,y} f_{yb} / \gamma_{m1} \qquad \textbf{(4-47)}$$

III. Transversal/Main beam

Step-1 **Classification of cross section**

Out stand flange

$$\frac{C}{tf} \leq \beta * \varepsilon \qquad \textbf{(4-48)}$$

Where, $\beta = 10$, $\beta = 11$, $\beta = 15$, for class 1,2 and 3 respectively.

Webs

$$\frac{d}{t_w} \leq \beta * \varepsilon \qquad \textbf{(4-49)}$$

Where, $\beta = 72$, $\beta = 83$, $\beta = 124$, for class 1,2 and 3 respectively

Step-2 **Member resistance**

Bending resistance

Section modulus depending up on their class is:

$$W_{pl,y} - class1 \,\&\, class2$$
$$W_{el,y} - class3$$
$$W_{eff,y} - class4$$

$$M_{c,Rd} = W_y f_y / \gamma_{m0} \quad \textbf{(4-50)}$$

Lateral torsional buckling resistance

Follow the same step from section ii used to check the lateral torsional buckling resistance.

Shear resistance

$$V_{c,Rd} = V_{pl,Rd} = A_v * \left(fy / \sqrt{3} \right) / \gamma_{m1} \quad \textbf{(4-51)}$$

The flow charts of the deign procedure to be followed are shown on the next chapter.

Chapter 5 COMPUTER PROGRAM FOR DESIGN OF COMPOSITE TIMBER- COLD FORMED STEEL JOIST SLAB

Input Data:

The input data consists of: Slab size, OSB/Plywood strength and density, steel strength for CFS and IPE sections, the modulus of elasticity of steel, web height, flange width, lip length and thickness of the CFS section are also inputted if user option is selected.

Moreover, input data includes live load and super imposed load depending on the use of the slab.

Analysis:

Having material properties, slab size and loads imposed on the slab as an input data, the joist spacing is calculated using equation (4-8) to (4-10). In addition to this, the following are computed:

- The moment capacity, shear capacity, local transversal load resistance and buckling moment resistance of CFS section are calculated using equation (4-38), (4-39), (4-41),(4-42) and (4-47).
- The moment capacity and shear capacity of transversal and main beam using equation (4-50) and (4-51).

Designing:

The light weight slab is designed for the critical moments obtained from the analysis and the sufficient sections which can resist the applied load are selected from the program or the inputted section by the user is checked if its sufficient or not.

Total amount of materials used

After the sufficient section is selected, the total amount of materials used are obtained by determining the number of joists needed and their adjacent weights and weights of the OSB or plywood and the transversal beam if any and main beam for the purpose of making the design economical.

The above algorithm is summarized in flow charts shown on Figure 5.1 to Figure 5.5.

5.1 Flow Chart for Timber Design

The cross sectional properties should satisfy elastic strength and service load requirement. For a medium span bending is the most critical criteria whereas for long span deflection governs the design and shear of heavily loaded short span members. All this criteria should be checked on design.

Timber has a different material property in two main directions parallel and perpendicular to the grain. Even though the normal stresses due to bending are parallel to grain direction, support conditions may impose stresses that are perpendicular to the grain direction. In addition to the primary stresses these stresses should be checked in the design against the permissible values.

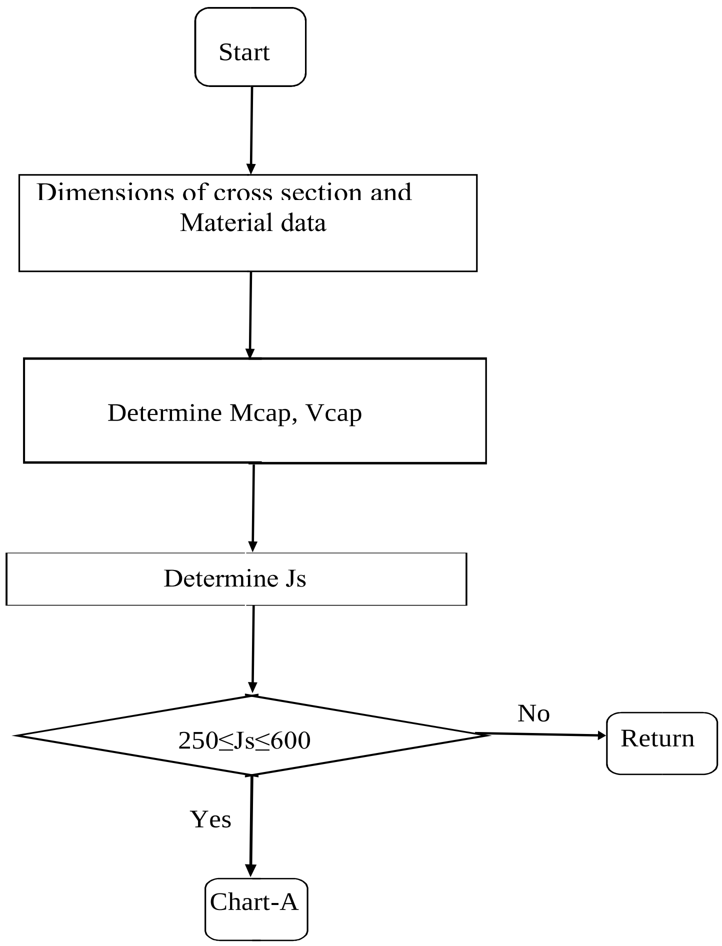


Figure 5-1 Flow chart for OSB/Plywood

5.2 Flow Chart for Cold-Formed Steel Joists

In the design of cold-formed steel flexural members consideration should first be given to the moment-resisting capacity and the stiffness of the member. Second the webs of beams should be checked for shear, combined bending and shear, web crippling, and combined bending and web crippling.

In addition to the design features discussed above, the moment-resisting capacity of the member may be limited by lateral-torsional buckling of the beam, particularly when the open section is fabricated from thin material and laterally supported at relatively large intervals. For this reason adequate bracing should be provided.

Unlike hot-rolled heavy steel sections, in the design of thin-walled cold-formed steel beams, special problems such as shear lag and flange curling are also considered to be important matters due to the use of thin material. Furthermore, the design of flexural members can be even more involved if the increase of steel mechanical properties due to cold work is to be utilized.

Design feature

1. Bending strength and deflection
2. Design of webs for shear, combined bending and shear, web crippling
3. Lateral torsional buckling

Chart A

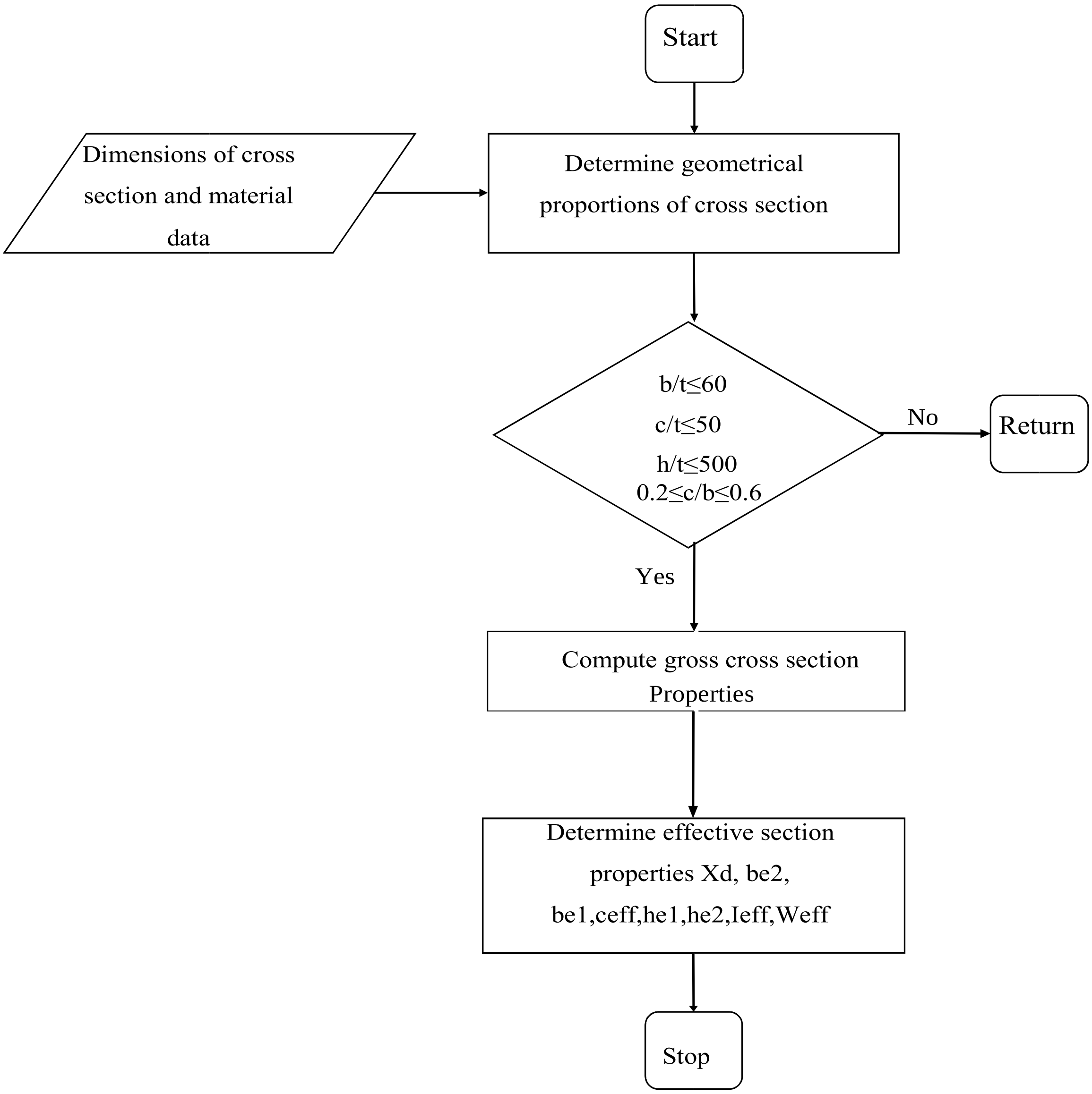


Figure 5-2 Flow chart for cold formed steel joist

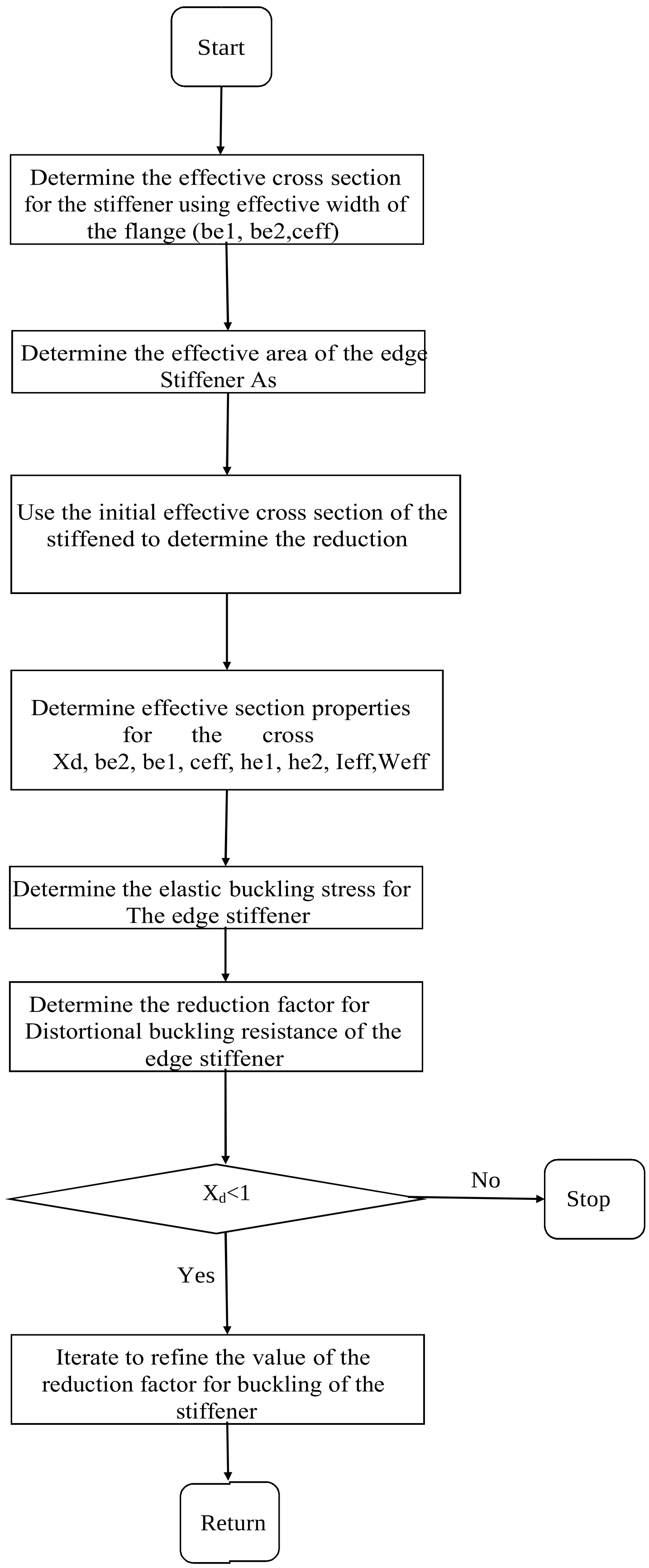


Figure 5-3 Flow chart for cold formed steel joist

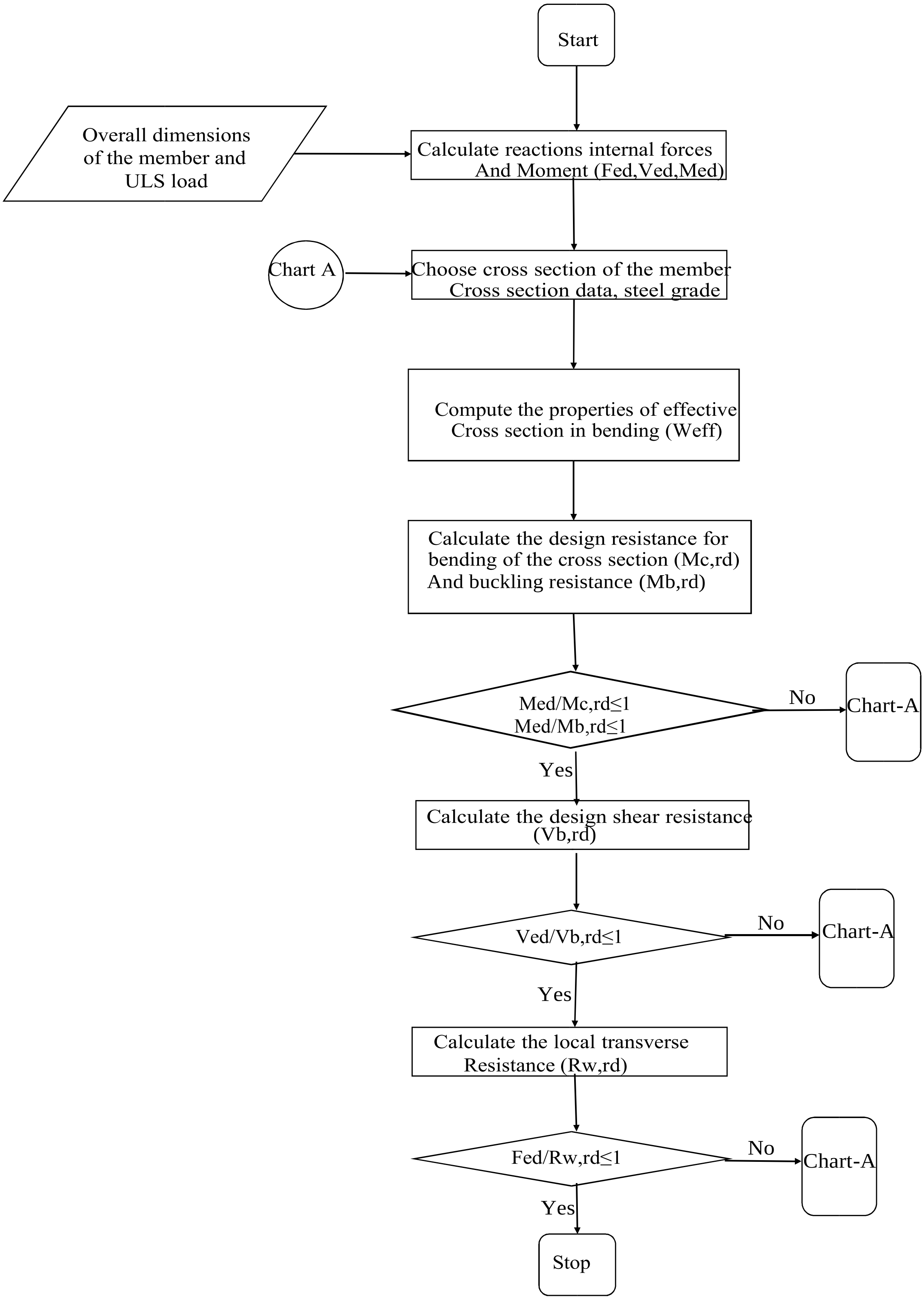
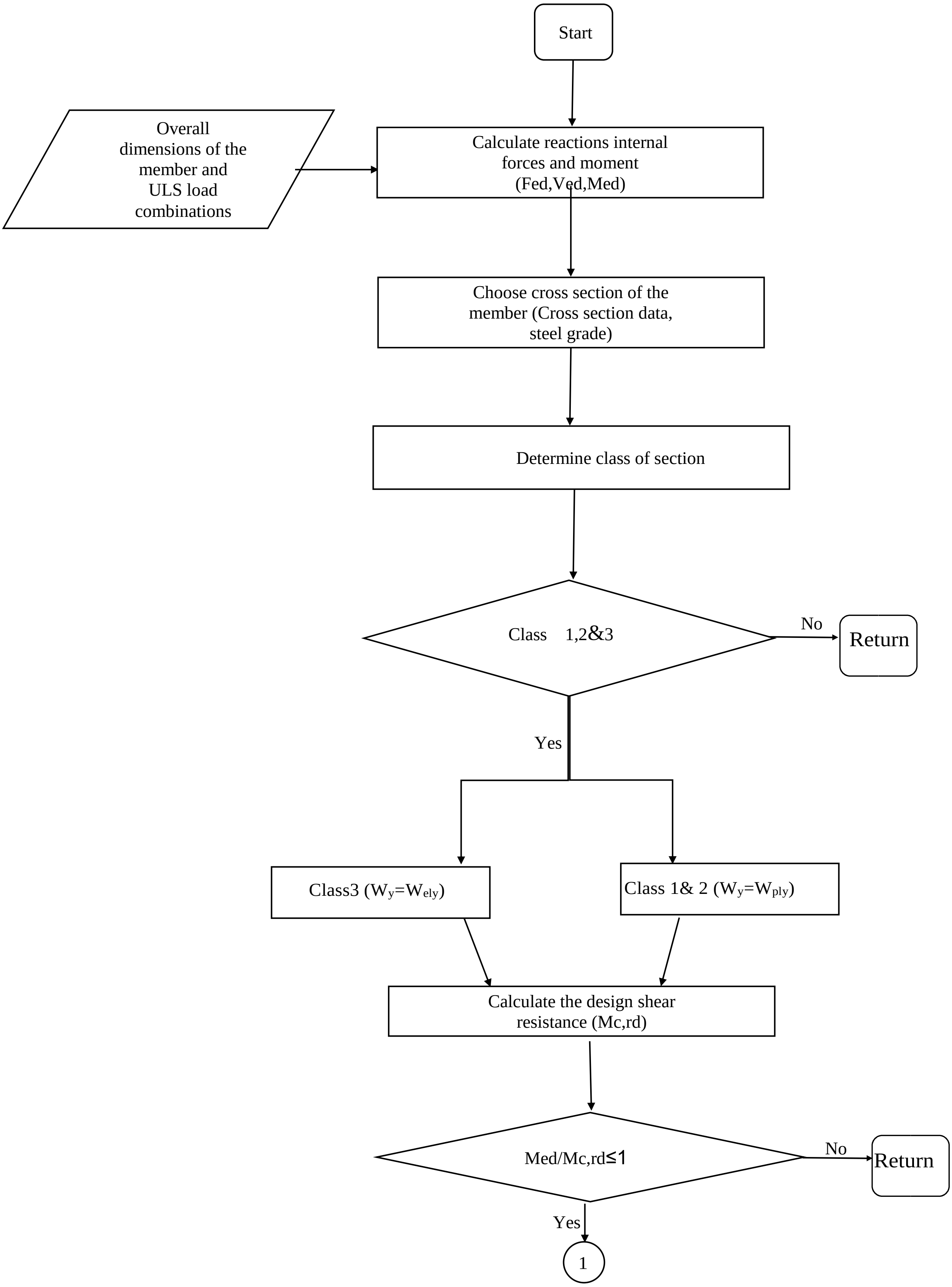


Figure 5-4 Flow Chart for cold formed steel joist

5.3 Flow Chart for transversal and main beam



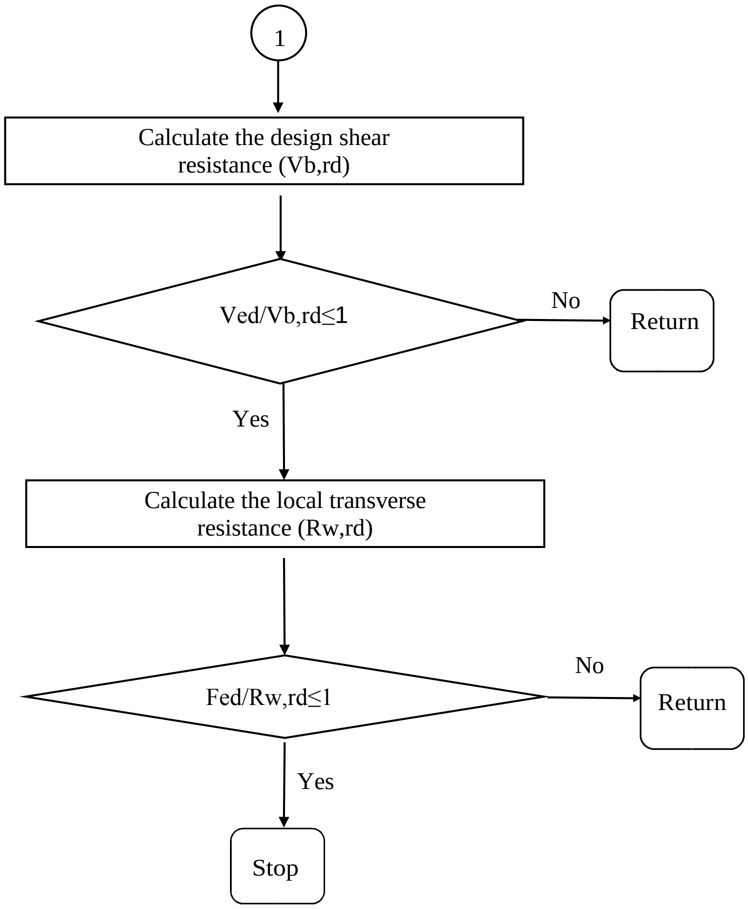


Figure 5-5 Flow Chart for transversal and main beam

5.4 Limitations

The developed program has the following limitations:

- It is developed for plain channel CFS sections.
- The database for the capacity of OSB sheathing is according to OSB design manual [3].
- The type of steel sections for transversal and main beam on the database is group of IPE sections according to the European standardization.
- The database for the grade of steel for CFS is stated on Annex B
- Connection design is not included here in.
- The composite action effect is not considered.

5.5 Graphical User Interface

A friendly graphical user interface is incorporated to advance the practice of the program. The procedure to follow while using this program is described below

5.5.1 Program procedure

The program designs a composite floor system (OSB/Plywood with cold formed floor joist). As shown on the interface the designer inputs:

Step 1 (general)

- Slab size
- Live load
- Super imposed load

Step 2 (OSB/Plywood)

I. Default (OSB data provided)
II. User
    - Bending stress parallel to the grain
    - Shear stress parallel to the grain

- Modulus of elasticity
- Load duration(k_3)
- Density

Step 3 (CFS Joist)

- Yield strength (f_y)
- Modulus of elasticity (E)
- Poisson's ratio (v)

I. Default (fed from database)

II. User

- Height of web (H)
- Width of flange (B)
- Lip length (C)
- Thickness (t)
- Corner radius (r)

Step 4 (Transversal and main beam)

- Yield strength (f_y)
- Modulus of elasticity (E)
- Poisson's ratio (v)

Transversal beam is added by checking if the out put is economical or not and comparing different results by thier weight

The designing programe is programed on Microsoft Visual Studio C#. consisting seven classes each for a specific purpose. The code with the classes is attached on Annex C.

The user interfaces for the design of light weight slab are shown on Figure.

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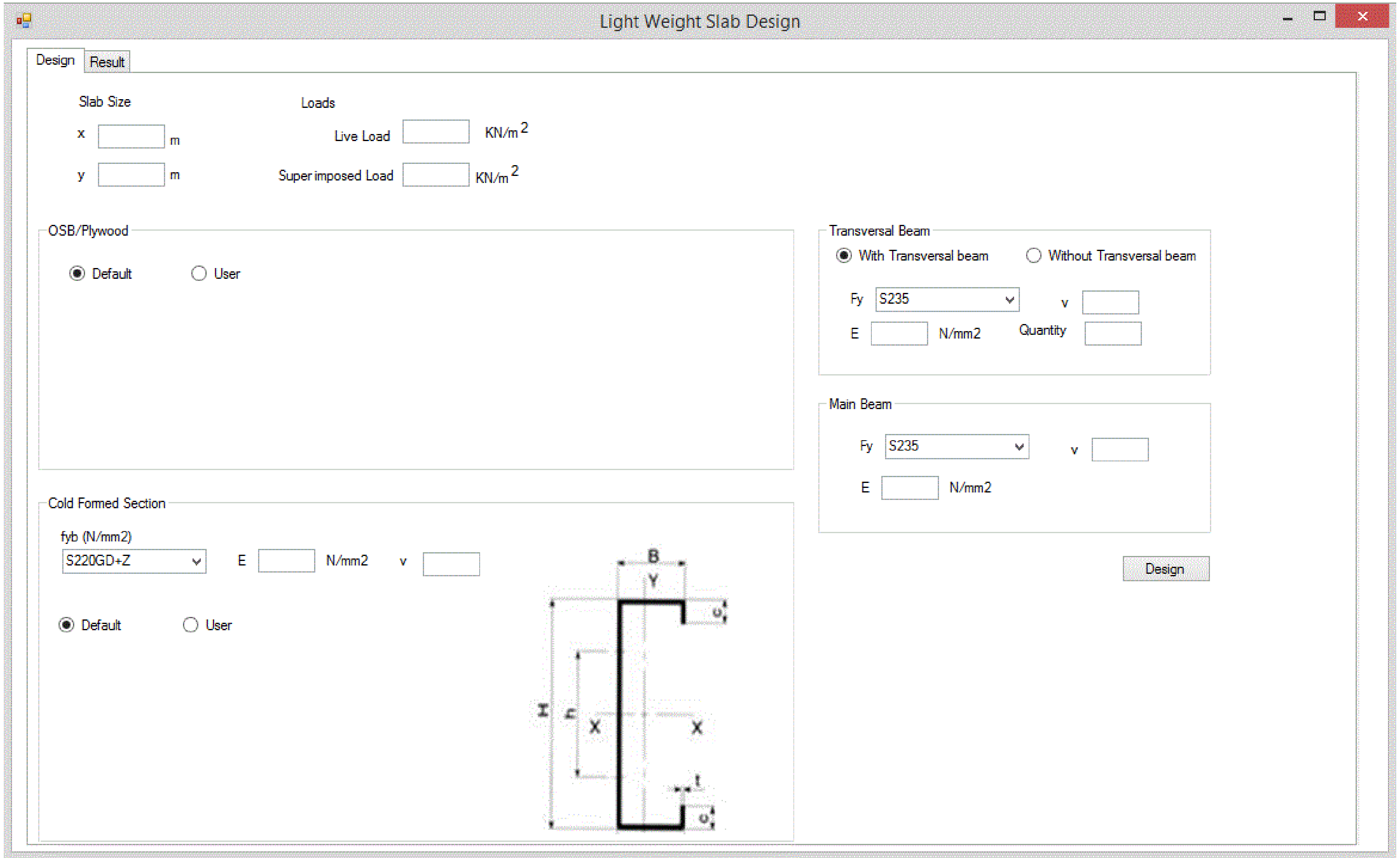

Figure 5-6 Design interface of program

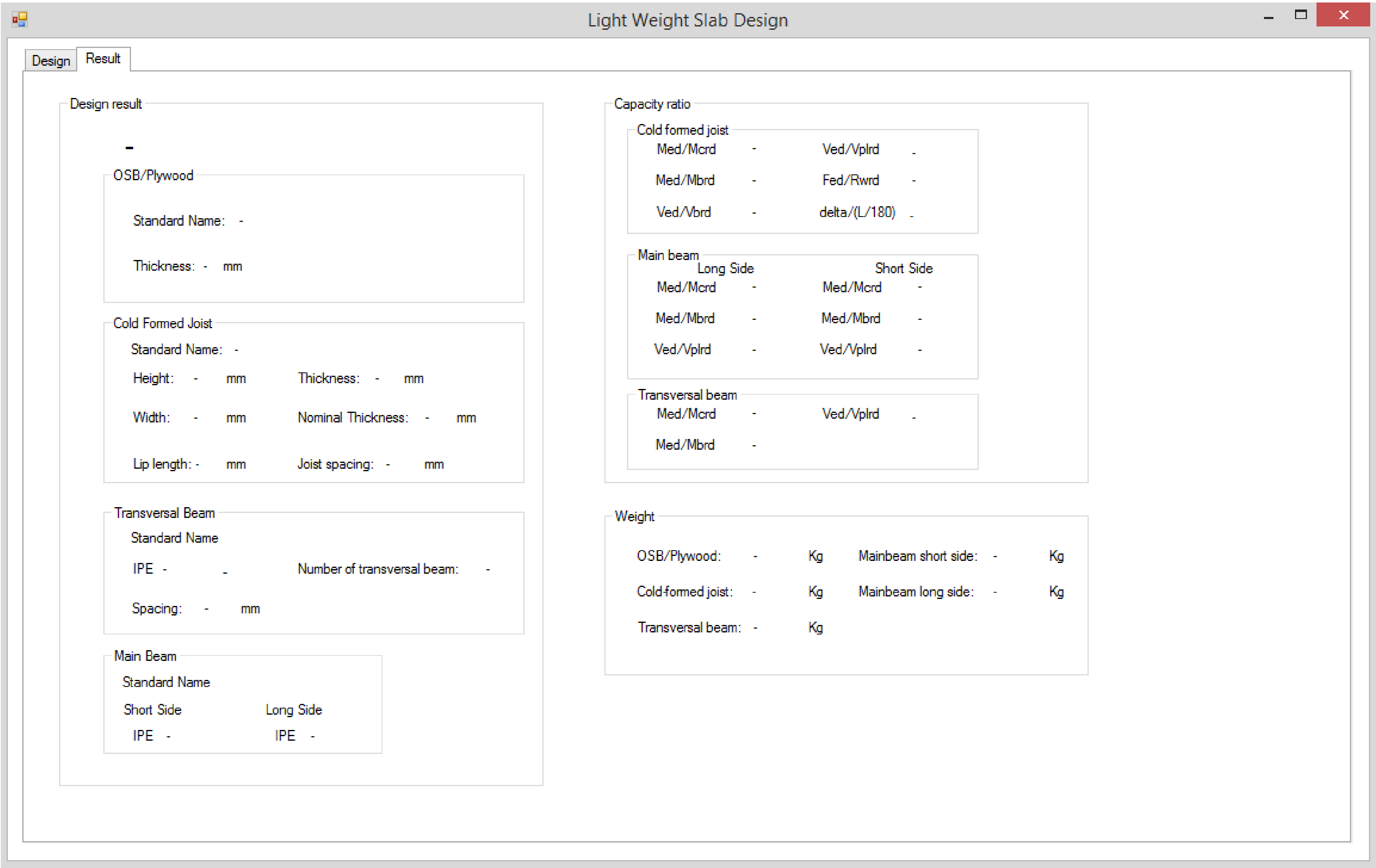

Figure 5-7 Result interface of program

5.6 Design Example

Design of a slab 1 m x 2 m using the slab system proposed on this paper.

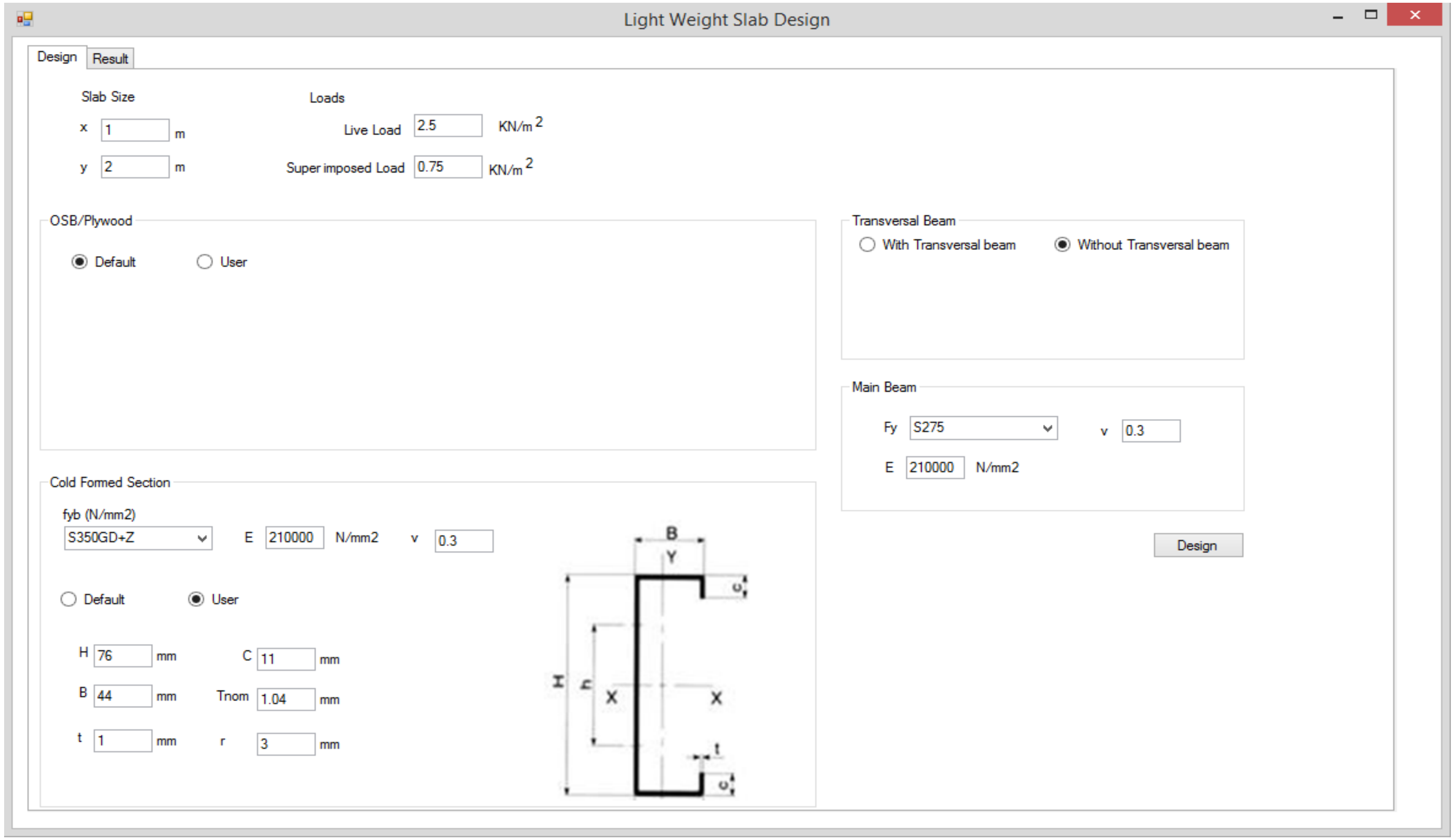

Figure 5-8 Input data

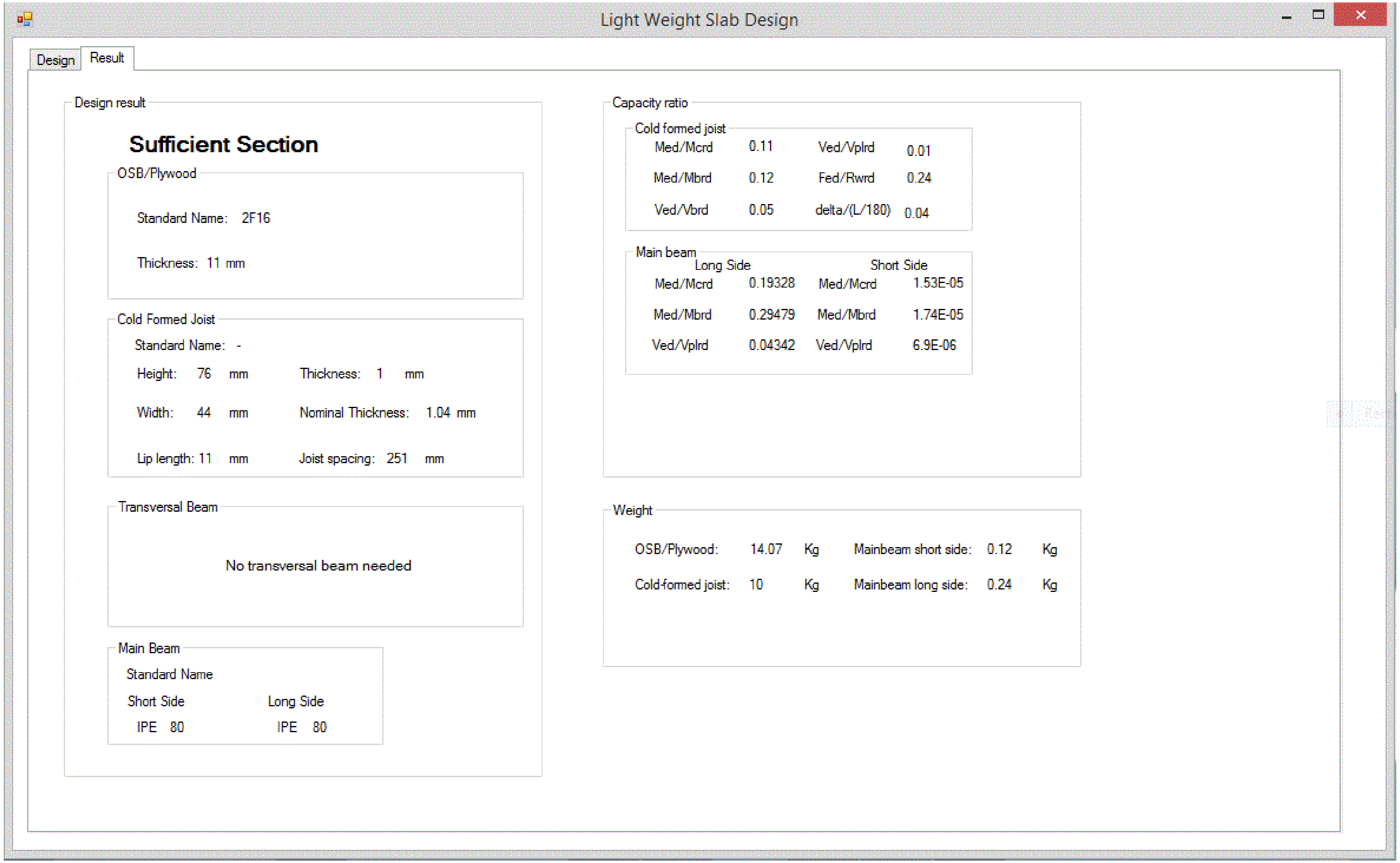

Figure 5-9 Output result

5.7 Verification of computer program

The program is verified analytically. The results of the program is compared with results from the analytic calculation on table 5-1 and Table 5-2 below and the capacity ratios are compared using graphs shown below on Figure 10, Figure 11 and Figure 12. Which shows the results are at close proximity which verifies the program.

Table 5-1 CFS Floor Joist result comparison

	Analytic Result	Program Result
Med	0.15423	0.1547
V_{ed}	0.617	0.6188
F_{ed}	0.617	0.6188
M_{crd}	1.389	1.3897
M_{brd}	1.272	1.28669
V_{brd}	11.894	11.893
R_{wrd}	2.547	2.548

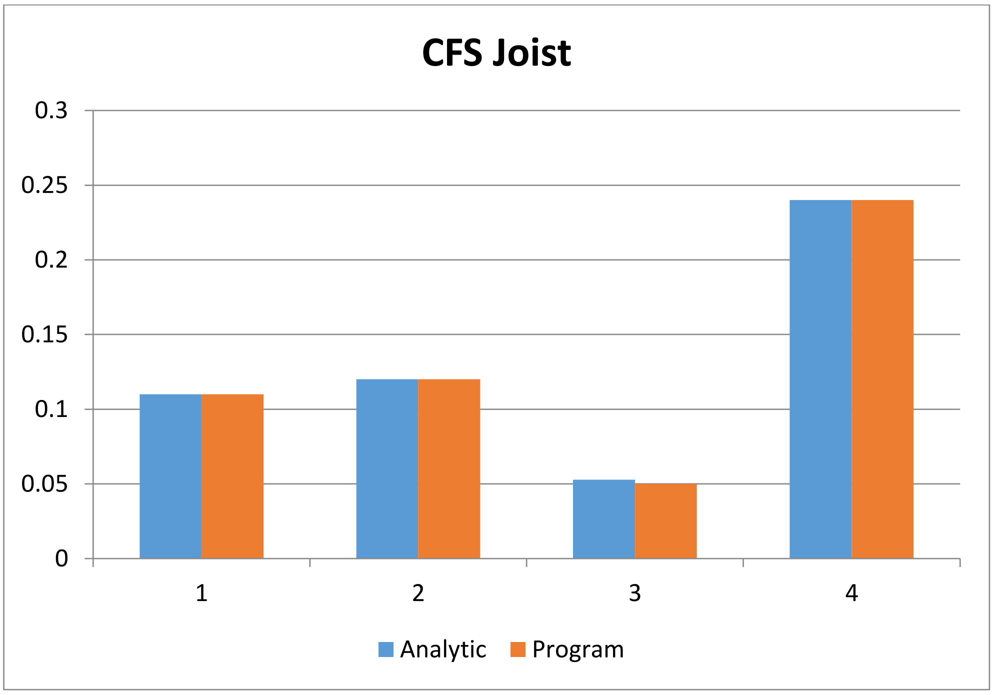

Figure 5-10 Capacity ratio comparison of CFS floor joist

Table 5-2 Main beam result comparison

	Analytic Result	Program Result	Analytic Result	Program Result
	Long side		Short side	
Med	1.2344	1.233	0.000099	0.000097
Ved	2.4688	2.466	0.0003973	0.000389
Mcrd	6.38	6.38	6.38	6.38
Mbrd	4.07	4.183	5.544	5.5898
Vplrd	56.738	56.7988	56.738	56.788

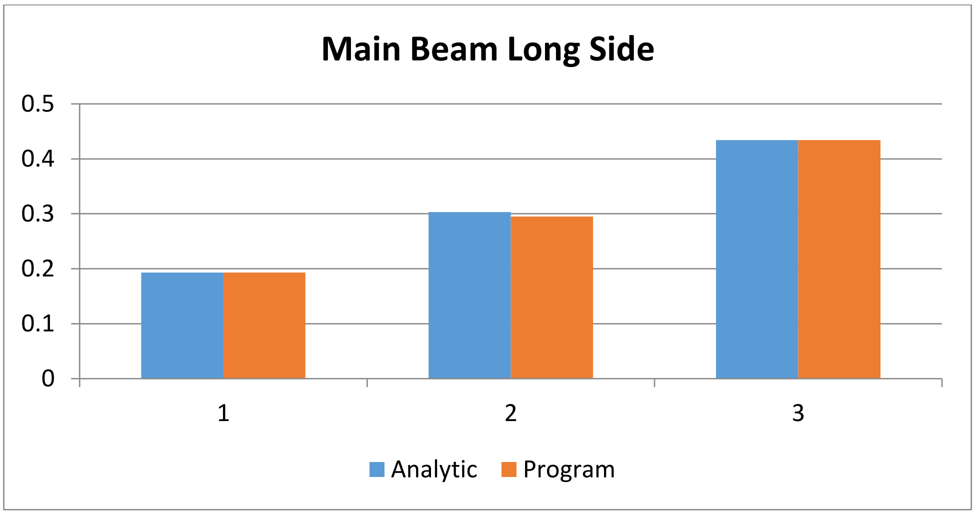

Figure 5-11 Capacity ratio comparison of main beam short side

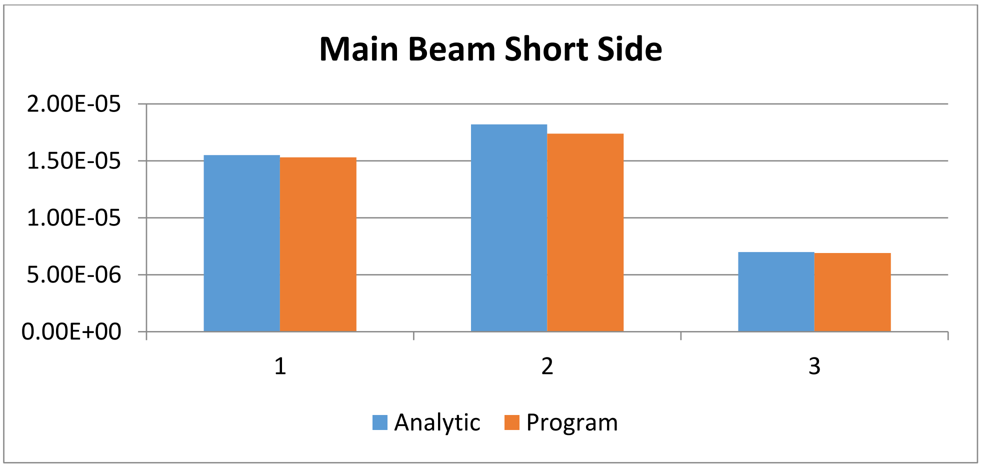

Figure 5-12 Capacity ratio comparison of main beam long side

Chapter 6 CONCLUSION AND RECOMMENDATION

6.1 Conclusion

This research has addressed the local buckling effect on CFS sections. Furthermore, provided an analysis and designing program for the proposed slab system, which is a composite timber CFS floor joist slab. Channel sections were only addressed on this study.

Base on this study the following conclusions are drawn

- Even though most CFS sections are prone to local buckling, there are cases where the section yields before failure. Therefore all CFS sections are not class four sections as assumed on Eurocode 3 Part 1-3 which results in a conservative and uneconomical design.
- A computer program for the analysis and design of composite timber CFS floor joist slab is developed.

6.2 Recommendation

Based on this study all CFS sections are not class 4 and the classification for hot rolled sections cannot be used to classify CFS sections as discussed on chapter 3. Therefore, recommendation is made on reviewing the classification of CFS sections on the code for more efficient and economical design of CFS sections to gain the full advantages these sections uphold.

Since the popularity of CFS sections is growing rapidly, the knowhow of how these sections behave is important. For the efficient use of these sections as a primary load bearing elements further research can be done on:

- The effect of composite action of timber with CFS joists and connection design
- Update or modify program for other section types and arrangement of joist since the program is at our disposal.
- Investigate the flexural behavior of built-up CFS sections, stiffened CFS sections and Z sections joists.

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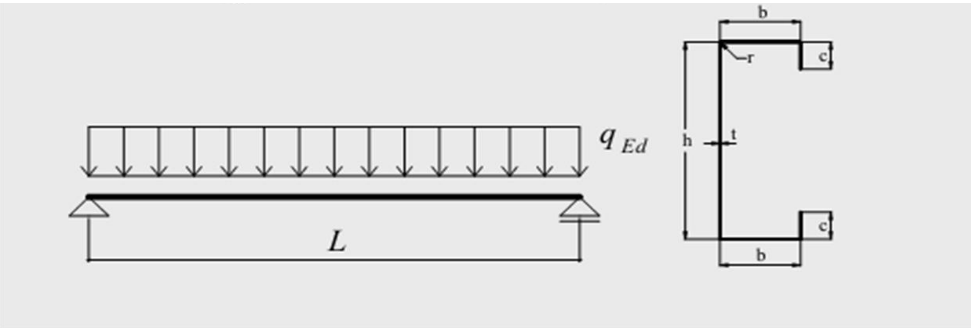
ANNEX-A
Worked Example

Worked example

Design a slab 1x2 m using the slab system proposed on this paper.



Given

Span of joist=1m

Super imposed load= $0.75\, {KN}/{m^2}$

Self-weight of joist= $0.014028 {KN}/{m}$

Self-weight of OSB sheeting= $0.069 {KN}/{m^2}$

Imposed load= $2.5 {KN}/{m^2}$

Basic Data of cold formed steel floor joist

The dimensions of the trial cross section and the material properties:

Total height h=76mm

Total width of flange in compression b_1 =44mm

Total width of flange in tension b_2 =44mm^2

Total width of edge fold c=11mm

Internal radius r =3mm

Thickness t=1mm

Nominal thickness t_{nom} =1.04

Basic yield strength $f_{yb} = 350 \, {}^{N}\!/_{mm^2}$

Modulus of elasticity $E = 210000 \, {}^{N}\!/_{mm^2}$

Poisons ratio $v = 0.3$

Partial factors $r_{Mo} = 1.00$

Solution1-OSB sheeting design

Applied loading on the joist at the ULS

OSB panel is designed per meter width

$$q_G = (q_{Gosb} + si) = (0.069 \, {}^{KN}\!/_{m^2} + 0.75 \, {}^{KN}\!/_{m^2})1m = 0.8190 \, {}^{KN}\!/_{m}$$

$$q_Q = 2.5 \, {}^{KN}\!/_{m}$$

$$q_d = \gamma_G q_G + \gamma_Q q_Q = 1.35 * 0.819 + 1.5 * 2.5 = 4.8556 \, {}^{KN}\!/_{m}$$

From OSB design manual- the capacities of an OSB panel selected are

2F16 OSB panel

Thickness 11 mm

Self-weight $0.069 \, {}^{KN}\!/_{m^2}$

Moment capacity $65 \, {}^{Nmm}\!/_{mm}$

Shear capacity $2.3 \,{}^{N}\!/\!_{mm}$

Bending stiffness $140000 \,{}^{Nmm^2}\!/\!_{mm}$

Consider 1m OSB panel with 4 supports

$$Js = \sqrt{\frac{M_{cap} * 1000mm}{(9/128) * q_d}} = \sqrt{\frac{65*1000mm}{0.07*4.8556}} = 436mm$$

$$J_s \leq \frac{\alpha * V_{cap}}{q_d * \alpha} = \frac{2.3*1000}{0.625*4.8556} = 757.89mm$$

$$J_s \leq \sqrt[3]{\frac{0.003*E*I}{q_d * \alpha}} = \sqrt[3]{\frac{0.003*140000*1000}{4.8556*(1/185)}} = 251.99mm$$

Taking the minimum J_s for the purpose of panel to CFS connection 250mm is taken as the joist spacing.

The selected OSB is sufficient with a joist spacing of 250 mm.

Solution2-cold formed joist design

Applied loading on the joist at the ULS

$$q_G = q_{Gjoist} + (q_{Gosb} + si) * j_s = 0.014028 + (0.069 + 0.75) * 0.25 = 0.2188 \,{}^{KN}\!/\!_{m}$$

$$q_d = \gamma_G q_G + (\gamma_Q q_Q) * j_s = 1.35 * 0.2188 + (1.5 * 2.5 * 0.25) = 1.2343 \,{}^{KN}\!/\!_{m}$$

Resistance of cross section

Web height $hp = h - tnom = 76 - 1.04 = 74.96mm$

Width of flange in compression and tension ($b_1 = b_2$)
$bp1 = bp2 = b - tnom = 44 - 1.04 = 42.96mm$

Width of edge fold $cp = C - tnom = 11 - (1.04/2) = 10.48mm$

Checking of geometric proportions

$$b/t \leq 60$$
$$b/t = 44/1 = 44 < 60 - ok$$
$$c/t \leq 50$$
$$c/t = 11/1 = 11 < 50 - ok$$
$$h/t \leq 500$$
$$h/t = 76/1 = 76 < 500 - ok$$

$$0.2 \leq c/b \leq 0.6$$
$$c/b = 11/44 = 0.25 < 0.6 - ok$$

The influence of rounding of the corners is neglected if

$$r/t \leq 5$$
$$r/t = 3/1 = 3 < 5 - ok$$
$$r/b1 \leq 0.1$$
$$r/b1 = 3/44 = 0.0682 < 0.1 - ok$$

Effective section properties of the flange and lip in compression

The stress ratio: $\psi = 1$ for uniform compression

$K_\sigma = 4$ for internal compression element (table source)

$$\varepsilon = \sqrt{235/f_{yb}}$$
$$\varepsilon = \sqrt{235/350} = 0.819$$

The relative slenderness:

$$\lambda_{p,b} = \frac{bp1/t}{28.4 * \varepsilon * \sqrt{K_\sigma}}$$

$$\lambda_{p,b} = \frac{42.96/1}{28.4 * 0.819 * \sqrt{4}} = 0.9234$$

The width reduction factor is:

$$\rho = \frac{\lambda_{p,b} - 0.055(3+\psi)}{\lambda_{p,b}{}^2} = \frac{0.9234 - 0.055(3+1)}{0.9234^2} = 0.825 < 1$$

The effective width is:

$$b_{eff} = \rho * bp1 = 0.825 * 42.96 = 35.442mm$$
$$b_{e1} = b_{e2} = 0.5 * b_{eff} = 17.72mm$$

Effective width of the edge fold

$$c_p / bp1 = 10.48/42.96 = 0.2439$$

If $c_p / bp1 \leq 0.35$, then $K_\sigma = 0.5$

$$\lambda_{p,c} = \frac{c_p/t}{28.4 * \varepsilon * \sqrt{K_\sigma}}$$

$$\lambda_{p,c} = \frac{10.48/1}{28.4 * 0.819 * \sqrt{0.5}} = 0.6372$$

$$\rho = \frac{\lambda_{c,p} - 0.188}{\lambda_{c,b}{}^2} = \frac{0.6372 - 0.188}{0.6372^2} = 1.106 < 1$$

$$\rho = 1$$

The effective width is: $c_{eff} = \rho * c_p = 1 * 10.48 = 10.48$

Effective area of the edge stiffener

$$A_s = t(b_{e2} + c_{eff}) = 1(17.72 + 10.48) = 28.2mm^2$$

The elastic critical buckling stress for the edge stiffener

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s}$$

$$b_1 = bp1 - \frac{b_{e2}{}^2 * t/2}{(b_{e2} + c_{eff})t} = 42.96 - \frac{17.72^2 * 1/2}{(17.72 + 10.48)1} = 37.39mm$$

$$K = \frac{Et^3}{4(1-v^2)} * \frac{1}{b_1{}^2 * h_p + b_1{}^3} = \frac{210000*1^3}{4(1-0.3^2)} * \frac{1}{37.39^2 * 74.96 + 37.39^3} = 0.3673 N/mm^2$$

$$I_s = \frac{b_{e2} * t^3}{12} + \frac{c_{eff}{}^3 * t}{12} + b_{e2} * t\left[\frac{c_{eff}{}^2}{2(b_{e2} + c_{eff})}\right]^2 + c_{eff} * t\left[\frac{c_{eff}{}^2}{2} - \frac{c_{eff}{}^2}{2(b_{e2} + c_{eff})}\right]^2$$

$$= \frac{17.72*1^3}{12} + \frac{10.48^3 *1}{12} + 17.72*1\left[\frac{10.48^2}{2(17.72 + 10.48)}\right]^2 + 10.48*1*$$

$$\left[\frac{10.48^2}{2} - \frac{10.48^2}{2(17.72 + 10.48)}\right]^2 = 278.212mm^4$$

$$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s} = \frac{2\sqrt{0.3673*210000*278.212}}{28.2} = 328.54 N/mm^2$$

$$\lambda_d = \sqrt{f_{yb}/\sigma_{cr,s}} = \sqrt{350/328.54} = 1.032$$

Thickness reduction factor χ_d for the edge stiffener

For $0.65 \le \lambda_d \le 1.38$

$$\chi_d = 1.47 - 0.723\lambda_d = 1.47 - (0.723*1.032) = 0.7238$$

Iterate to refine the value by modifying ρ

$$\sigma_{com,Ed,i} = \chi_d * f_{yb}/\gamma_{mo}$$

Iteration 1 initial values

$$\chi_d = 0.7238$$
$$b_{e2} = 17.72mm$$
$$c_{eff} = 10.48mm$$

Iteration 2

$$\chi_d = 0.71328$$
$$b_{e2} = 19.691mm$$
$$c_{eff} = 10.48mm$$

Iteration n final values

$$\chi_d = 0.7128$$
$$b_{e2} = 19.782mm$$
$$c_{eff} = 10.48mm$$

Final values of effective properties for flange and lip in compression are:

$$b_{e1} = 17.72mm$$
$$b_{e2} = 19.782mm$$
$$c_{eff} = 10.48mm$$
$$t_{red} = 1*0.7128 = 0.7128mm$$

Effective section properties of the web

The position of the neutral axis with regard to the flange in compression

$$h_c = \frac{c_p(h_p - c_p/2) + b_{p2}*h_p + h_p^2/2 + c_{eff}^2*\chi_d/2}{c_p + b_{p2} + h_p + b_{e1} + (b_{e2} + c_{eff})\chi_d} = 40.55mm$$

The stress ratio:

$$\psi = \frac{h_c - h_p}{h_c} = -0.8486$$

For $-1 < \psi < 0$

$$K_\sigma = 7.81 - 6.29\psi + 9.78\psi^2 = 20.19$$

The relative slenderness:

$$\lambda_{p,h} = \frac{h_p/t}{28.4 * \sqrt{235/f_{yb}} * \sqrt{K_\sigma}} = 0.7169$$

$$\rho = \frac{\lambda_{p,b} - 0.055(3+\psi)}{\lambda_{p,b}^{\ 2}} = \frac{0.7169 - 0.055(3 - 0.8486)}{0.7169^2} = 1.16466 < 1$$

The effective width of the zone in compression of the web is:

$$h_{eff} = \rho * h_c = 40.55mm$$

For ψ <0

$$h_{e1} = 0.4 * h_{eff} = 16.22mm$$

Near the neutral axis:

$$h_{e2} = 0.6 * h_{eff} = 24.33mm$$

The effective width of the web is:

Near the flange in compression:

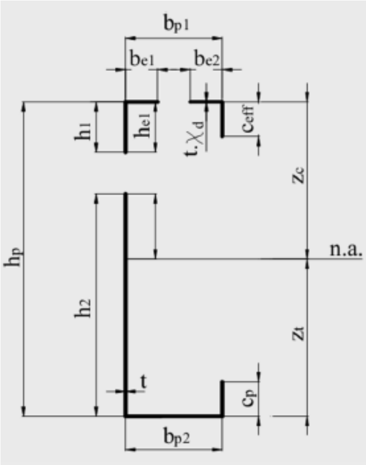
$$h_1 = h_{e1} == 16.22mm$$

Near the flange in tension:

$$h_2 = h_p - (h_c - h_{e2}) = 58.74mm$$

Effective section property

Effective cross section area:



$$A_{eff} = t\left[c_p + b_{p2} + h_1 + h_2 + b_{e1} + (b_{e2} + c_{eff})\chi_d\right] = 167.69mm^2$$

Position of the neutral axis with regard to the flange in compression:

$$Z_c = \frac{t\left[c_p(h_p - {c_p}/{2}) + b_{p2} * h_p + {h_1^{\,2}}/{2} + h_2(h_p - {h_2}/{2}) + ({c_{eff}}^2\,{\chi_d}/{2})\right]}{A_{eff}} = 40.561mm$$

Position of the neutral axis with regard to the flange in tension:

$$Z_t = h_p - Z_c = 34.3699mm$$

Second moment of area:

$$I_{eff,y} = \frac{h_1^{\,3}t}{12} + \frac{h_2^{\,3}t}{12} + \frac{b_{p2}t^3}{12} + \frac{c_p^{\,3}t}{12} + \frac{b_{e2}t^3}{12} + \frac{b_{e2}(\chi_d t)^3}{12} + \frac{c_{eff}^{\,3}(\chi_d t)}{12} +$$
$$c_p t(Z_t - {c_p}/{2})^2 + b_{p2}tZ_t^{\,2} + h_2 t(Z_t - h_2/2)^2 + h_1 t(Z_c - h_1/2)^2 + b_{e1}tZ_c^{\,2} +$$
$$b_{e2}(\chi_d t)Z_c^{\,2} + c_{eff}(\chi_d t)(Z_c - {c_{eff}}/{2})^2$$
$$I_{eff,y} = 164112.4708mm^4$$

Effective section modulus:

- With regard to the flange in compression

$$W_{eff,y,c} = \frac{I_{eff,y}}{Z_c} = 3970.6491 mm^3$$

- With regard to the flange in tension

$$W_{eff,y,t} = \frac{I_{eff,y}}{Z_t} = 4880.1453 mm^3$$

$$W_{eff,y} = \min(W_{eff,y,c}, W_{eff,y,t}) = 3970.6491 mm^3$$

Check bending resistance at ULS

$$M_{c,Rd} = {W_{eff,y} f_{yb}} / {\gamma_{mo}} = 1.389 KNm$$

Verification of bending resistance

$$M_{Ed} = {q_d * L^2} / 8 = 1.2343 * 1^2 / 8 = 0.15423 KNm$$

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{0.15423}{1.389} = 0.11 \leq 1 - OK!$$

Check of shear resistance at ULS

Design shear buckling resistance

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi} t f_{bv}}{r_{mo}}$$

$$\lambda_w = \frac{0.346 * hp}{t}\sqrt{\frac{f_{yb}}{E}} = 1.0588$$

For λ_w >0.83

$$f_{bv} = {0.48 f_{yb}} / {\lambda_w} = 185.67 \, N / mm^2$$

$$V_{b,Rd} = 11.894 KN$$

Verification of shear resistance

$$V_{Ed} = F_{Ed} = q_d * L/2 = 1.2343 * 1/2 = 0.617KN$$

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{0.617}{11.894} = 0.0519 - OK!$$

Check of local transverse resistance at ULS

The following criteria should be satisfied

$$h_w/t \le 200$$
$$h_w/t = 74.96 < 200 - ok$$
$$r/t \le 6$$
$$r/t = 3 \le 6 - ok$$
$$45° \le \phi \le 90°$$
$$\phi = 90° - ok$$

The local transverse resistance of the web

The bearing length is: $S_s = 110m$

For $S_s / t > 60$

$$R_{w,Rd} = \frac{k_1 k_2 k_3 [5.92 - \frac{h_w / t}{132}][0.71 + 0.015 S_s / t] t^2 f_{yb}}{\gamma_{m1}}$$
$$k = f_{yb} / 228 = 1.535$$
$$k_1 = 1.33 - 0.33k = 0.823$$
$$k_2 = 1.15 - 0.15r / t = 0.7$$
$$k_3 = 0.7 + 0.3(\phi / 90)^2 = 1$$
$$R_{w,Rd} = 2.547KN$$

Verification of local transverse force

$$\frac{F_{Ed}}{R_{w,Rd}} = \frac{0.617}{2.547} = 0.24 - OK$$

Buckling strength

Lateral torsional buckling

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}{}^{2} - \lambda_{LT}{}^{2}}}$$, but $\chi_{LT} \leq 1$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}{}^{2}]$$

$\alpha_{LT} = 0.34$ -buckling curve b

$$\lambda_{LT} = \sqrt{\frac{W_{eff,Y} * f_{yb}}{M_{cr}}}$$

$$M_{cr} = \frac{1.127 * \pi^{2} EI_{z}}{L^{2}} \sqrt{\frac{I_{w}}{I_{Z}} + \frac{L^{2} GI_{t}}{\pi^{2} EI_{z}}} = 7423065.34 Nmm$$

$$\lambda_{LT} = 0.42605$$

$$\chi_{LT} = 0.9156$$

$$M_{b,Rd} = \chi_{LT} W_{eff,y} f_{yb} / \gamma_{m1} = 1.272 KNm$$

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{0.15423}{1.272} = 0.12 - OK$$

The section is sufficient and no transversal beam needed for the given section and length.

Solution 3 Main beam design

IPE 80

Total height h=80mm Iy=80.1 cm^{4}

Total width of flange B=46mm Iz=8.49 cm^{4}

Thickness of web tw=3.8mm It=0.689 cm^{4}

Thickness of flange tf=5.2mm Iw=0.118* $10^{3} cm^{6}$

radius r =5mm

iz=1.05cm

A= 7.64 cm^2

Wpl,y=23.2 cm^3

Steel grade S275

Wel,y=20 cm^3

Factored load

Main beam on the short side

Since there is no transversal beam the beam is designed for its own weight

$$q_{dms} = 1.35 * q_{Gm} = 1.35 * 0.5886\, N/m = 0.7946 * 10^{-3}\, KN/m$$

$$M_{Ed} = q_{dms} * L_s^2 / 8 = 0.7946 * 10^{-3} * 1^2 / 8 = 0.099 * 10^{-3}\, KNm$$

$$V_{Ed} = q_{dms} * L_s / 2 = 0.7946 * 10^{-3} * 1/2 = 0.3973 * 10^{-3}\, KN$$

$$F_{Ed} = q_{dms} * L_s / 2 = 0.7946 * 10^{-3} * 1/2 = 0.3973 * 10^{-3}\, KN$$

Main beam on the long side

$$q_{dml} = 1.35 * q_{Gm} + (F_{edj} / (Js * 1000)) = 1.35 * 0.5886 / 1000\, kN/m + (0.617 / 0.25) = 2.4688\, KN/m$$

$$M_{Ed} = q_{dms} * L_s^2 / 8 = 2.4688 * 2^2 / 8 = 1.2344\, KNm$$

$$V_{Ed} = q_{dms} * L_s / 2 = 2.4688 * 2/2 = 2.4688\, KN$$

$$F_{Ed} = q_{dms} * L_s / 2 = 2.4688 * 2/2 = 2.4688\, KN$$

Out stand flange

$$\frac{C}{tf} = \frac{46 - 3.8 - (2*5)}{2*5.2} = 3.096 \leq 10 * \varepsilon$$

$$3.096 \leq 9.2$$

Flange is class1

Webs

$$\frac{d}{t_w} = \frac{80-(2*5.2)-(2*5)}{3.8} = 15.68 \leq 72*\varepsilon$$

$$15.68 \leq 66.24$$

Web is class 1

Therefore the whole section is class 1.

Member resistance

Bending resistance

Section modulus depending up on their class is:

$$W_{pl,y} - class1\,\&\,class2$$

$$M_{c,Rd} = {W_y f_y}/{\gamma_{m0}} = 23.2*10^3*275/1 = 6.38KNm$$

Main beam on the short side-

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{0.000099}{6.38} = 1.55*10^{-5} - OK$$

Main beam on the long side-

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{1.2344}{6.38} = 0.193 - OK$$

Lateral torsional buckling resistance

Follow the same step from section ii used to check the lateral torsional buckling resistance.

Shear resistance

$$V_{c,Rd} = V_{pl,Rd} = {A_v * \left(f_y / \sqrt{3}\right)} \Big/ {\gamma_{m1}}$$

$$A_v = A - 2 * b * t_f + (t_w + 2r)t_f \geq \eta h_w t_w$$
$$A_v = 7.64 * 10^2 - 2 * 4.6 * 5.2 + (3.8 + 2 * 5) * 5.2 = 357.36 \geq \eta h_w t_w$$
$$357.36 \geq 264.48$$
$$A_v = 357.36 mm^2$$

$$V_{c,Rd} = V_{pl,Rd} = {357.36 * \left(275 / \sqrt{3}\right)} \Big/ {1} = 56.738 KN$$

Main beam on the short side-

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{0.3973 * 10^{-3}}{56.738} = 0.007 * 10^{-3} - OK!$$

Main beam on the long side-

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{2.4688}{56.738} = 0.0434 - OK!$$

Buckling strength

Lateral torsional buckling *long side*

$$\lambda_{LT} = \sqrt{\frac{W_Y * f_y}{M_{cr}}}$$
$$M_{cr} = \frac{1.127 * \pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_Z} + \frac{L^2 GI_t}{\pi^2 EI_z}} = 5909030.21 Nmm$$
$$\lambda_{LT} = 1.039$$
$$\phi_{LT} = 1.1279$$
$$\chi_{LT} = 0.638$$

$$M_{b,Rd} = {\chi_{LT} W_{pl,y} f_y} \Big/ {\gamma_{m1}} = 4.07 KNm$$

Lateral torsional buckling *Short side*

$$\lambda_{LT} = \sqrt{\frac{W_Y * f_y}{M_{cr}}}$$

$$M_{cr} = \frac{1.127 * \pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_Z} + \frac{L^2 GI_t}{\pi^2 EI_z}} = 13441060.92 Nmm$$

$$\lambda_{LT} = 0.6889$$

$$\phi_{LT} = 0.7886$$

$$\chi_{LT} = 0.85279$$

$$M_{b,Rd} = \chi_{LT} W_{pl,y} f_y / \gamma_{m1} = 5.544 KNm$$

Main beam on the short side-

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{0.000099}{5.544} = 0.1819 * 10^{-4} - OK$$

Main beam on the long side-

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{1.2344}{4.07} = 0.3033 - OK$$

Deflection check

Main beam short side-

$$\Delta_m = \frac{5 * q_d * L^4}{384 * E * I} = \frac{5 * 0.7506 * 10^{-3} * 1000^4}{384 * 210000 * 801000} = 5.81 * 10^{-5} mm$$

$$\Delta = L/200 = 1000/250 = 4mm$$

OK!

Main beam long side-

$$\Delta_m = \frac{5 * q_d * L^4}{384 * E * I} = \frac{5 * 2.025 * 2000^4}{384 * 210000 * 801000} = 2.5mm$$

$$\Delta = L/200 = 2000/250 = 8mm$$ OK!

Use IPE 80 for the short and long beam.

Sufficient sections

OSB-2F16 with thickness of 11 mm

CFS joist-H76B44T3 with joist spacing of 250 mm

Main beam IPE-80 for short side and long side beam

ANNEX-B

Table

Table B- 1 Depth to breadth ratio of solid and laminated rectangular sections

Degree of lateral support	Maximum depth to breadth ratio
No lateral support	2
Ends position	3
Ends held in position and member held in line as by purlins or tie rods at centers not more than 30 times breadth of the member	4
Ends held in position and compression edge held in line as by direct connection of sheathing deck or joist	5
Ends held in position and compression edge held in line as by direct connection of sheathing, deck or joists, together with adequate bridging or blocking spaced at intervals not exceeding 6 times the depth	6
Ends held in position and both edges held firmly in line	7

Table B- 2 Modification factor K_2 for obtaining stresses and moduli

Property	K_2
Bending parallel to grain	0.8
Tension parallel to grain	0.8
Compression parallel to grain	0.6
Compression perpendicular to grain	0.6
Shear parallel to grain	0.9
Mean and minimum modulus of elasticity	0.8

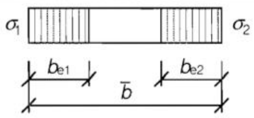
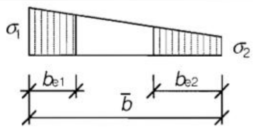
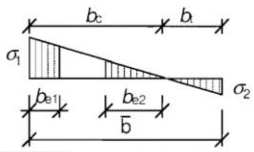
Table B- 3 Modification factor K_3 for duration loading

Duration of loading	K_3
Long term	1
Medium term	1.25
Short term	1.5
Very short term	1.74

Table B- 4 OSB standard sections

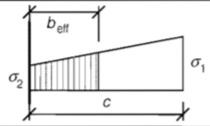
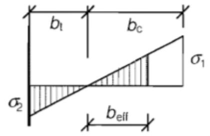
	Thickness	Self weight Kpa	Bending moment resistance Nmm/mm	planer Shear resistance due to bending N/mm	Bending stiffness EIo Nmm^2/mm
2F16	11	0.069	65	2.3	140000
2F16	12	0.079	95	2.9	220000
2F20	15	0.1	152	3.6	500000
2F24	18	0.12	228	4.2	820000
1F16	15	0.1	95	3.1	300000
1F20	15	0.1	143	3.7	360000
1F24	18	0.12	219	4.3	720000
1F32	22	0.14	2380	6.1	2100000
1F48	28.5	0.18	684	9.5	4400000

Table B- 5 Effective width of internal compression elements

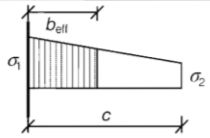
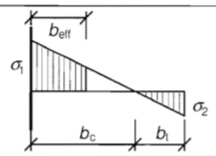
Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$\psi = 1$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0,5\ b_{eff}$ $\quad b_{e2} = 0,5\ b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0,4\ b_{eff}$ $\quad b_{e2} = 0,6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	AC1) $-1 > \psi \geq -3$ (AC1
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98\ (1 - \psi)^2$

Table B- 6 Effective width of outstand compression elements

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$1 > \psi \geq 0$: $b_{eff} = \rho\, c$
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$1 > \psi \geq 0$: $b_{eff} = \rho\, c$
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578\,/\,(\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,1\psi^2$	23,8

Table B- 7 Shear buckling strength f_{bv}

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support
λw≤0.83	0.58fyb	0.58fyb
0.83<λw<1.4	0.48fyb/λw	0.48fyb/λw
λw≥1.4	$0.67fyb/λw^2$	0.48fyb/λw

Table B- 8 Nominal values of basic yield strength f_{yb} and f_u for CFS

Type of steel	Grade	F_{yb}(N/mm^2)	F_u(N/mm^2)
Continuous hot dip zinc coated carbon steel sheet	S220GD+Z	220	330
	S250GD+Z	250	250
	S280GD+Z	280	360
	S320GD+Z	320	390
	S350GD+Z	350	420
Hot rolled flat products made of high yield strength steels for cold forming	S 315 MC	315	390
	S 355 MC	355	430
	S 420 MC	420	480
	S 460 MC	460	520
	S 500 MC	500	550
	S 550 MC	550	600
	S 600 MC	600	650
	S 650 MC	650	700
	S 700 MC	700	750

ANNEX-C
Source Code

Class OSB

```
namespace DesignParameters

{

    public class OSBDesign
    {
        public double sn{ get; set; }
        public double Gko{ get; set; }
        public double Mcap { get; set; }
        public double Vcap{ get; set; }
         public double Js { get; set; }
        public double qd { get; set; }
        public double EIo { get; set; }
        public double t { get; set; }
        public double Qk { get; set; }
        public double Superimposed { get; set; }
        public double E { get; set; }
        public double I { get; set; }
        public double BStress { get; set; }
        public double Sstress { get; set; }
        public double b { get { return 1000; } set { } }
        public double  k3 { get; set; }
        public double k5 { get; set; }
        public double k6 { get; set; }
        public double k7 { get; set; }
        public double k8 { get; set; }
        public double c { get; set; }
        public double A { get; set; }
          public double getqd()
        {
            this.qd = 1.35 * (this.Superimposed + Gko) + 1.5 * this.Qk;
            return this.qd;        }
      public double getMcap()
        {
            if (t <= 72)
                this.k7 = 1.17;
            else if (72 < t && t < 300)
                this.k7 = Math.Pow(300 / t, 0.11);
            else
                k7 = 0.81 * (Math.Pow(t, 2) + 92300 / (Math.Pow(t, 2) + 56800));
            this.Mcap = BStress * 0.8 * (k3/4) * k7 * b * Math.Pow(t, 2) / (6000);
            return this.Mcap;
        }
        public double getVcap()
        {
            this.Vcap = 0.6 * Sstress * (k3 / 4) * b * t / 1000;
            return this.Vcap;
        }
        public double getEIo()
        {
            this.EIo = E * b * Math.Pow(t, 3) / 12 / 1000;
            return this.EIo;
        }
        public double getJs()// support length of the osb
                var Js11 = Math.Floor(Math.Pow((Mcap*1000 / ((9 * this.qd) / 128)), 0.5));
                var Js21 =Math .Floor ((Vcap*1000 / ((5 * this.qd)/8)));
                var Js31 =Math .Floor ( Math.Pow((185*0.003*EIo*1000 / (this.qd)), 0.3333));
            List<double> mylist = new List<double>();
            mylist.Add(Js11);
            mylist.Add(Js21);
            mylist.Add(Js31);
            this.Js = Math.Min(Js11, Math.Min(Js21, Js31));
            this.Js = mylist.Where(e=>(e > 250 && e < 600) || (this.Js > 250 && this.Js < 600)).FirstOrDefault();
```

```
        this.sn = 3;
        if (this.Js == 0)
        {
            var Js12 =Math .Floor ( Math.Pow((Mcap*1000 / (0.08 * this.qd)), 0.5));
            var Js22 =Math .Floor ((Vcap*1000 / (0.6* this.qd)));
            var Js32 = Math.Floor(Math.Pow((144.93 * 0.003* EIo*1000 / (this.qd)), 0.3333));
            mylist = new List<double>();
            mylist.Add(Js12);
            mylist.Add(Js22);
            mylist.Add(Js32);
            this.Js = Math.Min(Js12, Math.Min(Js22, Js32));
            this.Js = mylist.Where(e => (e > 250 && e < 600) || (this.Js > 250 && this.Js <  600)).FirstOrDefault();
            this.sn = 4;
        }
        else if (this.Js ==0)
        {
            var Js13 = Math.Floor(Math.Pow((Mcap*1000 / (0.0772 * this.qd)), 0.5));
            var Js23 = Math.Floor((Vcap *1000/ (0.607 * this.qd)));
            var Js33 = Math.Floor(Math.Pow((153.84 * 0.003* EIo *1000/ (this.qd)), 0.3333));
            mylist = new List<double>();
            mylist.Add(Js13);
            mylist.Add(Js23);
            mylist.Add(Js33);
            this.Js = Math.Min(Js13, Math.Min(Js23, Js33));
            this.Js = mylist.Where(e => (e > 250 && e < 600) || (this.Js > 250 && this.Js < 600)).FirstOrDefault();
            this.sn = 5;
        }
        if (this.Js > 600)
            this.Js = 600;
        else
            this.Js = this.Js;
        return this.Js;
    }
     }
}
```

Class Effective section properties

```
public class EffectiveSectionProperties
    {
        public double Be1 { get; set; }
        public double Be2 { get; set; }
        public double Beff { get; set; }
        public double Ceff { get; set; }
        public double H1 { get; set; }
        public double H2 { get; set; }
        public double Ieff { get; set; }
        public double Weff { get; set; }
        public double As { get; set; }
        public double Aeff { get; set; }
        public double L { get; set; }
        public double Lj { get; set; }
        public double Xd { get; set; }
        public double Xlt { get; set; }
        public double Xd1 { get; set; }
        public double Jn { get; set; }
        public double Tn { get; set; }
        public GeometricProportions gp;
        public OSBDesign od;
        public EffectiveSectionProperties(GeometricProportions gpr, OSBDesign osb)
        {
            gp = gpr;
            od = osb;
        }
        #region Flange and Lip
        public double getEffectiveWidth()
        {
            var kbf = 4;
```

```
            var stressRatio = 1;
            var e = Math.Sqrt(235 / gp.Fyb);
            var rs = (gp.Bp / gp.t) / (28.4 * e * Math.Sqrt(kbf));
            var row = (rs - 0.055 * (3 + stressRatio)) / Math.Pow(rs, 2);
            this.Beff = row * gp.Bp;
            this.Be1 = 0.5 * this.Beff;
          return this.Be1;
        }
        public double getiteratexd()
        {
            double lastAnswer = 1, delta;
            do

{
                var fyb1 = lastAnswer * gp.Fyb;
                var kbf = 4;
                var stressRatio = 1;
                var e = Math.Sqrt(235 / fyb1);
                var rs = (gp.Bp / gp.t) / (28.4 * e * Math.Sqrt(kbf));
                var row = (rs - 0.055 * (3 + stressRatio)) / Math.Pow(rs, 2);
                this.Beff = row * gp.Bp;
                this.Be2 = 0.5 * this.Beff;
                double kbf1 = 0;
                var ratio = gp.Cp / gp.Bp;
                if (ratio <= 0.35)
                    kbf1 = 0.5;
                else if (ratio > 0.35 && ratio <= 0.6)
                    kbf1 = 0.5 + (0.83 *(Math.Pow (Math.Pow((ratio - 0.35), 2),0.3333)));
                var e1 = Math.Sqrt(235 / gp.Fyb);
                var rs1 = (gp.Cp / gp.t) / (28.4 * e1 * Math.Sqrt(kbf1));
                var row1 = Math.Min (((rs1 - 0.188) / Math.Pow(rs1, 2)),1);
                this.Ceff = row1 * gp.Cp;
                var As = gp.t * (this.Be2 + this.Ceff);
                var b1 = gp.Bp - ((Math.Pow(this.Be2, 2) * gp.t) / (2 * As));
                var Is = (this.Be2 * (Math.Pow(gp.t, 3)) / 12) +
                        ((Math.Pow(this.Ceff, 3)) * gp.t / 12) +
                        ((this.Be2 * gp.t) * (Math.Pow((Math.Pow(this.Ceff, 2)) / (2 * (this.Be2 + this.Ceff)), 2))) +
                        (this.Ceff * gp.t) * (Math.Pow(((this.Ceff / 2) - (Math.Pow(this.Ceff, 2) / (2 * (this.Be2 + this.Ceff)))),
2));
                var K = (gp.E * Math.Pow(gp.t, 3)) / ((4 * (1 - Math.Pow(gp.v, 2)) * ((Math.Pow(b1, 2) * gp.Hp) +
(Math.Pow(b1, 3)))));
                //Bs = Buckling Stress
                var Bs = (2 * (Math.Sqrt(K * gp.E * Is))) / As;
                var Rs = Math.Sqrt(gp.Fyb / Bs);
                if (Rs <= 0.65)
                    this.Xd = 1;
                else if (Rs > 0.65 && Rs < 1.38)
                    this.Xd = 1.47 - (0.723 * Rs);
                else
                    this.Xd = 0.66 / Rs;
                delta = Math .Abs (Xd - lastAnswer);
                lastAnswer = Xd;
            }
            while (delta>0 );
            return this.Xd;
        }
        #endregion
        #region Web
        public double getH1()
        {
            this.getEffectiveWidth();
            this.getiteratexd();
            var hc = (gp.Cp * (gp.Hp - gp.Cp / 2) + gp.Bp * gp.Hp + Math.Pow(gp.Hp, 2) / 2 + Math.Pow(this.Ceff, 2) *
this.Xd / 2) /
                      (gp.Cp + gp.Bp + gp.Hp + this.Be1 + (this.Be2 + this.Ceff) * this.Xd);
            var stressRatio = (hc - gp.Hp) / hc;
            double Kbf = 0;
            if (stressRatio > 0 && stressRatio < 1)
```

```
            Kbf = 8.2 / (1.05 + stressRatio);
        else if (stressRatio == 0)
            Kbf = 7.81;
        else if (stressRatio == 1)
            Kbf = 4;
        else if (stressRatio > -1 && stressRatio < 0)
            Kbf = 7.81 - 6.29 * stressRatio + 9.78 * Math.Pow(stressRatio, 2);
        else if (stressRatio == -1)
            Kbf = 23.9;
        else
            Kbf = 5.98 * Math.Pow(1 - stressRatio, 2);
        var e = Math.Sqrt(235 / gp.Fyb);
        var rs = (gp.Hp / gp.t) / (28.4 * e * Math.Sqrt(Kbf));
        var row = Math .Min (((rs - 0.055 * (3 + stressRatio)) / Math.Pow(rs, 2)),1);
        var heff = row * hc;
        double he1 = 0;
        double he2 = 0;
        if (stressRatio >= 0 && stressRatio < 1)
        {
            he1 = 2 * heff / (5 - stressRatio);
            he2 = heff - he1;
        }
        else if (stressRatio < 0)
        {
            he1 = 0.4 * heff;
            he2 = 0.6 * heff;
        }
        else if (stressRatio == 1)
        {
            he1 = 0.5 * heff;
            he2 = he1;
        }
        this.H1 = he1;
        this.H2 = gp.Hp - (hc - he2);
        return this.H1;
    }
    #endregion
    #region Effective section property
    public double getAeff()
    {
        this.Aeff = (gp.t * (gp.Cp + gp.Bp + this.H1 + this.H2 + this.Be1 + ((this.Be2 + this.Ceff) * this.Xd)));
        return this.Aeff;
    }
    public double getweff()
    {
        gp.getIz();
        this.getH1();
        this.getAeff();
        var Zc = gp.t * ((gp.Cp * (gp.Hp - (gp.Cp / 2))) + (gp.Bp * gp.Hp) + (this.H2 * (gp.Hp - (this.H2 / 2))) + 
(Math.Pow(this.H1, 2))
            + (Math.Pow(this.Ceff, 2) * this.Xd / 2)) / this.Aeff;
        var Zt = gp.Hp - Zc;
        this.Ieff = ((Math.Pow(this.H1, 3) * gp.t / 12) + (Math.Pow(this.H2, 3) * gp.t / 12) + (Math.Pow(gp.Bp, 3) * 
gp.t / 12) + (Math.Pow(gp.Cp, 3) * gp.t / 12) + (Math.Pow(gp.t, 3) * this.Be1 / 12)
            + (Math.Pow((gp.t * this.Xd), 3) * this.Be2 / 12) + (Math.Pow(this.Ceff, 3) * (gp.t * this.Xd) / 12) + (gp.Cp 
* gp.t * Math.Pow((Zt - (gp.Cp / 2)), 2)) + (gp.Bp * gp.t * Math.Pow(Zt, 2)) + (this.H2 * gp.t * Math.Pow((Zt - 
(this.H2 / 2)), 2))
            + (this.H1 * gp.t * Math.Pow((Zc - (this.H1 / 2)), 2)) + (this.Be1 * gp.t * (Math.Pow(Zc, 2))) + (this.Be2 * 
this.Xd * gp.t * (Math.Pow(Zc, 2))) + (this.Ceff * this.Xd * gp.t * (Math.Pow((Zc - (this.Ceff / 2)), 2))));
        var Weffy = this.Ieff / Zc;
        var weffz = this.Ieff / Zt;
        this.Weff = Math.Min(Weffy, weffz);
        return this.Weff;// mm3
    }
    #endregion

    public double getLj()
    {
```

```
            this.L = Math.Min(gp.a, gp.b);
            this.Lj = this.L / (Tn+1);
            return this.Lj;
        }
        #region lateral torsional buckling reduction factor
        public double getXlt()
        {
            this.getLj();
            gp.getIz();
            gp.getIw();
            gp.getIt();
            var Mcr = (1.127 * Math.Pow(Math.PI, 2) * gp.E * gp.Iz / Math.Pow(this.Lj, 2)) * (Math.Sqrt((gp.Iw / gp.Iz) +
(Math.Pow(this.Lj, 2) * gp.G * gp.It) / (Math.Pow(Math.PI, 2) * gp.E * gp.Iz)));
            var RSlt = Math.Sqrt(this.Weff * gp.Fyb / Mcr);
            var Olt = 0.5 * (1 + (0.34 * (RSlt - 0.2)) + Math.Pow(RSlt, 2));
            var X = 1 / (Olt + (Math.Sqrt(Math.Pow(Olt, 2) - Math.Pow(RSlt, 2))));
            if (X <= 1)
                this.Xlt = 1 / (Olt + (Math.Sqrt(Math.Pow(Olt, 2) - Math.Pow(RSlt, 2))));
            else
                this.Xlt = 1;
            return this.Xlt;
        }
        #endregion
    }
}
```

Class Final Design

```
public class finaldesign
    {
        public double Mcrd {get;set;}
        public double Mbrd { get; set; }
        public double Vcrd {get;set;}
        public double Vplrd {get;set;}
        public double Vbrd {get;set;}
        public double Rwrd {get;set;}
        public double fyrd {get;set;}
        public double qd { get; set; }
        public double Med { get; set; }
        public double Ved { get; set; }
        public double Fed { get; set; }
        public double delta { get; set; }
        public double def { get; set; }
        public GeometricProportions gp;
        public EffectiveSectionProperties esp;
        public OSBDesign od;
        public finaldesign(GeometricProportions gpr, OSBDesign osb, EffectiveSectionProperties eff)
        {
            gp = gpr;
            esp=eff;
            od = osb;
        }
        #region Moment capacity check
       public double getMcrd()
          {
           this.Mcrd=esp.getweff()*gp.Fyb/(gp.rmo*1000000);
           return this.Mcrd;

          }
        #endregion
        #region Shear capacity check
        public double getVbrd()
            {
                var Rsw=(0.346*gp.Hp/gp.t)*Math.Sqrt (gp.Fyb/gp.E);
                double fbv=0;
                if (Rsw<=0.83)
                    fbv=0.58*gp.Fyb;
```

```
        else
            fbv=0.48*gp.Fyb/Rsw;
        this.Vbrd=(gp.Hp*gp.t*fbv)/(gp.rmo*1000);
        return this. Vbrd;
    }
public double getVplrd()
    {
        this.Vplrd=gp.Hp*gp.t*gp.Fyb*Math.Sqrt(3)/((gp.rmo*1000));
        return this.Vplrd;
    }
#endregion
#region Lateral torsional buckling check
public double getMbrd()
{
    this.Mbrd = esp.getXlt()*esp.Weff * gp.Fyb / (gp.rm1*1000000);
    return this.Mbrd;
}
#endregion
#region Combined bending and shear check
public double getfyrd()
{
    if (Ved>0.5*Vplrd)
    {
        var row =Math.Pow(((2*Ved/Vplrd)-1),2);
        this.fyrd = (1 - row) * gp.Fy;
    }
    else
        this.fyrd=gp.Fy;
    return this.fyrd;
    }
#endregion
#region Factored load
public double getqd()
{
    var qg = (gp.Gkj) + ((gp.Superimposed + od.Gko) * od.getJs() / 1000); // in KN/m
    var qq = gp.Qk * od.getJs()/1000;
    this. qd = 1.35 * qg + 1.5 * qq;
    return this.qd;
}
public double getMed()
{
    this.Med = getqd() * Math.Pow((esp.getLj()/1000), 2)/8;
    return this.Med;
}
public double getVed()
{
    this.Ved = qd * esp.getLj() / 2000;
    return this.Ved;
}
public double getFed()
{
    this.Fed = qd * esp.Lj / 2000;
    return this.Fed;}
public double getdelta()
{
    this.esp.getweff();
    var w = gp.Qk * od.Js / 1000;// KN/m=N/mm
    this.delta = (5*w*Math .Pow (esp.getLj (),4))/(384*gp.E*esp.Ieff);
    this.def = esp.getLj() / 180;
    return this.delta;
}
public double getRwrd()
{
    var k = gp.Fyb / 228;
    var k1 = 1.33 - 0.33 * k;
    var k2 = 1.15 - (0.15 * gp.r / gp.t);
    if (k2 <= 1 && k2 >= 0.5)
        k2 = 1.15 - (0.15 * gp.r / gp.t);
```

```
        else if (k2 < 0.5)
            k2 = 0.5;
        else
            k2 = 1;
        var k3 = 0.7 + (0.3 * Math.Pow(gp.angle / 90, 2));
        var k4 = 1.22 - 0.22 * k;
        var k5 = Math.Min((1.06 - (0.06 * gp.r / gp.t)), 1);
        var c = 1.5 * gp.Hp;
        if (c <= 1.5 * gp.Hp)
            if (gp.Ss / gp.t <= 60)
                this.Rwrd = (k1 * k2 * k3 * (5.92 - (gp.Hp / (gp.t * 132))) * (1 + (0.01 * gp.Ss / gp.t)) * gp.Fyb *
Math.Pow(gp.t, 2)) / (1000*gp.rm1);
            else
                this.Rwrd = (k1 * k2 * k3 * (5.92 - (gp.Hp / (gp.t * 132))) * (0.71 + (0.015 * gp.Ss / gp.t)) * gp.Fyb *
Math.Pow(gp.t, 2)) /(1000* gp.rm1);
        else
            if (gp.Ss / gp.t <= 60)
                this.Rwrd = (k3 * k4 * k5 * (14.7 - (gp.Hp / (gp.t * 49.5))) * (1 + (0.007 * gp.Ss / gp.t)) * gp.Fyb *
Math.Pow(gp.t, 2)) / gp.rm1;
            else
                this.Rwrd = (k3 * k4 * k5 * (14.7 - (gp.Hp / (gp.t * 49.5))) * (0.75 + (0.011 * gp.Ss / gp.t)) * gp.Fyb *
Math.Pow(gp.t, 2)) / (gp.rm1 * 1000);
        return this.Rwrd;
    }
    #endregion
  }
}
```

Class Transversal/Main beam design

```
public class Transversal beam
  {
    public double Gkt { get; set; }//own weight of the section
    public double L { get; set; }
    public string Naming { get; set; }
    public double H { get; set; }
    public double B { get; set; }
    public double d { get; set; }
    public double tw { get; set; }
    public double tf { get; set; }
    public double r { get; set; }
    public double A { get; set; }
    public double Iy { get; set; }
    public double iy { get; set; }
    public double Iz { get; set; }
    public double iz { get; set; }
    public double Iw { get; set; }
    public double It { get; set; }
    public double Wely { get; set; }
    public double Wply { get; set; }
    public double Wy { get; set; }
    public double fy { get; set; }
    public double E { get; set; }
    public double v { get; set; }
    public double G
    {
        get
        {
            return E / (2 * (1 + v));
        }
    public double Mcrd { get; set; }
    public double Mbrd { get; set; }
    public double Vplrd { get; set; }
    public double Ved { get; set; }
    public double Mcom { get; set; }
    public double Olt { get; set; }
    public double Rslt { get; set; }
```

```
        public double Xlt { get; set; }
        public double Xlti { get; set; }
        public double del { get; set; }
        public double qdt { get; set; }
        public double Med { get; set; }
        public double Fed { get; set; }
        public double Ts { get; set; }
        public finaldesign fd;
        public OSBDesign od;
        public GeometricProportions gp;
        public EffectiveSectionProperties esp;
        public Transversalbeam(GeometricProportions gpr, OSBDesign osb, EffectiveSectionProperties eff)
        {
            od = osb;
            fd = new finaldesign(gpr, osb, eff);
            gp = gpr;
            esp = eff;
        }
        public double getL()
        {
            this.L =Math.Max (gp.a,gp.b);
            return this.L ;
        }
        public double getqdt()
        {
            this.qdt =(2*fd.Fed/(od.Js/1000)  ) + (1.35 * Gkt); // KN/m
            return this.qdt;
        }
        #region moment capacity check
        public double getWy()
        {
            double web=( H-(2*r)-(2*tf))/tw;
            double flange=(B-(2*r) -tw )/(2*tf );
            var e=Math.Sqrt(235 /fy);
            if (web <= 72 * e)
                if (flange <= 9 * e)
                    this. Wy  = Wply;
                else if (flange <= 10 * e && flange>9*e)
                    this.Wy = Wply;
                else if (flange <= 14 * e && flange > 10 * e)
                    this.Wy = Wely;
            else if (web <= 83 * e && web >72*e)
                if (flange <= 10 * e && flange > 9 * e)
                    this.Wy = Wply;
                else if (flange <= 14 * e && flange > 10 * e)
                    this.Wy = Wely;
             else if (web <= 124* e && web >83*e)
                 if (flange <= 14 * e)
                     this.Wy = Wely;
            return this.Wy ;
        }
        public double getMcrd()
          {
             this.Mcrd = this.getWy() * fy / (gp.rmo*1000000);
             return this.Mcrd ;
          }
        public double getOlt()
        {
            var Mcr = (1.127 * Math.Pow(Math.PI, 2) * E * Iz / Math.Pow(this.getL(), 2)) * (Math.Sqrt((Iw*1000000 / Iz)
+ (Math.Pow(this.getL(), 2) * G * It) / (Math.Pow(Math.PI, 2) * E * Iz)));
            this.Rslt  = Math.Sqrt(this.getWy()  * fy  / Mcr);
            if(H /B <=2)
                this.Olt = 0.5 * (1 + (0.21 * (this.Rslt - 0.2)) + Math.Pow(this.Rslt, 2));
            else
                this. Olt = 0.5 * (1 + (0.34 * (this.Rslt - 0.2)) + Math.Pow(this.Rslt , 2));
            return this. Olt;
        }
```

```
        public double getMbrd()
        {
            this.getOlt();
            var X = 1 / (Olt + (Math.Sqrt(Math.Pow(Olt, 2) - Math.Pow(this.Rslt , 2))));
            if (X <= 1)
                this.Xlti = 1 / (Olt + (Math.Sqrt(Math.Pow(Olt, 2) - Math.Pow(this.Rslt, 2))));
            else
                this. Xlti = 1;
            var f = 1 - (0.03 * (1 - (2 * Math.Pow((Rslt - 0.8), 2))));
            if (f<1)
            this. Xlt =Xlti/f;
            else
                this.Xlt = Xlti;
            this.Mbrd =this.Xlt*this.getWy() *fy/(gp.rm1*1000000)  ;
            return this.Mbrd;
        }
        #endregion
        #region shear capacity check
        public double getVplrd()
        {
            var Av = A- (2 * B * tf) + (tw + (2 * r)) * tf;
            if (Av >= (H - tf) * tw)
                this.Vplrd = Av * fy / (Math.Sqrt(3) * gp.rmo*1000);
            else
                this.Vplrd = (H - tf) * tw * fy / (Math.Sqrt(3) * gp.rmo*1000);
            return this.Vplrd;
        }
        #endregion
        #region combined bending and shear check
        public double getMcom()
        {
            this.getVplrd();
            this.getVed();
            if (Ved > 0.5 * Vplrd)
            {
                var row = Math.Pow(((2 * Ved / Vplrd) - 1), 2);
                var fyrd = (1 - row) * fy ;
                this.Mcom = fyrd / fy * this.getMcrd();
            }
            else
                this.Mcom = this.getMcrd();
            return this.Mcom;
        }
        #endregion
        #region deflection check
        public double getdel()
        {
            this.del = 5 * this.getqdt() * Math.Pow(this.getL(), 4) / (E * Iy * 384);
            return this.del;
        }
        public double getMed()
        {
            this.Med = this.getqdt() * Math.Pow((this.getL() / 1000), 2) / 8;
            return this.Med;
        }
        public double getVed()
        {
            this.Ved = this.getqdt() *this.getL() / 2000;
            return this.Ved;
        }
        public double getFed()
        {
            this.Fed = this.getqdt() * this.getL() / 2000;
            return this.Fed;
        }
        #endregion
    }
}
```