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Comparative Design Analysis of Stainless-Steel Hollow Sections

Masters Degree project in MPSEB

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CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2026
www.chalmers.se

MASTER'S THESIS ACEX30 2026

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of Stainless-Steel Hollow Sections**

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*Master's Thesis in the Master's Programme Structural Engineering and Building
Technology*

ANDREW ROSENBERG

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Examiner: Mozhdeh Amani, Structural Engineering, Architecture and Civil Engineer-
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Examensarbete ACEx30
Institutionen för Arkitektur och Samhällsbyggnadsteknik
Chalmers Tekniska Högskola, 2026

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Cover:
A cluster of stainless steel hollow sections.
Department of Architecture and Civil Engineering
Göteborg, Sweden, 2026

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ABSTRACT

Stainless steel hollow structural sections (HSS) are becoming increasingly popular in modern construction due to their favourable design properties. However, differences among international design standards can lead to variations in predicted member capacities and structural efficiency. A comparative analysis of different design approaches for stainless steel hollow sections based on four major structural standards: AISC 370-21, ASCE 8-22, Eurocode 1993-1-4, and the second-generation Eurocode (EN 1993-1-4:2025), was conducted to focus on how each standard addresses the determination of the bending resistance, global slenderness, and local slenderness, considering austenitic, ferritic, and duplex stainless-steel grades.

A literature review of the different design standards and baseline information about stainless steel and its applications in structural engineering was first conducted and then a design tool for a parametrized study was implemented. Different design methods include the effective width method and direct strength method found in ASCE 8-22, effective width method from AISC 370-21 and Eurocode 1993 1-4, and the continuous strength method in both AISC 370-21 and Eurocode 1993 1-4 generation 2.

Bending resistance calculations carried out across numerous sections of varying, height, width, thickness, and steel grades were compared to the slenderness of each section and the change in bending resistance for increasing slenderness is emphasized. Strength enhancements due to cold worked stainless steel was investigated and noted to play an important role in strength determinations when comparing standards. ASCE and AISC design codes become much more comparable with these strength enhancements factored in. The continuous strength method and direct strength method are dependent on global slenderness and does not work for all size sections. The comparison highlights the importance of knowing which method and standard is most applicable to a given situation.

Overall methods found in AISC 370-21 showed higher resistances than ASCE 8-22 but similar in results and methodology to both generations of EN1993 1-4.

Key words:

Stainless Steel, Hollow sections, Comparative analysis, Bending resistance, American Society of Civil Engineers, American Institute of Steel Construction, Eurocode,

Komparativ designanalys av ihåliga sektioner i rostfritt stål
Examensarbete inom masterprogrammet Konstruktionsteknik och byggnadsteknologi

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SAMMANFATTNING

Ihåliga profiler av rostfritt stål (HSS) blir alltmer populära i moderna konstruktioner då de har många gynsamma egenskaper. Skillnader mellan internationella konstruktionsstandarder kan leda till variationer i önskad bärförmåga och strukturell effektivitet. En jämförande analys av olika konstruktionsmetoder för hålsektioner av rostfritt stål baserad på fyra större strukturella standarder: AISC 370-21, ASCE 8-22, Eurocode 1993-1-4 och andra generationens Eurocode (EN 1993-1-4:2025), genomfördes. Fokus låg vid hur varje standard behandlar bestämning av böjhållfasthet, global slankhet och lokal slankhet, med beaktande av austenitiska, ferritiska och duplex rostfria stålklasser.

Till att börja med genomfördes en litteraturstudie av olika konstruktionsstandarder och grundläggande information om rostfritt stål och dess tillämpningar inom konstruktionsteknik. Detta implementerades sedan i ett konstruktionsverktyg för en parametriserad studie. De olika konstruktionsmetoderna inklude effektiva breddmetoden och direkta hållfasthetsmetoden som finns i ASCE 8-22, effektiva breddmetoden från AISC 370-21 och Eurocode 1993 1-4, och kontinuerlig hållfasthetsmetod i både AISC 370-21 och Eurocode 1993 1-4 generation 2.

Beräkningar av böjmotstånd utförda över ett flertal sektioner av varierande höjd, bredd, tjocklek och stålqualiteter jämfördes med varje sektioners slankhet, och förändringen i böjmotstånd för ökande slankhet betonades. Ökad bärförmåga i samband med kallbearbetat rostfritt stål på grund av kallbearbetat rostfritt stål undersöktes och noterades spela en viktig roll i hållfasthetsbestämningar vid jämförelse av standarder. ASCE- och AISC-konstruktionskoder blir mycket mer jämförbara med dessa hållfasthetsförbättringar. Den kontinuerliga hållfasthetsmetoden och den direkta hållfasthetsmetoden är beroende av global slankhet och fungerar inte för alla sektioners storlekar. Jämförelsen belyser vikten av att veta vilken metod och standard som är mest lämplig för en viss situation.

Sammanfattningsvis visade metoderna i AISC 370-21 högre böjmotstånd än ASCE 8-22 men liknande resultat och metodologi som båda generationerna av EN1993 1-4.

Nyckelord:

Rostfritt stål, Hålfprofiler, Jämförande analys, Böjmotstånd, American Society of Civil Engineers, American Institute of Steel Construction, Eurocode,

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Preface

This comparison study of stainless steel hollow sections was conducted from January 2026 to June 2026. This work is part of a larger comparison study of design standards from the Steel Construction Institute based in the UK. The study was carried out at the Department of Structural Engineering at Chalmers University of Technology, Sweden and is a collaboration between Chalmers and the Steel Construction Institute. I would like to thank my supervisor and examiner on this project, Mozhdeh Amani for the time and support she has given me. I would also like to thank the two representatives from the Steel Construction Institute, Nancy Baddoo and Francisco Meza whose feedback and insight have been immensely valuable.

Gothenburg, June 2026
Andrew Rosenberg

Notations

Below is the list of all included variables occurring in this report.

Roman Upper Case Letters

Symbol	Description	Unit
A_g	Gross cross-sectional area	mm^2
A_{corner}	Total corner area of RHS section	mm^2
A_{re}	External corner area	mm^2
A_{ri}	Internal corner area	mm^2
B	Overall width of RHS section	mm
B_c	Material parameter found in ASCE 8-22	–
C	Ratio of corner flange area to full cross section area	–
D	Outer diameter of CHS section	mm
E	Young's modulus of elasticity	MPa
E_{sh}	Strain hardening modulus	MPa
F_e	Elastic buckling stress	MPa
F_n	Adjusted global flexural yield stress	MPa
F_u	Ultimate tensile strength	MPa
F_y	Yield strength	MPa
$F_{y,avg}$	Average enhanced yield strength due to cold forming	MPa
$F_{y,corner}$	Yield strength in corner regions	MPa
$F_{y,rd}$	Yield strength for circular sections	MPa
$F_{y,wall}$	Yield strength in flat wall regions	MPa
F_{yc}	Tensile yield stress at corners	MPa
G	Shear modulus	MPa
H	Overall height of RHS section	mm
I	Second moment of area for CHS	mm^4
I_e	External corner moment contribution	mm^4
I_i	Internal corner moment contribution	mm^4
I_y	Major axis moment of inertia	mm^4
I_z	Minor axis moment of inertia	mm^4
L_b	Unbraced member length	mm
L_p	Limiting laterally unbraced length	mm
L_u	Reference unbraced length	mm
M_{cr}	Critical buckling moment	kNm
M_{crl}	Critical local buckling moment	kNm
M_n	Nominal moment capacity	kNm
M_{ne}	Nominal flexural strength for lateral torsional buckling	kNm
M_{nl}	Nominal flexural strength for local buckling	kNm
M_p	Plastic moment capacity	kNm
M_y	Elastic moment capacity	kNm
R_{pg}	Bending strength reduction factor	–
S	Elastic section modulus	mm^3
S_{ec}	Effective section modulus in compression	mm^3
S_{ef}	Effective section modulus	mm^3

S_{et}	Effective section modulus in tension	mm ³
S_y	Elastic section modulus about major axis	mm ³
W_{ef}	Effective section modulus in Eurocode	mm ³
W_{el}	Elastic section modulus in Eurocode	mm ³
W_{pl}	Plastic section modulus in Eurocode	mm ³
Z	Plastic section modulus	mm ³

Roman Lower Case Letters

Symbol	Description	Unit
b	Section width	mm
b_{flange}	Flat width of flange	mm
b_{web}	Flat height of web	mm
b_{re}	Distance to centroid of external corner	mm
b_{ri}	Distance to centroid of internal corner	mm
f_y	Yield stress	MPa
f_{ya}	Enhanced yield stress due to cold work	MPa
h	Section height	mm
h_{re}	Distance to centroid of external corner	mm
h_{ri}	Distance to centroid of internal corner	mm
k	Plate buckling coefficient	–
n	Ramberg–Osgood material parameter	–
r	Internal corner radius	mm
r_e	External corner radius	mm
r_i	Internal corner radius	mm
t	Section thickness	mm

Greek Upper Case Letters

Symbol	Description	Unit
Λ	Limiting deformation capacity parameter	–

Greek Lower Case Letters

Symbol	Description	Unit
α	Bending coefficient	—
ϵ	Eurocode material parameter	—
ϵ_{corner}	Strain induced in RHS corners	—
ϵ_{csm}	Failure strain in CSM	—
ϵ_f	Minimum elongation after rupture	—
ϵ_{rnd}	Strain induced in CHS	—
ϵ_u	Ultimate strain	—
ϵ_{wall}	Strain induced in flat wall regions	—
ϵ_y	Yield strain	—
γ_{M0}	Partial safety factor for resistance	—
γ_{M1}	Partial safety factor for instability resistance	—
γ_{M2}	Partial safety factor for cross-sectional resistance	—
λ	General slenderness parameter	—
λ_f	Flange slenderness ratio	—
λ_l	Local slenderness parameter	—
λ_{LT}	Global slenderness for lateral torsional buckling	—
λ_{pf}	Compact flange slenderness limit	—
λ_{pw}	Compact web slenderness limit	—
λ_{rf}	Non-compact flange slenderness limit	—
λ_{rw}	Non-compact web slenderness limit	—
λ_w	Web slenderness ratio	—
ν	Poisson's ratio	—
ϕ	Resistance reduction factor (LRFD)	—

1 Introduction

1.1 Background

Stainless steel hollow structural sections (HSS) are increasingly popular in modern construction due to their favourable strength-to-weight ratio, torsional resistance, corrosion resistance, and aesthetic appeal. The use of stainless-steel grades such as austenitic, ferritic, and duplex in HSS design brings unique mechanical properties, making it possible to tailor solutions to meet specific structural demands and environmental conditions. Stainless steel HSS require less energy for transportation and installation, which reduces the ecological footprint of a project. In addition, they are made from recycled materials, which makes their production sustainable. Stainless steel is different than carbon steel and exhibits different material properties, namely the stress strain behaviour. The stress strain curve of carbon steel is linear elastic up to the yield stress while stainless steel is much more non-linear with a more gradual curve and the exact yield stress is much harder to define. Therefore, stainless steel “yield” strengths are generally defined for a particular offset permanent strain (conventionally the 0.2% strain). Austenitic steel is used for structural applications that require good strength, corrosion resistance, and material formability. Duplex steel compared to Austenitic stainless steel, has a higher strength, corrosion resistance and resistance to stress cracking induced by corrosion and can be used where a better performance is necessary. Different countries use different design codes and standards, which often come with discrepancies and differences in practices that will lead to alternative solutions for the same problem. AISC 370-21 and ASCE/SEI 8-22 are codes used primarily in the United States that consider load resistance factor design (LRFD) and allowable stress design (ASD) design rules while focusing on the unique mechanical behaviour of stainless steel and modifying the buckling/resistance formulations relative to carbon steel accordingly. The Eurocode provides additional supplements (EN1993 1-4) for stainless steel materials while still highlighting standard procedures for regular carbon steel. The second generation of Eurocode aims to address more advanced testing data.

1.2 Aim

This thesis aims to conduct a comparative analysis of design approaches for stainless steel hollow sections based on four major structural standards: AISC 370-21, ASCE 8-22, Eurocode 1993-1-4, and the second-generation Eurocode (EN 1993-1-4:2025). This study will focus on how each standard determines bending resistance with considerations for austenitic, ferritic, and duplex stainless-steel grades.

1.3 Objectives

The objective of the thesis is to investigate each of the structural standards and how differences in design factors such as global and local slenderness, bending resistance, and cross sectional classes are affected by different parameters such as steel grade (austenitic, ferritic, and duplex), cross sectional profile, and loading conditions. A parametric study will be carried out to evaluate these differences, and the results will be analysed and interpreted in relation to variations in material modelling, resistance factors, partial safety factors, and slenderness treatment adopted by the four standards.

1.4 Methods

The objective of the thesis will be carried out using the following methodology

1. A literature review of the different design standards and background information on stainless steel
2. Development of a tool for a parameterized study of Stainless Steel HSS sections for different cross-sectional classes, slenderness ratios and loading conditions
3. Design calculations for Stainless Steel HSS sections will be carried out across the four standards, examining various slenderness ranges. A comprehensive comparative analysis of design outcomes will then be performed, highlighting the strengths and limitations of each standard.
4. Finite Element Analysis(FEA): FEA will be conducted to examine buckling behaviour and local deformations across different stainless-steel grades. This analysis will provide additional insights for comparing the standards.

1.5 Limitations

Due to the complexity of the task and the given time constraints the following limitations and simplifications will be considered...

1. Resistance values are calculated for beams considering only pure bending. No considerations for shear or tension will be taken.
2. The material properties for each grade are according to the code specifications, which means there is a slight difference between the Eurocode material properties (which also depend on wall thickness), and the US properties.
3. The Ferritic stainless-steel grade is not included in AISC 370, so it will not be included in the ASCE 8 study either.
4. Beams are assumed to be laterally restrained so considerations of Lateral torsional buckling (LTB) have not been taken.
5. to account for this a limit of $h/b \leq 3$ has been applied to RHS using AISC 370, both with standard properties and the Continuous Strength Method (CSM)
6. CSM for hollow sections in flexure in AISC 370 specifies a limit of $Lb \leq 0.5Lp$, but this is derived from I-sections and is too conservative when applied to hollow sections, and has been ignored and instead the $h/b \leq 3$ limit is used

2 Theory

2.1 Stainless Steel Types

Stainless steel is a family of steels with notable heat resistant and corrosion resistant properties. In order to be considered stainless steel it must contain a minimum of 10.5% chromium content and a maximum of 1.2% carbon content. Similar to carbon steel there are various types of stainless steels known as grades with varying levels of corrosion resistance and strength. The different mechanical properties are controlled through additional alloyed elements that allow for applications for resistance in different types of corrosive environments. Selecting the right stainless steel grade is important to maximize economic benefit and avoid unnecessary costs and usage of alloying metals. (AISC Design Guide 27 2022).

There are 5 main grades of stainless steel but the primary focus is on the three most common grades referenced in design code, Austenitic, Ferritic, and Duplex stainless steels. Each group provides different levels of corrosion resistance and strength.

2.1.1 Austenitic Stainless Steels

The most widely used alloys of austenitic stainless steel are based on additions of 17 to 18% chromium and 8 to 11% nickel. Austenitic stainless steels have a face-centred cubic atomic structure compared to the body-centred structure of carbon steels. This difference in crystal structure provides austenitic stainless steels higher ductility, formability, and corrosion resistance. They are also easily cold formed, and readily weldable. They tend to show increased toughness and lower risk of brittle failure over a wide range of temperatures. The cold forming process provides further strengthening but they can also be further enhanced by adding higher levels of chromium or molybdenum and nitrogen to provide extra resistance in corrosive environments. (AISC Design Guide 27 2022)

2.1.2 Ferritic Stainless Steels

Most ferritic stainless steels contain between 10.5% and 18% chromium with little to no nickel additions. They share the same body-centred cubic atomic structure as carbon steels and therefore generally considered less ductile, less formable, and less weldable than either the austenitic or duplex stainless steels. They can be strengthened by cold working but to a more limited degree than the austenitic stainless steels. They can still be used in a broad range of corrosive environments providing a good resistance to stress corrosion cracking, and their corrosion performance can be further enhanced by additions of molybdenum. (AISC Design Guide 27 2022)

2.1.3 Duplex Stainless Steels

Duplex stainless steels have a mixed microstructure of austenite and ferrite, and so are sometimes called austenitic-ferritic stainless steels. They typically contain 20 to 26% chromium, 1 to 8% nickel, 0.05 to 5% molybdenum, and 0.05 to 0.3% nitrogen. They provide higher strength levels than austenitic stainless steels and are suitable for a broad range of corrosive environments. Although duplex stainless steels have good ductility, their higher strength and greater springback makes them less formable than

austenitic stainless steels. These improved properties and reduced workability often means duplex steel is more expensive to implement. They can also be strengthened by cold working. They have good weldability and better resistance to chloride stress corrosion cracking and crevice corrosion than comparable austenitic stainless steels. (AISC Design Guide 27 2022)

2.2 Material Behaviour

The biggest difference in stainless steel compared to carbon steel is that it exhibits different stress strain behaviour. Mainly in the shape of the stress strain curve. Carbon steel has a linear elastic stress strain curve up to the yield stress while stainless steel is non linear and the stress strain curve is much more gradual with the exact yield stress being much harder to define. Therefore, stainless steel “yield” strengths are generally defined for a particular offset permanent strain (conventionally the 0.2% strain). After yielding Austenitic and duplex stainless steel exhibit significant additional strain hardening. Strength levels are further enhanced by cold forming. Figures 2.1 and 2.2 show stress strain curves for different kinds of stainless steels while figure 2.3 defines the 0.2 proof strength used for yielding.

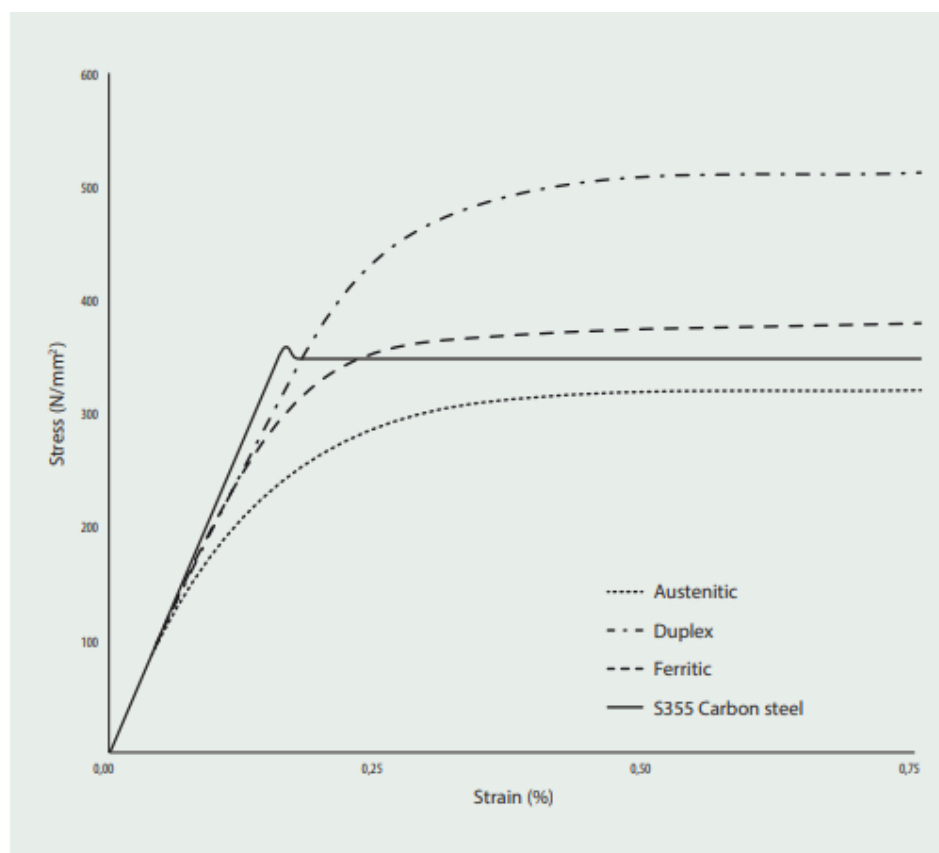


Figure 2.1: Stress-strain curves for stainless steel and carbon steel from 0 to 0,75% strain (Design Manual for Structural Stainless Steel 4th Edition)

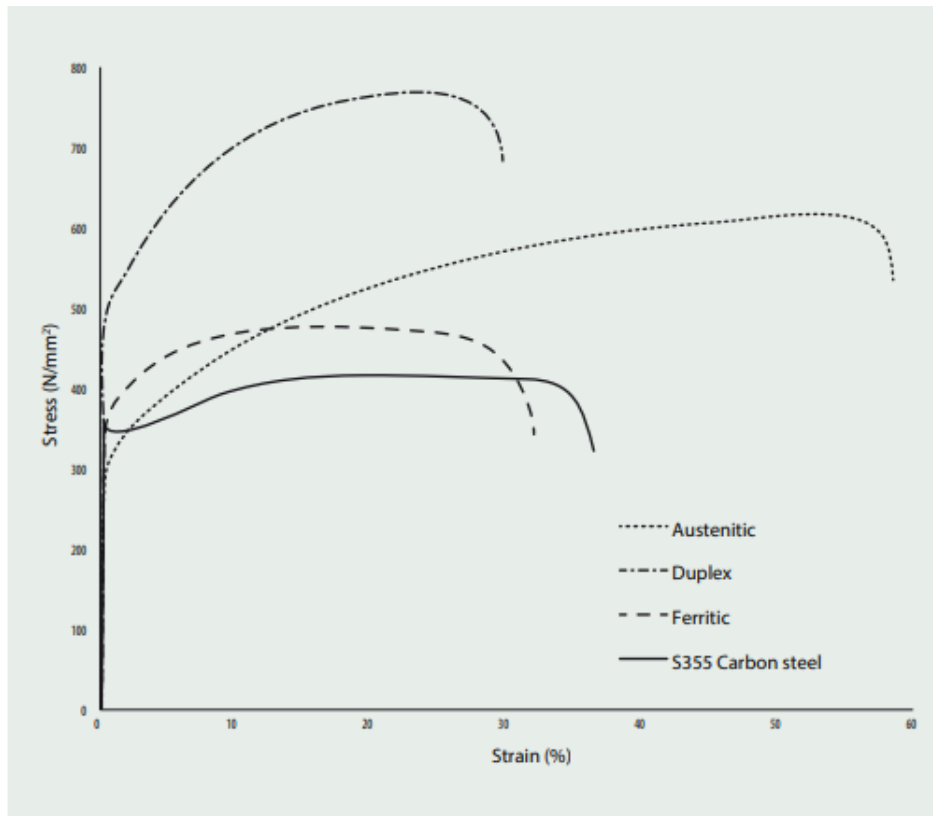


Figure 2.2: Full range stress strain curves for stainless steel and carbon steel (Design Manual for Structural Stainless Steel 4th Edition))

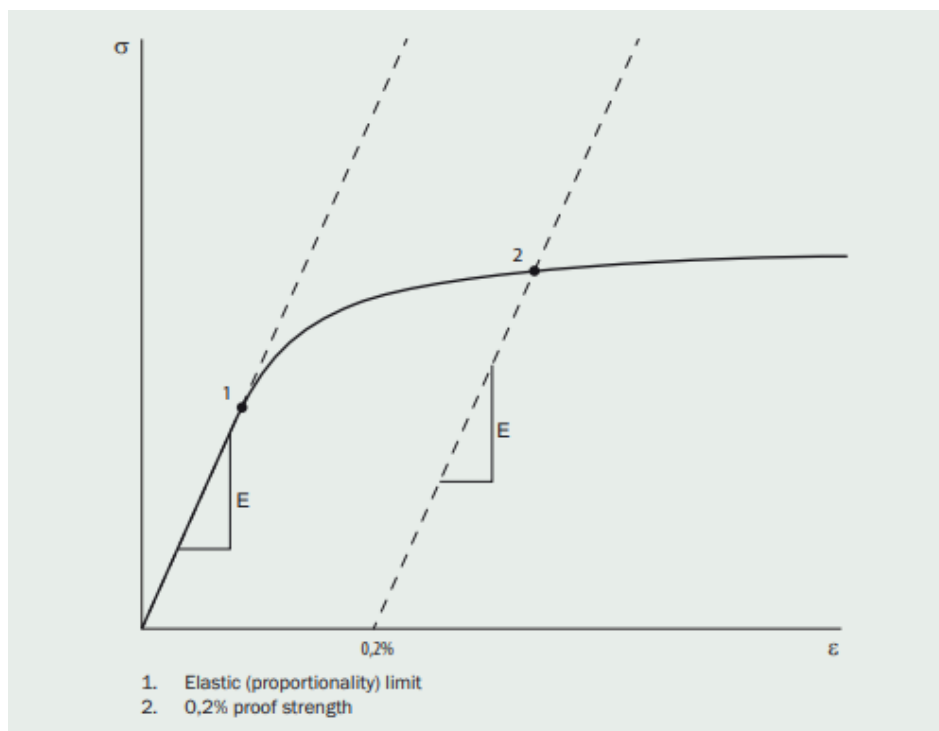


Figure 2.3: Definition of the 0,2% proof strength (Design Manual for Structural Stainless Steel 4th Edition)

2.3 Hollow Steel Sections

Hollow Steel Sections(HSS) are closed sections that come in rectangular, square or circular shapes and are fabricated by welding or rolling steel plates into hollow tubes, which are then cut to specific lengths and used as structural members. The closed shape of HSS provides excellent strength and resistance to bending, torsion, and compression.

2.3.1 Geometric considerations

Rectangular Hollow Sections(RHS) have an unequal width to height ratio while Square Hollow Sections(SHS) have equal width and height. Both account for contributions of a "flange" and a "web" and utilize effective widths and effective heights to determine the strength. These sections are created by welding open plate sections together to create closed a closed geometry. Because of this, plate behaviour is often controlling when accounting for local buckling. This differs from Circular Hollow Sections (CHS) with just dimensions consisting of a diameter and a thickness that uses an effective area approach instead. Circular sections exhibit shell behaviour and are primarily controlled by compression design.

Sizes are classified into cross sectional classes based on height to thickness ratios and can be either considered slender, stocky, or have an intermediate slenderness.

2.3.2 Manufacturing of HSS and Cold Forming

In order to properly create the shape, forming loads are usually required. Forming loads are the mechanical forces required to permanently shape flat steel into tubular profiles and these are generally higher for stainless steels than carbon steels and have different spring back properties. Spring back is the elastic recovery that occurs after a metal is bent or formed and the forming load is removed. Therefore in order to produce the proper shape, some over bending is required. Duplex stainless steels require approximately twice the forming loads used compared to austenitic materials, and as a result the possible range of duplex sections is more limited. Furthermore, because of the lower ductility in the duplex material, larger bending radii should be used. (Design Manual for Structural Stainless Steel 4th Edition)

Stainless steel is generally available in the "annealed condition". This process involves the material being heated up and maintained at that temperature for a time period, and then rapidly cooled. Annealing returns the material to a soft and workable state to allow for easily formed sections. However, strength levels of stainless steels, especially the austenitic grades, are enhanced during cold forming operations and during various fabrication techniques such as roller levelling, and flattening.

Associated with this enhancement, is a reduction in ductility but this normally is of slight consequence due to the initially high values of ductility. Especially for the austenitic stainless steels. The price of cold worked stainless steel is slightly higher than the equivalent annealed material, depending on the grade, product form and level of cold working. As stainless steel is cold worked, it tends to exhibit increasing non-symmetry of tensile and compressive behaviour and anisotropy (different stress-strain characteristics parallel and transverse to the rolling directions). The degree of asymmetry and anisotropy depends on the grade, level of cold working and manufacturing route. Structural sections of thickness above 3 mm are not made from

heavily cold worked material and the differences in stress strain behaviour for such sections due to non symmetry and anisotropy are not large. Non-linear material behaviour has a more significant effect. Anisotropy and non-symmetry are more significant in the design of lighter gauge, heavily worked sections.

For cold worked material, the compression strength in the longitudinal direction is less than the tensile strength in both the transverse and longitudinal directions. During the fabrication of a section by cold forming, plastic deformations occur which result in a significant increase in the 0,2% proof strength. A strength enhancement of about 50% is typical in the cold formed corners of cross sections; the strength of the material in the flat faces also increases. (Design Manual for Structural Stainless Steel 4th Edition)

2.4 Design codes

2.4.1 US AISC 370-21

The American Institute of Steel Construction(AISC) and the Steel Construction Institute in the UK have published AISC 370-21 which is the primary design code for the United States in structural stainless steel. It uses Load Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) for the design rules of structural members, connections, and welded members. It recognizes unique mechanical behaviour of stainless steels and modifies buckling/resistance formulations relative to carbon steel design and sets forth criteria for the design, fabrication, and erection of structural stainless steel buildings and other structures in most loading applications. It does not however take into consideration seismic applications.

In the United States, hot-rolled stainless steel structural members that typically have no reductions from cross-sectional buckling (due to the common use of thicker steel) do not have an approved standard. However significant design guidance is provided in AISC Design Guide 27.

2.4.2 US ASCE 8-22

The American Society of Civil Engineers provides similar guidance in its published ASCE 8-22. This standard is more specific for cold formed hollow and thin walled shapes, and provides an update to ASCE 8-02 to align the design of cold-formed stainless steel structures with the newest stainless steel research and practice. It integrates design methods such as the effective width method(EWM) and the direct strength method (DSM).

This standard also aims to coordinate with AISC 370; however there exist minor differences between the two. The biggest difference is that the scope of stainless steel alloys is broader in this standard than the current scope of AISC 370 and there is greater attention given here to cross-section buckling and issues related to the use of thin coils and steel sheets. This standard also provides corrosion requirements specific to stainless steel structures that greatly exceed those of typical carbon steel applications. (ASCE 8-22 preface)

2.4.3 Eurocode Generation 1 2020

En 1993 1-4 focuses on stainless steel structures and supplements EN1993 1-1 and 1-3 which for designs of carbon steel structures

2.4.4 Eurocode Generation 2 2025

EN1993 1-4 2025 is the second generation code that updates and expands on the previous generation of code while specifically addressing more advanced testing data and reliability calibration. It also introduces new design methods such as the continuous strength method (CSM).

3 Methods

As stated in the introduction the objective of the thesis is to investigate how differences in design across the four structural standards affect the global and local slenderness, compression resistance, bending resistance, and cross sectional classes for hollow section profiles.

After conducting a literature review of the different design standards and baseline information about stainless steel and its applications in structural engineering, the following methods were used for carrying out this objective:

- Development of a tool for a parametrized study of stainless steel HSS sections
- Design Calculations and a comparative analysis
- Verification using Finite Element Analysis

The initial comparisons of this study for the compression resistance of columns based on the American design codes ASCE 8-22 and AISC 370-21 were made by the Steel Construction Institute,(SCI) so it is necessary to refer to the publication "Design comparison of stainless steel HSS columns in AISC 370 and ASCE 8" for additional information. This study and how it relates to the holistic comparison of HSS for stainless steel across the four standards will be later discussed in chapter 5.

3.1 Parametric Study Overview

In order to develop a proper parametric study input parameters across the four standards-ASCE 8-22, AISC-370, Eurocode 1, and Eurocode 2-must remain consistent. The rectangular, square, and circular sections used remain consistent with each set of profiles having the same height width and thickness, or diameter and thickness for each analysis. The same structural grades of steel will also be used. Sections will be examined in both the annealed condition and when the strength is enhanced due to cold forming in fabrication. Each profile is expected to have the same resistance across each standard however due to various resistance and safety factors and other input parameters unique to a particular standard this may not be the case. A range of slenderness ratios from stocky sections to slender sections will also be investigated for additional accuracy verification.

3.1.1 Summary of Computational Procedure

The computational procedure consists of different steps that are applied to every section. The input parameters are taken from the provided geometries in the code and converted to metric units. Despite working with American design codes, all calculations will be performed in metric units. The same sectional properties are then calculated for every standard and applied to the code specific formulas. The final bending resistances are determined and are put into comparison plots for analysis.

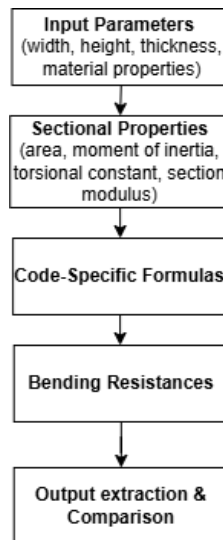


Figure 3.1: Summary of computational procedure used

3.1.2 Software used

initial design calculations were done using Mathcad Prime for an example section to verify the process and then the example was carried over to Microsoft Excel and scaled up to include all the section sizes and material properties.

3.1.3 Validation checks and processing of results

In order to ensure all resistances calculated are accurate, selected sections were verified with examples from the standards and cross checked with the previous study from SCI. FEM analysis will also be used for additional verification

3.2 Resistance Predications and Comparative Analysis

The bending resistance will be taken assuming the sections are beams and subject to only uniform bending. No considerations for shear or tension will be taken. The beam is taken as restrained with no risk of lateral torsional buckling. However determinations of global slenderness will still be made for comparisons. These resistance predictions will be determined using the following methods

- F_y as given in the product standard (i.e. in the annealed condition)
- F_y enhanced due to cold-forming
- Effective width method(EWM) and Direct strength method(DSM) in ASCE 8
- Continuous Strength Method (CSM)

3.2.1 Yield strength in the annealed condition

The bending resistance is calculated for each standard first considering the cross sectional properties and the yield strength of each steel grade in the annealed condition. No strength enhancements due to cold forming is applied. The material properties used for each standard is given in table 3.1 below.

Table 3.1: Material properties for stainless steel from various standards

		AISC	ASCE	Eurocode Gen 1	Eurocode Gen 2
Yield Strength F_y (MPa)	Austenitic EN 1.4301	205	205	Nominal Thickness t	
				$t \leq 8$ mm	$t \leq 13.5$ mm
	Duplex EN 1.4462	450	450	230	210
Modulus of Elasticity E (MPa)	Austenitic EN 1.4301	193 000	193 000	200 000	200 000
	Duplex EN 1.4462	200 000	200 000	200 000	200 000
Shear Modulus G (MPa)	Austenitic EN 1.4301	76 923	76 923	76 923	76 923
	Duplex EN 1.4462	76 923	76 923	76 923	76 923

The included range of cross sectional sizes considered were provided by AISC DG 27 2nd edition and listed in table 3.2 below. The sectional property tables are given in imperial units and were later converted to metric units.

Table 3.2: Range of included sections

Rectangular HSS			Round HSS		
Number of sections	Range of sizes (in.)	Range of thicknesses (in.)	Number of sections	Range of diameters (in.)	Range of thicknesses (in.)
274	1 – 32	0.06 – 0.625	128	1 – 7.5	0.032 – 0.375

The cross sectional properties needed are the area of the section and the moment of inertia about the major axis of bending. The properties were calculated considering the effects of round corners where the radius of the outer corner is equal to twice the thickness of the member. Although considerations for global lateral torsional buckling were not taken, the torsional constant and moment of inertia about the minor axis were calculated as well to find the global slenderness value.

$$\begin{aligned}
 A_{ri} &= \left(1 - \frac{\pi}{4}\right) * r_i^2 \\
 A_{re} &= \left(1 - \frac{\pi}{4}\right) * r_e^2 \\
 A_g &= (h * b) - (b - 2 * t) * (h - 2 * t) - 4 * A_{re} + 4 * A_{ri} \\
 h_{ri} &= \frac{h - 2t}{2} - \left(\frac{10 - 3\pi}{12 - 3\pi}\right) r_i \\
 b_{ri} &= \frac{b - 2t}{2} - \left(\frac{10 - 3\pi}{12 - 3\pi}\right) r_i \\
 h_{re} &= \frac{h}{2} - \left(\frac{10 - 3\pi}{12 - 3\pi}\right) r_e \\
 b_{re} &= \frac{b}{2} - \left(\frac{10 - 3\pi}{12 - 3\pi}\right) r_e \\
 I_e &= \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3(12 - 3\pi)}\right) r_e^4 \\
 I_i &= \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3(12 - 3\pi)}\right) r_i^4 \\
 I_y &= \frac{b h^3}{12} - \frac{(b - 2t)(h - 2t)^3}{12} - 4 \left(I_e + A_{re} h_{re}^2\right) + 4 \left(I_i + A_{ri} h_{ri}^2\right)
 \end{aligned}$$

$$I_z = \frac{h b^3}{12} - \frac{(h - 2t)(b - 2t)^3}{12} - 4 \left(I_e + A_{re} b_{re}^2 \right) + 4 \left(I_i + A_{ri} b_{ri}^2 \right)$$

where

h = section height, mm

b = section width, mm

t = section thickness, mm

$r_i = t$ is the internal radius of the section, mm

$r_e = r_i + t$ the external radius of the section, mm

for circular sections the area and moment of inertia are calculated as follows:

$$A_g = \frac{\pi}{4} \cdot \left(D^2 - (D - 2 \cdot t)^2 \right)$$

$$I = \frac{\pi}{4} \cdot \left(\left(\frac{D}{2} \right)^4 - \left(\frac{D}{2} - t \right)^4 \right)$$

D is the outer diameter of the section, mm

Global slenderness for Lateral Torsional buckling in AISC is first calculated by

$$\lambda_{LT} = \sqrt{\frac{M_p}{M_{Cr}}}$$

where M_p is the plastic moment capacity and M_{cr} is the critical buckling moment which is dependent on the unbraced length of the member. Localized properties of the cross section through cross sectional classification are split up into classes for the flange and classes for the web. The worst case scenario is considered. The classification limits for both AISC and Eurocode are shown in table 3.4 below.

Table 3.3: Cross section classification table

	AISC		Eurocode	
	Limits	Classification	Limits	Classification
Classification of flange	$\lambda_f < \lambda_{pf}$	Compact	$\lambda_f \leq 33\varepsilon$	CSC class 1
			$\lambda_f \leq 35\varepsilon$	CSC class 2
	$\lambda_f \leq \lambda_{rf}$	Non-Compact	$\lambda_f \leq 35\varepsilon$	CSC class 3
	$\lambda_f > \lambda_{rf}$	Slender	$\lambda_f > 35\varepsilon$	CSC class 4
Classification of web	$\lambda_w < \lambda_{pw}$	Compact	$\lambda_w \leq 72\varepsilon$	CSC class 1
			$\lambda_w \leq 76\varepsilon$	CSC class 2
	$\lambda_w \leq \lambda_{rw}$	Non-Compact	$\lambda_w \leq 90\varepsilon$	CSC class 3
	$\lambda_w > \lambda_{rw}$	Slender	$\lambda_w > 90\varepsilon$	CSC class 4

where

$$\lambda_f = \frac{b_{flange}}{t}$$

and

$$\lambda_w = \frac{b_{web}}{t}$$

b_{flange} is the flat width of the flange equivalent to $b - 3t$ and b_{web} is the flat height of the web equivalent to $h - 3t$. with the limits defined as:

$$\lambda_{pf} = 1.17 \sqrt{\frac{E}{F_y}}$$

$$\lambda_{rf} = 1.24 \sqrt{\frac{E}{F_y}}$$

$$\lambda_{pw} = 2.54 \sqrt{\frac{E}{F_y}}$$

$$\lambda_{rw} = 3.01 \sqrt{\frac{E}{F_y}}$$

$$\epsilon = \sqrt{\frac{235}{F_y} * \frac{E}{210000}}$$

Moment resistance for each standard is determined based on these classes and by taking the smallest value from either the flange or the web and is listed in table 3.4

Table 3.4: Moment resistance calculation table according to cross sectional class

	AISC		Eurocode	
	Classification	Moment Resistance	Classification	Moment Resistance
flange	Compact	M_p	CSC class 1	M_p
			CSC class 2	M_p
	Non-Compact	$\min \left(M_p, M_p - (M_p - M_y) \cdot \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right)$	CSC class 3	M_y
	Slender	$F_y * S_{eff}$	CSC class 4	$F_y * W_{ef}$
web	Compact	M_p	CSC class 1	M_p
			CSC class 2	M_p
	Non-compact	$\min \left(M_p, M_p - (M_p - M_y) \cdot \frac{\lambda_w - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right)$	CSC class 3	M_y
	Slender	$\min(F_y * S_y * R_{pg}, S_{ef} * F_y * R_{pg})$	CSC class 4	$F_y * W_{ef}$

where $M_p = F_y * W_{pl}$ the plastic moment, kNm

$M_y = F_y * W_{el}$ the elastic moment, kNm

$S_y = W_y$ the elastic section modulus, mm^3

$S_{ef} = W_{ef}$ the effective section modulus, mm^3

R_{pg} = bending strength reduction factor

3.2.2 Yield strength enhanced due to cold-forming

Each standard provides guidelines for enhancing the yield strength F_y , due to cold forming. In the American codes ASCE 8 and AISC370-21 the average yield stress $F_{y,a}$ of stainless steel cold-form HSS is determined on one of the following methods

1. Full section tensile tests according to the American Society for Testing Materials (ASTM) A370
2. Stub column tests for the American Iron and Steel Institute (AISI) S902
3. Computed using equations provided by each standard.

The enhanced yield strength due to cold forming was determined using the computational method with the equations below for each standard and compared to the annealed condition.

According to AISC section B4.3:

$$F_{y,avg} = \frac{F_{y,corner}A_{corner} + F_{y,wall}(A_g - A_{corner})}{A_g} \leq F_u \quad (3.1)$$

for rectangular HSS or

$$F_{y,avg} = F_{y,rd} \quad (3.2)$$

for round HSS

where

$A_{corner} = (\pi * t)(2 * r + t) + 16 * t^2$ which is the total corner area including a region of length $2t$ extending around the perimeter of the cross section on both sides of each corner, mm^2

A_g = gross section of member, mm^2

$F_{y,corner} = 0.85 \frac{F_y}{\varepsilon_y^n} * (\varepsilon_{corner} + \varepsilon_y)^n$ and $F_y \leq F_{y,corner} \leq F_u$ which is the tensile yield stress of corners of RHS (MPa)

$F_{y,rd} = 0.85 \frac{F_y}{\varepsilon_y^n} * (\varepsilon_{rd} + \varepsilon_y)^n$ and $F_y \leq F_{y,rd} \leq F_u$ which is the tensile yield stress of CHS (MPa)

$F_{y,wall} = 0.85 \frac{F_y}{\varepsilon_y^n} * (\varepsilon_{wall} + \varepsilon_y)^n$ and $F_y \leq F_{y,wall} \leq F_u$ which is the tensile yield stress of the flat parts of RHS (MPa)

$$n = \frac{\log\left(\frac{F_y}{F_u}\right)}{\log\left(\frac{\varepsilon_y}{\varepsilon_u}\right)}$$

$\varepsilon_{corner} = \frac{t}{2(2r+t)}$ strain induced in the corner of RHS

$\varepsilon_{rd} = \frac{t}{2(2D+t)}$ strain induced in CHS

$\varepsilon_u = 1 - \frac{F_y}{F_u} \leq f$ the ultimate strain

$\varepsilon_f = 50$ mm, the minimum elongation after rupture

$\varepsilon_{wall} = \frac{t}{900} + \frac{\pi t}{2(B+H+t)}$ the strain induced in the flat parts of RHS

$\varepsilon_y = 0.002 + \frac{F_y}{E}$

B = overall width of RHS, mm

H = overall height of RHS, mm

D = overall diameter of CHS, mm

r = internal radius of the corner which is taken as 2t, mm

E = modulus of elasticity of stainless steel (MPa)

F_u = minimum tensile stress (MPa)

F_y = minimum yield stress (MPa)

For Eurocode generation 1 (EN 1993 1-4), table 2-2 provides nominal strength values for stainless steels in the cold worked condition for various grades while in generation 2 (EN 1993 1-4:2025) table 5.3 provides equations for calculation of $F_{y,avg}$ that is calculated similarly to AISC Section B4.3

According to ASCE 8-22 section 1.3.5.2

the strength increase from cold work is calculated by

$$F_{ya} = C * F_{yc} + (1 - C) * F_y \leq F_u$$

where

$F_{yc} = B_c * F_u$ the tensile yield stress of corners

$$B_c = 0.85 \left(\frac{\varepsilon_{corner}}{\varepsilon_y} + 1 \right)^n$$

C = ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange for flexural members

3.2.3 Effective Width Method & Direct Strength Method

According to section 6.3.1 in ASCE 8-22 the effective width method (EWM) calculates the nominal flexural strength for local buckling according to equation 3.3

$$M_{nl} = S_{ec} * F_n \leq S_{et} * F_y \quad (3.3)$$

where

S_{ec} = effective section modulus calculated at the extreme fiber compressive stress of F_n

F_n = the adjusted global flexural yield stress defined in section 6.2

S_{et} = effective section modulus calculated at the extreme fiber tension stress of F_y

In section 6.3.2 the direct strength method (DSM) calculates the nominal flexural strength, M_{nl} for local buckling. This method differs from the effective method and considers the interaction of local buckling and global buckling using the following equations

For $\lambda_l \leq 0.667$:

$$M_{nl} = M_{ne} \quad (3.4)$$

For $\lambda_l > 0.667$:

$$M_{nl} = \left[1 - 0.2 \left(\frac{M_{crl}}{M_{ne}} \right)^{0.4} \right] \left(\frac{M_{crl}}{M_{ne}} \right)^{0.4} M_{ne} \quad (3.5)$$

where

$$\lambda_l = \sqrt{\frac{M_{ne}}{M_{crl}}}$$

M_{ne} = Nominal flexural strength for lateral torsional buckling

M_{crl} = critical local buckling moment

3.2.4 Continuous Strength Method

The continuous strength method (CSM) that is found in ASIC 370-21 and the second generation of Eurocode is an alternate design method for determining the strength of HSS members. It is permitted to be used when the given section satisfies the slenderness requirements for each standard. It incorporates a bilinear material model with linear strain hardening. This means that stress will increase beyond the yield strength of the material with increasing strain up to failure.

In section 2.3 of appendix 2 in ASIC 370-21 and similarly in B.5 of Eurocode 1-4:2025 The normalized deformation capacity is defined as the ratio of the cross section failure strain (ε_{csm}) to the yield strain (ε_y). It is determined based on the geometry of the section as follows:

For RHS

The CSM method in AISC is applicable for flexural members if

$$h/b \leq 3$$

and

$$\lambda_l \leq 1.6$$

where L_b is the length of the member, and L_p is the limiting laterally unbraced length for the limit state of yielding. λ_l is the cross-sectional slenderness defined by

$$\lambda_l = \sqrt{\frac{F_y}{F_{el}}} \quad (3.6)$$

For RHS

$$F_{el} = \frac{k\pi^2 Et^2}{12(1 - \nu^2)b^2} \quad (3.7)$$

For CHS

$$F_{el} = \frac{E}{\sqrt{3(1 - \nu^2)}} \frac{2t}{D} \quad (3.8)$$

where

D = cross section diameter

b = width of the most slender element of the cross section

t = design thickness of the most slender element for RHS and the design wall thickness for CHS

ν = Poissons ratio = 0.3

k = plate buckling coefficient

if the standard is applicable the deformation capacity ($\frac{\varepsilon_{csm}}{\varepsilon_y}$) is determined by

if $\lambda_l \leq 0.68$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \min\left(\frac{0.25}{\lambda_l^{3.6}}, \Lambda, \frac{0.10(1 - \frac{F_y}{F_u})}{\varepsilon_y}\right) \quad (3.9)$$

if $\lambda_l > 0.68$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.222}{\lambda_l^{1.05}}\right) * \frac{1}{\lambda_l^{1.05}} \quad (3.10)$$

For CHS

According to table A-2.1.1 for circular sections the standard is applicable for flexural members if

$$\lambda_l \leq 0.6$$

The deformation capacity can be determined by

if $\lambda_l \leq 0.30$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \min\left(\frac{4.44 * 10^{-3}}{\lambda_l^{4.5}}, \Lambda, \frac{0.10 * (1 - \frac{F_y}{F_u})}{\varepsilon_y}\right) \quad (3.11)$$

if $\lambda_l > 0.30$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \left(1 - \frac{0.224}{\lambda_l^{0.342}}\right) * \frac{1}{\lambda_l^{0.342}} \quad (3.12)$$

where $\Lambda = 15$

The moment capacity for HSS is then determined according to section 2.6 for flexural strength as follows

if $\frac{\varepsilon_{csm}}{\varepsilon_y} < 1.0$

$$M_n = \frac{\varepsilon_{csm}}{\varepsilon_y} M_y \quad (3.13)$$

if $\frac{\varepsilon_{csm}}{\varepsilon_y} > 1.0$

$$M_n = M_p * \left(1 + \frac{E_{sh}}{E} \frac{S}{Z} \left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1\right) - \frac{\left(1 - \frac{S}{Z}\right)}{\left(\frac{\varepsilon_{csm}}{\varepsilon_y}\right)^\alpha}\right) \quad (3.14)$$

where

$$M_p = F_y Z$$

$$M_y = F_y S$$

S = elastic section modulus about axis of bending (mm^3)

Z = plastic section modulus about axis of bending (mm^3)

α = bending coefficient from Table A-2.6.1

3.3 Safety Factors

every standard provides safety factors that reduces the calculated resistance by a flat amount in order to ensure a safe design capable of handling additional unexpected loading. In the American design codes the limit state philosophy known as "Load Resistance Factor Design"(LRFD) is used and in Eurocode partial safety factors are

used instead. These factors vary for different loading conditions and cross section types which will guarantee some level of difference in final resistances. These factors are summarized in the table below. To make better comparisons and better understand the differences between standards these design factors will not be considered.

Eurocode		ASCE 8-22 and AISC 370-21	
$\gamma M0$	1.10	AISC 370-21 (ϕ)	0.90
$\gamma M1$	1.10	ASCE 8-22 RHS (ϕ)	0.90
$\gamma M2$	1.25	ASCE 8-22 CHS (ϕ)	0.95

3.4 Predication verification using Finite Element Analysis

In order to perform additional verifications using finite element analysis. The software ABAQUS will be used to model buckling behaviour and determine actual failure loads of selected example sections and compared with the predicted resistances across the four standards. This will provide additional insight about the accuracy of each design standard and their most practical applications.

4 Results

4.1 ASCE 8 (EWM vs DSM)

ASCE 8 local buckling of CHS beams is addressed through the calculation of an effective area. Neither EWM nor DSM is applicable here. Because of this, the ASCE 8 comparison will focus only on square and rectangular sections.

The DSM takes into consideration the interaction between global and local slenderness, so the bending resistance of the section according to DSM is directly related to the unbraced length of the member, L_y . When L_y is small compared to L_u as shown in 4.1 the beam is stocky. For this given global slenderness, The EWM will be about 60% smaller compared to the direct strength method for cross sections with a small local slenderness. As L_y increases, the resistance predictions of the two methods become closer together until it reaches the point where L_y is equal to L_u as shown in 4.2 and the resistances are the same for the two methods for these same stocky sections. At larger cross-sectional slenderness values, the effective width method will be greater than the direct strength method for duplex grades. The greater bending resistances predicted by the EWM can be attributed to the fact that the DSM buckling curve is lower than the EWM buckling curve. Figure 4.3 shows that when the resistances are plotted against the global beam slenderness values for each section with varying unbraced lengths. The importance of the role that global slenderness plays in DSM is further emphasized.

Another difference is that in DSM the reduction in strength is applied to the entire cross section where with EWM it is applied only to the flat portions of the cross-sectional elements which is consistent with plate-like behaviour.

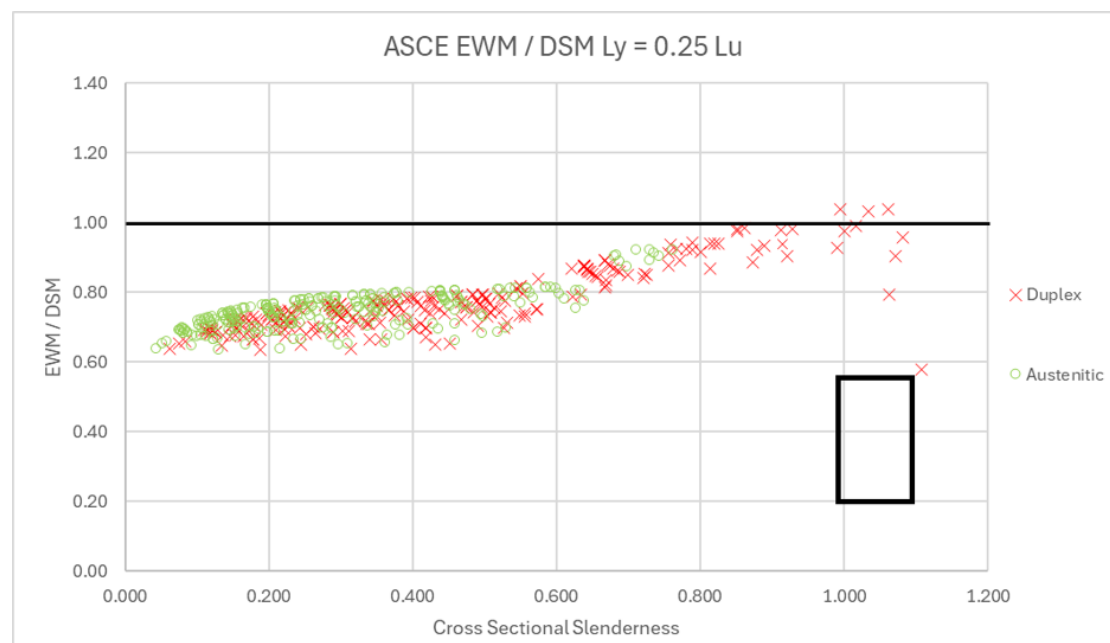


Figure 4.1: ASCE 8-22 comparison of effective width method and direct standard method for $L_y = 0.25 L_u$

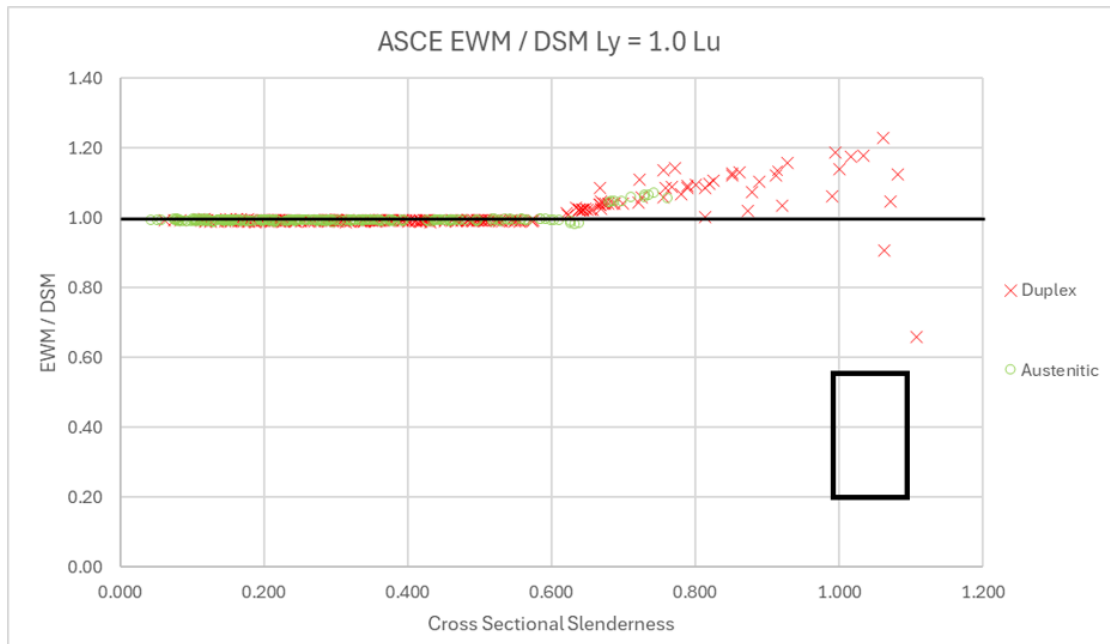


Figure 4.2: ASCE 8-22 comparison of effective width method and direct standard method for $L_y = L_u$

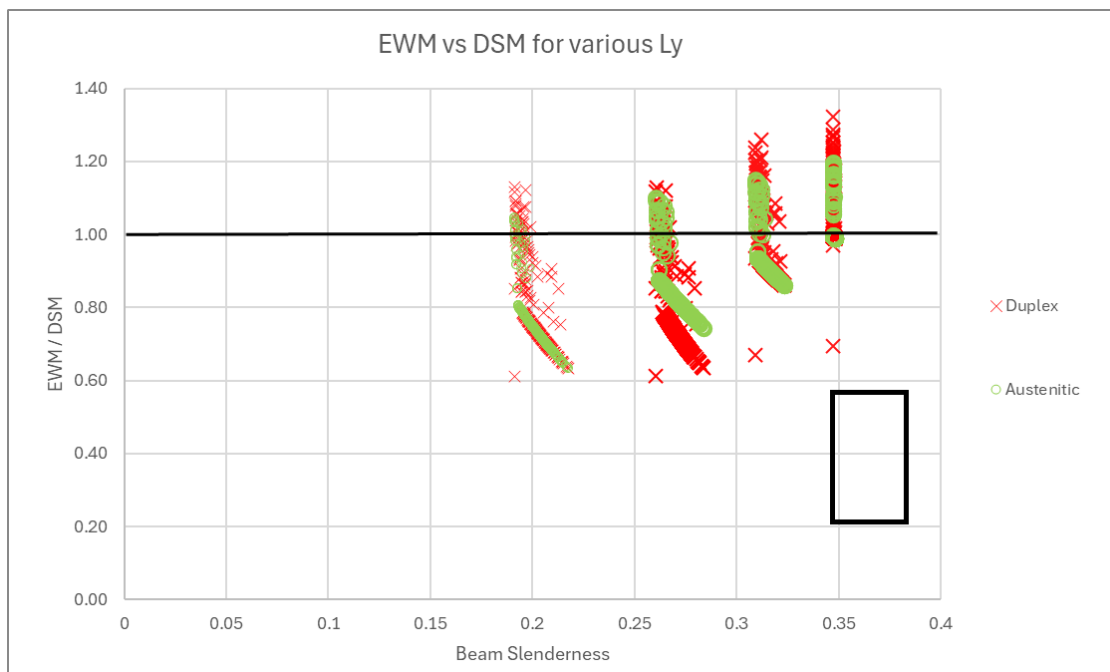


Figure 4.3: Global slenderness comparison of the effective width method vs direct strength method

4.2 AISC 370-21 Comparisons

4.2.1 Nominal Thickness vs. Design Thickness

The design thickness value used in AISC 370-21 is 95% of the original nominal thickness value of the section. Figure 4.4 shows that this should result in about a 5% increase in strength for the nominal thickness resistances compared to the design thickness resistances. This behaviour appears for stockier sections both rectangular and circular however some slender duplex sections will be pushed into smaller cross sectional classes by the reduced thickness which results in a smaller bending resistance when using the design thickness. This decrease for slender sections is about 2% less for some rectangular sections and 1% for circular sections. The change due to increased slenderness is not that significant however using the design thickness can still result in a safer more conservative design.

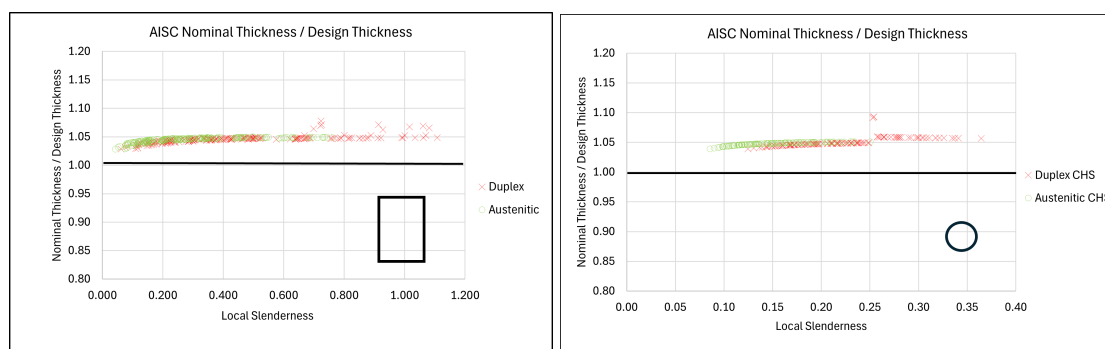


Figure 4.4: AISC 370-21 Comparison of nominal vs design thickness

4.2.2 Nominal Thickness vs Continuous Strength Method

The effective width method for beams in flexure from chapter F of AISC 370-21 using the nominal thickness results in a bending resistance that is about 10-15% lower than the continuous strength method for stockier rectangular sections, but only about 5-10% lower for circular sections. For increasing slenderness rectangular sections tend to decrease in bending resistance for the standard approach compared to CSM and show more variability in results due to plate like behaviour. While circular sections form a more uniform curve and since stresses are more equally distributed the resulting bending resistances using the standard approach will be higher than CSM for more slender sections.

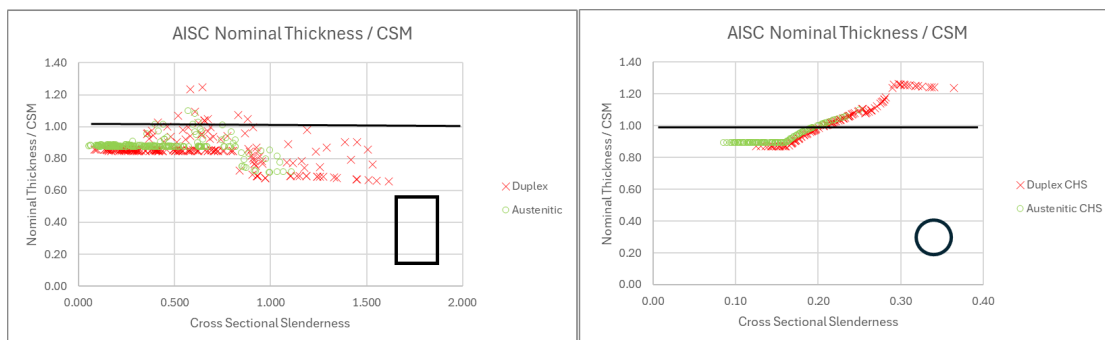


Figure 4.5: Effective width method for nominal thickness compared to the continuous strength method

4.3 ASCE 8-22 vs AISC 370-21 (based on F_y)

Figure 4.6 shows that all cross sections for both rectangular and circular using the EWM in ASCE 8-22 is more conservative than using the EWM in AISC 370. For more slender rectangular sections in duplex steel however, where local buckling will likely occur the post yield strength for both standards become more similar. For all circular sections ASCE will result in a consistent 20% decrease in strength. When compared to CSM, EWM in ASCE shows a similar trend to EWM in AISC with the bending resistances for rectangular sections being lower than CSM while circular sections have the same uniform curve as in figure 4.5. The similarity between the two effective width methods compared to CSM occurs because both standards use similar buckling curves. This pattern is consistent for DSM as well shown in figure 4.8. Overall CSM will result in larger less conservative predictions across both standards.

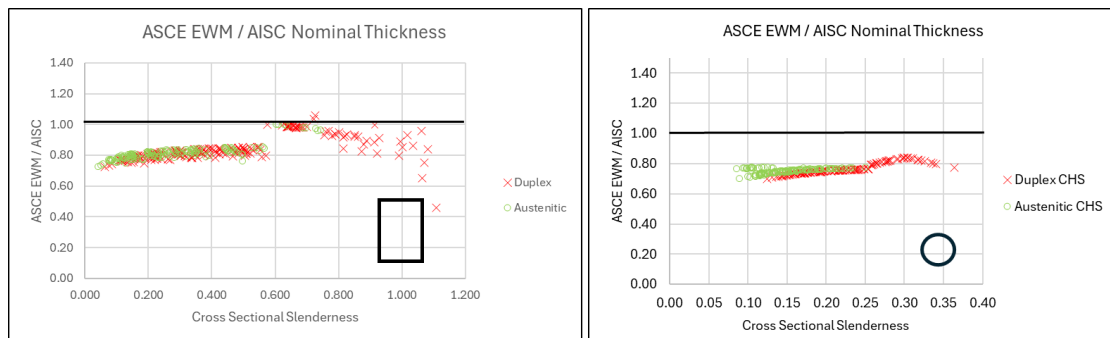


Figure 4.6: ASCE 8-22 compared with AISC 370-21

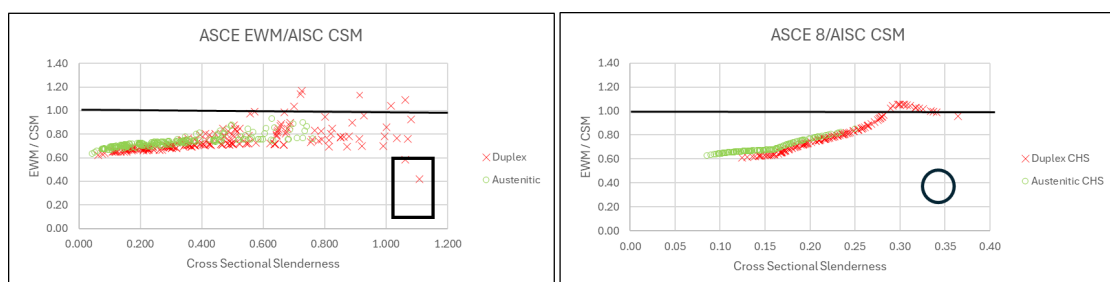


Figure 4.7: ASCE 8-22 effective width method compared to AISC 370-21 continuous strength method

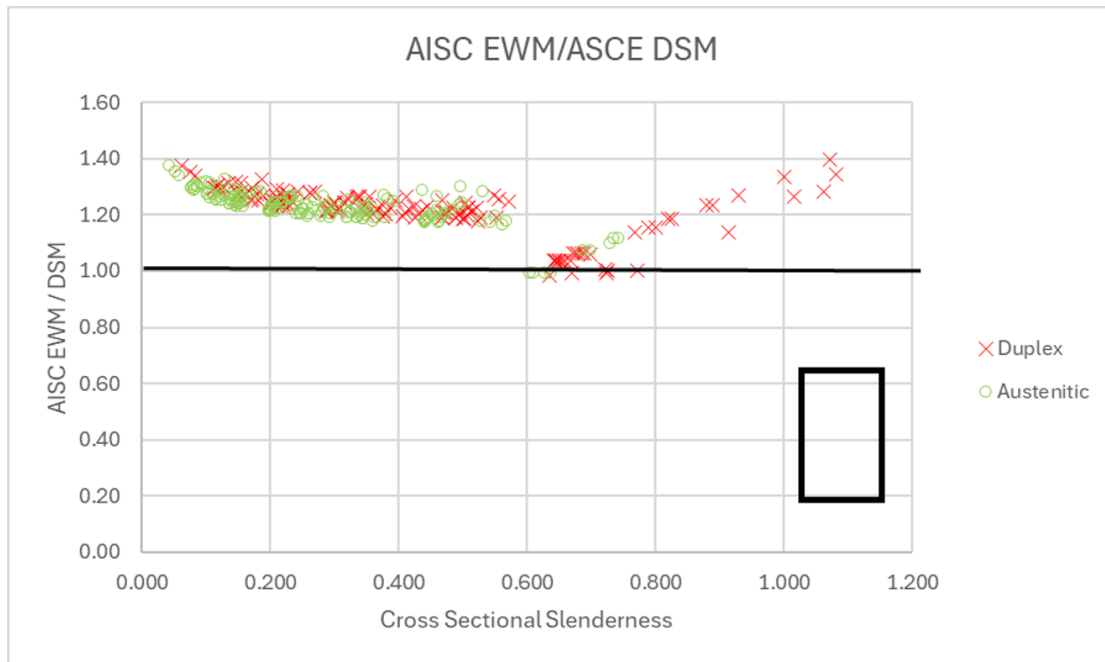


Figure 4.8: AISC 370-21 effective width method compared to the ASCE 8-22 direct strength method

4.4 ASCE 8 vs AISC 370 (based on $F_{y.avg}$)

4.4.1 Yield Strength Comparisons

Both American standards provide guidelines for how to increase yield strength of a section due to cold working processes during fabrication. Eurocode generation 1 does not provide any guidelines and the ones given in Eurocode generation 2 are the same as AISC 370 so it will not be compared. For stockier rectangular sections in austenitic steel ASCE 8 results in about a 10-20% increase in strength than AISC. Duplex steel is much smaller and the two standards become equal much quicker for this material. Both steel grades will move towards equal yield strengths for both standards with increasing local slenderness. For circular sections, ASCE 8 does not provide guidelines for calculating the increased yield strength, $F_{y.avg}$ and therefore figure 4.9 compares the standard yield strength F_y in ASCE 8, against $F_{y.avg}$ in AISC 370. The comparison shows that the increase in yield strength is much less conservative for austenitic steel grades and the difference between F_y and $F_{y.avg}$ can be as much as 35% for stocky sections. For duplex the increase in yield strength is significantly less. For stockier sections with a slenderness of 0.2 or less the difference is less than 5% which is not significant. The more slender sections in AISC show an underestimation of yield strength by up to 10% and will result in a more conservative estimate of overall bending resistance.

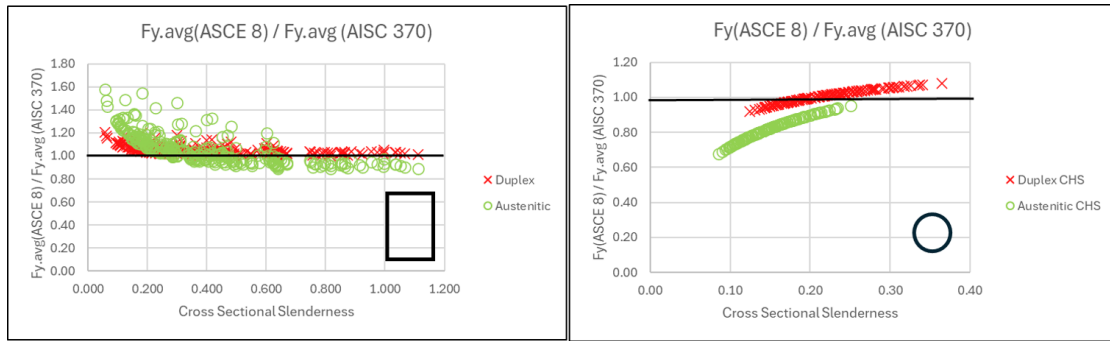


Figure 4.9: Comparison of $F_{y,avg}$ for ASCE and AISC

4.4.2 Bending Resistance Comparisons

Bending resistance comparisons become much more consistent when the yield strength enhancements due to cold working are introduced. Figure 4.10 shows the ASCE comparison and both austenitic and duplex steels highlight this consistent trend. The EWM is more conservative than the DSM but this difference decreases with increasing slenderness. For ASCE EWM vs AISC in figure 4.11 the resistances are similar and the variation is less than 5% and can be considered non impactful. There are some duplex sections that do not follow this trend however it is likely due to those sections falling under a worse cross sectional class causing the resistance for AISC to drop. Figure 4.12 shows the comparison of DSM versus CSM and DSM shows to be higher for stockier sections up but the two methods converge for increasing slenderness. Finally, figure 4.13 highlights that both AISC methods are quite comparable with a consistent 5-10% difference between them across all sections. Overall resistances predicted by AISC align much closer to resistances predicted by ASCE when using an enhanced yield strength and the DSM is observed to be the most efficient method across all standards for stockier sections.

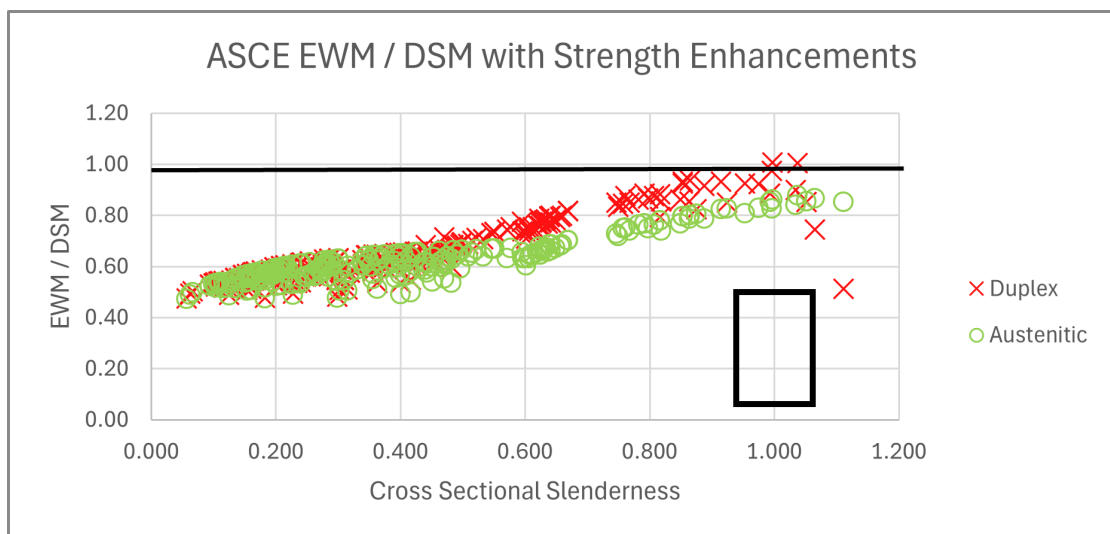


Figure 4.10: ASCE effective width method compared to the direct strength method including the yield strength enhancements

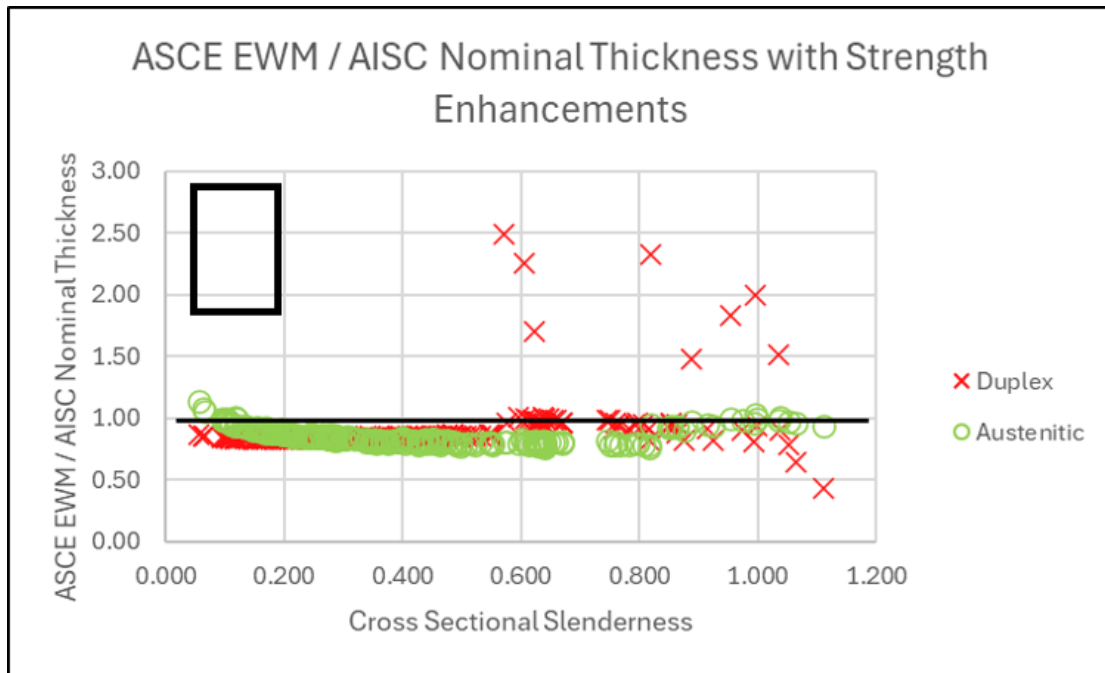


Figure 4.11: ASCE effective width method compared to AISC including the yield strength enhancements

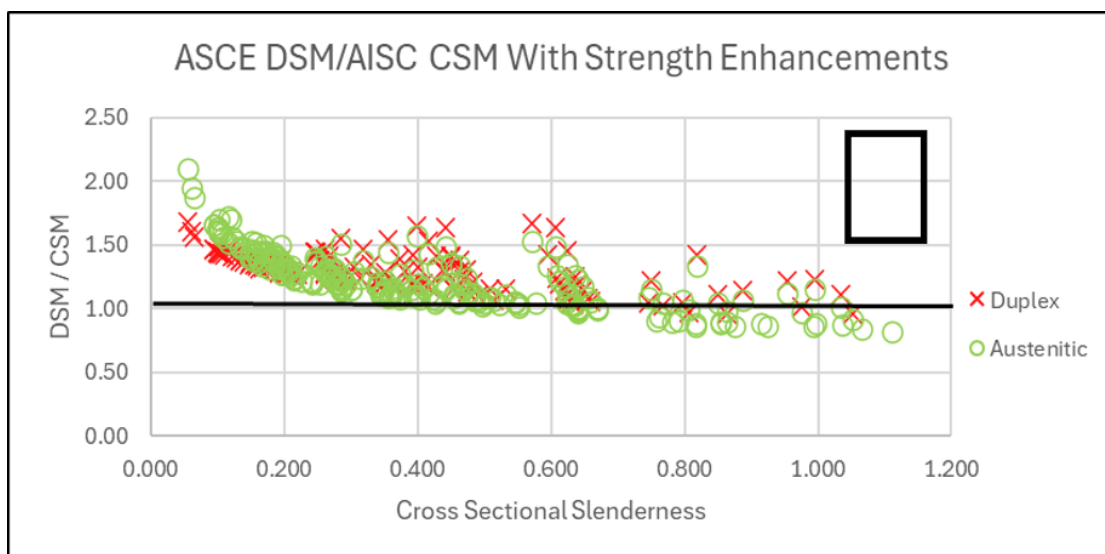


Figure 4.12: ASCE direct strength method compared to AISC continuous strength method include yield strength enhancements

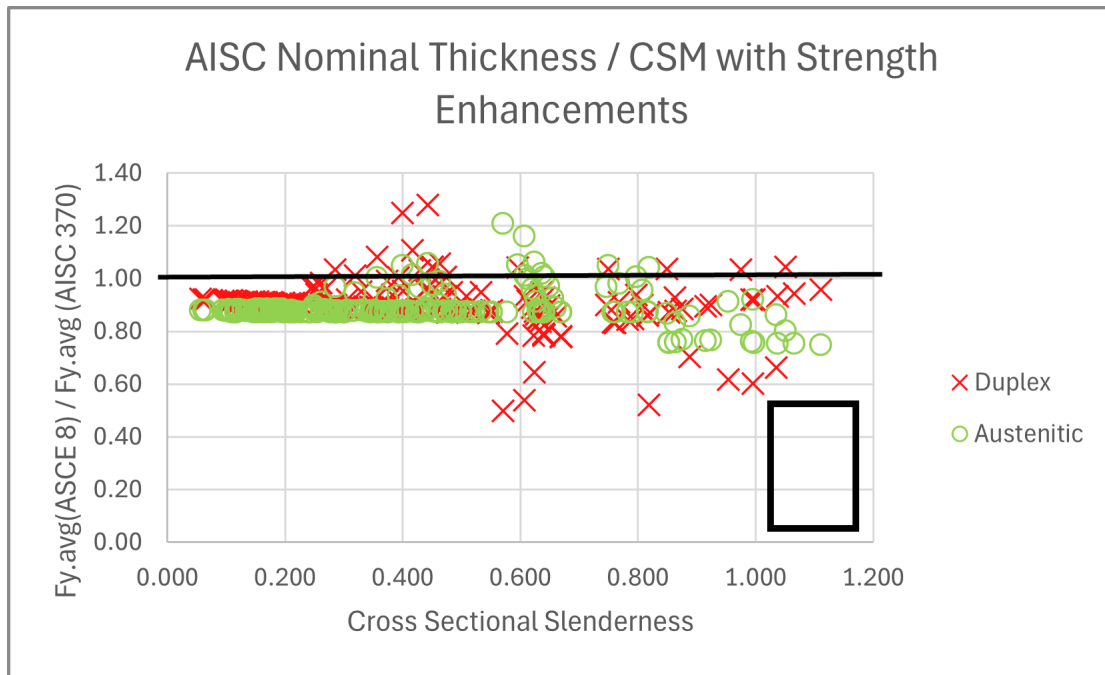


Figure 4.13: AISC comparison of effective width method with nominal thickness to the continuous strength method with yield strength enhancements

4.5 Eurocode Comparisons

4.5.1 Rectangular Sections

EN1993 1-4 is considered to be generation 1 and was last amended in 2020. In 2025, generation 2 was introduced however changes for the standard method appear to be very small with most sections showing the same resistance for generation 1 and generation 2 while other sections only show a 1-2% difference as highlighted in figure 4.14. When comparing AISC370-21 and Eurocode, the results are also mostly identical particularly for stockier sections. More slender sections using duplex steel still show a drop of about 10% however likely due to the difference in yield stress (F_y) used by each standard. These similarities can be attributed to the fact that Eurocode uses a similar process of cross sectional classification and determination of moment for local buckling without much consideration for lateral torsional buckling. This is in contrast to the two ASCE methods as shown in figure 4.16 however, as global buckling and the unbraced length of the member plays a much bigger role. The continuous strength method comparison in figure 4.17 shows that AISC appears to be overall less conservative but large variations in the trend across sections suggest that this particular method is not well developed to capture the plate like behaviour used in rectangular sections.

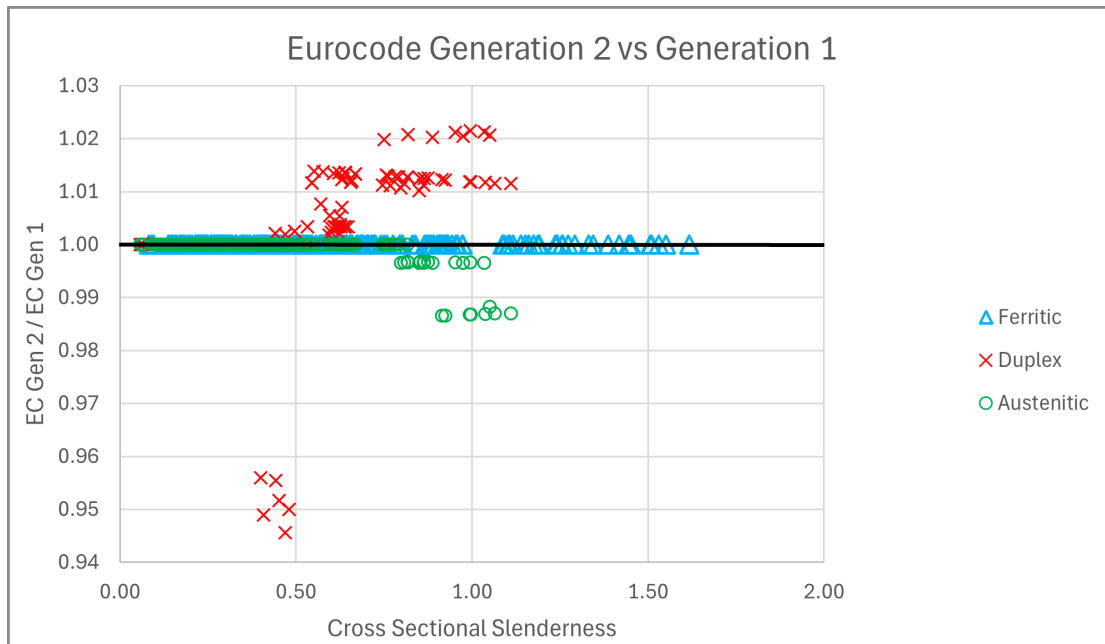


Figure 4.14: EN1993 1-4 generation 1 compared with EN1993 1-4 (2025) generation 2

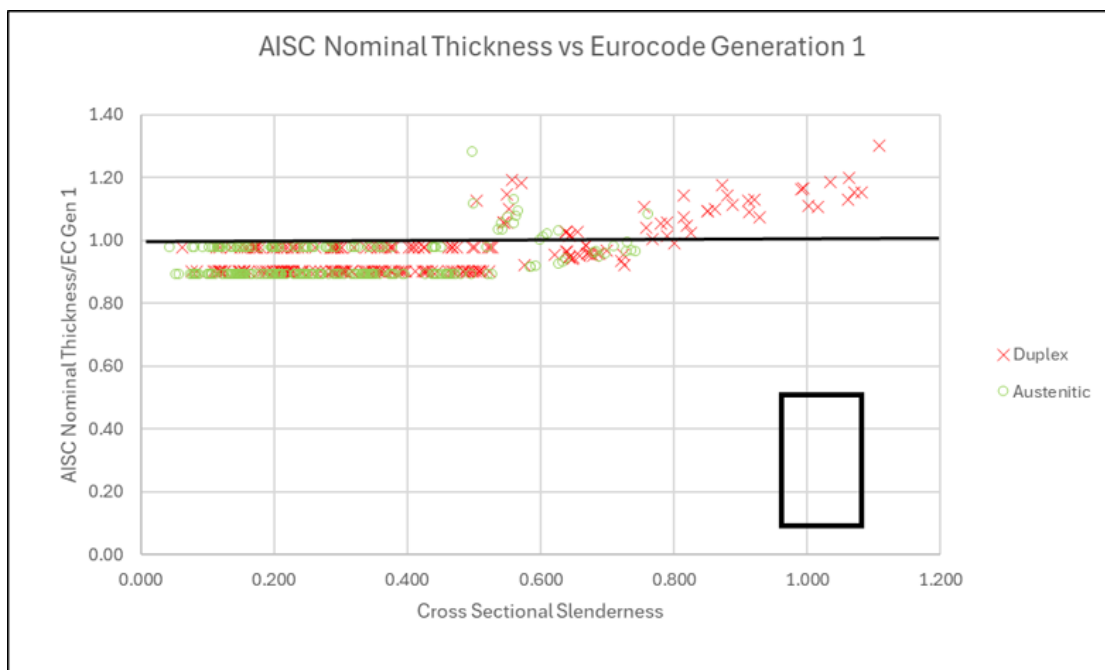


Figure 4.15: AISC effective width method with nominal thickness to EN1993 1-4 generation 1

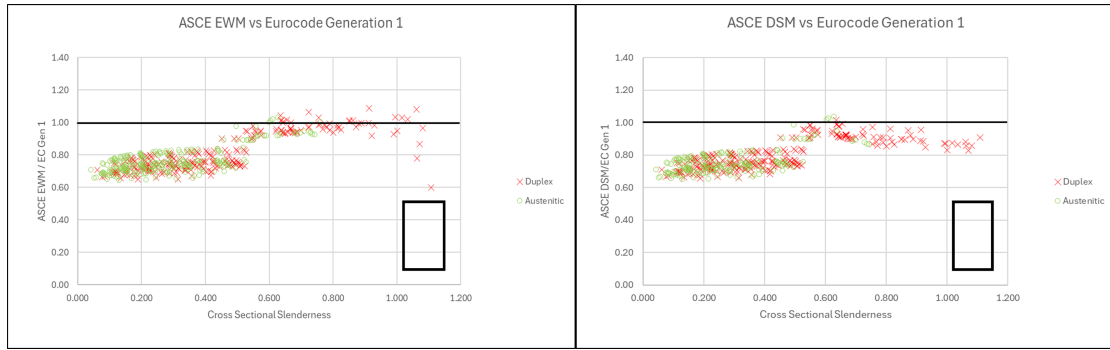


Figure 4.16: ASCE methods compared with EN1993 1-4 generation 1

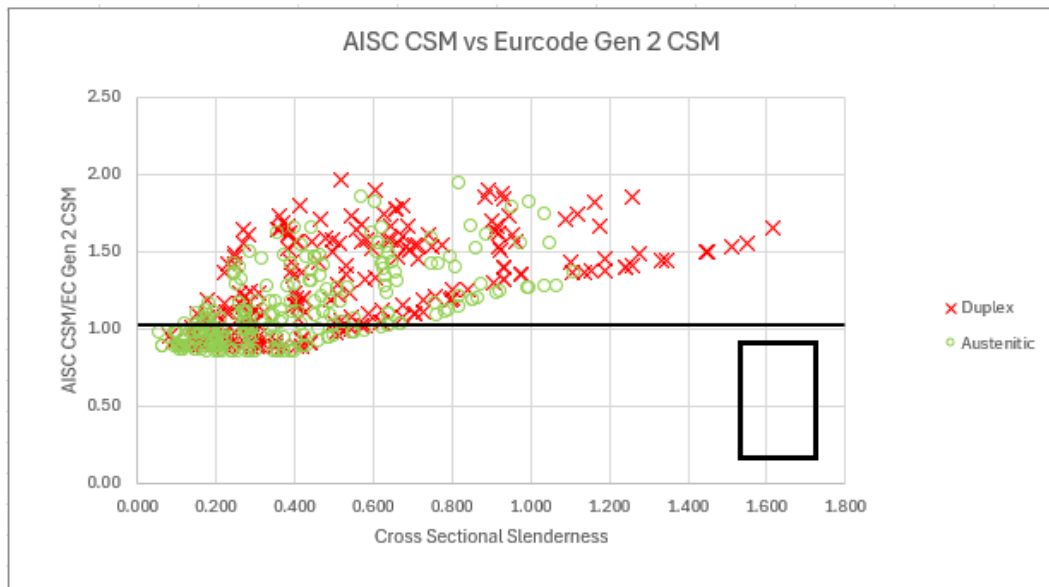


Figure 4.17: AISC continuous strength method compared with continuous strength method in EN1993 1-4 (2025) generation 2

4.5.2 Circular Sections

For CHS both AISC and ASCE effective width methods when compared to the first generation of Eurocode exhibit more conservative results for both austenitic and duplex steels. Austenitic steels appear to be more consistent, with ASCE EWM being about 35% less and AISC EWM being 5-10% less. Duplex steel is effected much more by cross sectional slenderness as the resistance changes considerably for more slender sections. This can mainly be explained by both ASCE and AISC using a lower yield strength than Eurocode as well as AISC having stricter requirements for cross sectional classes which determine when the elastic section modulus or the plastic section modulus can be used for calculation of the bending moment resistance. ASCE does not specify cross sectional classes and therefore always takes the elastic section modulus conservatively for determination of moment resistance.

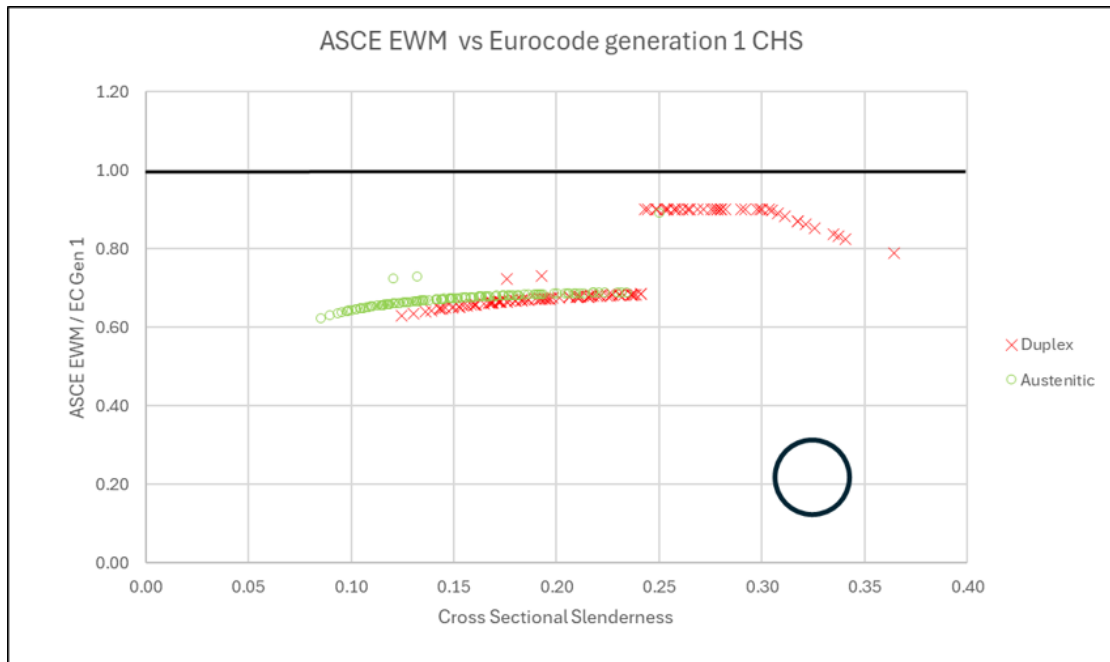


Figure 4.18: ASCE effective width method compared with Eurocode generation 1 for CHS

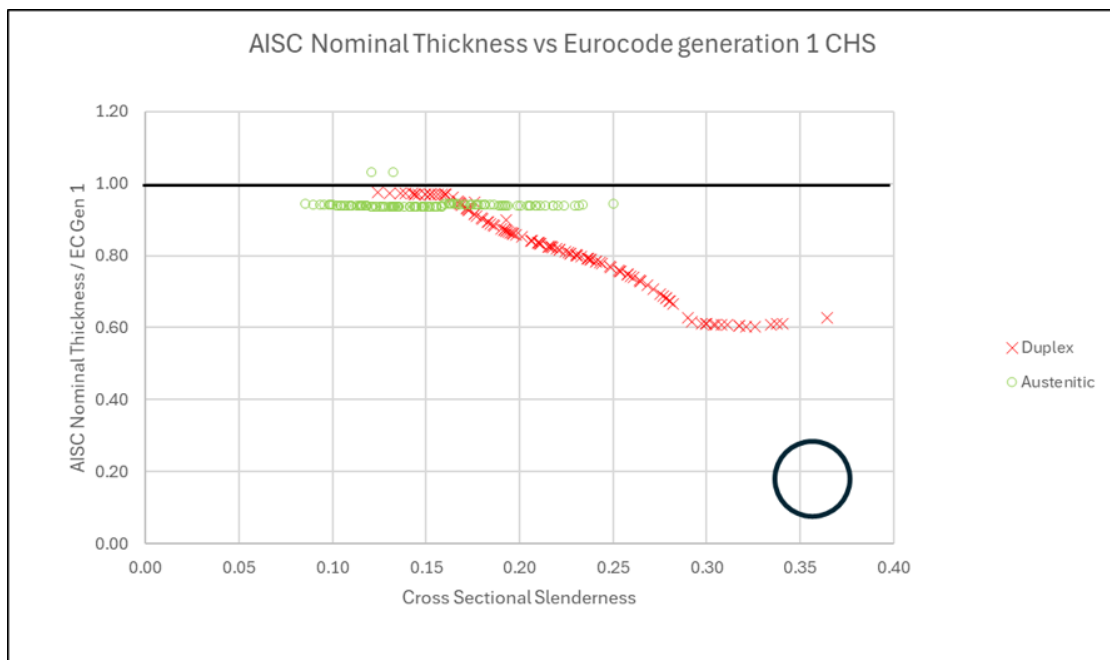


Figure 4.19: AISC effective width method compared with Eurocode generation 1 for CHS

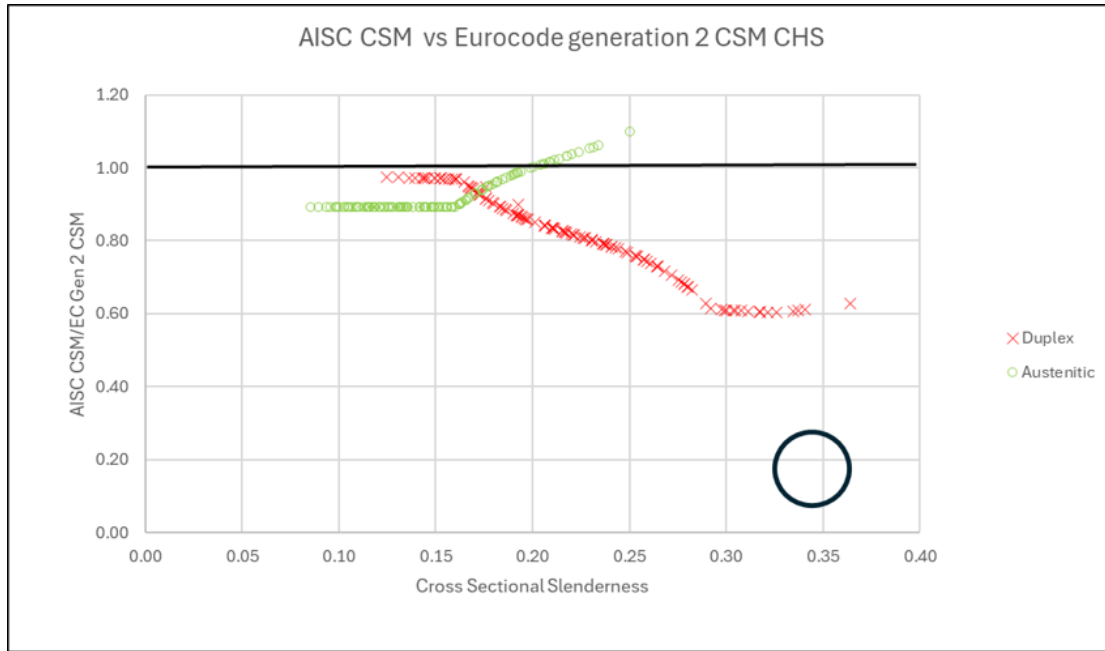


Figure 4.20: AISC continuous strength method compared with Eurocode generaion 2 continuous strength method for CHS

4.6 FEM Analysis

The FEM analysis was performed using 5 sample cross sections of varying cross sectional slenderness listed below in table 4.1 and the beam was modeled by using $h + t$ as the height and $b + t$ as the width of the section. The length was taken to be five meters across all sections. The beam is simply supported at both ends and supported at the top to prevent lateral movements. Stiffeners were placed at both ends and in the middle to prevent lateral torsional buckling. A nonlinear analysis was then conducted using 200 iterations to observe buckling behaviour after failure. Figure 4.21 shows this buckling behaviour. The maximum moment was found by observing the Load Proportionality Factor (LPF) at each load increment until the maximum value was found. The graph of this LPF factor for each load increment is shown in figure 4.23 for the example section $HSS24 \times 16 \times 0.312$. This factor was then multiplied by a reference moment calculated using a uniformly distributed reference load across the top of the beam and the distance from the load to the support. Alternatively the maximum moment can be determined by cutting the section in the midspan of the beam at the load increment where the max LPF factor occurs and taking the value directly as shown in figure 4.22. These two methods were shown to be similar.

Table 4.1: Sample Sections for FEM analysis

Section	h (mm)	b (mm)	t (mm)	Austenitic Slenderness	Duplex Slenderness
HSS4×2×0.375	101.600	50.800	9.525	0.043	0.062
HSS24×8×0.375	609.600	203.200	9.525	0.377	0.549
HSS14×8×0.250	355.600	203.200	6.350	0.452	0.658
HSS20×8×0.250	508.000	203.200	6.350	0.499	0.726
HSS24×16×0.312	609.600	406.400	7.925	0.743	1.082

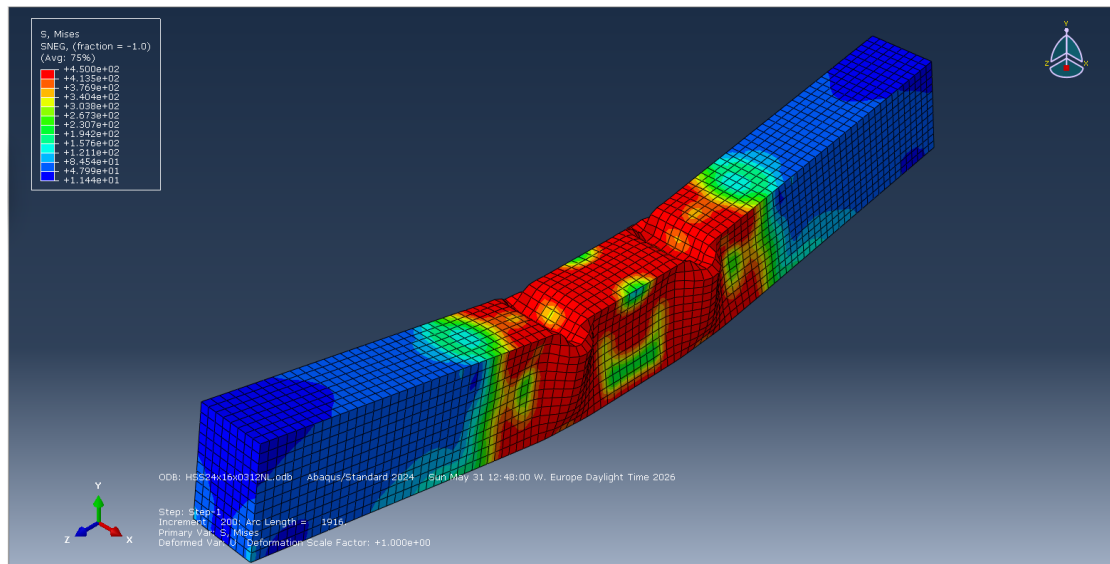


Figure 4.21: Buckling behaviour after long time for HSS24x16x0.312 Duplex section

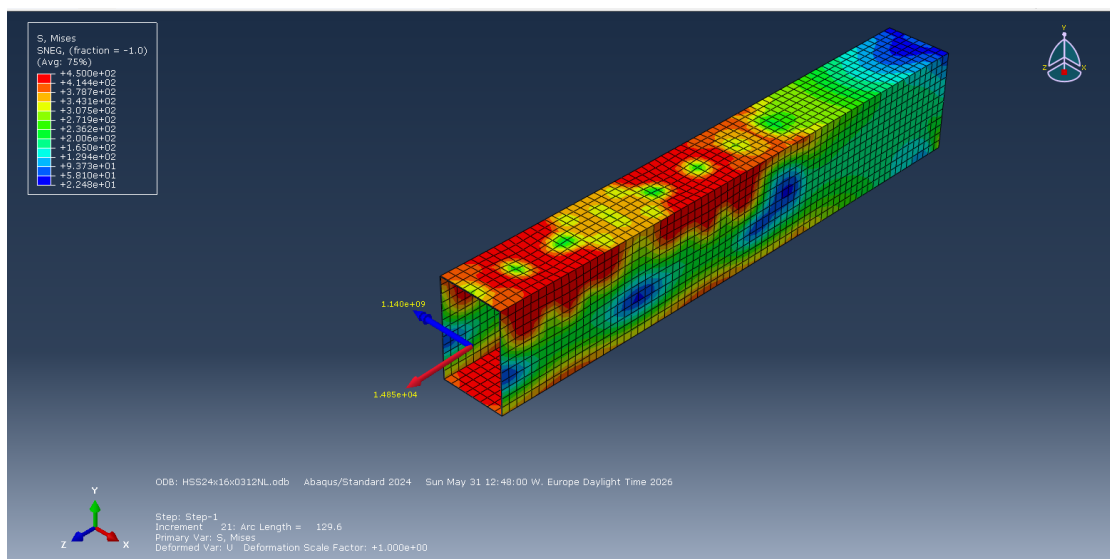


Figure 4.22: FEM max moment for HSS24x16x0.312 Duplex section

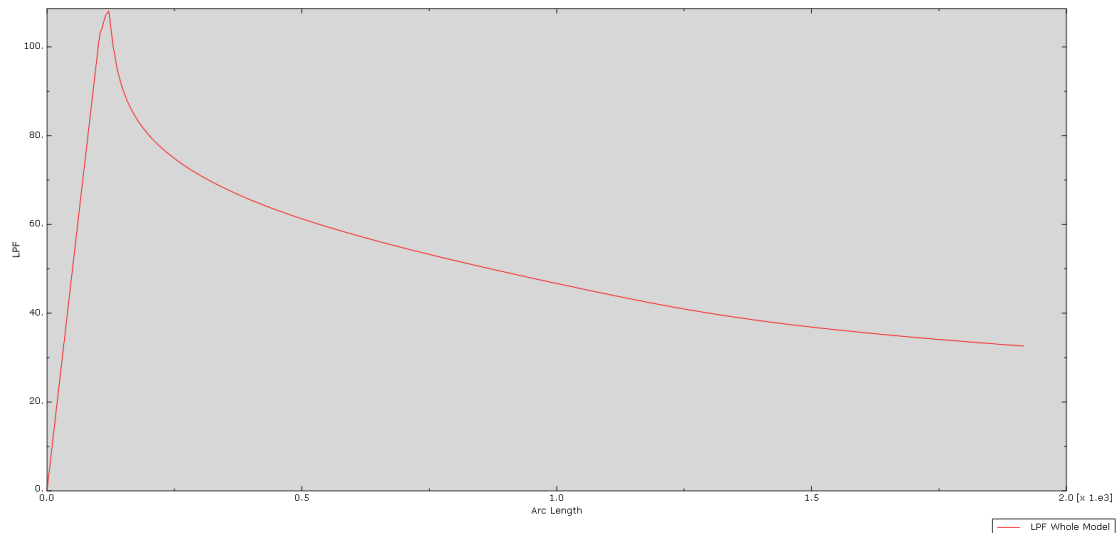


Figure 4.23: LPF factor for HSS24x16x0.312 Duplex section

4.6.1 FEM Moment Resistance Comparisons

The moment resistances determined from the FEM analysis were then compared to the different design methods found in the three standards. Both stainless steel grades were compared and the results are shown in 4.24 and 4.25 for Austenitic and Duplex respectively. The direct strength method gives the largest resistances values for Austenitic and the largest for duplex sections of small cross sectional slenderness. The EWM from ASCE8-22 gives consistently the smallest resistances across both steel grades. Eurocode resistances are the closest to the real resistances from FEM but still slightly conservative with AISC being the next closest.

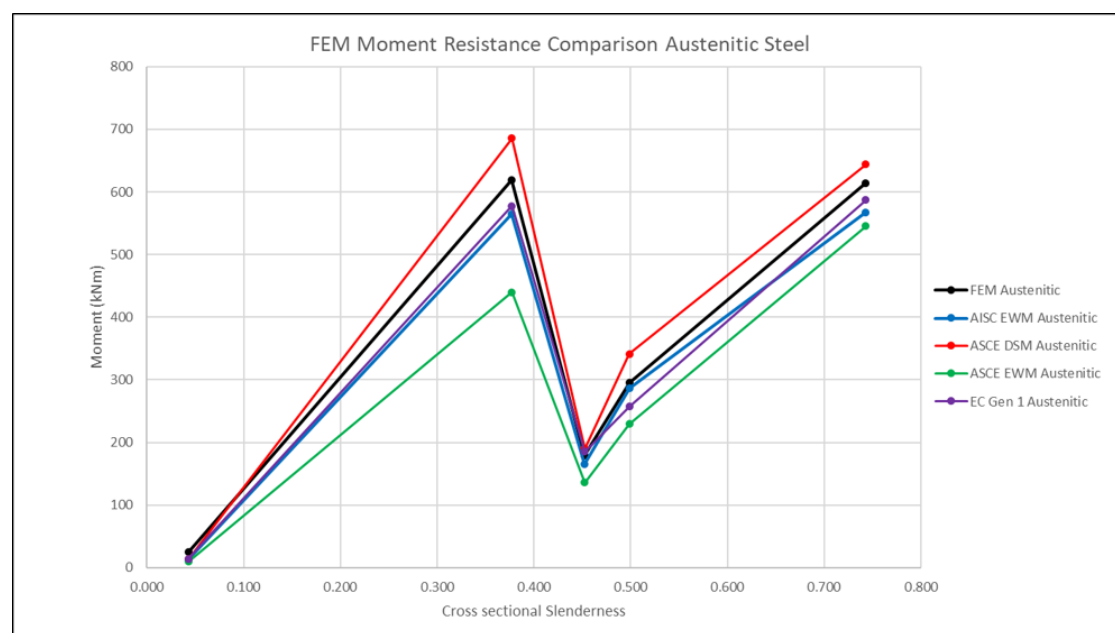


Figure 4.24: FEM moment resistance comparison for Austenitic steel

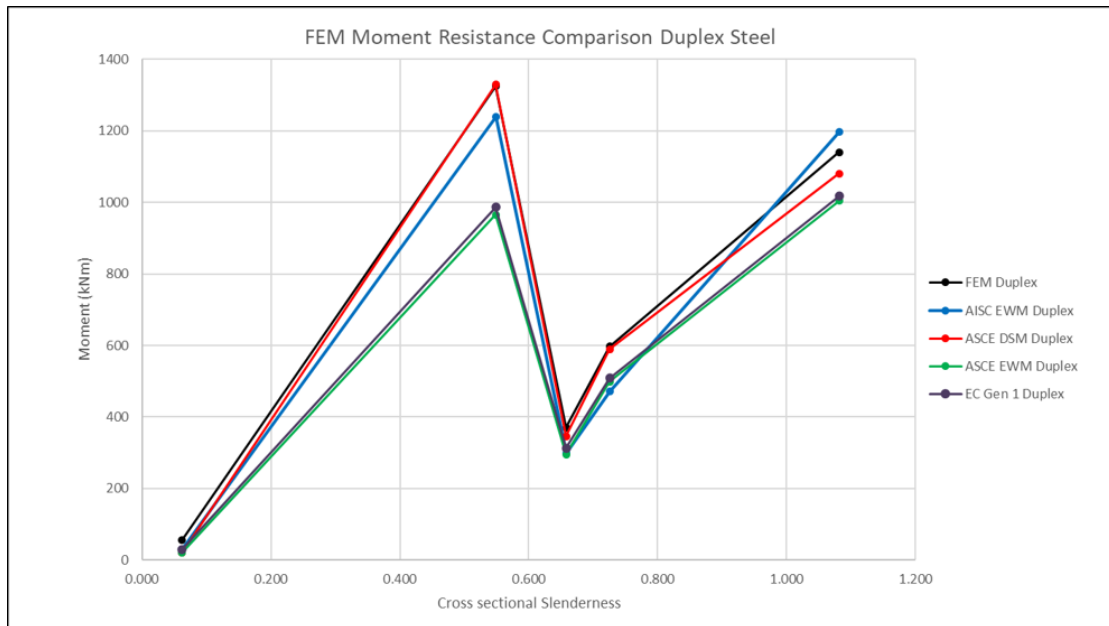


Figure 4.25: FEM moment resistance comparison for Duplex steel

5 Discussion

5.1 Column comparisons

The results from the published study on columns by SCI remain consistent with comparisons made about the bending resistances of these same sections. The important differences to note between ASCE 8 and AISC 370 from this study are as follows

First, In ASCE 8 the EWM consistently predicts higher resistances of about 20% than the DSM.

Second, although the design thickness in AISC 370 is only 5% lower than the nominal thickness, The difference in resistance can go up to 10% for sections governed by local buckling. This is because the reduced thickness affects both the cross-sectional area and the slenderness of the elements.

When comparing ASCE 8 and AISC 370 directly, and when global buckling governs, AISC 370 consistently predicts higher resistances. For RHS, AISC 370 can give resistances up to 20 % greater than ASCE 8 when both standards use EWM. This difference is because of the higher global buckling curve adopted in AISC 370.

For DSM, the difference from AISC 370 becomes around 15% due to the lower resistance factor and the lower global buckling curve used in ASCE 8. There are also big differences between ASCE 8 EWM and AISC 370 CSM for columns with short column slenderness due to inconsistencies with how ASCE 8 treats inelastic local buckling of sections.

The effect of cold-work also contributes to differences between the standards. For columns AISC 370 generally predicts higher average yield strengths ($F_{y,avg}$) than ASCE 8, because it was developed for cold-rolled sections, whereas the equations in ASCE 8 reflect the behaviour of press-braked sections.

For resistances of RHS calculated using $F_{y,avg}$, AISC 370 shows resistances about 60 % higher than ASCE 8 for stocky austenitic sections, and up to 30% higher for duplex sections. For more slender sections the difference in resistance becomes less due the higher effective width curve in ASCE 8. For CHS, AISC 370 predicts resistances about 64 % higher than ASCE 8 for austenitic stainless steel, and about 15% for duplex steel.

5.2 Beam comparisons

5.2.1 ASCE 8-22 Comparisons

When designing a stainless-steel beam with ASCE 8 in bending it is ideal to use the direct strength method for beams with smaller unbraced lengths or smaller cross sections as it will yield higher resistances but as the global slenderness and size of the cross section increases (thus increased local slenderness) it becomes more ideal to consider the effective width method instead. When unsure about intermediate values it is necessary to use both methods and compare the differences and choose the best method for a given loading scenario.

5.2.2 AISC Nominal VS Design

The comparison between nominal and design thicknesses in AISC 370-21 demonstrates that the use of the reduced design thickness results in safer and more conservative predictions. For stockier sections, the expected reduction in resistance is proportional to the 5% reduction in thickness. However for more slender sections that are pushed into smaller cross sectional classes by the reduced thickness, it is about 7% less for some Duplex rectangular sections and 6% for circular sections. Although this additional reduction is relatively small, it highlights the relationship of section thickness to effective cross sectional area and slenderness and their influence on local buckling behaviour.

5.2.3 AISC Nominal VS CSM

The EWM from AISC 370-21 consistently shows lower resistances than the continuous strength method for rectangular sections with increased variability in more slender sections due to local plate instability. Circular sections have a smaller reduction in resistance for the standard approach compared with CSM. Circular sections will have more uniformly distributed stresses resulting in higher bending resistances using the standard approach than CSM for more slender sections. This comparison suggests that CSM is not well developed to capture plate behaviour and is better suited for shell behaviour that occurs in circular sections.

5.2.4 ASCE 8-22 vs AISC 370-21 (based on F_y)

When comparing the two standards, ASCE 8-22 and AISC 370-21 for a standard yield strength (F_y) most cross sections both rectangular and circular are more conservative for the EWM in ASCE 8-22 than in the EWM in AISC 370. For more slender rectangular sections in duplex steel however, where local buckling will likely occur, the post yield strength for both standards is similar. Circular sections in ASCE show a consistent decrease in strength. When compared to CSM, EWM in ASCE shows a similar trend to EWM in AISC with the bending resistances for rectangular sections being lower than CSM while circular sections have the same uniform curve. The similarity between the two effective width methods compared to CSM occurs because both standards use similar buckling curves. Overall CSM will result in larger less conservative predictions across both standards and is better able to capture the strain hardening capacity of stainless steel. This method is most applicable for circular sections of large slenderness values where local buckling is more critical than global. Austenitic steel is a more predictable material with efficient strength to weight ratios

and is more economical. The duplex sections analysed are more variable and are largely dependent on the size of the cross section. While it is generally considered less economical it can provide an increased strength if larger loading conditions are present.

5.2.5 ASCE 8 vs AISC 370 (based on $F_{y.avg}$)

The ability to cold form stainless steel is an important property that allows for yield strength enhancements. Both American standards as well as the second generation of Eurocode provide guidelines for how to increase yield strength of a section due to cold working processes. Compared to columns which show that ASCE 8-22 has consistently lower yield strength predictions compared with AISC 370, Beams are more varied and the cross sectional slenderness plays a bigger role. Stockier rectangular sections in ASCE 8-22 show higher strengths than AISC for both austenitic and duplex steel however the differences for duplex steel are much smaller. As slenderness increases, both grades will move towards equal yield strengths for both standards.

ASCE 8-22 does not provide guidelines for increasing the yield strength due to cold working when designing for circular sections and therefore it is necessary to compare the standard yield strength in ASCE 8-22, against the enhanced one in AISC 370. The comparison shows that the increase in yield strength has a very pronounced effect for austenitic steel grades with a small slenderness while duplex, that increase in yield strength is significantly less. More slender sections in AISC show an underestimation of yield strength. When looking at bending resistance using the enhanced yield strength, the EWM is more conservative than the DSM but is highly dependent on cross sectional slenderness. ASCE EWM and AISC EWM show similar resistances with the variation being small enough to be considered non impactful. Comparing DSM against CSM shows that DSM is higher for stockier sections but the two methods converge for increasing slenderness. Finally the strength enhancements allow for the two AISC methods to become quite comparable with a small difference between them across all sections. Overall resistances predicted by AISC align much closer to resistances predicted by ASCE when using an enhanced yield strength and the DSM is observed to be the most efficient method across all standards for stockier sections. It highlights how the American design standards for stainless steel were developed with cold working fabrication in mind. While the cold working process is considerably less economical compared with standard hot rolling methods it will allow for the design of these members to best capture their intended behaviour.

5.2.6 Eurocode Comparisons

When designing with the standard method the difference between the two generations of Eurocode appear to be very small with most sections showing the same resistance for generation 1 and generation 2 while other sections only showing slight variations mostly in duplex steel. Comparing AISC370-21 and Eurocode, the results highlight that both codes use methods of cross sectional classification and determination of moment for local buckling without much consideration for lateral torsional buckling. Any difference can primarily be attributed to the difference in yield stress F_y between the two standards. This is in contrast to the two ASCE methods as global buckling and the unbraced length of the member plays a much bigger role. The continuous strength method in AISC is less conservative but not well developed to capture the plate like

behaviour used in rectangular sections which reinforces this methods preferred application on circular sections.

Circular sections designed using AISC or ASCE effective width methods are more conservative for both kinds of steel than the first generation of Eurocode. Austenitic steel is more consistent with little to no drop in resistance for an increased cross sectional slenderness. Duplex steel is much more dependent on cross sectional slenderness. This can mainly be explained by both ASCE and AISC using a lower yield strength than Eurocode as well as AISC having stricter requirements for cross sectional classes which determine when the elastic section modulus or the plastic section modulus can be used for calculation of the bending moment resistance. ASCE does not specify cross sectional classes and therefore always takes the elastic section modulus conservatively for determination of moment resistance.

5.2.7 FEM Comparison

The FEM analysis gives meaningful insights about each design standard and how they can best be applied to different situations. The Direct strength method has the highest resistances and is usually greater than reality. This can lead to unsafe designs where the predicted design resistance is higher than what the beam can actually support. This only occurs however for beams with small unbraced lengths. Larger unbraced lengths will bring the resistance of the DSM down and closer to reality. ASCE EWM is the most conservative and will always lead to a safer design but it is so low that it could raise the cost of a given project considerably if using beams with large cross sectional slenderness. ASCE methods can be worth while if the design loads are low and only stocky sections are used as the difference in the bending resistance from reality is much smaller. Eurocode methods are closest to reality and provide the most efficient results. Meaning the size and cost of the section best fit the situation and given loading conditions. This could suggest that stainless steel hollow sections are more well studied in Europe and their behaviour is best captured through their methods.

5.3 Further development and future work

While the results from this study are overall satisfactory, there are still ample opportunities to improve the process and continue future work with this. Only moment resistance on beams in pure bending were considered and the results of axially loaded columns has already been published. Other loading effects such as tension, shear and combined bending with axial loading can be studied to better understand more differences between standards. The work can also continue to examine additional grades of stainless steel and more kinds of geometry. It may also prove beneficial to identify which parameter (unbraced length, width, thickness, steel grade, etc.) plays the biggest role in determining bending resistance and which standard is the most sensitive to changes in this parameter.

A more extensive FEM analysis can be conducted to better understand the real behaviour of beams and model different situations that account for lateral torsional buckling and varying degrees of global slenderness. The analysis can also be expanded to the additional loading effects and verified against hand calculations.

Lastly to further verify accuracy of the bending resistances it is necessary to obtain additional reference values from stainless steel manufacturers and conduct an error

analysis to determine margins of error on hand calculations and identify areas of improvement. Many sections, particular of duplex grade have sizes that fall outside the observed correlation between standards.

6 Conclusion

The aim of this master's thesis was to compare the bending resistances of various stainless steel hollow sections to identify differences in the design process of four different structural standards. Each design method showed to have strong areas in which its application is most beneficial and weak areas where results proved inconsistent or worse than the application of a different design method. The design process does however need some additional analysis, and validation for specific cross sections. The methods studied were however limited to only the ultimate bending resistance of the beams. More consideration of all potential failure modes could lead to different observations and results than the ones achieved here.

The method of modelling and simulating in finite element proved to be more difficult and sensitive than expected and could use some further development. The design standards provide little guidance about boundary conditions and it is necessary to connect the designs to a real world application to best verify accuracy. Additionally only five example sections were used for finite element analysis and running the same simulations for more sections and materials could provide more insights into differences.

Overall the comparison demonstrates that AISC 370-21 predicts mostly higher resistances than ASCE 8-22 but similar or less than Eurocode and highlights key differences such as global buckling curves, local buckling formulations, resistance factors, and the calculation of $F_{y,avg}$ in international building standards.

References

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Appandix A - ASCE Bending Calculations

ASCE 8-22 Bending for HSS Sections

members without holes

1-Material Properties

$$F_y := 205 \frac{N}{mm^2} \quad E := 193000 \frac{N}{mm^2} \quad G := 74231 \text{ MPa} \quad \mu := 0.3 \quad \text{Table C-3}$$

$$\alpha := 1.44 \quad \beta_0 := 0.35 \quad \beta_1 := 0.44 \quad \beta_2 := 0.61 \quad \text{Table 6.1}$$

2- Cross-sectional properties Example section HSS32x16x0.375

$$h := 812.8 \text{ mm} = 812.8 \text{ mm} \quad b := 406.4 \text{ mm} = 406.4 \text{ mm} \quad t := 9.525 \text{ mm} = 9.525 \text{ mm}$$

$$r_i := t \quad b_{\text{flange}} := b - 3 \cdot t = 377.825 \text{ mm} \quad b_{\text{web}} := h - 3 \cdot t = 784.225 \text{ mm} \quad r_e := r_i + t$$

$$A_{ri} := \left(1 - \frac{\pi}{4}\right) \cdot r_i^2 = 19.47 \text{ mm}^2 \quad A_{re} := \left(1 - \frac{\pi}{4}\right) \cdot r_e^2 = 77.88 \text{ mm}^2$$

$$A_g := h \cdot b - (b - 2 \cdot t) \cdot (h - 2 \cdot t) - 4 \cdot A_{re} + 4 \cdot A_{ri} = (2.263 \cdot 10^4) \text{ mm}^2$$

$$h_{ri} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i \quad b_{ri} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$h_{re} := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e \quad b_{re} := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$I_e := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_e^4 \quad I_i := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_i^4$$

$$I_y := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot h_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot h_{ri}^2) = (2.005 \cdot 10^9) \text{ mm}^4$$

$$I_z := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t) \cdot (b - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot b_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot b_{ri}^2) = (6.926 \cdot 10^8) \text{ mm}^4$$

Appendix A.1 - ASCE Bending Calculations RHS

$$S_y := \frac{I_y}{\frac{h}{2}} = (4.933 \cdot 10^6) \text{ mm}^3$$

$$Z_y := \frac{b \cdot h^2}{4} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - 4 \cdot A_{re} \cdot h_{re} + 4 \cdot A_{ri} \cdot h_{ri} = (6.015 \cdot 10^6) \text{ mm}^3$$

$$r_c := r_i + \frac{t}{2} = 14.288 \text{ mm}$$

$$b_m := b - t \quad h_m := h - t$$

$$A_h := b_m \cdot h_m - (4 - \pi) \cdot r_c^2 = (3.186 \cdot 10^5) \text{ mm}^2$$

$$h_0 := (b_m + h_m - 4 \cdot r_c) \cdot 2 + 2 \cdot \pi \cdot r_c = (2.376 \cdot 10^3) \text{ mm}$$

$$J := \frac{4 \cdot A_h^2 \cdot t}{h_0} = (1.628 \cdot 10^9) \text{ mm}^4$$

3-yielding and plastic bending moment:

$$W_{el} := S_y \quad W_{pl} := Z_y$$

$$M_y := W_{el} \cdot F_y = (1.011 \cdot 10^3) \text{ kN} \cdot \text{m} \quad M_p := W_{pl} \cdot F_y = (1.233 \cdot 10^3) \text{ kN} \cdot \text{m}$$

Considerations for Global Buckling (Lateral Torsional Buckling) 6.2.1, 6.2.1.4, 6.2.2

Assumption: member under uniform bending

$$M_{\max} := 10 \text{ kN} \cdot \text{m} \quad M_A := M_{\max} \quad M_B := M_{\max} \quad M_C := M_{\max}$$

$$C_b := 12.5 \cdot \frac{M_{\max}}{2.5 \cdot M_{\max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 1$$

$$L_u := \frac{0.12 \cdot C_b \cdot \pi}{F_y \cdot S_y} \cdot \sqrt{E \cdot G \cdot J \cdot I_y} = 80.618 \text{ m}$$

$$L_y := L_u \cdot 1 = 80.618 \text{ m} \quad \text{Assumption: An unbraced length less than } L_u$$

$$K_y := 1 \quad \text{Assumption: pinned-pinned}$$

Appendix A.1 - ASCE Bending Calculations RHS

$$F_{cre} := \frac{C_b \cdot \pi}{K_y \cdot L_y \cdot S_y} \cdot \sqrt{E \cdot G \cdot J \cdot I_y} = (1.708 \cdot 10^3) \frac{N}{mm^2}$$

$$\lambda_e := \sqrt{\frac{F_y}{F_{cre}}} = 0.346$$

$$F_n := \begin{cases} F_y & \text{if } L_y \leq L_u \\ \begin{cases} F_y & \text{if } \lambda_e \leq \beta_0 \\ 1.2 \cdot (\beta_1)^{\lambda_e} \cdot F_y & \text{else if } 1.6 \geq \lambda_e > \beta_0 \\ \beta_2 \cdot F_{cre} & \text{else if } \lambda_e > 1.6 \end{cases} & \text{else if } L_y > L_u \end{cases} = 205 \text{ MPa}$$

Inelastic Reserve Strength:

$$M_P := F_y \cdot Z_y = (1.233 \cdot 10^3) \text{ kN} \cdot m$$

$$M_{cre} := S_y \cdot F_{cre} = 8426.404 \text{ kN} \cdot m$$

$$M_{ne.G} := \begin{cases} M_P - (M_P - M_y) \cdot \frac{\sqrt{\frac{M_y}{M_{cre}}} - 0.23}{0.12} & \text{if } M_{cre} > 8.16 \cdot M_y \\ S_y \cdot F_n & \text{else} \end{cases} = (1.018 \cdot 10^3) \text{ kN} \cdot m$$

$$\lambda_{l,EWM} := \sqrt{\frac{M_{ne.G}}{M_{cre}}} = 0.348$$

Appendix A.1 - ASCE Bending Calculations RHS

Local Buckling, Effective Width Method, 6.3.1, A1, A1.2

Flange $w_f := b - 3 \cdot t = 377.825 \text{ mm}$ $k_f := 4$

Elastic buckling stress of the flange plate:

$$F_{\text{cr},f} := \left(\frac{k_f \cdot \pi^2 \cdot E}{12 (1 - \mu^2)} \right) \cdot \left(\frac{t}{w_f} \right)^2 = 443.448 \frac{N}{\text{mm}^2}$$

Effective width of the flange

$f := F_n = 205 \text{ MPa}$ $\lambda_f := \sqrt{\frac{f}{F_{\text{cr},f}}} = 0.68$

$\rho_f := \text{if } \lambda_f > 0.673 \quad = 0.995$

$$\left\| \begin{array}{l} \left(1 - \frac{0.22}{\lambda_f} \right) \\ \lambda_f \\ \text{else} \\ 1 \end{array} \right\|$$

$b_{e,f} := w_f \cdot \rho_f = 375.888 \text{ mm}$

$\frac{b_{e,f}}{2} = 187.944 \text{ mm}$

$b_{\text{ineff},f} := w_f - b_{e,f} = 0.002 \text{ m}$

$h_{\text{NA},1} := \frac{h}{2} + \frac{b_{\text{ineff},f} \cdot t \cdot \left(\frac{t}{2} \right)}{A_g - b_{\text{ineff},f} \cdot t} = 406.404 \text{ mm}$ new depth of the neutral axis after the flange reduction

$h_{d1} := \frac{b_{\text{ineff},f} \cdot t \cdot \left(\frac{t}{2} \right)}{A_g - b_{\text{ineff},f} \cdot t} = 0.004 \text{ mm}$ shift of the neutral axis downwards due to the flange reduction

Effective width of the web

$w_w := h - 3 \cdot t = 784.225 \text{ mm}$

$f_1 := \frac{\left(\frac{w_w}{2} \right)}{\frac{h}{2}} \cdot F_n = 197.793 \text{ MPa}$

$f_2 := -\frac{\left(\frac{w_w}{2} \right)}{\frac{h}{2}} \cdot F_n = -197.793 \text{ MPa}$

$\psi := \left| \frac{f_2}{f_1} \right| = 1$

$k_w := 4 + 2 \cdot (1 + \psi)^3 + 2 \cdot (1 + \psi) = 24$

Appendix A.1 - ASCE Bending Calculations RHS

Elastic buckling stress of web plate:

$$F_{cr1.w} := \left(\frac{k_w \cdot \pi^2 \cdot E}{12 (1 - \mu^2)} \right) \cdot \left(\frac{t}{w_w} \right)^2 = 617.581 \frac{N}{mm^2}$$

$$\lambda_w := \sqrt{\frac{f_1}{F_{cr1.w}}} = 0.566$$

$$\rho_w := \begin{cases} \text{if } \lambda_w > 0.673 \\ \left(1 - \frac{0.22}{\lambda_w} \right) \\ \text{else} \\ 1 \end{cases} = 1$$

$$b_{e.w} := w_w \cdot \rho_w = 784.225 \text{ mm}$$

$$b_1 := \frac{b_{e.w}}{3 + \psi} = 196.056 \text{ mm}$$

$$b_2 := \begin{cases} \text{if } \frac{h}{b} \leq 4 \\ \text{if } \psi > 0.236 \\ \frac{b_{e.w}}{2} \\ \text{else if } \psi \leq 0.236 \\ b_{e.w} - b_1 \\ \text{else if } \frac{h}{b} > 4 \\ \frac{b_{e.w}}{(1 + \psi)} - b_1 \end{cases} = 392.113 \text{ mm}$$

$$h_c := \frac{h}{2} + h_{d1} = 406.404 \text{ mm}$$

$$b1 := \begin{cases} \text{if } (b_1 + b_2) > h_c \\ \frac{h_c}{2} \\ \text{else} \\ b_2 \end{cases} = 0.203 \text{ m}$$

$$b2 := \begin{cases} \text{if } (b_1 + b_2) > h_c \\ \frac{h_c}{2} \\ \text{else} \\ b_2 \end{cases} = 0.203 \text{ m}$$

$$b_{ineff.w} := \begin{cases} \text{if } (b1 + b2) < h_c \\ h_c - (b1 + b2) \\ \text{else} \\ 0 \end{cases} = 0 \text{ m}$$

$$h_{NA.2} := h_{NA.1} + \frac{2 \cdot b_{ineff.w} \cdot t \cdot \left(b_1 + \frac{b_{ineff.w}}{2} + t \right)}{A_g - b_{ineff.f} \cdot t - 2 \cdot t \cdot b_{ineff.w}} = 406.404 \text{ mm}$$

new depth of the neutral axis after the flange and web reduction

$$h_{d2} := h_{NA.1} - h_{NA.2} = 0 \text{ m}$$

shift of the neutral axis downwards due to the web reduction

Appendix A.1 - ASCE Bending Calculations RHS

Sectional Properties of the effective cross section:

$$I_{\text{ineff}} := b_{\text{ineff.f}} \cdot \frac{t^3}{12} + b_{\text{ineff.f}} \cdot t \cdot \left(h_{\text{NA.2}} - \frac{t}{2} \right)^2 + 2 \cdot \left(\frac{t \cdot b_{\text{ineff.w}}^3}{12} + t \cdot b_{\text{ineff.w}} \cdot \left(h_{\text{NA.2}} - \frac{b_{\text{ineff.w}}}{2} - (b_1 + h_{\text{d1}} + h_{\text{d2}} - t) \right)^2 \right) =$$

$$I_{\text{eff}} := I_y - I_{\text{ineff}} = (2.002 \cdot 10^9) \text{ mm}^4 \quad I_{\text{ineff}} = (2.976 \cdot 10^6) \text{ mm}^4$$

$$S_{\text{ec}} := \frac{I_{\text{eff}}}{h - h_{\text{NA.2}}} = 4925252.932 \text{ mm}^3 \quad S_{\text{et}} := \frac{I_{\text{eff}}}{h_{\text{NA.2}}} = (4.925 \cdot 10^6) \text{ mm}^3$$

The nominal flexural strength for local buckling (effective width method)

$$M_{\text{nl.EWM}} := \begin{cases} S_{\text{ec}} \cdot F_n \leq S_{\text{et}} \cdot F_y \\ \left| S_{\text{ec}} \cdot F_n \right| \\ \text{else} \\ \left| S_{\text{et}} \cdot F_y \right| \end{cases} = 1009.658 \text{ kN} \cdot \text{m}$$

Local Buckling, Direct Stress Method, 6.3.2, B.3.3.2

$$M_{\text{crl.f}} := S_y \cdot F_{\text{crl.f}} = (2.187 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$w_c := \frac{w_w}{2} \quad w_t := \frac{w_w}{2} \quad \psi_{\text{DSM}} := \left| \frac{w_t}{w_c} \right| = 1$$

$$k_{\text{w.DSM}} := 4 + 2 \cdot (1 + \psi_{\text{DSM}})^3 + 2 \cdot (1 + \psi_{\text{DSM}}) = 24$$

$$\text{Elastic buckling stress of web plate: } F_{\text{crl.w.DSM}} := \left(\frac{k_{\text{w.DSM}} \cdot \pi^2 \cdot E}{12 (1 - \mu^2)} \right) \cdot \left(\frac{t}{w_w} \right)^2 = 617.581 \frac{\text{N}}{\text{mm}^2}$$

$$F_{\text{n.web}} := \frac{h}{w_w} \cdot F_{\text{crl.w.DSM}} = 640.084 \text{ MPa}$$

$$M_{\text{crl.w}} := S_y \cdot F_{\text{n.web}} = (3.157 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$M_{\text{crl}} := \min(M_{\text{crl.f}}, M_{\text{crl.w}}) = (2.187 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$S_y = (4.933 \cdot 10^6) \text{ mm}^3$$

Appendix A.1 - ASCE Bending Calculations RHS

$$\lambda_I := \sqrt{\frac{M_{ne.G}}{M_{crI}}} = 0.682$$

$$M_{nl.DSM} := \begin{cases} \text{if } \lambda_I > 0.667 & = (1.007 \cdot 10^3) \text{ kN} \cdot \text{m} \\ \left\| \left(1 - 0.2 \cdot \left(\frac{M_{crI}}{M_{ne.G}} \right)^{0.4} \right) \cdot \left(\frac{M_{crI}}{M_{ne.G}} \right)^{0.4} \cdot M_{ne.G} \right. \\ \text{else} \\ \left\| M_{ne.G} \end{cases}$$

$$M_{n.EWM} := \min(M_{ne.G}, M_{nl.EWM}) = (1.01 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$M_{n.DSM} := \min(M_{ne.G}, M_{nl.DSM}) = (1.007 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$\beta_f := 1$$

$$\beta_w := \frac{h}{b_{web}} = 1.036$$

$$\psi_f := 1$$

$$\psi_w := -1$$

$$K_{ss.f} := 4$$

$$K_{f.f} := 6.97$$

$$K_{ss.w} := 23.9$$

$$K_{f.w} := 39.6$$

$$F_{el.w.ss} := K_{ss.w} \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{t}{b_{web}} \right)^2 = 615.008 \text{ MPa}$$

$$F_{el.w.f} := K_{f.w} \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{t}{b_{web}} \right)^2 = (1.019 \cdot 10^3) \text{ MPa}$$

Appendix A.1 - ASCE Bending Calculations RHS

$$F_{el.f.ss} := K_{ss.f} \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{t}{b_{flange}} \right)^2 = 443.448 \text{ MPa}$$

$$F_{el.f.f} := K_{f.f} \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \mu^2)} \cdot \left(\frac{t}{b_{flange}} \right)^2 = 772.709 \text{ MPa}$$

$$F_{el.p.ss} := \min(\beta_f \cdot F_{el.f.ss}, \beta_w \cdot F_{el.w.ss}) = 443.448 \text{ MPa}$$

$$F_{el.p.f} := \min(\beta_f \cdot F_{el.f.f}, \beta_w \cdot F_{el.w.f}) = 772.709 \text{ MPa}$$

$$\phi := \frac{\beta_f \cdot F_{el.f.ss}}{\beta_w \cdot F_{el.w.ss}} = 0.696$$

$$a_w := \min\left(0.53, 0.63 - 0.1 \cdot \frac{h}{b}\right) = 0.43$$

$$\xi := 0.53 - \frac{a_w}{\phi} = -0.088 \quad \text{must be more than 0 and less than 1}$$

$$F_{el} := F_{el.p.ss} + \xi \cdot (F_{el.p.f} - F_{el.p.ss}) = 414.445 \text{ MPa}$$

$$\lambda_{cs} := \sqrt{\frac{F_y}{F_{el}}} = 0.703$$

ASCE 8-22 Bending for CHS Sections using effective width method

Material Properties Austenitic EN 1.4301

$$F_y := 205 \frac{N}{mm^2} \quad F_u := 515 \frac{N}{mm^2} \quad E := 193000 \frac{N}{mm^2} \quad C := 0.65 \quad \lambda := 0.1$$

Example section HSS7.5x0.375

Sectional Properties

$$D := 190.5 \text{ mm} \quad t := 9.525 \text{ mm} \quad \frac{D}{t} = 20$$

$$A_g := \frac{\pi}{4} \cdot (D^2 - (D - 2 \cdot t)^2) = (5.415 \cdot 10^3) \text{ mm}^2$$

$$I := \frac{\pi}{4} \cdot \left(\left(\frac{D}{2} \right)^4 - \left(\frac{D}{2} - t \right)^4 \right) = (2.223 \cdot 10^7) \text{ mm}^4$$

$$\text{if} \left(\frac{D}{t} \leq 0.881 \cdot \frac{E}{F_y}, \text{"Ok"}, \text{"Not ok"} \right) = \text{"Ok"}$$

$$L_y := \sqrt{\frac{\lambda^2 \cdot \pi^2 \cdot E \cdot I}{A_g \cdot F_y}} = 617.626 \text{ mm}$$

$$W_{el} := \frac{\pi \cdot (D^4 - (D - 2 \cdot t)^4)}{32 \cdot D} = (2.334 \cdot 10^5) \text{ mm}^3 \quad W_{pl} := \frac{D^3 - (D - 2 \cdot t)^3}{6} = (3.123 \cdot 10^5) \text{ mm}^3$$

$$K_c := \left(\frac{\left((1 - C) \cdot \left(\frac{E}{F_y} \right) \cdot \left(\frac{D}{t} \right) + 5.882 \cdot C \right)}{8.93 - 3.048 \cdot C} \right) = 2.921$$

$$F_n := \text{if} \left(\frac{D}{t} \leq 0.112 \cdot \frac{E}{F_y} \right) = (2.05 \cdot 10^8) \text{ Pa}$$

$$\begin{aligned} & \parallel F_y \\ & \text{else if } \frac{D}{t} \leq 0.881 \cdot \frac{E}{F_y} \\ & \parallel K_c \cdot F_y \\ & \text{else} \\ & \parallel 0 \end{aligned}$$

$$M_n := F_n \cdot W_{el} = 47.849 \text{ kN} \cdot \text{m}$$

$$\phi M_n := 0.95 \cdot M_n = 45.456 \text{ kN} \cdot \text{m}$$

Appendix B - AISC Bending Calculations

AISC 370-21 Bending for HSS Sections

Members without holes

Material Properties

$$F_y := 205 \text{ MPa} \quad E := 193000 \frac{\text{N}}{\text{mm}^2} \quad G := 74321 \text{ MPa} \quad \mu := 0.3 \quad \text{Table C-3}$$

$$\alpha := 1.44 \quad \beta_0 := 0.35 \quad \beta_1 := 0.44 \quad \beta_2 := 0.61 \quad \text{Table 6.1} \quad \phi_{\text{LRFD}} := 0.9$$

2- Cross-Sectional Properties

Example section HSS32x16x0.375

$$h := 812.8 \text{ mm} = 812.8 \text{ mm} \quad b := 406.4 \text{ mm} = 406.4 \text{ mm} \quad t := 9.525 \text{ mm} = 9.525 \text{ mm}$$

$$r_i := t \quad b_{\text{flange}} := b - 3 \cdot t = 377.825 \text{ mm} \quad b_{\text{web}} := h - 3 \cdot t = 784.225 \text{ mm} \quad r_e := r_i + t$$

$$A_{ri} := \left(1 - \frac{\pi}{4}\right) \cdot r_i^2 = 19.47 \text{ mm}^2 \quad A_{re} := \left(1 - \frac{\pi}{4}\right) \cdot r_e^2 = 77.88 \text{ mm}^2$$

$$A_g := h \cdot b - (b - 2 \cdot t) \cdot (h - 2 \cdot t) - 4 \cdot A_{re} + 4 \cdot A_{ri} = 22629.219 \text{ mm}^2$$

$$h_{ri} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i \quad b_{ri} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$h_{re} := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e \quad b_{re} := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$I_e := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_e^4 \quad I_i := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_i^4$$

$$I_y := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot h_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot h_{ri}^2) = (2.005 \cdot 10^9) \text{ mm}^4$$

$$I_z := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t) \cdot (b - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot b_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot b_{ri}^2) = (6.926 \cdot 10^8) \text{ mm}^4$$

Appendix B.1 - AISC Bending Calculations

$$S_y := \frac{I_y}{\frac{h}{2}} = (4.933 \cdot 10^6) \text{ mm}^3$$

$$Z_y := \frac{b \cdot h^2}{4} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - 4 \cdot A_{re} \cdot h_{re} + 4 \cdot A_{ri} \cdot h_{ri} = 6015473.752 \text{ mm}^3$$

$$r_c := r_i + \frac{t}{2} = 14.288 \text{ mm}$$

$$b_m := b - t \quad h_m := h - t$$

$$A_h := b_m \cdot h_m - (4 - \pi) \cdot r_c^2 = (3.186 \cdot 10^5) \text{ mm}^2$$

$$h_0 := (b_m + h_m - 4 \cdot r_c) \cdot 2 + 2 \cdot \pi \cdot r_c = (2.376 \cdot 10^3) \text{ mm}$$

$$J := \frac{4 \cdot A_h^2 \cdot t}{h_0} = (1.628 \cdot 10^9) \text{ mm}^4$$

3-yielding and plastic bending moment:

$$W_{el} := S_y \quad W_{pl} := Z_y$$

$$M_y := W_{el} \cdot F_y = (1.011 \cdot 10^3) \text{ kN} \cdot \text{m} \quad M_p := W_{pl} \cdot F_y = (1.233 \cdot 10^3) \text{ kN} \cdot \text{m}$$

4 - Global Slenderness Properties

$$C_1 := 1$$

$$L_u := \frac{0.12 \cdot C_1 \cdot \pi}{F_y \cdot S_y} \cdot \sqrt{E \cdot G \cdot J \cdot I_y} = 80.666 \text{ m}$$

$$L := 5 \text{ m}$$

$$M_{cr} := C_1 \cdot \frac{\pi}{L} \cdot \sqrt{E \cdot I_z \cdot G \cdot J} = 79909.874 \text{ kN} \cdot \text{m}$$

$$\lambda_{LT} := \sqrt{\frac{M_p}{M_{cr}}} = 0.124$$

Local Slenderness properties

$$\text{if} \left(\frac{h}{b} \leq 3, \text{ "The standard is applicable", "NA"} \right) = \text{ "The standard is applicable"}$$

Appendix B.1 - AISC Bending Calculations

$$\lambda_f := \frac{b_{\text{flange}}}{t} = 39.667$$

$$\lambda_w := \frac{b_{\text{web}}}{t} = 82.333$$

$$\lambda_{pf} := 1.17 \cdot \sqrt{\frac{E}{F_y}} = 35.899$$

$$\lambda_{rf} := 1.24 \cdot \sqrt{\frac{E}{F_y}} = 38.047$$

classification of flange

```
class.flange := if (λf ≤ λpf)           = "Slender"
                || "Compact"
                else if λf ≤ λrf
                || "Non-Compact"
                else if λf > λrf
                || "Slender"
```

$$\lambda_{rw} := 3.01 \cdot \sqrt{\frac{E}{F_y}} = 92.357$$

$$\lambda_{pw} := 2.54 \cdot \sqrt{\frac{E}{F_y}} = 77.936$$

classification of web

```
class.web := if (λw ≤ λpw)           = "Non-Compact"
              || "Compact"
              else if λw ≤ λrw
              || "Non-Compact"
              else if λw > λrw
              || "Slender"
```

1-Yielding

$$M_{n.1} := M_p = (1.233 \cdot 10^3) \text{ kN} \cdot \text{m}$$

Appendix B.1 - AISC Bending Calculations

2-Flange Local Buckling

$$F_{el.f} := \left(\frac{4 \cdot \pi^2 \cdot E}{12 (1 - \mu^2)} \right) \cdot \left(\frac{t}{b_{flange}} \right)^2 = 443.448 \frac{N}{mm^2} \quad \text{elastic buckling stress of the flange}$$

$$b_{e.flange} := \begin{cases} \text{if } 0.772 \cdot b_{flange} \cdot \left(1 - 0.1 \cdot \sqrt{\frac{F_{el.f}}{F_y}} \right) \cdot \sqrt{\frac{F_{el.f}}{F_y}} < b_{flange} \\ 0.772 \cdot b_{flange} \cdot \left(1 - 0.1 \cdot \sqrt{\frac{F_{el.f}}{F_y}} \right) \cdot \sqrt{\frac{F_{el.f}}{F_y}} \\ \text{else} \\ b_{flange} \end{cases} = 365.9 \text{ mm}$$

$$b_{ineff.f} := \begin{cases} \text{if } b_{flange} - b_{e.flange} > 0 \\ b_{flange} - b_{e.flange} \\ \text{else} \\ 0 \end{cases} = 0.012 \text{ m}$$

$$h_{NA.1} := \frac{h}{2} - \frac{b_{ineff.f} \cdot t \cdot \left(\frac{h}{2} - \frac{t}{2} \right)}{A_g - b_{ineff.f} \cdot t} = 0.404 \text{ m}$$

$$I_{ineff.f} := b_{ineff.f} \cdot \frac{t^3}{12} + b_{ineff.f} \cdot t \cdot \left(h - h_{NA.1} - \frac{t}{2} \right)^2 = (1.851 \cdot 10^{-5}) \text{ m}^4$$

$$I_{eff1} := I_y - I_{ineff.f} = 1986070891.9 \text{ mm}^4 \quad S_{e.f} := \frac{I_{eff1}}{h - h_{NA.1}} = 4862741.661 \text{ mm}^3$$

$$M_{n.2} := \begin{cases} \text{if class.flange} = \text{"Compact"} \\ M_p \\ \text{else if class.flange} = \text{"Non-Compact"} \\ \min \left(M_p, M_p - (M_p - M_y) \cdot \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \\ \text{else if class.flange} = \text{"Slender"} \\ F_y \cdot S_{e.f} \end{cases} = 996.862 \text{ kN} \cdot \text{m}$$

Appendix B.1 - AISC Bending Calculations

3-Web Local Buckling

$$\alpha_w := \frac{2 \cdot b_{web} \cdot t}{b_{flange} \cdot t}$$

$$R_{pg} := \min \left(1 - \left(\frac{\alpha_w}{6.75 + 2.12 \cdot \alpha_w} \cdot \left(\frac{\lambda_w}{\lambda_{rw}} - 1 \right) \right), 1 \right) = 1$$

$$M_{n.3} := \begin{cases} \text{if class.web} = \text{"Compact"} & = 1165.471 \text{ kN} \cdot \text{m} \\ \parallel M_p \\ \text{else if class.web} = \text{"Non-Compact"} \\ \parallel \min \left(M_p, M_p - (M_p - M_y) \cdot \frac{\lambda_w - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \\ \text{else if class.web} = \text{"Slender"} \\ \parallel \min (F_y \cdot S_y \cdot R_{pg}, S_{e.f} \cdot F_y \cdot R_{pg}) \end{cases}$$

$$M_n := \min (M_{n.1}, M_{n.2}, M_{n.3}) = 996.862 \text{ kN} \cdot \text{m} \quad \phi_{LRFD} \cdot M_n = 897.176 \text{ kN} \cdot \text{m}$$

Appendix B.1 - AISC Bending Calculations

AISC 370-21 Bending for CHS Sections

Example section HSS7.5x0.375

Sectional Properties

$$D := 190.5 \text{ mm} \quad t := 9.525 \text{ mm} \quad \frac{D}{t} = 20$$

$$A_g := \frac{\pi}{4} \cdot (D^2 - (D - 2 \cdot t)^2) = (5.415 \cdot 10^3) \text{ mm}^2$$

$$I := \frac{\pi}{4} \cdot \left(\left(\frac{D}{2} \right)^4 - \left(\frac{D}{2} - t \right)^4 \right) = (2.223 \cdot 10^7) \text{ mm}^4 \quad \lambda := 0.1$$

$$L_y := \sqrt{\frac{\lambda^2 \cdot \pi^2 \cdot E \cdot I}{A_g \cdot F_y}} = 617.626 \text{ mm} \quad \lambda_r := 0.31 \cdot \frac{E}{F_y} = 291.854$$

$$S_{y.chs} := \frac{\pi \cdot (D^4 - (D - 2 \cdot t)^4)}{32 \cdot D} = (2.334 \cdot 10^5) \text{ mm}^3$$

$$Z_{y.chs} := \frac{D^3 - (D - 2 \cdot t)^3}{6} = (3.123 \cdot 10^5) \text{ mm}^3$$

$$M_{y.chs} := S_{y.chs} \cdot F_y = 47.849 \text{ kN} \cdot \text{m} \quad M_{p.chs} := Z_{y.chs} \cdot F_y = 64.011 \text{ kN} \cdot \text{m}$$

$$\text{class} := \begin{cases} \text{if } \left(\frac{D}{t} \leq 0.08 \cdot \frac{E}{F_y} \right) & \text{= "Compact"} \\ \parallel \text{"Compact"} \\ \text{else if } \frac{D}{t} \leq 0.31 \cdot \frac{E}{F_y} & \\ \parallel \text{"Non-Compact"} \\ \text{else if } \frac{D}{t} > 0.31 \cdot \frac{E}{F_y} & \\ \parallel \text{"Slender"} \end{cases}$$

$$M_{n.chs} := \begin{cases} \text{if class = "Compact"} & = 64.011 \text{ kN} \cdot \text{m} \\ \parallel M_{p.chs} \\ \text{else if class = "Non-Compact"} & \\ \parallel \left(\frac{0.068 \cdot \lambda_r}{\left(\frac{D}{t} \right)} + 1 \right) \cdot F_y \cdot S_{y.chs} \\ \text{else if class = "Slender"} & \\ \parallel 0 \end{cases}$$

AISC 370-21 Bending with continuous strength method

Rectangular Sections

Material Properties

$$F_y := 205 \text{ MPa} \quad E := 193000 \frac{\text{N}}{\text{mm}^2} \quad G := 76920 \text{ MPa} \quad \mu := 0.3 \quad \text{Table C-3}$$

$$F_u := 515 \text{ MPa} \quad \varepsilon_y := \frac{F_y}{E} = 0.001062 \quad E_{sh} := \frac{F_u - F_y}{0.16 \left(1 - \frac{F_y}{F_u} \right) - \varepsilon_y} = 3255 \text{ MPa}$$

$$\alpha := 2 \quad \beta_0 := 0.35 \quad \beta_1 := 0.44 \quad \beta_2 := 0.61 \quad \text{Table 6.1} \quad \phi_{LRFD} := 0.9$$

2- Cross-Sectional Properties

$$t_{nominal} := 9.525 \text{ mm} = 9.525 \text{ mm}$$

$$h := 812.8 \text{ mm} = 812.8 \text{ mm} \quad b := 406.4 \text{ mm} = 406.4 \text{ mm} \quad t := t_{nominal} \cdot 0.95 \quad \text{design thickness}$$

$$r_i := t \quad b_{flange} := b - 3 \cdot t = 379.254 \text{ mm} \quad b_{web} := h - 3 \cdot t = 785.654 \text{ mm} \quad r_e := r_i + t$$

$$A_{ri} := \left(1 - \frac{\pi}{4} \right) \cdot r_i^2 = 17.572 \text{ mm}^2 \quad A_{re} := \left(1 - \frac{\pi}{4} \right) \cdot r_e^2 = 70.286 \text{ mm}^2$$

$$A_g := h \cdot b - (b - 2 \cdot t) \cdot (h - 2 \cdot t) - 4 \cdot A_{re} + 4 \cdot A_{ri} = 21526.094 \text{ mm}^2$$

$$h_{ri} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i \quad b_{ri} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$h_{re} := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e \quad b_{re} := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$I_e := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)} \right) \cdot r_e^4 \quad I_i := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)} \right) \cdot r_i^4$$

$$I_y := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot h_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot h_{ri}^2) = (1.91 \cdot 10^9) \text{ mm}^4$$

$$I_z := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t) \cdot (b - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot b_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot b_{ri}^2) = (6.605 \cdot 10^8) \text{ mm}^4$$

Appendix B.2 - AISC Bending CSM

$$S_y := \frac{I_y}{\frac{h}{2}} = (4.7 \cdot 10^6) \text{ mm}^3$$

$$Z_y := \frac{b \cdot h^2}{4} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - 4 \cdot A_{re} \cdot h_{re} + 4 \cdot A_{ri} \cdot h_{ri} = 5727768.253 \text{ mm}^3$$

Applicability check of CSM Table A-2.1.1

$$L_b := 5 \text{ m} \quad L_p := \sqrt{\frac{\lambda^2 \cdot \pi^2 \cdot E \cdot I_z}{A_g \cdot F_y}} = 16.885 \text{ m} \quad \lambda := 1.0 \quad k := 23.9$$

$$F_{el} := \frac{k \cdot \pi^2 \cdot E \cdot t^2}{12 \cdot (1 - \mu^2) \cdot b_{web}^2} = 553.028 \text{ MPa} \quad \lambda_l := \sqrt{\frac{F_y}{F_{el}}} = 0.609 \quad \Lambda := 15$$

if $(L_b \leq 0.5 \cdot L_p, \text{"Ok"}, \text{"Not ok"}) = \text{"Ok"}$

if $(\lambda_l \leq 1.6, \text{"Ok"}, \text{"Not ok"}) = \text{"Ok"}$

If both conditions are true then standard is applicable

$$M_y := S_y \cdot F_y = 963.597 \text{ kN} \cdot \text{m} \quad M_p := Z_y \cdot F_y = (1.174 \cdot 10^3) \text{ kN} \cdot \text{m}$$

$$\begin{aligned} \text{def_capacity} &:= \text{if } \lambda_l \leq 0.68 &= 1.5 \\ &\left\| \min \left(\frac{0.25}{\lambda_l^{3.6}}, \Lambda, \frac{0.10 \cdot \left(1 - \frac{F_y}{F_u} \right)}{\varepsilon_y} \right) \right. \\ &\text{else if } \lambda_l > 0.68 \\ &\left\| \left(1 - \frac{0.222}{\lambda_l^{1.05}} \right) \cdot \frac{1}{\lambda_l^{1.05}} \right. \end{aligned}$$

$$\begin{aligned} M_n &:= \text{if } \text{def_capacity} < 1.0 &= 1087.563 \text{ kN} \cdot \text{m} \\ &\left\| \text{def_capacity} \cdot M_y \right. \\ &\text{else if } \text{def_capacity} > 1.0 \\ &\left\| M_p \cdot \left(1 + \frac{E_{sh}}{E} \cdot \frac{S_y}{Z_y} \cdot (\text{def_capacity} - 1) - \frac{\left(1 - \frac{S_y}{Z_y} \right)}{\text{def_capacity}^\alpha} \right) \right. \end{aligned}$$

$$\phi_{LRFD} \cdot M_n = 978.806 \text{ kN} \cdot \text{m}$$

Appendix B.2 - AISC Bending CSM

CHS Sections using continuous strength method

Example section HSS7.5x0.375

Sectional Properties

$$D := 190.5 \text{ mm} \quad t = 9.05 \text{ mm}$$

$$\frac{D}{t} = 21.053$$

$$A_{g.chs} := \frac{\pi}{4} \cdot (D^2 - (D - 2 \cdot t)^2) = (5.158 \cdot 10^3) \text{ mm}^2$$

$$I := \frac{\pi}{4} \cdot \left(\left(\frac{D}{2} \right)^4 - \left(\frac{D - 2 \cdot t}{2} \right)^4 \right) = (2.128 \cdot 10^7) \text{ mm}^4$$

$$L_y := \sqrt{\frac{\lambda^2 \cdot \pi^2 \cdot E \cdot I}{A_g \cdot F_y}} = (3.031 \cdot 10^3) \text{ mm}$$

$$S_{y.chs} := \frac{\pi \cdot (D^4 - (D - 2 \cdot t)^4)}{32 \cdot D} = (2.234 \cdot 10^5) \text{ mm}^3$$

$$Z_{y.chs} := \frac{D^3 - (D - 2 \cdot t)^3}{6} = (2.982 \cdot 10^5) \text{ mm}^3$$

$$F_{el.chs} := \frac{E}{\sqrt{3 \cdot (1 - \mu^2)}} \cdot \left(\frac{2 \cdot t}{D} \right) = 11097 \frac{\text{N}}{\text{mm}^2}$$

$$\lambda_{l.chs} := \sqrt{\frac{F_y}{F_{el.chs}}} = 0.14$$

$$\begin{aligned} \text{def_capacity.chs} := & \text{if } \lambda_{l.chs} \leq 0.3 &= 15 \\ & \left\| \min \left(\frac{0.00444}{\lambda_{l.chs}^{4.5}}, \Lambda, \frac{0.10 \cdot \left(1 - \frac{F_y}{F_u} \right)}{\varepsilon_y} \right) \right\| \\ & \text{else if } \lambda_{l.chs} > 0.3 \\ & \left\| \left(\frac{1 - 0.224}{\lambda_{l.chs}^{0.342}} \right) \cdot \frac{1}{\lambda_{l.chs}^{0.342}} \right\| \end{aligned}$$

$$M_{y.chs} := S_{y.chs} \cdot F_y = 45.803 \text{ kN} \cdot \text{m} \quad M_{p.chs} := Z_{y.chs} \cdot F_y = 61.125 \text{ kN} \cdot \text{m}$$

Appendix B.2 - AISC Bending CSM

$$M_{n.chs} := \begin{cases} \text{if } def_capacity.chs < 1.0 \\ \quad def_capacity.chs \cdot M_{y.chs} \\ \text{else if } def_capacity.chs > 1.0 \\ \quad M_{p.chs} \cdot \left(1 + \frac{E_{sh}}{E} \cdot \frac{S_{y.chs}}{Z_{y.chs}} \cdot (def_capacity.chs - 1) - \frac{\left(1 - \frac{S_{y.chs}}{Z_{y.chs}} \right)}{def_capacity.chs^\alpha} \right) \end{cases} = 71.9 \text{ kN} \cdot \text{m}$$

$$\phi M_n := 0.95 \cdot M_{n.chs} = 68.277 \text{ kN} \cdot \text{m}$$

Appendix C - Eurocode Bending Calculations

EN 1993 1-4 Generation 1 (amd.2020)- HSS Sections

Members without holes:

1-Material Properties $F_y := 210 \text{ MPa}$ $E := 200000 \frac{N}{mm^2}$ $\mu := 0.3$

$$G := \frac{E}{2 \cdot (1 + \mu)} = (7.692 \cdot 10^4) \text{ MPa}$$

$$\varepsilon := \sqrt{\frac{235 \text{ MPa}}{F_y} \cdot \frac{E}{210000 \text{ MPa}}} = 1.032$$

2- Cross-Sectional Properties

$h := 508 \text{ mm} = 508 \text{ mm}$ $b := 508 \text{ mm} = 508 \text{ mm}$ $t := 12.7 \text{ mm} = 12.7 \text{ mm}$

$r_i := t$ $b_{\text{flange}} := b - 3 \cdot t = 469.9 \text{ mm}$ $b_{\text{web}} := h - 3 \cdot t = 469.9 \text{ mm}$ $r_e := r_i + t$

$$A_{ri} := \left(1 - \frac{\pi}{4}\right) \cdot r_i^2 = 34.613 \text{ mm}^2$$

$$A_{re} := \left(1 - \frac{\pi}{4}\right) \cdot r_e^2 = 138.453 \text{ mm}^2$$

$$A_g := h \cdot b - (b - 2 \cdot t) \cdot (h - 2 \cdot t) - 4 \cdot A_{re} + 4 \cdot A_{ri} = (2.475 \cdot 10^4) \text{ mm}^2$$

$$h_{ri} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$b_{ri} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$h_{re} := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$b_{re} := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$I_e := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_e^4$$

$$I_i := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_i^4$$

$$I_y := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot h_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot h_{ri}^2) = (1.003 \cdot 10^9) \text{ mm}^4$$

$$I_z := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t) \cdot (b - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot b_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot b_{ri}^2) = (1.003 \cdot 10^9) \text{ mm}^4$$

Appendix C.1 - EN 1993 1-4 Bending Calculations Gen 1

$$S_y := \frac{I_y}{h} = (3.949 \cdot 10^6) \text{ mm}^3$$

$$Z_y := \frac{b \cdot h^2}{4} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - 4 \cdot A_{re} \cdot h_{re} + 4 \cdot A_{ri} \cdot h_{ri} = (4.57 \cdot 10^6) \text{ mm}^3$$

$$r_c := r_i + \frac{t}{2} = 19.05 \text{ mm}$$

$$b_m := b - t \quad h_m := h - t$$

$$A_h := b_m \cdot h_m - (4 - \pi) \cdot r_c^2 = (2.45 \cdot 10^5) \text{ mm}^2$$

$$h_0 := (b_m + h_m - 4 \cdot r_c) \cdot 2 + 2 \cdot \pi \cdot r_c = (1.948 \cdot 10^3) \text{ mm}$$

$$J := \frac{4 \cdot A_h^2 \cdot t}{h_0} = (1.565 \cdot 10^9) \text{ mm}^4$$

3-yielding and plastic bending moment:

$$W_{el} := S_y \quad W_{pl} := Z_y$$

$$M_y := W_{el} \cdot F_y = 829.38 \text{ kN} \cdot \text{m} \quad M_p := W_{pl} \cdot F_y = 959.679 \text{ kN} \cdot \text{m}$$

4-cross-sectional classification:

$$\lambda_f := \frac{b_{flange}}{t} = 37$$

$$\lambda_w := \frac{b_{web}}{t} = 37$$

$$\lambda_{f1} := 33 \cdot \varepsilon = 34.068$$

$$\lambda_{f2} := 35 \cdot \varepsilon = 36.132$$

$$\lambda_{f3} := 37 \cdot \varepsilon = 38.197$$

$$\lambda_{w1} := 72 \cdot \varepsilon = 74.33$$

$$\lambda_{w2} := 76 \cdot \varepsilon = 78.459$$

$$\lambda_{w3} := 90 \cdot \varepsilon = 92.912$$

classification of flange

```
class.flange := if (λf ≤ λf1) = "Class 3"
                || "Class 1"
                else if λf ≤ λf2
                || "Class 2"
                else if λf ≤ λf3
                || "Class 3"
                else
                || "Class 4"
```

classification of web

```
class.web := if (λw ≤ λw1) = "Class 1"
              || "Class 1"
              else if λw ≤ λw2
              || "Class 2"
              else if λw ≤ λw3
              || "Class 3"
              else
              || "Class 4"
```

Appendix C.1 - EN 1993 1-4 Bending Calculations Gen 1

$$\text{class.section} := \begin{cases} \text{if } (\text{class.web} > \text{class.flange}) \\ \parallel \text{ class.web} \\ \text{else if } \text{class.web} \leq \text{class.flange} \\ \parallel \text{ class.flange} \end{cases} = \text{"Class 3"}$$

5- Calculations of the effective widths:

5-1-Flange

$$\psi_f := 1 \quad k_{\sigma f} := 4$$

$$\lambda_{pf} := \frac{\lambda_f}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma f}}} = 0.631$$

$$\rho_f := \min \left(1, \frac{0.772 \cdot \lambda_{pf} - 0.079}{\lambda_{pf}^2} \right) = 1$$

$$b_{\text{flange.eff}} := \begin{cases} \text{if } \text{class.flange} = \text{"Class 4"} \\ \parallel \rho_f \cdot b_{\text{flange}} \\ \text{else} \\ \parallel b_{\text{flange}} \end{cases} = 469.9 \text{ mm}$$

$$b_{\text{ineff.f}} := b_{\text{flange}} - b_{\text{flange.eff}} = 0 \text{ m}$$

$$h_{\text{NA.1}} := \frac{h}{2} - \frac{b_{\text{ineff.f}} \cdot t \cdot \left(h - \frac{t}{2} \right)}{A_g - b_{\text{ineff.f}} \cdot t} = 254 \text{ mm}$$

the height is measured from bottom of the section

$$h_{d1} := \frac{h}{2} - h_{\text{NA.1}} = 0 \text{ mm} \quad \text{shift of the NA due to the flange reduction}$$

$$I_{\text{ineff.f}} := b_{\text{ineff.f}} \cdot \frac{t^3}{12} + b_{\text{ineff.f}} \cdot t \cdot \left(h - h_{\text{NA.1}} - \frac{t}{2} \right)^2 = 0 \text{ m}^4$$

$$I_{\text{eff1}} := I_y - I_{\text{ineff.f}} = (1.003 \cdot 10^9) \text{ mm}^4 \quad S_{e.f} := \frac{I_{\text{eff1}}}{h - h_{\text{NA.1}}} = (3.949 \cdot 10^6) \text{ mm}^3$$

5-2-Web

$$\psi_w := \frac{-(h_{NA.1} - 3 \cdot t)}{h - h_{NA.1} - 3 \cdot t} = -1$$

$$k_{\sigma w} := 7.81 - 6.29 \cdot \psi_w + 9.78 \cdot \psi_w^2 = 23.88$$

$$\lambda_{pw} := \frac{\lambda_w}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma w}}} = 0.258 \quad \rho_w := \min\left(1, \frac{0.772 \cdot \lambda_{pw} - 0.079}{\lambda_{pw}^2}\right) = 1$$

$$b_{web,eff} := \begin{cases} \text{if class.web} = \text{"Class 4"} & = 469.9 \text{ mm} \\ \parallel \rho_w \cdot b_{web} \\ \text{else} \\ \parallel b_{web} \end{cases}$$

$$b_{ineff.w} := b_{web} - b_{web,eff} = 0 \text{ m}$$

$$b_{e1} := 0.4 \cdot b_{web,eff} = 187.96 \text{ mm}$$

$$b_{e2} := 0.6 \cdot b_{web,eff} = 281.94 \text{ mm}$$

$$h_{NA.2} := h_{NA.1} - \frac{2 \cdot \left(b_{ineff.w} \cdot t \cdot \left(b_{e2} + \frac{b_{ineff.w}}{2} + h_{NA.1}\right)\right)}{A_g - b_{ineff.f} \cdot t - 2 \cdot t \cdot b_{ineff.w}} = 254 \text{ mm} \quad \text{new depth of the neutral axis after the flange and web reduction}$$

$$h_{d2} := h_{NA.1} - h_{NA.2} = 0 \text{ mm} \quad \text{shift of the neutral axis downwards due to the web reduction}$$

$$I_{ineff} := b_{ineff.f} \cdot \frac{t^3}{12} + b_{ineff.f} \cdot t \cdot \left(h - h_{NA.2} - \frac{t}{2}\right)^2 + 2 \cdot \left(\frac{t \cdot b_{ineff.w}^3}{12} + t \cdot b_{ineff.w} \cdot \left(\frac{b_{ineff.w}}{2} + b_{e2} + h_{d1} + h_{d2}\right)^2\right) = 0 \text{ m}^4$$

$$I_{eff2} := I_y - I_{ineff} = (1.003 \cdot 10^9) \text{ mm}^4$$

$$S_{ec} := \frac{I_{eff2}}{h - h_{NA.2}} = (3.949 \cdot 10^6) \text{ mm}^3 \quad S_{et} := \frac{I_{eff2}}{h_{NA.2}} = (3.949 \cdot 10^6) \text{ mm}^3$$

Appendix C.1 - EN 1993 1-4 Bending Calculations Gen 1

$$W_{\text{eff}} := \min(S_{\text{ec}}, S_{\text{et}}) = (3.949 \cdot 10^6) \text{ mm}^3$$

$$M_{\text{Rk}} := \begin{cases} \text{if (class.section = "Class 3")} & \\ \parallel W_{\text{el}} \cdot F_y & \\ \text{else if (class.section = "Class 4")} & \\ \parallel W_{\text{eff}} \cdot F_y & \\ \text{else} & \\ \parallel W_{\text{pl}} \cdot F_y & \end{cases} = 829.38 \text{ kN} \cdot \text{m}$$

EN 1993 1-4 Generation 2 (amd.2025)- HSS Sections

Members without holes:

1-Material Properties $F_y := 210 \text{ MPa}$ $E := 200000 \frac{N}{mm^2}$ $\mu := 0.3$

$$G := \frac{E}{2 \cdot (1 + \mu)} = (7.692 \cdot 10^4) \text{ MPa}$$

$$\varepsilon := \sqrt{\frac{235 \text{ MPa}}{F_y} \cdot \frac{E}{210000 \text{ MPa}}} = 1.032$$

2- Cross-Sectional Properties

$h := 508 \text{ mm} = 508 \text{ mm}$ $b := 508 \text{ mm} = 508 \text{ mm}$ $t := 12.7 \text{ mm} = 12.7 \text{ mm}$

$r_i := t$ $b_{\text{flange}} := b - 3 \cdot t = 469.9 \text{ mm}$ $b_{\text{web}} := h - 3 \cdot t = 469.9 \text{ mm}$ $r_e := r_i + t$

$$A_{ri} := \left(1 - \frac{\pi}{4}\right) \cdot r_i^2 = 34.613 \text{ mm}^2$$

$$A_{re} := \left(1 - \frac{\pi}{4}\right) \cdot r_e^2 = 138.453 \text{ mm}^2$$

$$A_g := h \cdot b - (b - 2 \cdot t) \cdot (h - 2 \cdot t) - 4 \cdot A_{re} + 4 \cdot A_{ri} = (2.475 \cdot 10^4) \text{ mm}^2$$

$$h_{ri} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$b_{ri} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i$$

$$h_{re} := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$b_{re} := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e$$

$$I_e := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_e^4$$

$$I_i := \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3 \cdot (12 - 3 \cdot \pi)}\right) \cdot r_i^4$$

$$I_y := \frac{b \cdot h^3}{12} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot h_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot h_{ri}^2) = (1.003 \cdot 10^9) \text{ mm}^4$$

$$I_z := \frac{h \cdot b^3}{12} - \frac{(h - 2 \cdot t) \cdot (b - 2 \cdot t)^3}{12} - 4 \cdot (I_e + A_{re} \cdot b_{re}^2) + 4 \cdot (I_i + A_{ri} \cdot b_{ri}^2) = (1.003 \cdot 10^9) \text{ mm}^4$$

Appendix C.2 - EN 1993 1-4 Bending Calculations Gen 2

$$S_y := \frac{I_y}{h} = (3.949 \cdot 10^6) \text{ mm}^3$$

$$Z_y := \frac{b \cdot h^2}{4} - \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - 4 \cdot A_{re} \cdot h_{re} + 4 \cdot A_{ri} \cdot h_{ri} = (4.57 \cdot 10^6) \text{ mm}^3$$

$$r_c := r_i + \frac{t}{2} = 19.05 \text{ mm}$$

$$b_m := b - t \quad h_m := h - t$$

$$A_h := b_m \cdot h_m - (4 - \pi) \cdot r_c^2 = (2.45 \cdot 10^5) \text{ mm}^2$$

$$h_0 := (b_m + h_m - 4 \cdot r_c) \cdot 2 + 2 \cdot \pi \cdot r_c = (1.948 \cdot 10^3) \text{ mm}$$

$$J := \frac{4 \cdot A_h^2 \cdot t}{h_0} = (1.565 \cdot 10^9) \text{ mm}^4$$

3-yielding and plastic bending moment:

$$W_{el} := S_y \quad W_{pl} := Z_y$$

$$M_y := W_{el} \cdot F_y = 829.38 \text{ kN} \cdot \text{m} \quad M_p := W_{pl} \cdot F_y = 959.679 \text{ kN} \cdot \text{m}$$

4-cross-sectional classification:

$$\lambda_f := \frac{b_{flange}}{t} = 37$$

$$\lambda_w := \frac{b_{web}}{t} = 37$$

$$\lambda_{f1} := 33 \cdot \varepsilon = 34.068$$

$$\lambda_{f2} := 35 \cdot \varepsilon = 36.132$$

$$\lambda_{f3} := 37 \cdot \varepsilon = 38.197$$

$$\lambda_{w1} := 72 \cdot \varepsilon = 74.33$$

$$\lambda_{w2} := 76 \cdot \varepsilon = 78.459$$

$$\lambda_{w3} := 99 \cdot \varepsilon = 102.203$$

classification of flange

```
class.flange := if (λf ≤ λf1) = "Class 3"
                || "Class 1"
                else if λf ≤ λf2
                || "Class 2"
                else if λf ≤ λf3
                || "Class 3"
                else
                || "Class 4"
```

classification of web

```
class.web := if (λw ≤ λw1) = "Class 1"
              || "Class 1"
              else if λw ≤ λw2
              || "Class 2"
              else if λw ≤ λw3
              || "Class 3"
              else
              || "Class 4"
```

Appendix C.2 - EN 1993 1-4 Bending Calculations Gen 2

$$\text{class.section} := \begin{cases} \text{if (class.web} > \text{class.flange)} & = \text{"Class 3"} \\ \parallel \text{class.web} \\ \text{else if class.web} \leq \text{class.flange} \\ \parallel \text{class.flange} \end{cases}$$

5- Calculations of the effective widths:

5-1-Flange

$$\psi_f := 1 \quad k_{\sigma f} := 4$$

$$\lambda_{pf} := \frac{\lambda_f}{27.7 \cdot \varepsilon \cdot \sqrt{k_{\sigma f}}} = 0.647$$

$$\rho_f := \min \left(1, \frac{0.772 \cdot \lambda_{pf} - 0.079}{\lambda_{pf}^2} \right) = 1$$

$$b_{\text{flange.eff}} := \begin{cases} \text{if class.flange} = \text{"Class 4"} & = 469.9 \text{ mm} \\ \parallel \rho_f \cdot b_{\text{flange}} \\ \text{else} \\ \parallel b_{\text{flange}} \end{cases}$$

$$b_{\text{ineff.f}} := b_{\text{flange}} - b_{\text{flange.eff}} = 0 \text{ m}$$

$$h_{\text{NA.1}} := \frac{h}{2} - \frac{b_{\text{ineff.f}} \cdot t \cdot \left(h - \frac{t}{2} \right)}{A_g - b_{\text{ineff.f}} \cdot t} = 254 \text{ mm}$$

the height is measured from bottom of the section

$$h_{d1} := \frac{h}{2} - h_{\text{NA.1}} = 0 \text{ mm} \quad \text{shift of the NA due to the flange reduction}$$

$$I_{\text{ineff.f}} := b_{\text{ineff.f}} \cdot \frac{t^3}{12} + b_{\text{ineff.f}} \cdot t \cdot \left(h - h_{\text{NA.1}} - \frac{t}{2} \right)^2 = 0 \text{ m}^4$$

$$I_{\text{eff1}} := I_y - I_{\text{ineff.f}} = (1.003 \cdot 10^9) \text{ mm}^4 \quad S_{e.f} := \frac{I_{\text{eff1}}}{h - h_{\text{NA.1}}} = (3.949 \cdot 10^6) \text{ mm}^3$$

5-2-Web

$$\psi_w := \frac{-(h_{NA.1} - 3 \cdot t)}{h - h_{NA.1} - 3 \cdot t} = -1$$

$$k_{\sigma w} := 7.81 - 6.29 \cdot \psi_w + 9.78 \cdot \psi_w^2 = 23.88$$

$$\lambda_{pw} := \frac{\lambda_w}{27.7 \cdot \varepsilon \cdot \sqrt{k_{\sigma w}}} = 0.265 \quad \rho_w := \min \left(1, \frac{0.772 \cdot \lambda_{pw} - 0.079}{\lambda_{pw}^2} \right) = 1$$

$$b_{web,eff} := \begin{cases} \text{if class.web} = \text{"Class 4"} & = 469.9 \text{ mm} \\ \parallel \rho_w \cdot b_{web} \\ \text{else} \\ \parallel b_{web} \end{cases}$$

$$b_{ineff.w} := b_{web} - b_{web,eff} = 0 \text{ m}$$

$$b_{e1} := 0.4 \cdot b_{web,eff} = 187.96 \text{ mm}$$

$$b_{e2} := 0.6 \cdot b_{web,eff} = 281.94 \text{ mm}$$

$$h_{NA.2} := h_{NA.1} - \frac{2 \cdot \left(b_{ineff.w} \cdot t \cdot \left(b_{e2} + \frac{b_{ineff.w}}{2} + h_{NA.1} \right) \right)}{A_g - b_{ineff.f} \cdot t - 2 \cdot t \cdot b_{ineff.w}} = 254 \text{ mm} \quad \text{new depth of the neutral axis after the flange and web reduction}$$

$$h_{d2} := h_{NA.1} - h_{NA.2} = 0 \text{ mm} \quad \text{shift of the neutral axis downwards due to the web reduction}$$

$$I_{ineff} := b_{ineff.f} \cdot \frac{t^3}{12} + b_{ineff.f} \cdot t \cdot \left(h - h_{NA.2} - \frac{t}{2} \right)^2 + 2 \cdot \left(\frac{t \cdot b_{ineff.w}^3}{12} + t \cdot b_{ineff.w} \cdot \left(\frac{b_{ineff.w}}{2} + b_{e2} + h_{d1} + h_{d2} \right)^2 \right) = 0 \text{ m}^4$$

$$I_{eff2} := I_y - I_{ineff} = (1.003 \cdot 10^9) \text{ mm}^4$$

$$S_{ec} := \frac{I_{eff2}}{h - h_{NA.2}} = (3.949 \cdot 10^6) \text{ mm}^3 \quad S_{et} := \frac{I_{eff2}}{h_{NA.2}} = (3.949 \cdot 10^6) \text{ mm}^3$$

$$W_{eff} := \min(S_{ec}, S_{et}) = (3.949 \cdot 10^6) \text{ mm}^3$$

Appendix C.2 - EN 1993 1-4 Bending Calculations Gen 2

$$M_{Rk} := \begin{cases} \text{if (class.section = "Class 3")} & \\ \parallel W_{el} \cdot F_y & \\ \text{else if (class.section = "Class 4")} & \\ \parallel W_{eff} \cdot F_y & \\ \text{else} & \\ \parallel W_{pl} \cdot F_y & \end{cases} = 829.38 \text{ kN} \cdot \text{m}$$

$$M_{RD} := \frac{M_{Rk}}{1.1} = 753.981 \text{ kN} \cdot \text{m}$$

Appendix D - Strength Enhancements

Appendix D - Strength Enhancements

ASCE 8-22 Yield strength enhancements

(due to cold forming for flexural members)

Material Properties Austenitic EN 1.4301

$$F_y := 205 \frac{N}{mm^2} \quad E := 193000 \frac{N}{mm^2} \quad F_u := 515 \frac{N}{mm^2}$$

$$F_{uv} := F_u \quad F_{yv} := F_y \quad F_{yf} := F_y$$

$$\alpha := 1.44 \quad \beta_0 := 0.35 \quad \beta_1 := 0.44 \quad \beta_2 := 0.61 \quad \text{Table 6.1}$$

Sectional Properties

Example section HSS32x16x0.375

$$h := 812.8 \text{ mm}$$

$$b := 406.400 \text{ mm}$$

$$t := 9.525 \text{ mm} \quad r_i := t \quad \text{assume interior corner } r_i = t$$

$$r_e := r_i + t = 19.05 \text{ mm}$$

$$A_{\text{flange}} := b \cdot t = (3.871 \cdot 10^3) \text{ mm}^2 \quad \text{area of top flange (simplified)}$$

$$A_{\text{corner}} := \frac{\pi \cdot t \cdot (2 \cdot t + r_i)}{2} + 8 \cdot t^2 = (1.153 \cdot 10^3) \text{ mm}^2 \quad \text{area of the top flange corners}$$

$$C := \frac{A_{\text{corner}}}{A_{\text{flange}}} = 0.3$$

For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange;

$$\varepsilon_c := \frac{t}{(2 \cdot (2 \cdot r_i + t))} = 0.167$$

$$\varepsilon_u := 1 - \frac{F_{yv}}{F_{uv}} = 0.602 \quad \varepsilon_{p0.2} := 0.002 + \frac{F_{yv}}{E} \quad n_p := \frac{\log\left(\frac{F_{yv}}{F_{uv}}\right)}{\log\left(\frac{\varepsilon_{p0.2}}{\varepsilon_u}\right)} = 0.174$$

$$B_c := 0.85 \cdot \left(\frac{\varepsilon_c}{\varepsilon_{p0.2}} + 1\right)^{n_p} = 1.712$$

$$F_{yc} := B_c \cdot F_{yv}$$

$$F_{ya} := C \cdot F_{yc} + (1 - C) \cdot F_{yf} = 249 \frac{N}{mm^2}$$

AISC 370-21 & EN 1993 1-4 2025 Yield strength enhancements

due to cold forming for flexural members

Example section HSS32x16x0.375

$$A_z := \left(1 - \frac{\pi}{4}\right) \cdot r_e^2 = 77.88 \text{ mm}^2 \quad A_{ep} := \left(1 - \frac{\pi}{4}\right) \cdot r_i^2 = 19.47 \text{ mm}^2$$

$$A_g := h \cdot b - (h - 2 \cdot t) \cdot (b - 2 \cdot t) - 4 \cdot A_z + 4 \cdot A_{ep} = (2.263 \cdot 10^4) \text{ mm}^2 \quad \text{(gross cross sectional area accounting for corners)}$$

$$h_z := \frac{h}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e \quad h_{ep} := \frac{h - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i \quad b_{flange} := h - 3 \cdot t = 784.225 \text{ mm}$$

$$b_z := \frac{b}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_e \quad b_{ep} := \frac{b - 2 \cdot t}{2} - \frac{(10 - 3 \cdot \pi)}{(12 - 3 \cdot \pi)} \cdot r_i \quad b_{web} := b - 3 \cdot t = 377.825 \text{ mm}$$

$$A_{corner} := \pi \cdot t \cdot (2 \cdot t + r_i) + 16 \cdot t^2 = 2306.679 \text{ mm}^2 \quad \text{area of all 4 corners}$$

$$\epsilon_{corner} := \frac{t}{2 \cdot (2 \cdot r_i + t)} = 0.17 \quad \epsilon_y := \frac{F_y}{E} + 0.002 = 0.003 \quad \epsilon_u = 0.602$$

$$\epsilon_{wall} := \frac{t}{900 \text{ mm}} + \frac{\pi \cdot t}{2 \cdot (b + h - 2 \cdot t)} = 0.023$$

$$n := \frac{\log\left(\frac{F_y}{F_u}\right)}{\log\left(\frac{\epsilon_y}{\epsilon_u}\right)} = 0.174$$

$$F_{y,corner} := 0.85 \cdot \frac{F_y}{\epsilon_y^n} \cdot (\epsilon_{corner} + \epsilon_y)^n = 351.015 \frac{N}{\text{mm}^2}$$

if $(F_y \leq F_{y,corner} \leq F_u, \text{"ok"}, \text{"not ok"}) = \text{"ok"}$

$$F_{y,wall} := 0.85 \cdot \frac{F_y}{\epsilon_y^n} \cdot (\epsilon_{wall} + \epsilon_y)^n = 253.24 \frac{N}{\text{mm}^2}$$

if $(F_y \leq F_{y,wall} \leq F_u, \text{"ok"}, \text{"not ok"}) = \text{"ok"}$

$$F_{y,avg} := \frac{F_{y,corner} \cdot A_{corner} + F_{y,wall} \cdot (A_g - A_{corner})}{A_g} = 263.206 \frac{N}{\text{mm}^2}$$

Appendix D - Strength Enhancements

HSS7.5×0.375 CHS section for AISC

$D := 190.5 \text{ mm}$ $t = 9.525 \text{ mm}$

$$\epsilon_{\text{rnd}} := \frac{t}{2 \cdot (D - t)} = 0.026$$

$$F_{y,\text{rnd}} := 0.85 \cdot \frac{F_y}{\epsilon_y^n} \cdot (\epsilon_{\text{rnd}} + \epsilon_y)^n \quad \text{Eqn. B4-4} \quad \text{if } (F_y \leq F_{y,\text{rnd}} \leq F_u, \text{"ok"}, \text{"not ok"}) = \text{"ok"}$$

$$F_{y,\text{rnd}} = 258.499 \frac{\text{N}}{\text{mm}^2}$$

