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MEng MINOR DISSERTATION

Assessing possible economic or operational benefits of class 3 webs without stiffeners vs class 4 webs with stiffeners in steel plate girders

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4 May 2021

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LIST OF ABBREVIATIONS AND SYMBOLS

The following symbols and abbreviations are used throughout this minor dissertation with any deviations or additional nomenclature noted in the text where they appear.

Symbol	Description	Units
ANSI/AISC 360-05	Specifications for structural steel buildings by the American Institute of Steel Construction	-
A_f	Flange area	mm^2
A_{tf}	Area of top flange	mm^2
A_{bf}	Area of bottom flange	mm^2
A_s	Area of steel section; area of stiffener or pair of stiffeners	mm^2
A_{tot}	Total cross-sectional area	mm^2
$A_{tot.min}$	Minimum required total cross-sectional area	mm^2
A_v	Shear area	mm^2
A_w	Web area	mm^2
b	Plate width	mm
b_f	Breadth of flange	mm
b_{ad}	Adapted plate width	mm
b_{opt}	Optimum plate width	mm
B_r	Factored bearing resistance of member or component	kN
BS5950-1	BS5950-1:2000: British Standard for structural use of steelwork in building- Part 1: Code of practice for design of rolled and welded sections	-
CAN/CSA-S16-01	Limit state design of steel structures by the Canadian Standards Association	-
E	Elastic modulus of steel ($E = 200 \times 10^3$ MPa)	MPa
EN1993-1-5	Eurocode 3-Design of steel Structures-Part 1-5: Plated structural elements	-
FCAW	Flux cored arc welding	-
f_{cri}	Inelastic critical plate-buckling stress in shear	MPa

f_y	Specified minimum yield stress	MPa
f_{ybf}	Yield strength of bottom flange	MPa
f_{yw}	Yield strength of bottom web	MPa
f_{ytf}	Yield strength of top flange	MPa
f_s	Ultimate shear stress	MPa
f_u	Ultimate strength	MPa
GMAW	Gas metal arc welding	-
Greenbook	Structural steel connections handbook by the Southern African Institute of Steel Construction: 1st Edition of 2012	
h	Girder depth	mm
h_w	Clear depth of the web between the flanges or between web fillets of rolled section (web depth)	mm
$h_{w.ad}$	Adapted web depth	mm
$h_{w.opt}$	Optimum web depth	mm
k_v	Shear buckling coefficient	-
I_x	Moment of inertia in the x axis	mm^4
I_y	Moment of inertia in the y axis	mm^4
L	Gross length; length of a member; beam span	mm
M_{el}	Elastic bending moment resistance	kNm
M_{fRd}	Bending moment resistance based on the flanges only method	kNm
M_r	Factored moment resistance of member component	kNm
M'_r	Reduced factored moment resistance	kNm
M_{pl}	Plastic moment resistance	kNm
M_u	Ultimate bending moment	kNm
M_y	Yield moment	kNm
s	Centre to centre distance between transverse web stiffeners	mm
SANS	South African National Standard	-

SANS 10162-1	SANS 10162-1:2011 Edition 2.1: South African national standard for the structural use of steel Part 1: Limit states design of hot rolled steelwork	-
SANS2001-CS1	SANS 2001-CS1: 2012 Edition 1.1: South African national standard for construction works: Part CS1: Structural steelwork	-
SANS 50025-2	SANS50025-2:2009 Edition 1: South African national standard for hot rolled products of structural steels Part 2: Technical delivery conditions for non-alloy structural steels	-
SASCH	Steel construction handbook by the Southern African Institute of Steel Construction: 8th Edition of 2016	-
SAW	Submerged arc welding	-
SMAW	Shielded metal arc welding	-
t	Plate thickness	mm
t_f	Flange thickness	mm
$t_{f.ad}$	Adapted flange thickness	mm
$t_{f.opt}$	Optimum flange thickness	mm
t_w	Web thickness	mm
$t_{w.ad}$	Adapted web thickness	mm
$t_{w.opt}$	Optimum web thickness	mm
V_r	Factored shear resistance of a member or a component	kN
W	Plate width	mm
$Z_{el,x}$	Elastic section modulus of steel about the principal x-axis	mm^3
$Z_{el,y}$	Elastic section modulus of steel about the principal y-axis	mm^3
$Z_{pl,x}$	Plastic section modulus of steel about the principal x-axis	mm^3
$Z_{pl,y}$	Plastic section modulus of steel about the principal y-axis	mm^3
β_f	Flange slenderness ratio equal to $(0.5 \cdot b_f) / t_f$	-
β_w	Web slenderness ratio equal to h_w / t_w	-
β_{wmax}	Maximum web slenderness ratio	-
ε_{tf}	Strain in top flange	-
ε_{ytf}	Yield strain of top flange	-

Δ	Displacement; deflection	mm
ϕ	Resistance factor for structural steel ($\phi = 0.9$)	-
Φ_{be}	Resistance factor for beam web bearing, end	-
Φ_{bi}	Resistance factor for beam web bearing, interior	-
ρ	Ratio of web area to flange area	-
τ_y	Shear yield stress	MPa
σ_r	Residual stress	MPa
σ_{yw}	Web yield stress	MPa
υ	Poisson's ratio ($\upsilon = 0.3$)	-

ABSTRACT

The aim of this research was to identify a possible range where the use of deep I-plate girders primarily under bending stresses with thick web plates and no web stiffeners has economical or operational benefits than adoption of plate girder cross-sections with slender web plates reinforced with intermediate transverse stiffeners. The research focuses mainly on optimisation for bending moment resistance. Shear capacities of the cross-sections were also computed and compared because shear forces share a close relationship with bending moments being a derivative of the moment.

The idea was that upon exhausting the capacity of the largest available rolled I-section, designers should not always tend to lean towards sections with slender web as is common practice encouraged by design standards to get the highest moment carrying capacity.

Design codes or standards adopted by different countries and sometimes regions provide structural designers with guidelines on acceptable design practice. The latest South African design standards encourage the use of thin web plates with stiffeners to guard against premature buckling of the web whilst realising high bending moment resistance. This is due to the increased lever arm and having more material in the flange plates as explained in SASCH (2016).

Though setting an upper limit on web slenderness β_w, the South African steel design standard, SANS10162-1 allows designers to ignore the limit if calculations prove that buckling of the compression flange into the web will not occur under factored load levels. SASCH (2016) alludes to the possibility of there being benefits that can be realised from using thicker web plates than the theoretically recommended ones but it does not give a comprehensive insight as to the nature and extent of those benefits. As shown in the following chapters, not much research has been done to assess the benefits of adopting compact webs in plate girders.

For a given cross-section area, different geometrical cross-section configurations were compared in an iterative process. The parameters under investigation were the moment resistance and shear carrying performance to mass ratios. Other design parameters of the elements were not to be compromised following recommended guidelines in the South African steel design standard, SANS10162-1. Additional knowledge gathered from research work detailed in the literature study was incorporated in the optimisation process but guidelines from SANS10162-1 took precedence.

The exercise showed that for cross-section area requirements below $35.6 \times 10^3 mm^2$ to carry moments, wholly class 3 sections provide a competitive alternative to sections with class 4 webs and flanges below class 3 classification. They resist a lesser moment, but the differences are of small magnitudes. The class 3 sections showed significant improvements in terms of shear capacity. Over larger spans, sections with class 4 webs with intermediate transverse stiffeners and of similar mass per unit length become heavier and more expensive because of the increase in the number of stiffeners. The labour requirements and consumables for fabrication also increase because of the stiffener requirements for sections with class 4 webs as shown in the cost estimates.

1. INTRODUCTION

1.1. Background

The evolution of the steel girder is closely linked to that of the transportation industry with the need to design wider, longer span bridges for highways and rail roads. Steel was adopted as an efficient material to meet these demands because of its appreciable high strength to weight ratio compared to other commonly used structural materials like reinforced concrete. This also allowed for bridges with higher clearance to passages below them should there be a need. Riveted plate girders were introduced to highways in the late 19th and early 20th centuries (Parsons Brinckerhoff, 2005). Since then, built up plate girders have been utilised for span and loading requirements in various structures like bridges, buildings, industrial sheds, industrial warehouses, and cranes.

During the early civilisation the cost of material was rarely a major factor as the cost of a structure was mainly determinant on labour costs, however in the modern day, life cycle costs, impact of extracting and processing raw materials from the environment and the increasing costs of steel as developing economies in need of more infrastructure are emerging, all that added to the continuous automation in production processes mean that the cost of material is now prioritised over fabrication and erection costs (Abspoel, 2015b). To that end the optimisation of load carrying elements like steel girders for weight per unit length and for reduced fabrication costs has become of paramount importance.

Plate girders are commonly I-sections that are built up from plates and optimised to primarily resist flexural stresses. The moment carrying capacity for a given cross-sectional area is enhanced by having as much material in the flanges, away from the neutral axis which is in the web. As such, designers and researchers have always looked at testing the limits on the maximum allowable lever arm between the flanges which is provided by the web. This results in deep girders with compact flanges but slender webs which are susceptible to buckling. Not much work has been put into determining if having slightly thicker webs might carry any economic or operational benefits.

Steel fabricators around the world can now rapidly and efficiently assemble plate girders using a mechanised process of welding the flanges and web together. In South Africa there are even some standard sections of plate girders that steel fabricators can quickly assemble on request with known parameters and properties that can be quickly utilised by designers, fabricators, and contractors alike that are listed in SASCH (2016). Riveted and bolted plate girders were more common than their welded counterparts before the improvement of welding techniques.

The plates making up the girders' cross-sections are usually thin steel plates, and the traditional approach encourages the use of slender webs with stiffeners to guard against plate buckling of the web to minimise material usage.

The study aims to try and establish if there is a range in which there are operational or economic benefits of utilising thicker web plates without stiffeners as opposed to thin web plates with stiffeners for plate girders primarily under bending and compare the shear parameters of the different cross-sections.

1.2. Purpose of Study

To determine a range where use of thick class 3 webs as opposed to thin class 4 webs with stiffeners as they are defined in SANS 10162-1 has economic or operational benefits in plate girder design primarily under bending.

1.3. Hypothesis

There exist wholly class 3 girder sections that might outperform girders with stiffened class 4 webs in flexure as per the design recommendations of SANS10162-1 and SASCH (2016).

1.4. Objectives

- Study flexure and shear strength characteristics of selected class 3 and class 4 web girders.
- Make recommendations on the economical use of class 3 webs without intermediate transverse stiffeners in fabricated girders in lieu of class 4 webs with stiffeners.

1.5. Limitations

Limitations of the study are listed below:

- The study is solely to give recommendations or conclusions on plate girder cross-section optimisation in comparison to the recommendations in the South African National Standards for steel design and other published, compatible supporting documents used by designers.
- The study focuses mainly on homogenous, prismatic, symmetrical I-section plate girder statically loaded as a simply supported member with maximum moment and minimum shear stresses at mid-span.
- Where present, tension field is assumed to be adequately anchored. That is, for a stiffened girder, the stiffeners, flanges, and connections between them have enough capacity to allow the web plate to carry more load after buckling in the diagonal tension field.
- Compression flange assumed to be fully restrained laterally
- The study only concentrates on Steel SANS 50025-2-S355JR commonly referred to as S355JR steel as it is the common strength grade for structural steel produced in South Africa. Properties can be found in SASCH (2016).

2. LITERATURE REVIEW

2.1. Introduction

Plate girders are flexural members specially fabricated to support heavy loads over relatively large spans when readily available rolled sections have insufficient load carrying capacity or stiffness. In its simplest form, a plate girder is made of two parallel flange plates fillet welded to a perpendicular web plate to form an I-section (Roberts & Narayanan, 2003).

Available hot-rolled I-sections are limited by the manufacturing plant available in the steel mills. For products like I-beams there are mainly two types of mills i.e., structural, and universal mills. In structural mills the steel is passed through multiple mill-stands with rollers oriented to shape the hot steel to the required section whilst in universal mills the hot steel passes backwards and forwards through the same mill with both horizontal and vertical rollers which are adjusted to give the desired shape (British Constructional Steelwork Association, 2012).

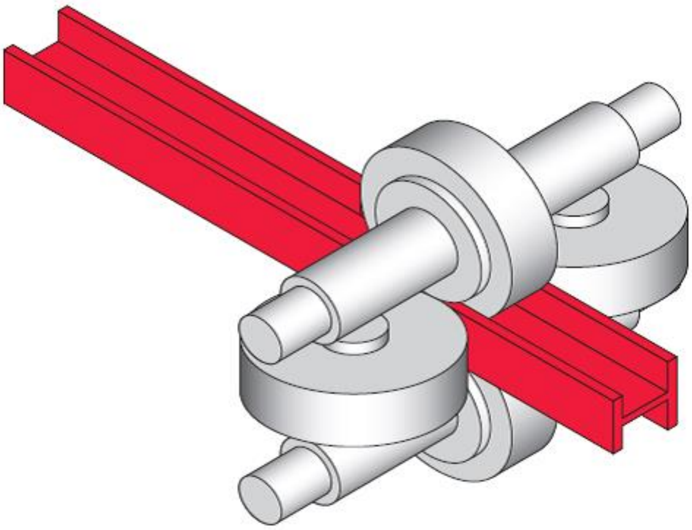


Figure 2-1: Rolling beams in a universal mill (British Constructional Steelwork Association, 2012).

The geometric properties of the flanges and webs in built-up welded plate girders are not as limited to the plant available in the mill as their hot rolled counterparts. Built-up sections can have fabrication limits like available plate sizes, handling, weldability of material which is a function of plate thickness and chemical composition, and handling. The latter is more of a concern at transportation and erection stages than inside the steel manufacturing or fabrication plant.

Other common built-up cross-sections are boxed sections which can be single or multi-celled, or composite with different materials making up the respective girder elements. The boxed girder consists of two flanges and two webs for a single-cell box and more than two webs in a multi-cell box girder as seen in figure 2-2.

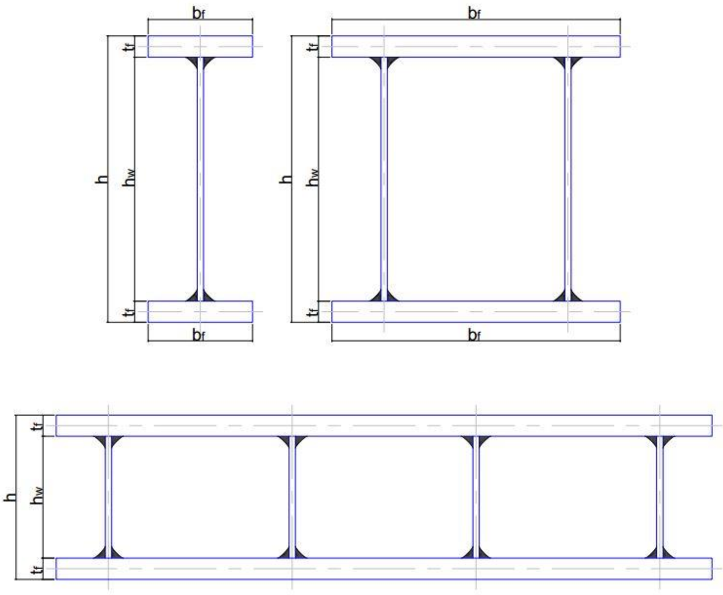


Figure 2-2: I-section, single-cell box and multi-cell box girder cross sections(Lui, 1999)

As plate girders are often employed on specific cases where readily available rolled sections are inadequate, a designer has the freedom to choose on cross-section profile, geometrical proportions of the elements to be assembled, connection types, and materials to utilise.

Different modifications can also be introduced to a cross-section to suit a certain application or to satisfy a design parameter either of the ultimate or serviceability limit state for example:

i. Transverse or longitudinal stiffeners can be added to the web plate to improve load carrying capacity and buckling resistance of the plate girder;
ii. Holes through the webs to accommodate services or to reduce overall weight of the girder primarily resisting moments;
iii. Hybrid girders with the common ones having flange plates made of a higher steel grade than the web plate resulting in a girder with improved bending moment resistance but overall lower mass of material and cross section size;
iv. Plate girders with corrugated web plates that are less susceptible to buckling and possessing a relatively higher transverse load carrying capacity compared to flat, thin plated webs.

The development of highly automated workshops in recent years has reduced the fabrication costs of I profile plate girders considerably but most box girders still have to be fabricated manually with consequently high fabrication costs (Roberts and Narayanan, 2003). In early civilisation up to 1960s it was the norm that any savings in material realised from optimisation of I-sections would not be significant because of fabrication costs (Bresler, et al., 1960). I-section plate girders have also proven to be easier to erect than box girders hence making them the most common plate girder section with designers, fabricators, and contractors. Riveted and bolted girders are falling behind their welded counterparts mainly because of the advancement in automated welding techniques in workshops around the world.

As a result of having many parameters that can be varied, understanding each element's contribution to the overall and local capacity of a structural member is critical for a designer to try and optimise a cross-section of the member for a given case.

For any given cross-sectional area to produce its greatest resistance to bending, the material making up the cross-section should be placed as far from the neutral axis as practicable. The I-profile gives the best shape to achieve this and the target is to put as much material in the flanges away from the neutral axis which is located in the web connecting the flanges (Brockenbrough, 2004).

The flanges resist the applied moment, and a deeper web corresponds to less axial flange force meaning higher moment carrying capacity. Proportioning of plate girders for higher moment carrying capacity is based on trying to find a balance for cross-sectional area with the deepest web possible and greatest flange area. In many instances, this results in a web plate of slender proportions prone to buckling at relatively low compressive action.

In South Africa, published South African National Standards (SANS) for structural designs are deemed acceptable design practices and provide designers with guidance on optimisation assumptions that can be followed during the design of steel plate girders. The code for design of steelwork adopts the limit state design approach and as expected advocates for use of slender webs to give higher moment carrying capacities. This is evident by the limit set in clause 14.3 for web slenderness β_w which allows for the web to be a class 4 slender element, and the allowance to go beyond this limit should calculations show that compression flange induced web buckling will not occur at factored load levels also found in the same clause.

2.2. Plate girder elements functions, and critical parameters

2.2.1. Primary Functions of Main Components

The primary response of the elements making up the I-section to external loading conditions are;

a) Flanges: to resist axial compressive and tensile forces arising from bending moments;
b) Web: to resist applied shear forces;
c) Web to flange welds: to resist longitudinal shear at interface;
d) Transverse stiffeners: to improve shear buckling resistance and shear bearing capacity;
e) Longitudinal stiffeners: to improve shear and bending resistance.

2.3. Plate girders under bending

According to Abspoel (2015b), material yielding, lateral torsional buckling, torsional buckling and combinations of these phenomena, or vertical buckling of the compressive flange into the web determine the moment carrying capacity of a plate girder section. This study focusses on a range of plate girders with cross sectional areas immediately above available hot rolled I-sections where it is expected that lateral and torsional restraint to the compression flange is usually provided by other elements or members commonly found in the structure like floor slabs, roofing systems and other bracing elements including walls in some cases.

2.3.1. Use of conventional plate buckling theory in design of plate girders

Since plate girders usually consist of very thin web plates, it was generally anticipated and accepted that buckling of the web limits the girder's moment carrying capacity. The conventional plate buckling theory predicts a critical load under which a thin plate subjected to edge stresses buckles out of its plane before reaching its ultimate capacity (Basler & Thurlimann, 1961).

The estimation for the buckling strength was based on the solutions derived for column buckling which gives a good measure of the column strength. However, unlike columns, plates continue to carry loads in a stable post buckling behaviour (Said El-Adly, et al., 2011). Due to post buckling strength of plates the buckling strength limit based on said approach proved not to be critical. Unlike in column buckling, the critical stress for plate buckling does not give an indication of its resistance but determines the stress level where a perfectly flat plate will initially buckle (Abspoel, 2015b). Based on post buckling behaviour of plates, as described by Winter (1952), it was generally accepted that a smaller factor of safety be adopted against web buckling compared to column buckling.

2.3.2. Introduction of flange induced web buckling in design of plate girders

The design of steel plate girders was revolutionised by Basler and Thurlimann (1961) based on their research on plate girder behaviour carried out in the late 1950s and early 1960s. They proposed that compression flange induced web buckling was a critical design aspect, and this was adopted and incorporated into many design standards/ codes around the world. It is still treated as such in modern day steel design guidelines in different countries and regions including SANS10162-1 for South Africa and EN1993-1-5 as adopted by some European nations. Chern (1969), Cooper (1971), Abspoel (2015b) also carried out research to determine the maximum allowable web slenderness as will be shown in the following discussion. This resulted in design guidelines promoting long lever arms between the flanges increasing moment capacity as the limits that can be reached before the web plate fails by compression flange induced buckling are tested.

The bending moment resistance formulas for plate girders with thin webs are based on the effective width theory introduced by Winter (1952). The theory was developed as an alternative to the linear plate buckling theory and accepts that the web buckles but also exhibits post buckling strength. Different ways of determining this effective area from the flanges and part of the web that participates in bending resistance have been developed by researchers as will be shown in the following discussion. As a result, there are different ways to calculate flexural capacity for plate girders with slender webs, but all follow the principle to try and test the limits of maximum allowable web slenderness whilst avoiding compression flange induced buckling.

Investigations carried out by Basler, et al. (1960) on welded plate girders showed that the concept of expressing the post buckling strength of plate girders as a percentage of the web buckling strength or applying a smaller factor of safety is refutable. They proposed that the concept should be replaced by a more accurate strength prediction which considers the influence of flanges and transverse stiffeners if any to the capacity of the plate girder.

Basler and Thurlimann (1961) carried out experiments to find out the likely cause of failure under bending for plate girder specimens denoted G1, G2, G3, G4 and G5. The girder geometries are shown in figure 2-3. They concluded that except for the remote possibility of a brittle fracture, vertical torsional or lateral buckling, vertical buckling of the compression flange into the web will always be the cause of failure for a statically loaded and symmetrically proportioned plate girder.

Nine tests were carried out on the five girders and the failure modes and respective loads are shown in figure 2-5.

The symbols used by Basler and Thurlimann (1961) in their experiments are as follows:

b is the depth of girder
c if half of the flange width
d is the thickeness of flange
t is the thickness of web
k is the buckling coeefficient
l is the buckling length of the column
r is the radius of gyration
A_w is the area of the web
A_f is the area of the flange
P is the load on test girders
P_{cr} is the critical load
P_p is the plastic load
P_u is the ultimate load
P_y is the yield load
P_u^{th} is the theoratical ultimate load
P_u^{ex} is the experimental ultimate load

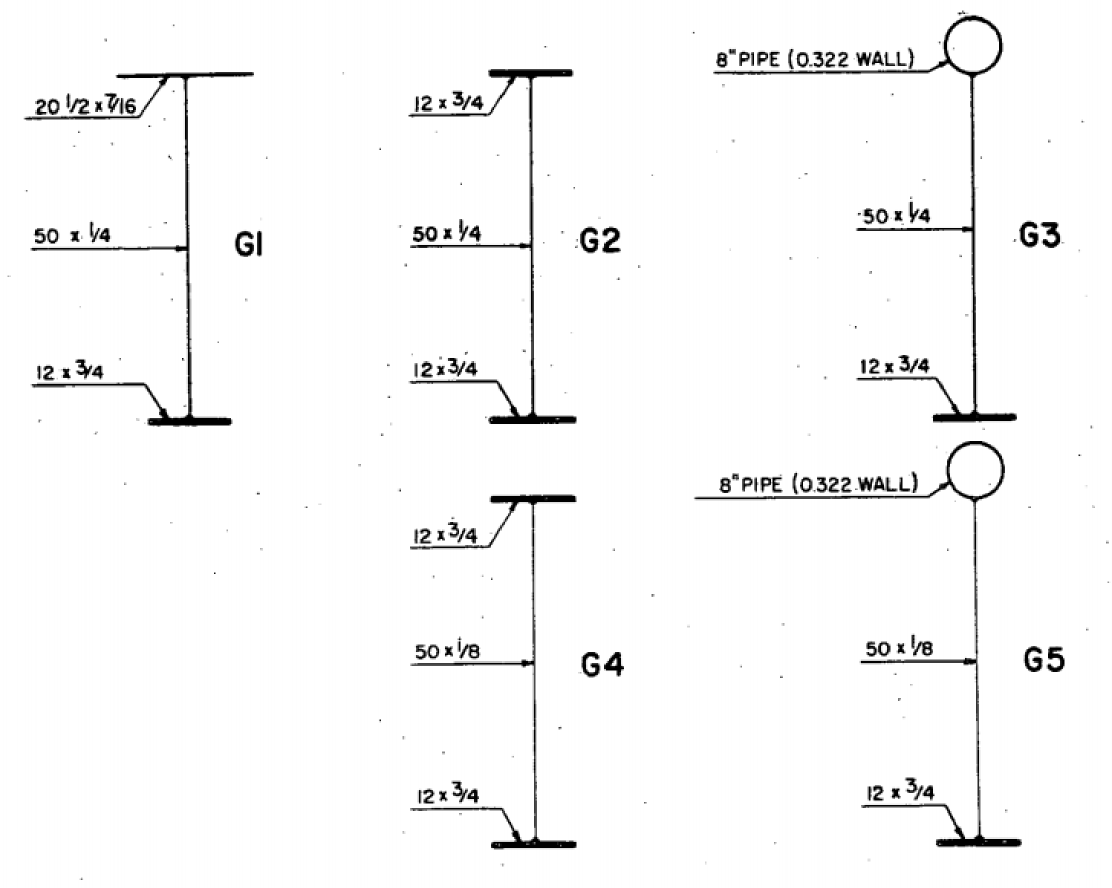


Figure 2-3: Cross sections of girders G1 to G5 (Basler & Thurlimann, 1961). Dimensions are in inches

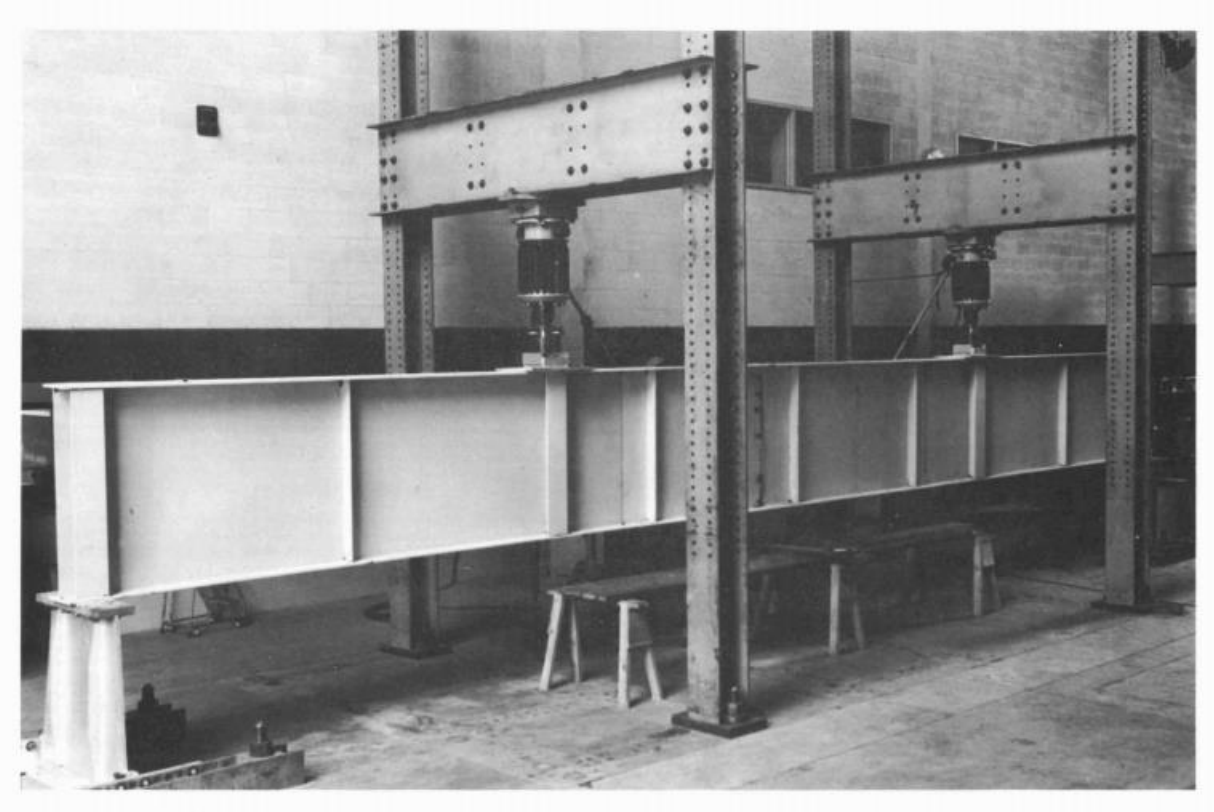


Figure 2-4: Overall view of test set up (Basler & Thurlimann, 1961)

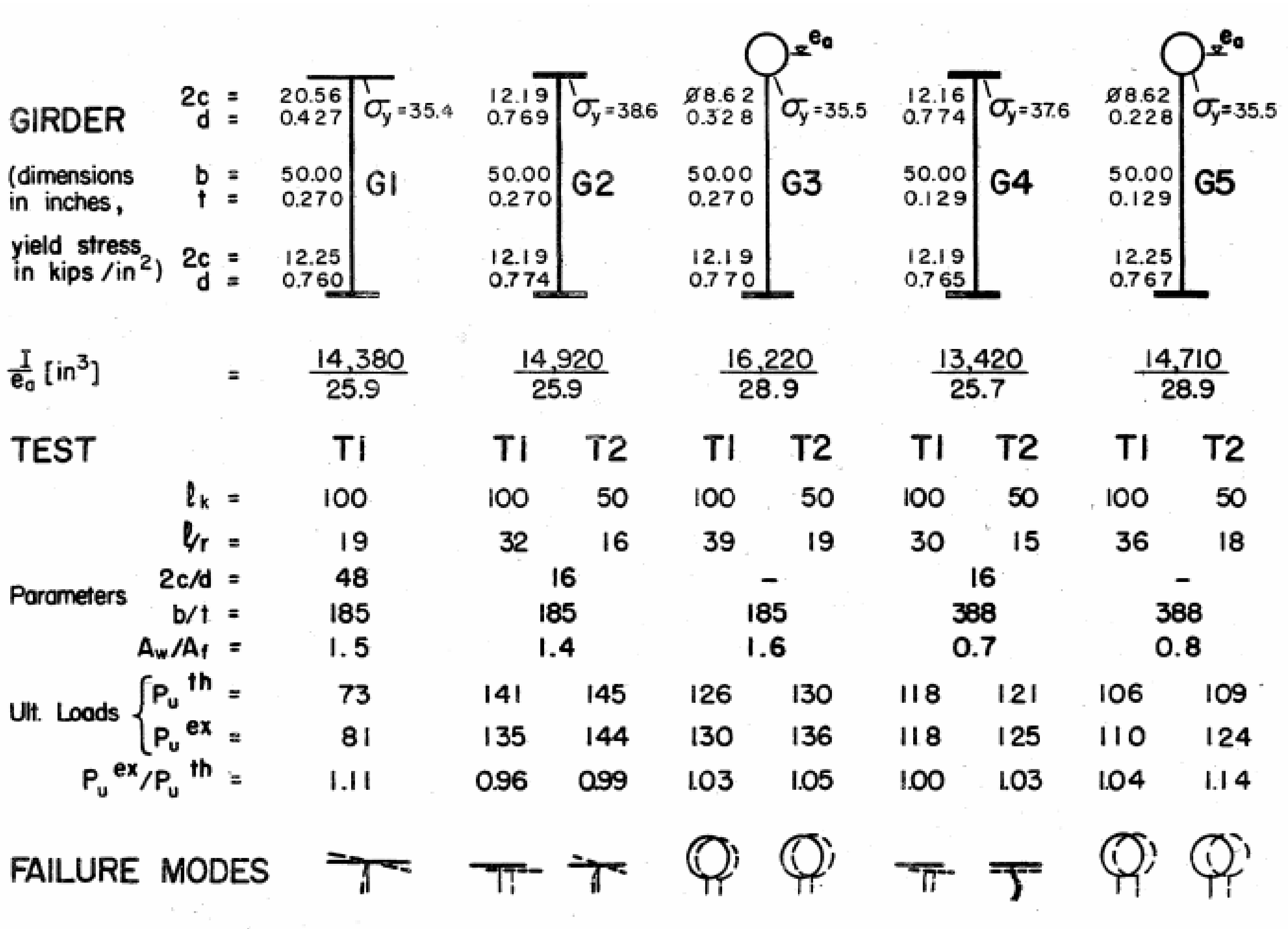


Figure 2-5: Different geometries of Basler and Thurlimann's test specimens, parameters, ultimate loads at failure, and their failure modes (Basler & Thurlimann, 1961)

Basler and Thurlimann (1961) then carried out a comparison between the static ultimate loads of the girders under bending. As shown in figure 2-6 , test girders with tubular flanges, G3 and G5 perfomed better when the ultimate load was presented as a fraction of yield load. This is because their extreme flange fibres are considerably further away from the neutral axis owing to the tubular shape than the parallel flange girders of the same web depth.

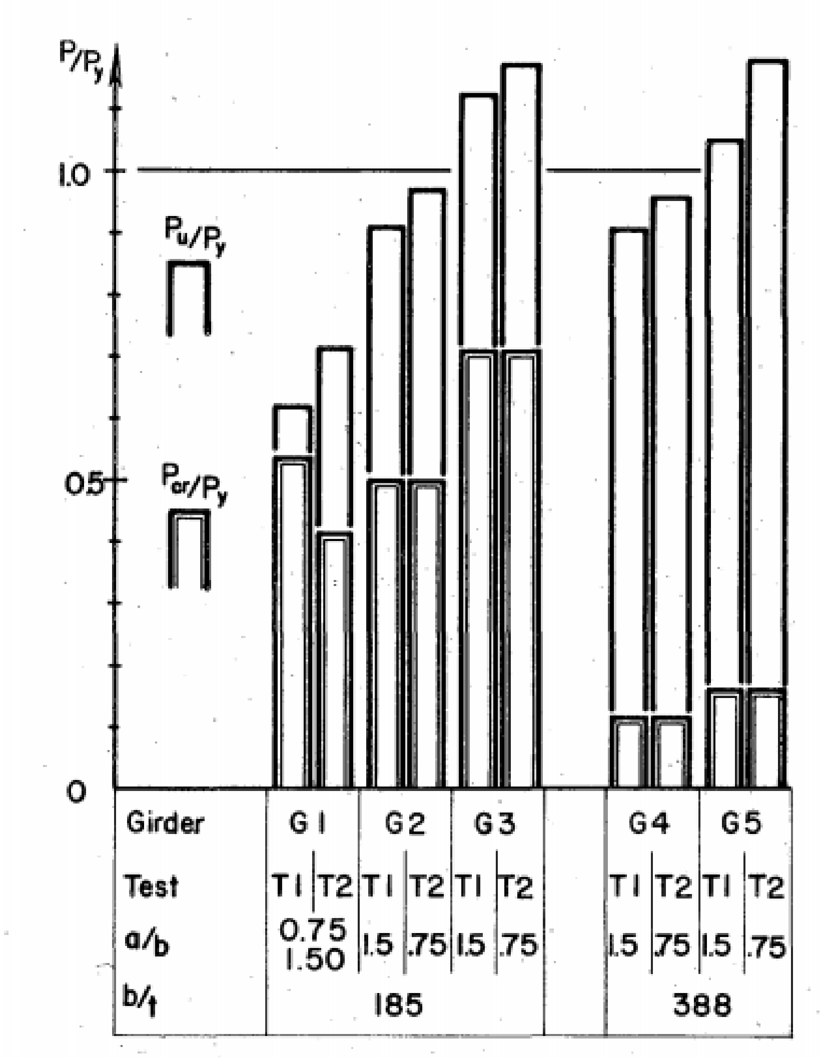


Figure 2-6: Loads on test girders as a fraction of yield load (Basler & Thurlimann, 1961)

Another comparison was made with the static ultimate loads on test girders expressed as a fraction of the plastic load as shown in figure 2-7. It can be seen that the difference as a result of the two shapes almost completely disappears for girders of similar web slenderness.

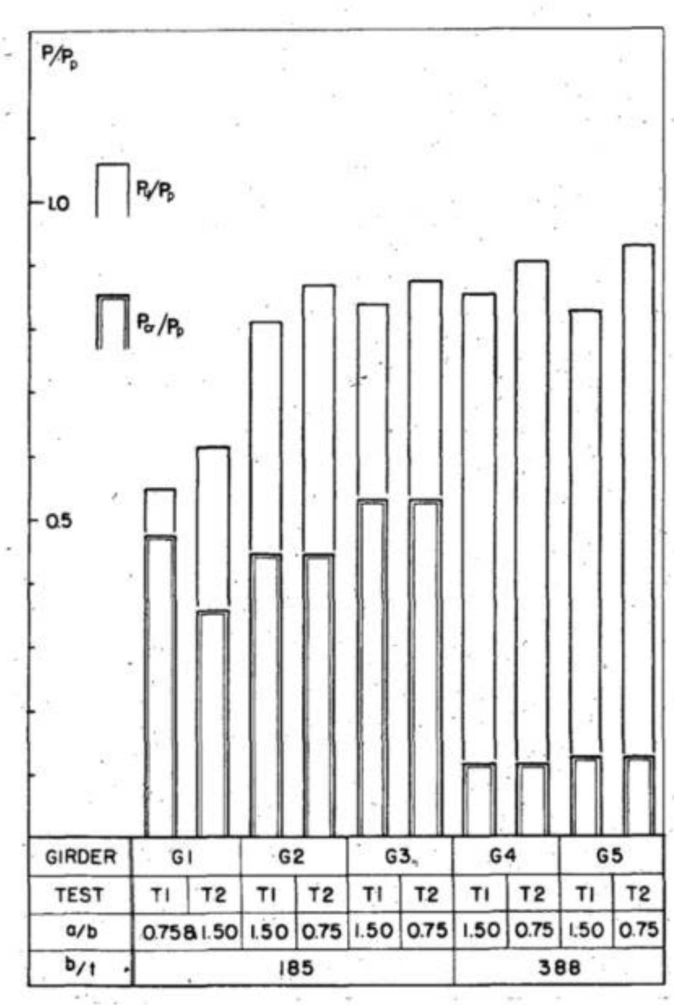


Figure 2-7: Loads on test girders as a fraction of plastic load (Basler & Thurlimann, 1961)

Basler and Thurlimann (1961) also made a comparison of the ultimate strength to the amount of steel needed to build the test section. As shown in figure 2-8 , the denominator for the ordinate is a product of cross-sectional area and the compression flange yield stress. The cross-sectional area is directly proportional to the weight and the significant yield strength ensures that a fair comparison can be drawn between girders with different yield levels.

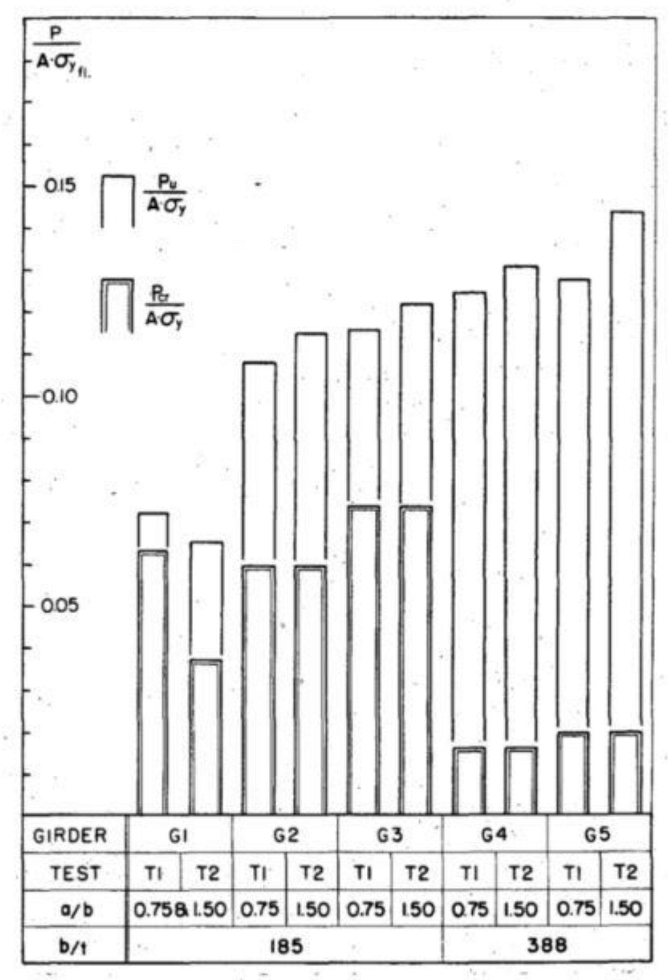


Figure 2-8: Load to test girders as a fraction of cross-sectional area and yield stress (Basler & Thurlimann, 1961)

Figure 2-8 shows that girders become more economical in bending as the web slenderness ratio increases. However, there obviously should be a limit otherwise the web will eventually become so weak that it will not be able to support the compression flange properly. Basler and Thurlimann (1961) got confirmation of this action in the second test of girder G4.

In all the conducted tests, the girders failed due to a failure of the compression flange. The compression flanges failed by either lateral buckling or lateral torsional buckling except for the the second test on girder G4 which failed by vertical buckling of the compression flange into the web. The failure modes are highlighted in table 2-1.

Table 2-1: Description of failure modes for tests conducted by Basler and Thurlimann (1961)

Girder	Test Number	Compression flange failure mode
G1	T1	Lateral torsional buckling
G2	T1	Lateral buckling
	T2	Lateral torsional buckling
G3	T1	Lateral buckling
	T2	Lateral buckling
G4	T1	Lateral buckling
	T2	Vertical buckling into web
G5	T1	Lateral buckling
	T2	Lateral buckling

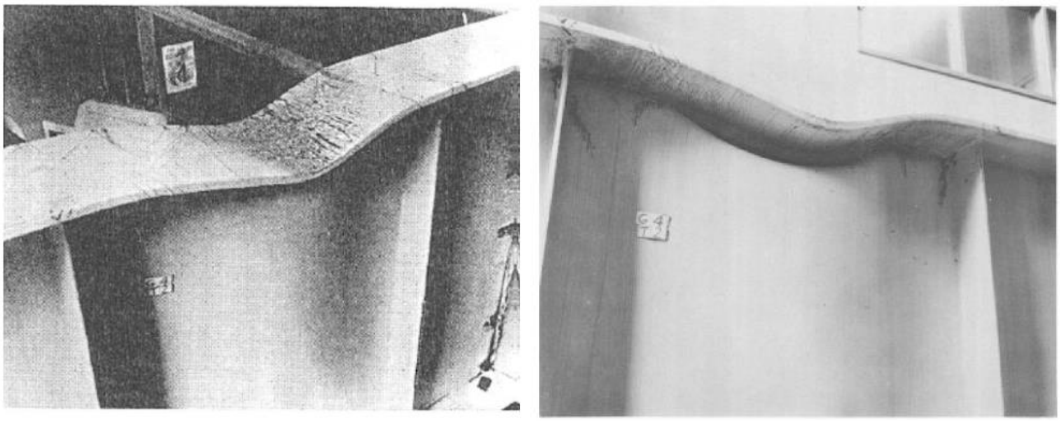


Figure 2-9: Test specimen G4-T2 showing vertical buckling of compression flange into the web (Basler & Thurlimann, 1961)

The properties and behaviour of test specimen G4-T2 were the ones used by Basler and Thurlimann (1961) to deduce equations for maximum allowable web slenderness. The tests proved that the initial linear buckling theory on which, up to 1960, most specifications and design practices were based, was very conservative and the effective width method as introduced by Winter in 1952 modelled the reality much better.

Vertical buckling of the compression flange into the web has since been adopted as the critical design criteria for plate girders under bending with researchers trying to find the critical web slenderness ratio under given circumstances to avoid such a failure. Finding the thinnest web plate for a cross-section that will not buckle became the priority to determine the best design practice to give an economic, practical, and safe plate girder. Various bending moment equations have been developed with researchers varying mostly on their choice of calculation for effective widths and coming up with more slender webs as will be shown by the theories discussed here.

Chern (1969), suggested that the flanges provide full fixity to the web of a welded plate girder and the maximum web slenderness suggested by Basler and Thurlimann (1961) can be increased however slightly by a magnitude of just 5.3%.

Cooper (1971) carried out experiments on two girder specimens and reproduced the test done by Basler and Thurlimann (1960) for their test specimen G4 as it failed by vertical buckling of the compression flange as a control. The two girders tested by Cooper (1971) had larger web slenderness ratios of $\beta_w = 444$, and $\beta_w = 751$ than the limit set by Basler and Thurlimann of $\beta_w = 368$ and both test specimens failed because of general yielding of the compression flange. Cooper then continued the tests on the plate girder with $\beta_w = 751$ and the control test specimen G4, until vertical buckling of the compression flange into the web occurred. The tests showed that the ultimate moments at failure were very close to those predicted by the equations derived by Basler and Thurlimann (1961) including for the girder with web slenderness that was too high when considering the limits proposed by Basler and Thurlimann (1961).

According to Schilling (1974), the bending strength and stiffness of both homogeneous and hybrid I-shaped sections vary with the web slenderness and the ratio of the web area to the total area. For both homogeneous and hybrid girders, the maximum bending strength and stiffness for a given cost are achieved by using the maximum permissible web slenderness and an optimum web to total area ratio $A_w / A_{tot} = 1/2$, for elastic strength and equal to 2/3 to mobilise plastic strength. For hybrid beams, the optimum area ratios are higher, and increase as the web to flange yield-strength ratio decreases. For both homogeneous and hybrid beams, the optimum web to total area ratio for stiffness I, is 3/4 (Schilling, 1974).

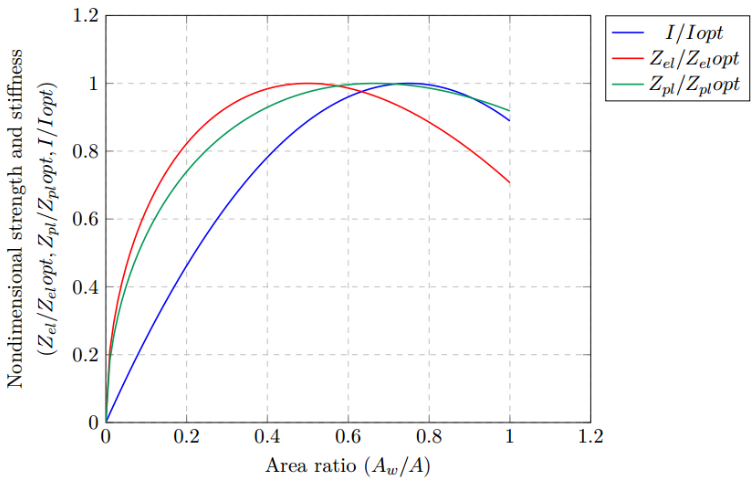


Figure 2-10: Strength and stiffness as a function of area ratio for I-beams d (Schilling, 1974).

For an area ratio equal to 0.63 the elastic section modulus and moment of inertia are both approximately 97.5% of their optimum values. The plastic section modulus is almost a maximum also at this value as can be seen in figure 2-10 and therefore a web area to total area ratio in the region of 0.63 gives an I-section that is stiff and has an appreciable flexural resistance (Schilling, 1974).

The limits on width to thickness for elements making up structural steel shapes under flexural compression in current structural steel design codes like ANSI/AISC 360-05, CAN/CSA S16-01, and EN1993-1 mostly ignore interaction between the elements (Ragheb & Wael, 2015). These limits were meant to ensure the elements avoid local buckling failure. This means that the limits for one element are not related to the dimensions of the other element making up the structural steel shape. Ragheb and Wael (2015) deduced that because of the flange to web interaction, the maximum width to thickness ratios in the mentioned codes was very conservative or over-estimated except for one case found in ANSI/AISC 360-05. In extension this includes the limits found in SANS10162-1 as it is based largely on CAN/CSA S16-01 and particularly adopts the limits in question from this Canadian code. This means the limits advising on the moment equations to adopt for a given element classification or the need for intermediate transverse stiffeners to avoid local buckling result in conservative designs. It can also be extrapolated that the proposed maximum web slenderness limits to avoid compression flange induced buckling are also conservative or over-estimated as they look at the problem as an individual plate buckling problem without considering the interaction between section elements and the dimensions of connected parts.

Abspoel (2016) set up experiments at Delft University of Technology with the aim of establishing higher maximum web slenderness ratios. The experiments were based on the guidelines set up from the work done by Basler and Thurlimann in the 1960s. However, the test specimens by Abspoel (2016) had web slenderness ratios much higher than the maximum value proposed by Basler and Thurlimann in 1961. From the experiments, Abspoel (2016) concluded that flange induced buckling is not a determining factor for maximum load capacity under bending. Abspoel (2016) concluded that the recommendations by Basler and Thurlimann on maximum web slenderness which were consequently adopted in most steel design codes of practices including the South African codes are too conservative to an extent that they can even be doubled. According to Abspoel (2016), the bending moment resistance of test girders is rather close to the bending moment resistance calculated based on taking the cross-section as consisting of only the flanges, but also close to the effective bending moment resistance considering partial yielding of the web as considered in the effective width theory.

Abspoel (2015b) determined that the web depth is the most influential geometric parameter on moment capacity of plate girders. This was after investigations on four independent geometric parameters being, the web height h_w, web thickness t_w, flange width b_f, and the flange thickness t_f. Starting with a reference cross-section of web area 1 000mm x 2mm and flange area 200mm x 10mm and using FEM-models, each parameter was increased by 10% one at a time with the others kept constant. All other properties were also kept constant. The increase in cross-sectional area and bending resistance for each step was recorded and is shown in table 2-2 .

Table 2-2: Determining the most influential geometric parameter (Abspoel, 2015b).

	Reference	**h_w +10%**	**t_w + 10%**	**b_f + 10%**	**t_f + 10%**
Relative moment resistance	100%	109.8%	101.5%	109.4%	109.3%
Relative cross-section area	100%	103.3%	103.3%	106.7%	106.7%

The results showed that an increase in the web height results in the highest increase in moment capacity with the lowest increase in cross-sectional area. Therefore, in terms of optimisation for moment carrying capacity the web height is the most influential geometric parameter.

Using plates of higher steel grade and hybrid girders with flanges made from plates of higher steel grade than the web plate also improves moment capacity of a girder. Nascimento et al (2021) investigated flange induced web buckling of plate girders numerically using finite element method by means of a general-purpose software ABAQUS. The study was conducted on girders with high strength steels S690 and S700 and hybrid girders. From their research, they concluded that there are two types of flange induced buckling being;

a. Global web-column like buckling and
b. Local buckling of flange into web.

Global buckling was expected to occur more in steel-concrete composite plate girders in the sagging moment region. This is due to the higher relative stiffness and strength provided by the composite action of the top flange. Local buckling occurs more in symmetrical homogeneous sections (Nascimento, et al., 2021).

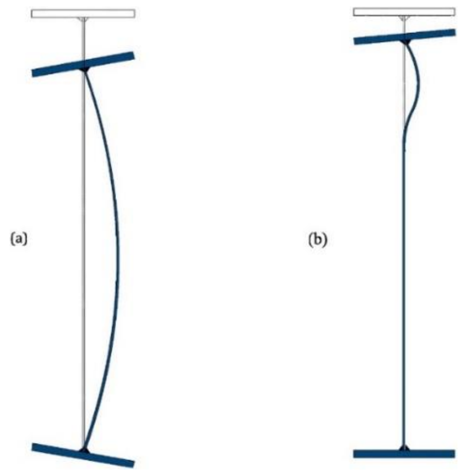


Figure 2-11: Global and local flange induced buckling modes, respectively (Nascimento, et al., 2021).

Results from the research carried out by Nascimento et al (2021) also indicated that hybrid girders are more prone to flange induced buckling than homogeneous girders for high strength steel girders. Flange induced buckling was generally reported after first yielding of the flange at load levels about 6-10% over the design loads stipulated in EN1993-1-5. Nascimento et al. (2021), concluded that the slenderness limits given in EN1993-1-5 for class 4 sections are very conservative for high strength steel girders.

Shear capacity of plate girders can be enhanced by various methods like use of corrugated webs. This research will however focus mainly on the bending moment aspect for plate girders of steel grade S355JR with properties shown in table 2-3.

Table 2-3: Mechanical properties of relevant steels as extracted from the SASCH (Southern African Institute of Steel Construction, 2016)

Grade	Ultimate tensile strength f_u	Yield stress f_y (minimum) for thickness t (mm)					Charpy V notch impact test	
		3<t ≤16	16<t≤40	40<t≤63	63<t≤80	80<t≤100	Test temp	Minimum average energy absorbed
	MPa	MPa	MPa	MPa	MPa	MPa	˚C	J
S355JR	470 to 360	355	345	335	325	315	20	27 (only if specified)
S355JO	470 to 360	355	345	335	325	315	0	27
S355J2	470 to 360	355	345	335	325	315	-20	27

Mela and Heinisuo (2014) researched on the economical use of high strength steel in steel beams. In their study they optimised the beams for cost and weight separately under the design rules set in EN1993-1-5 for beams under both bending and shear but ignoring lateral torsional buckling. Their study also included hybrid sections with flanges and webs of different steel grades and considered the contributions of factors like cost of fabrication, transportation, and erecting the beam on site to the overall cost of the structure. The study indicated that the cost savings of using higher steel grades, particularly S500 and S700 are limited compared to using S355 grade steel mainly because of restrictions by available plate sizes for the higher steel grades. Hybrid cross-sections had significant cost saving rewards and they proceeded to give a ranking on hybrid cross-section performance as shown in table 2-4. The web plate was always of steel grade equal to or lower than that of the flanges. The girder with best performance in terms of cost is ranked No. 1 going down to the worst performing girder ranked No. 14.

Table 2-4: Hybrid girders steel grade combinations ranking (Mela & Heinisuo, 2014).

Ranking	Top flange	Web	Bottom flange
1	S355	S355	S355
2	S500	S355	S500
3	S500	S500	S500
4	S700	S355	S700
5	S700	S500	S700
6	S700	S700	S700
7	S355	S355	S500
8	S355	S355	S700
9	S500	S355	S355
10	S700	S355	S355
11	S500	S355	S700
12	S500	S500	S700
13	S700	S355	S500
14	S700	S500	S500

Their study also showed that for the homogenous sections, ranked Nos. 1, 3 and 6 in table 2-4, optimisation for cost and optimisation for weight produced a section of almost equal weight. Optimisation for cost and weight methods also coincided for several of the other combinations of the steel grades tested as shown in figure 2-12. To that end, optimisation for weight per meter length of welded I-sections can be taken to directly represent cost optimisation also and it is the easier of the two methods with less uncertainties on the variables contributing to its function at a given time and location.

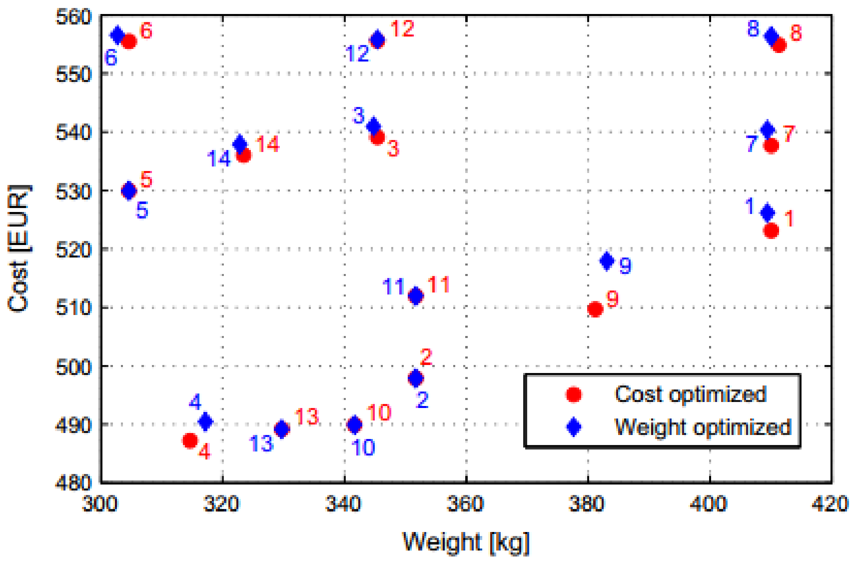


Figure 2-12: Minimum cost and minimum weight solutions with numbering corresponding to table 2-4 (Mela & Heinisuo, 2014).

Table 2-5: Cost contribution of the minimum cost design (Mela & Heinisuo, 2014).

Cost component	Contribution to total cost
Material	50%
Erecting	16%
Beam welding	11%
Painting	11%
Cutting	6%
Blasting	2%
Sawing	2%
Transport	< 1%

The models by Mela and Heinisuo (2014) also show that after the cost of the material, welding and painting are big contributors to the overall cost of a welded I-beam section. With the knowledge that use of stiffeners increases paint area, fabrication requirements and material, their use is expected to have a significant impact on cost of a welded I-section.

The South African steel design standards are based mostly on the work done by Basler and Thurlimann (1961) for the design of steel plate girders. They recommend the use of transverse stiffeners when a given web slenderness ratio is exceeded. Some of the research discussed above proved that there is room to increase this limit. Not much research can be found if for smaller values of required moment resistance however still exceeding the capacity of readily available hot-rolled sections there is a benefit in not jumping straight to the thinnest web plate possible but adopting a thicker web plate which does not require intermediate transverse stiffeners as prescribed by SANS 10162-1.

The benefits can be operational in terms of less fabrication needs as the welding of stiffeners is not as automated to appreciable standards compared to welding the flanges and web together. It also results in less areas that can be considered points of weakness that can arise from welding meaning increased rigidity. This can also translate to an economical benefit if the difference in steel weight between the two girders is not so significant meaning the one with less fabrication needs is more advantageous.

2.3.3. Connections

Connections between elements of a structure should be able to transmit the reactions mobilised in the loaded element to the receiving element which it connects to until the load is dissipated to the supports as per the design. Connections between members should also be cost effective as they contribute significantly to the total cost of a steel structure. According to Mela and Heinisuo (2014) as shown in table 2-5, welding of an I-section is approximately 11% of the total cost of a structure and erection which involves connecting different members on site is weighted at approximately 16% of the total cost of a steel structure.

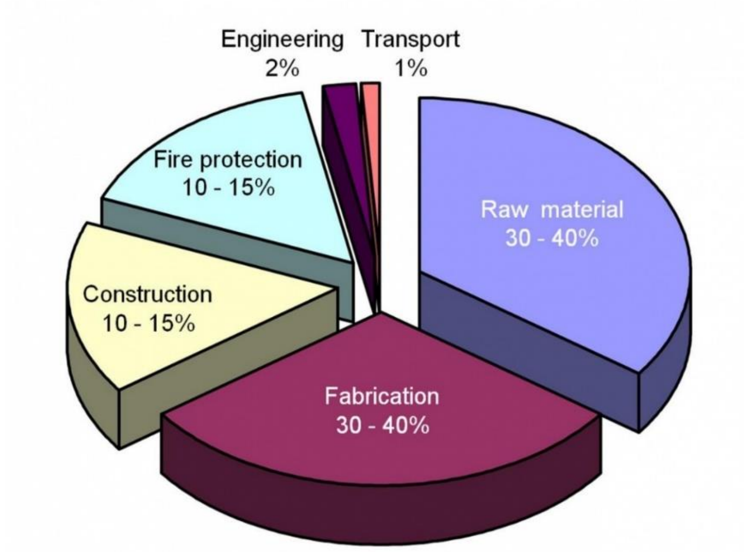


Figure 2-13: Breakdown cost of a steel frame for a typical multi-storey commercial building (British Constructional Steelwork Association, 2017)

The British Constructional Steelwork Association in 2017 estimated fabrication costs to be between 30% and 40% of the overall cost for a steel frame with the relative proportions of the other elements making up the total cost of the frame shown in figure 2-13.

This differs from the estimates by Mela and Heinisuo (2014) owing mainly to the different categories they use to break down the total cost of the structure. Combining sawing, cutting, and welding under fabrication for Mela and Heinisuo (2014) their estimate comes up to 19% for fabrication. Painting and blasting are taken to fall under fire protection and erecting equates to construction. This makes the data sets more comparable. Whilst most of the similar categories are within the range of each other, the differences in cost of material i.e., 30-40% by the British Constructional Steelwork Association (2017) and 50% by Mela and Heinisuo (2014) can be taken to explain the significant gap on their fabrication estimates as shown in table 2-6.

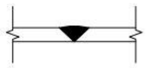
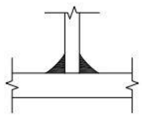
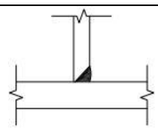
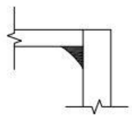
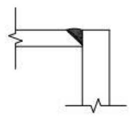
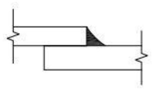
Table 2-6: Comparison between cost estimates by The British Constructional Steelwork Association (2017) and those by Mela and Heinisuo (2014)

British Constructional Steelwork Association (2017)		Mela and Heinisuo (2014)	
Raw material	30-40%	Material	50%
Fabrication	30-40%	Sawing, cutting, and welding	19%
Construction	10-15%	Erecting	16%
Fire protection	10-15%	Blasting and painting	17%
Engineering	2%	N/A	
Transport	1%	Transport	<1%

For the I-plate girders, connections between the web and flanges are commonly achieved by welding. The process of fusion welding or electric arc welding involves an electric arch jumping across a small gap between an electrode and two steel elements to be joined, causing a pool of molten metal sometimes referred to as the ‘weld pool’, to form. The parts are fused as the weld pool cools and solidifies after the electrode is removed. To protect the weld from the atmosphere, a shielding gas and or flux are utilised giving a weld of acceptable quality (De Clercq, et al., 2012).

Welds can be fillet welds or groove welds and they can also be described based on the configuration or location of the connection point between the two elements being joined as illustrated in table 2-7.

Table 2-7: Weld joint types (Khichade, 2015).

	Fillet	**Groove**
Butt	N.A.	
Tee		
Corner		
Lap		N.A.

The joint between the web plate and flange plate is the tee joint. The fillet weld is the more popular than the groove joint in South Africa.

Azad (1978) (cited in Tredoux, 2017) considered plates of thicknesses less than 6mm to be impractical for fabrication of built-up members. As welding technology has improved over time, fabricators have become comfortable welding plates as thin as 4mm. However, from interviews done by Tredoux (2017) with steel fabricators in South Africa, 5mm is the typical minimum thickness they are comfortable using for welded I-sections.

2.4. Conclusion

From the literature review no reports or research addressing the current study could be found thus supporting the hypothesis that there might exist wholly class 3 girder sections that might outperform girders with stiffened class 4 webs as per the design recommendations of SANS10162-1 and SASCH (2016).

3. THEORETICAL BACKGROUND TO DESIGN OF PLATE GIRDERS UNDER BENDING

3.1. Introduction

This chapter intends to discuss the various theories adopted to come up with most design equations for plate girders under bending.

3.1.1. Local plate buckling of thin plates

Thin plates experiencing in-plane compressive stresses tend to fail by buckling before they realise their ultimate load carrying capacity. The buckling mechanism mainly depends on the loading and the boundary conditions of the plate.

Before Basler and Thurlimann (1960) introduced the phenomenon of flange induced buckling of the web, local web buckling was assumed to set a clear limit on the allowable web slenderness for a plate girder. This then governed the minimum amount of material required in the web for optimum utilisation of available cross section material. A lot of effort was put in setting up these web buckling values which are summarised in various texts including a book by Timoshenko and Gere (1963).

Timoshenko and Gere (1963) give the general equation for the elastic critical stress where buckling occurs as:

$$\sigma_{cr} = k_\sigma \cdot \frac{\pi^2 E}{12(1-\upsilon^2)\left(\frac{b}{t}\right)^2} \tag{3.1}$$

Where:

k_σ is the buckling factor.

Conservatively looking at the web to be simply supported between the flanges, the buckling factor can be found from figure 3-1.

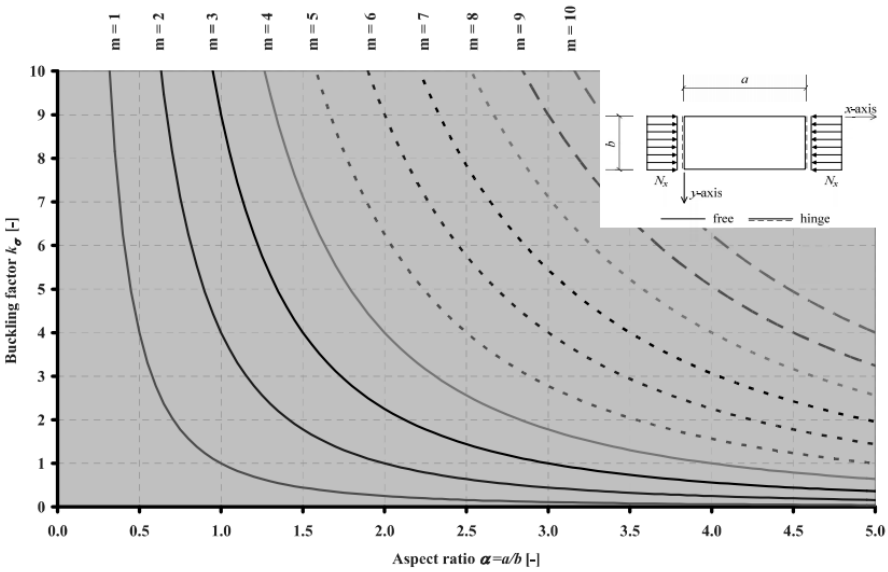


Figure 3-1: Plate buckling factor for a plate simply supported at two edges with compression applied through the supported edges (Abspoel, 2015b).

Where:

m is the number of half sine waves in the x axis direction.

α is the aspect ratio dependent on the plate geometric parameters a, and b as shown in figure 3-1 which is also represented by equation (3.2).

$$k_\sigma = \left(\frac{m \cdot b}{a}\right)^2 \quad (3.2)$$

For each aspect ratio α, the critical stress σ_{cr}, is determined by using the minimum value of the buckling factor k_σ. Figure 3-1 shows that, for a plate simply supported on two edges with compression applied through the same edges, the minimum buckling factor is when the number of buckles is equal to a unit.

The local buckling factor for the flange plate can be determined from figure 3-2.

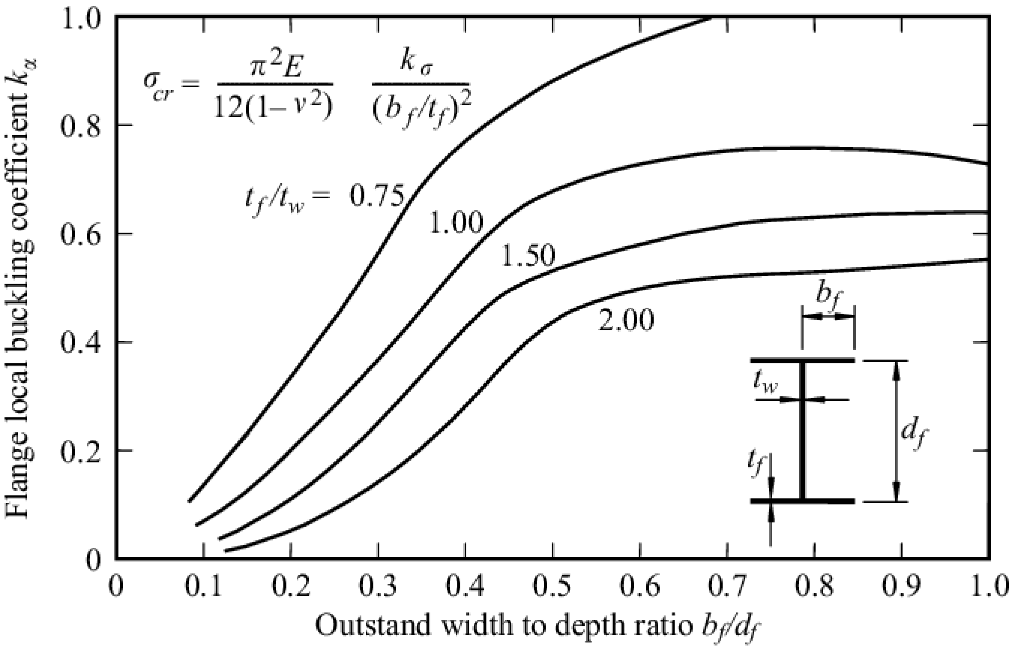


Figure 3-2: Local buckling factor for compression flange in an I-section under bending (Abspoel, 2015b).

For a plate element simply supported on all for edges k_σ is given as in figure 3-3.

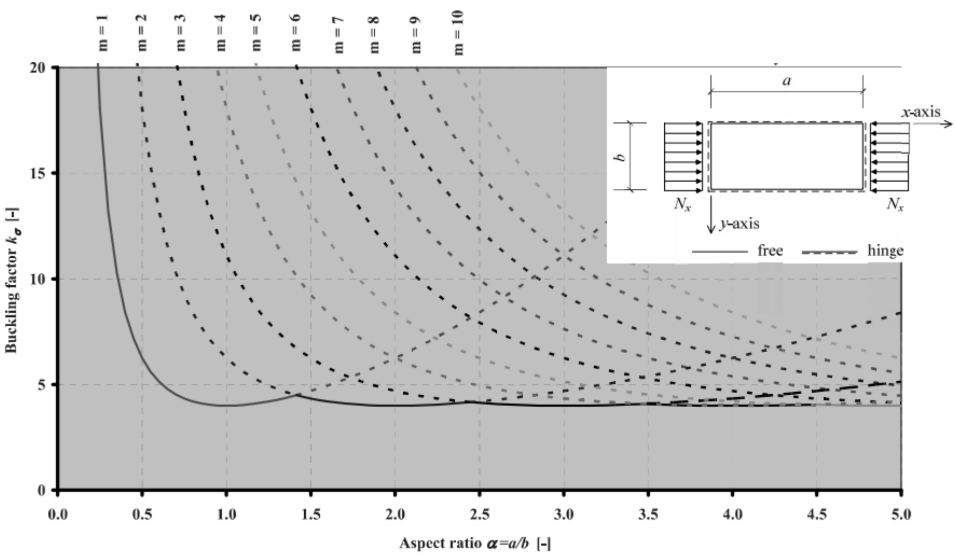


Figure 3-3: Plate buckling factor for a plate in compression, simply supported on all sides (Abspoel, 2015b).

From the graph k_σ is a minimum when m = α, at a value of $k_\sigma = k_\alpha = 4$.

The same method is used to determine critical stress for plates experiencing flexural actions and the buckling factor can be determined from figure 3-4.

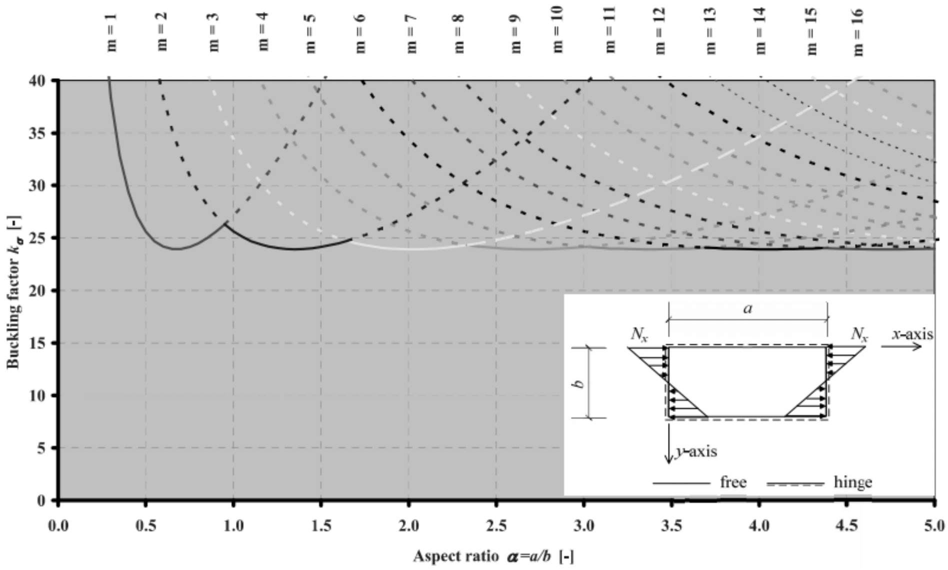


Figure 3-4: Plate buckling factor for a plate simply supported on all edges under bending (Abspoel, 2015b).

The buckling factor k_σ, for a simply supported plate under pure bending can also be calculated using equation (3.3). The equation considers the number of buckles m, the geometric properties being the length a, and the width b.

$$k_\sigma = \frac{\pi^2}{\left(\frac{a}{b}\right)^2} \times \frac{\left(m^2 + \frac{a^2}{b^2}\right)\left(m^2 + 4\frac{a^2}{b^2}\right)\left(m^2 + 9\frac{a^2}{b^2}\right)}{\sqrt{\left(m^2 + \frac{a^2}{b^2}\right)^2 \left(16m^2 \times \frac{6}{25}\right)^4 \left(m^2 + 9\frac{a^2}{b^2}\right)}} \qquad (3.3)$$

3.1.2. Flange induced web buckling in plate girders.

As mentioned earlier, the flanges primarily resist bending actions, and the web connects them whilst also resisting shear forces providing a lever arm between the parallel flanges. During post buckling behaviour of the web plate, the bending curvature of the plate girder produces compressive forces in the web plate (Khosiin, n.d.). The compression flange tends to collapse into the web which causes the web plate to buckle in a phenomenon called flange induced buckling.

Figure 3-5 shows an exaggeration of the deformed shape of the girder under bending and how the flanges in bending induce compressive stresses, F_3, in the web connecting them.

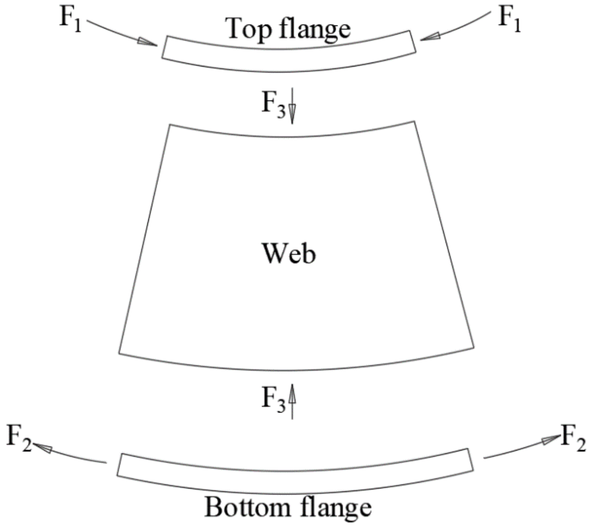


Figure 3-5: Compression of the web due to bending of flanges.

In a truss configuration, the internal members (verticals and diagonals) provide vertical rigidity by bracing the top chord and bottom chord in the vertical direction. Similarly, in an I-plate girder, the web can be taken to represent the truss internal members with the compression flange and tension flange taking up the roles of the top and bottom chords, respectively. The compression flange lacks rigidity in the vertical direction and relies on the web which will be inadequate if it is too slender. By setting an upper limit of web slenderness it is possible to avoid flange induced buckling occurring before the section's moment carrying capacity is attained.

The model to determine the maximum web slenderness was based on column buckling of the web by compressive stresses due to curvature of the plate girder. The general equation as derived by Basler (1959) is shown as equation (3.4).

$$\beta_{wmax} = \sqrt{\frac{\pi^2 E}{24 \cdot (1-\upsilon^2)} \cdot \frac{A_w}{A_f} \cdot \frac{1}{f_{ytf} \cdot \varepsilon_{tf}}} \tag{3.4}$$

To ensure that the flange fully yields to be able to get some deformation capacity, the strain in the flange is assumed to be above yield stain, $\varepsilon_{tf} \geq \varepsilon_{ytf} = \frac{f_{ytf}}{E}$. The residual stresses in the compression flange are then considered as to ensure that $\varepsilon_{tf} \geq \varepsilon_{ytf}$ giving the expression for strain as $\varepsilon_{tf} = \frac{f_{ytf}+\sigma_r}{E}$ which when substituted into equation (3.4) gives:

$$\frac{h_w}{t_w} \leq \beta_{wmax} = 0.67 \cdot \sqrt{\frac{A_w}{A_{tf}}} \cdot \sqrt{\frac{E^2}{f_{ytf} \cdot (f_{ytf} + \sigma_r)}} \tag{3.5}$$

According to Basler and Thurlimann (1961) the web slenderness is the most influential parameter in a girder subjected to bending and they proposed an upper limit for the ratio of the flange to web area as 0.5 which when substituted into equation (3.5) gives:

$$\beta_{wmax} = \frac{0.48E}{\sqrt{f_{ytf} \cdot (f_{ytf} \cdot \sigma_r)}} \quad (3.6)$$

By assuming a residual stress level of $\sigma_r = {f_{ytf}}/{2}$ for mild steel another simplification for maximum web slenderness from equation (3.4) becomes

$$\frac{h_w}{t_w} \leq \beta_{wmax} = 0.55 \cdot \frac{E}{f_{ytf}} \cdot \sqrt{\frac{A_w}{A_f}} = k \cdot \frac{E}{f_{ytf}} \cdot \sqrt{\frac{A_w}{A_f}} \quad (3.7)$$

Where:

k is a constant.

Equation (3.7) is adopted in EN1993-1-5 with the value of the constant dependent on the failure mechanism in question for the element.

SANS10162-1 also adopts the same equation with the constant taken as k ≅ 0.4, and the ratio of flange to web area equal to a unit.

3.1.3. Tension field action

For a plate girder with stiffeners connected to the web, the stiffened web plate can carry more load after buckling in the diagonal tension field (Parrott, 2011). This results in the girder behaving more like a truss as shown in figure 3-6 with tension regions in the buckled plate behaving like truss diagonals and stiffeners in compression. The tension field action may not be considered for girder end panels or internal panels with large openings (Mahachi, 2004).

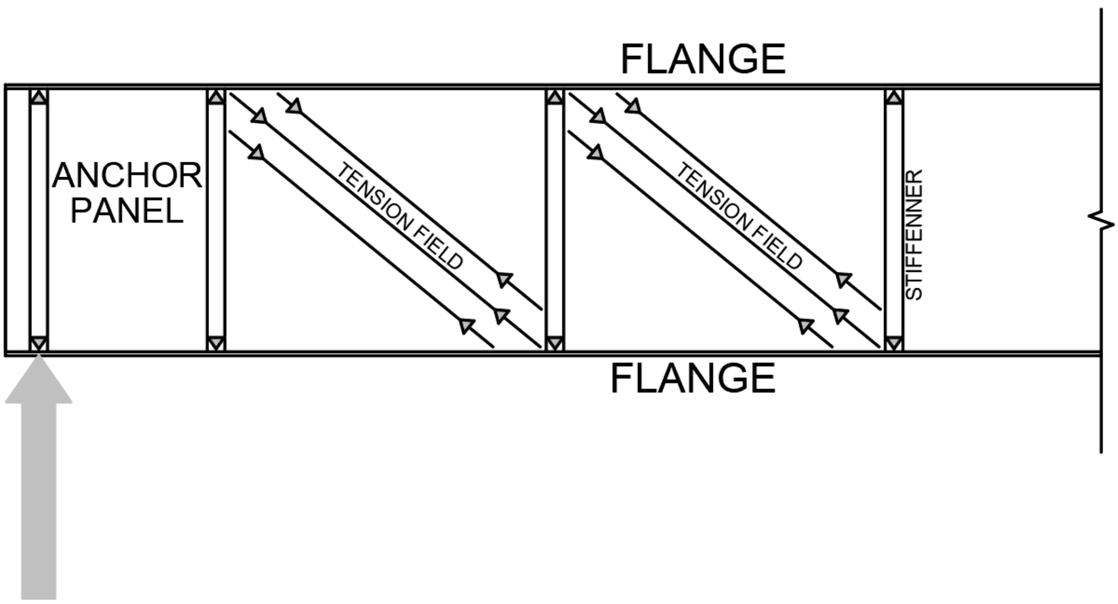


Figure 3-6: Tension field action in a welded girder with stiffeners (Parrott, 2011).

3.1.4. Moment resistance in plate girders

The compression flange is the primary support to bending actions hence its moment resistance capacity is critical. A girder's bending moment resistance is essentially governed by the stability of the compressed flange.

Because of this generalised understanding, earlier design codes like the BS5950-1 popularised the use of the 'flange only' method for calculating a plate girder's moment carrying capacity. The design standard states that if a plate girder experiences high shear load values of up to 60% its shear capacity, the entire shear stress can be taken as being resisted by the web with the two flanges resisting all stresses that result from the applied moments with a requirement that both flanges cannot be slender. If the applied shear is less than 60% which the code categorised as low, the moment equations available in the standard for hot rolled I-beam sections can be adopted. The method became very popular with steel designers as the formulae for calculating bending moment resistance were very simple and designers could quickly estimate the bending resistance of a plate girder by hand calculation but however it is very conservative (Lee & Chiew, 2013). The flange only method ignores any contribution to moment resistance by the web and any connections like the web-flange weld. The moment resistance according to the flange only method can be calculated as shown in equation (3.8).

$$M_{fRd} = A_f \cdot f_y \cdot \left(h_w + t_f\right) \quad (3.8)$$

Plate girder design for flexural capacity has however moved towards the more accurate, though more cumbersome effective width method. A plate girder with class 3 flanges and a class 4 web has a maximum moment resistance less than the factored yield moment ΦM_y, because the class 4 web buckles prematurely due to the compressive stresses it experiences through the edges where it connects to the flanges. At this point the bending stress distribution will no longer be linear and additional stresses are transferred to the compression flange due to the web deflecting laterally on its compression side.

Designers normally account for this phenomenon by multiplying the elastic bending moment resistance, M_{el}, by a reduction factor, ξ. SANS 10162 adopts an equation by Basler and Thurlimann (1961) which is based on the effective width theory with the aim to take post buckling capacity into account.

3.1.4.1. Effective width method

Also referred to as the 'effective modulus method', it follows the principle that stress is uniformly distributed over a plate cross-section up to the critical stress σ_{cr}, at this stage, the plate buckles and stresses are redistributed with an increase close to the edges of the plate and a reduction in stress in the middle.

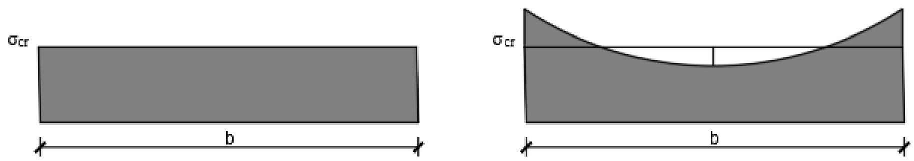


Figure 3-7 Stress distributions just before critical stress and at reaching critical stress.

At this buckling stress, the plate tends to withdraw itself from the loading and if the edges of the plates are supported, they then react stiffer than the middle area leading to the in-plane stresses distributing more to these areas and the adjacent elements.

Stress redistribution continues until yield strength of the cross section f_y, is attained at the cross-section edges as shown in figure 3-8.

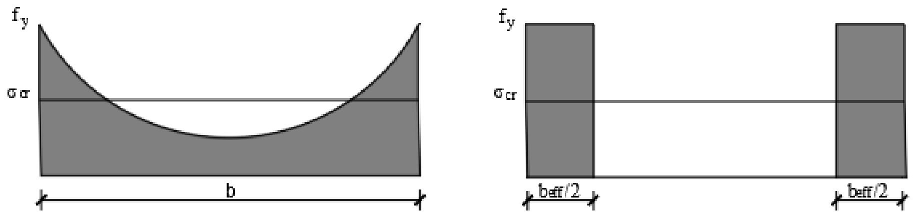


Figure 3-8 Stress distributions beyond critical stress and at effective stress distribution

The resistance of the plate remains the same and Von Karman (1910) estimated the effective width b_{eff} by equating the resistance at the critical stress σ_{cr}, to the value of the same at yield stress f_y, as shown in equation (3.9).

$$\sigma_{cr} \cdot b \cdot t = f_y \cdot b_{eff} \cdot t \tag{3.9}$$

⇨
$$\frac{b_{eff}}{b} = \frac{\sigma_{cr}}{f_y} \tag{3.10}$$

Substituting $f_y = \sigma_{cr}$ at effective stress distribution and $b=b_{eff}$ into equation (3.1) for elastic critical stress gives

$$f_y = k_\sigma \cdot \frac{\pi^2 E}{(1-\upsilon^2)\left(\frac{b_{eff}}{t}\right)^2} \tag{3.11}$$

⇨
$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}} \tag{3.12}$$

Equation (3.12) has since been modified several times with the latest version, adopted in EN1993-1-5 as developed by Winter et al. (1950) and further modified by Winter (1952) given as equation (3.13).

$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}} \cdot \left(1 - 0.22\sqrt{\frac{\sigma_{cr}}{f_y}}\right) \tag{3.13}$$

For plate girders it is assumed that since the web is a class 4 slender element, part of the compressive web is subjected to local buckling and can no longer be able to contribute to the effective section of the plate girder. Only the remaining effective section will act like an equivalent class 3 section contributing to the bending moment resistance at ultimate limit state (Lee & Chiew, 2013).

3.1.5. Moment resistance in plate girders according to Basler and Thurlimann (1961)

Basler and Thurlimann (1961) assumed an effective width of $30t_w$ for the part of the web under compression for a S235 steel grade web plate of web slenderness $\frac{h_w}{t_w} = 360$ as shown in figure 3-9.

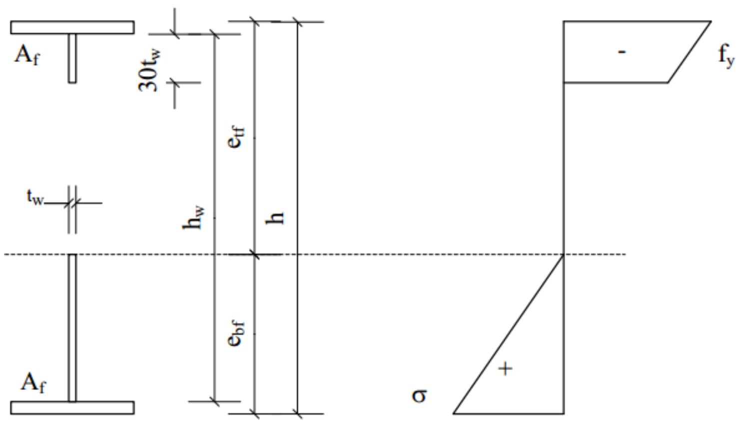


Figure 3-9: Effective cross section and stress distribution for I section under compression (Basler & Thurlimann, 1961).

Using equilibrium of parts in tension and in compression the neutral axis position in the web can be determined by assuming a linear distribution of stress. Basler and Thurlimann (1961) further gave several equations to determine bending moment resistance based on a reduction factor ξ. The equation adopted in SANS10162 is based on the formula for web slenderness shown here as equation (3.14) which can be seen to be related to proposals for web slenderness by Basler and Thurlimann (1961), shown earlier as equation (3.7) and also the reduction factor as shown in equation (3.15)

$$\frac{h_w}{t_w} > 5.7\sqrt{\frac{E}{f_y}} \tag{3.14}$$

$$\xi = 1 - 0.0005\frac{A_w}{A_f} \cdot \left(\frac{h}{t} - 5.7\sqrt{\frac{E}{f_y}}\right) \tag{3.15}$$

Basler calculated the elastic bending moment resistance as

$$M_{el} = A_f \cdot (h - t_f) \cdot f_y + {}^1/_6 \cdot t_w \cdot h_w^2 \cdot f_y \tag{3.16}$$

Which can be simplified to give equation (3.17)

$$M_{el} = A_f \cdot h \cdot f_y \left(1 + \frac{\rho}{6}\right) \tag{3.17}$$

By assuming $h\text{-}t_f \approx h$, $h_w \approx h$ and $A_w = h_w \cdot t_w \approx h \cdot t_w$ given $\rho = A_w/A_f$

This then gives the ultimate bending moment resistance as shown in equation (3.18)

$$M_u = M_{el}\left[1 - 0.0005 \cdot \frac{A_w}{A_f}\left(\frac{h}{t} - 5.7\sqrt{\frac{E}{f_y}}\right)\right] \tag{3.18}$$

For a plate girder with a compact web Basler adopted an equation from the work done by Haaijer and Thurlimann (1957) considering plastic behaviour for a web of slenderness β_w=53 giving the relationship

$$\frac{M_u}{M_{el}} = \frac{M_{pl}}{M_{el}} = \frac{A_f \cdot h \cdot f_y \left(1 + \frac{\rho}{4}\right)}{A_f \cdot h \cdot f_y \left(1 + \frac{\rho}{6}\right)} = \frac{\left(1 + \frac{\rho}{4}\right)}{\left(1 + \frac{\rho}{6}\right)} \tag{3.19}$$

3.1.6. Moment resistance in plate girders according to Hoglund (1961)

Hoglund (1971) proposed a different way to determine the effective width as shown in equations (3.20) and (3.21).

$$b_{e1} = 0.76 \cdot t_w \cdot \sqrt{\frac{E}{f_y}} \tag{3.20}$$

$$b_{e2} = 1.64 \cdot t_w \cdot \sqrt{\frac{E}{f_y}} \tag{3.21}$$

The widths b_{e1} and b_{e2} are shown diagramatically in figure 3-10.

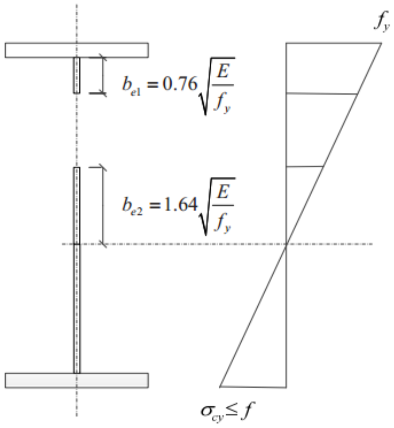


Figure 3-10: Effective cross section and stress distribution (Hoglund, 1971)

Hoglund (1971) then proposed the formula for ultimate bending moment resistance M_u as

$$M_u = M_{el}\left[1 - 0.15 \cdot \frac{A_w}{A_f}\left(1 - 4.8 \cdot \frac{t_w}{h_w} \cdot \sqrt{\frac{E}{f_y}}\right)\right] \quad (3.22)$$

For
$$\beta_w = \frac{h_w}{t_w} \geq 4.8\sqrt{\frac{E}{f_y}} \quad (3.23)$$

The formula for bending moment resistance by Hoglund (1971) is comparable and analogous to Basler's formula shown as equation (3.18).

3.1.7. Moment resistance in plate girders according to Herzog (1973)

Herzog (1973) cited in Abspoel (2015b) proposed another method to calculate the effective area and the ultimate bending moment resistance. The calculation considers the influences of lateral buckling, torsional buckling, and vertical buckling of the compressive flange into the web plate. The non-linear behaviour of the web is taken into account by reducing the maximum stress in the compressive part of the web to half of the yield strength as shown in fig 3-11.

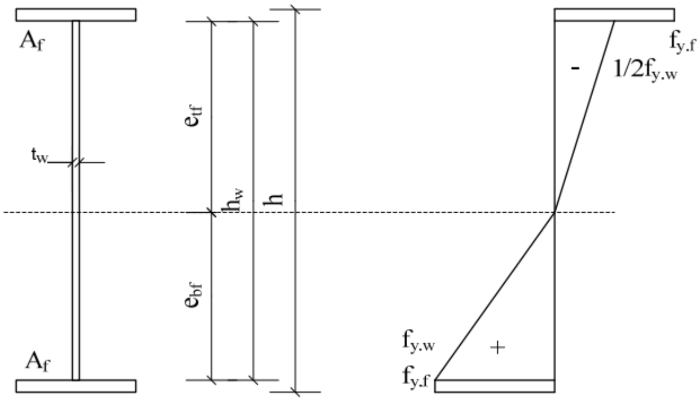


Figure 3-11: Herzog's plate girder stress distribution (Abspoel, 2015b)

Assuming a homogenous and symmetrical girder, the yield strength of the top flange is equal to the yield strength of the bottom flange, $f_{y.tf}= f_{y.bf}=f_{y.f}$ and the flange areas are equal, $A_{tf} =A_{bf} =A_f$, the height e_{tf}, of the compressive part of the web was determined as $e_{tf}= (2/3)\cdot h_w$.

Herzog (1973) then proposed an 'unmodified' bending moment resistance equation as

$$M_{Uo}^{o} = A_f \cdot f_{yf} \cdot \left(h_w + t_f\right) + \frac{1}{9} \cdot A_w \cdot f_{yw} \cdot h_w \tag{3.24}$$

Herzog (1973) adopted a factor, K_1 to multiply the unmodified bending moment resistance to account for torsional buckling of the compressive flange.

$$K_1 = \sqrt{\frac{16 \cdot t_f}{b}} < 1 \tag{3.25}$$

For $b/t_f \leq 16$ Herzog did not reduce the unmodified moment resistance for torsional buckling

To take the horizontal buckling into account Herzog (1973) adopted a second multiplying factor, K_2 calculated from equation (3.26)

$$K_2 = \sqrt{1 - \left(\frac{\lambda_y}{70}\right)^2} < 1 \tag{3.26}$$

With λ_y being a function of slenderness. This study assumes adequate lateral restraint therefore horizontal buckling is not critical.

To cater for the vertical buckling of the compressive flange into the web, Herzog (1973) adopted a third factor K_3 to multiply the unmodified bending moment resistance:

$$K_3 = 1.17 - \frac{d'}{2000 \cdot t_w} \tag{3.27}$$

With d’ being the spacing between longitudinal stiffeners. In the absence of longitudinal stiffeners d’= h_w giving

$$K_3 = 1.17 - \frac{h_w}{2000 \cdot t_w} = 1.17 - \frac{\beta_w}{2000} \qquad (3.28)$$

For web slenderness $\beta_w \leq 340$ the factor $K_3=1$ meaning vertical buckling of the compressive flange into the web is deemed not to be critical.

3.1.8. Moment resistance in plate girders according to Stark (1988)

Stark (1988) cited in Abspoel (2015b) developed a method to determine the ultimate moment capacity of a plate girder also based on the effective width method. The proposed formula is analogous to the one provided by Winter (1952) shown as equation (3.13). The only difference is a change of the factor 0.22 to 0.20 meaning the effective width as stated by Stark (1988) is slightly bigger. The effective width as proposed by Stark is shown in equation (3.29).

$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}} \cdot \left(1 - 0.20\sqrt{\frac{\sigma_{cr}}{f_y}}\right) \qquad (3.29)$$

The effective width at the top of the web is defined to be half of the effective width based on pure compression. Calculating the critical stress as shown in equation (3.1) with a buckling factor of k=4 gives the following expression

$$\sqrt{\frac{\sigma_{cr}}{f_y}} = \sqrt{k \cdot \frac{\pi^2 \cdot \frac{E}{f_y}}{12 \cdot (1-\upsilon^2) \cdot \left(\frac{b}{t}\right)^2}} = \sqrt{4 \cdot \frac{\pi^2 \cdot \frac{E}{f_y}}{12 \cdot (1-\upsilon^2) \cdot \left(\frac{b}{t}\right)^2}} = 1.90 \cdot \frac{\sqrt{\frac{E}{f_y}}}{\frac{b}{t}} \qquad (3.30)$$

Substituting equation (3.30) into (3.29) and accounting for the defined factor of half gives

$$b_{e1} = 0.95 \cdot t \cdot \sqrt{\frac{E}{f_y}} \cdot \left(1 - 0.38 \cdot \frac{\sqrt{\frac{E}{f_y}}}{\frac{b}{t}}\right) \qquad (3.31)$$

The total effective width b_{eff} of the web of a plate girder under pure bending is determined based on a critical stress σ_{cr} with a buckling factor k_σ=23.9 leading to equation (3.32).

$$\sqrt{\frac{\sigma_{cr}}{f_y}} = \sqrt{k \cdot \frac{\pi^2 \cdot \frac{E}{f_y}}{12 \cdot (1 - v^2) \cdot \left(\frac{b}{t}\right)^2}} = \sqrt{23.9 \cdot \frac{\pi^2 \cdot \frac{E}{f_y}}{12 \cdot (1 - v^2) \cdot \left(\frac{b}{t}\right)^2}} = 4.65 \cdot \frac{\sqrt{\frac{E}{f_y}}}{\frac{b}{t}} \tag{3.32}$$

Substitution of equation (3.32) into (3.29) gives:

$$b_{eff} = 4.65 \cdot t \cdot \sqrt{\frac{E}{f_y}} \cdot \left(1 - 0.93 \cdot \frac{\sqrt{\frac{E}{f_y}}}{\frac{b}{t}}\right) \tag{3.33}$$

The effective part of the bottom part of the web is calculated as $b_{e2} = b_{eff} - b_{e1}$ giving

$$b_{e2} = 3.69 \cdot t \cdot \sqrt{\frac{E}{f_y}} \cdot \left(1 - 1.07 \cdot \frac{\sqrt{\frac{E}{f_y}}}{\frac{b}{t}}\right) \tag{3.34}$$

Stark suggested the use of a much more simplified model with $b_{e1} = b_{e2} = 0.85 \cdot t_w \cdot \sqrt{\frac{E}{f_y}}$ as shown in figure 3-12.

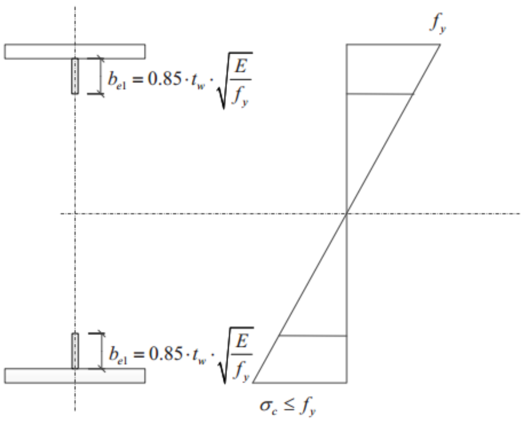


Figure 3-12: Stark's simplified effective width model

Stark did not provide an explicit equation to determine the ultimate moment resistance for a plate girder leaving it open for the use of iterations that adopt the suggested effective widths to determine the ultimate bending moment resistance of a cross section.

3.1.9. Moment resistance according to EN1993-1-5

Figure 3-13 shows how the effective width is calculated in EN1993-1-5.

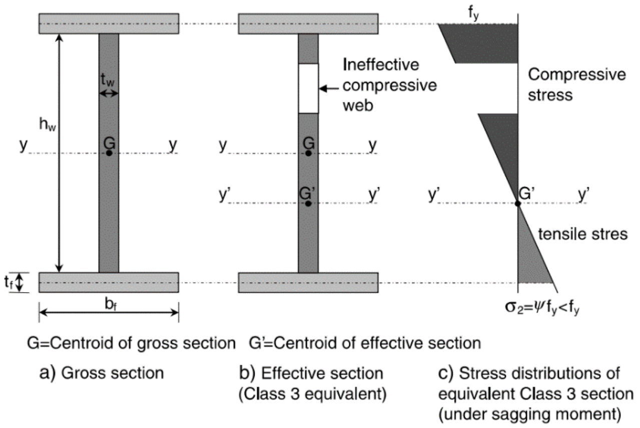


Figure 3-13: Effective cross section for EN1993-1-5 (Lee & Chiew, 2013)

The estimation of effective width in EN1993-1-5 as shown in figure 3-13 is very similar to the one proposed by Hoglund (1971). As part of the compressive web is ineffective in bending resulting in stress redistribution and changes in the effective area, the position of the centroid G tends to shift down to a new position G' under the action of a positive or sagging bending moment. This means that at the ultimate limit state the top extreme fibre will have yielded whilst the bottom extreme fibre remains in the elastic region of stresses (Lee & Chiew, 2013).

To compute the bending moment resistance of the equivalent class 3 section as determined from the effective area, EN1993-1-5 assumes the following:

i. There is a linear strain distribution in the web.
ii. The ultimate limit state is reached when yield strength f_y, is reached at the centroid of the compressive flange while stress at the centroid of the tensile flange will be less than f_y.

To compute the corresponding effective cross-section properties the following iterative steps with reference to figure 3-14 are to be followed.

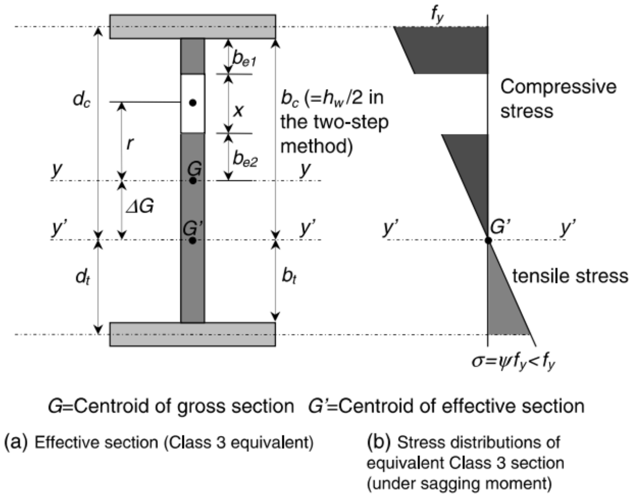


Figure 3-14: EN1993-1-5 effective cross section properties

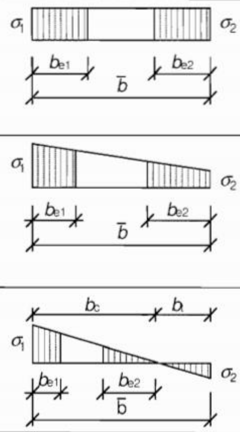
i. Compute the stress ratio ψ, assuming a linear stress distribution

$$\psi = {}^{\sigma_1}/_{\sigma_2} = -{}^{(b_t - t_f)}/_{(b_c - t_f)} \tag{3.35}$$

Where σ_1 and σ_2 are the direct stresses at the top and bottom flanges, respectively.

The assumption is that the section is subjected to a sagging moment with compressive stress taken as a positive and to start the calculation one could initially assume that the section is fully effective with b_t/b_c=1 and hence ψ=-1.

ii. Table 4.1 in EN1993-1-5 shown here as figure 3-15 is used together with the determined value of ψ to calculate the buckling factor k_σ.



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

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

Figure 3-15: Buckling factor for internal compression elements according to EN1993-1-5.

iii. Compute reduction factor ρ, from k_σ as determined from equation (3.36).

$$\rho = \frac{\overline{\lambda_p} - 0.055(3+\psi)}{\left(\overline{\lambda_p}\right)^2} \tag{3.36}$$

$\overline{\lambda_p}$ is the relative plate slenderness calculated as:

$$\overline{\lambda_p} = \frac{\left({h_w}/{t_w}\right)}{28.4\varepsilon \cdot \sqrt{k_\sigma}} \text{ and } \varepsilon = \sqrt{\frac{235}{f_y}}$$

iv. The effective breadth b_{eff}, is then taken as

$$b_{eff} = \rho b_c = b_{e1} + x + b_{e2} \tag{3.37}$$

with

$$b_{e1} = 0.4 b_{eff} = 0.4\rho b_c \tag{3.38}$$

$$b_{e2} = 0.6 b_{eff} = 0.6\rho b_c \tag{3.39}$$

$$x = (1-\rho) b_c \tag{3.40}$$

v. The location of the new centroid for the effective section G', can then be computed using the values of ρ, b_c, b_{e1}, and b_{e2} and hence the corresponding shift in the centroid location ΔG, as illustrated in figure 3-14.

vi. Based on ΔG, recalculate values of b_t and b_c and repeat the iteration from the first step until the change of ΔG is negligible and the values of b_t and b_c are converged.

vii. Calculate the second moment of area of the effective section I_{eff} based on the converged values of b_t, b_c, ΔG and other section dimensions.

viii. The effective section modulus of the section W_{eff}, and the bending resistance M_{cRd} are finally calculated as

$$W_{eff} = {I_{eff}}/{d_c} = {I_{eff}}/{\left({h_w}/{2} + {t_f}/{2} + \Delta G\right)} \tag{3.41}$$

$$M_{cRd} = W_{eff} \cdot f_y \tag{3.42}$$

3.1.10. Moment resistance according to Abspoel (2015b)

Abspoel (2015b) introduced a different formula for bending moment resistance of a plate girder M_{abs}, based on a simplified effective width method for the web and a simplified stress model as shown in figure 3-16.

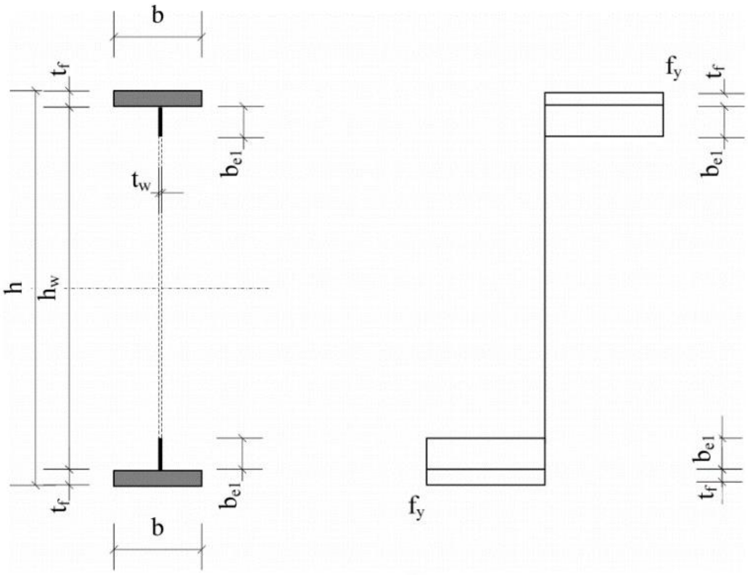


Figure 3-16: Abspoel effective cross section $A_{abs.eff}$ (Abspoel, 2015b)

The effective section is symmetrical, with contributions from the two flanges and two small effective widths of the webs that have a height equal to b_{e1}, as determined in EN1993-1-5 and computed from equation (3.38). The effective parts of the cross-section experience full plastic stress distribution (Abspoel, 2015b). Abspoel's proposed bending moment resistance M_{abs}, is computed from equation (3.43).

$$M_{abs} = f_y \cdot \left[b \cdot t_f \cdot \left(h_w + t_f \right) + b_{e1} \cdot t_w \cdot (h_w - b_{e1}) \right] \qquad (3.43)$$

Comparing with equation (3.8) for the flange only method, M_{abs} is an extension of the method to include a part of the web effective in resisting moments. Based on experiments carried on at Delft University of Technology, Abspoel (2015b) suggested the following for girders of steel grades S235 and S355:

i. A maximum web slenderness $\beta_{w.max}$ = 800 results in minimum cross-sectional area $A_{tot.min}$. A higher grade of steel also gives a smaller cross-sectional area.
ii. A minimum flange slenderness $\beta_{f.min}$ = 5 also results in minimum cross-sectional area $A_{tot.min}$.
iii. For a given minimum cross-sectional area $A_{tot.min}$, maximum web slenderness $\beta_{w.max}$ = 800, and minimum flange slenderness $\beta_{f.min}$ = 5, the optimal dimensions of a cross section can be determined from the following equations (3.44) to (3.47).

$$t_{w.opt} = \sqrt{\frac{1}{3 \cdot \beta_{w.max}}} \cdot \sqrt{A_{tot.min}} \tag{3.44}$$

$$h_{w.opt} = \sqrt{\frac{\beta_{w.max}}{3}} \cdot \sqrt{A_{tot.min}} \tag{3.45}$$

$$t_{f.opt} = \sqrt{\frac{1}{3 \cdot \beta_{f.min}}} \cdot \sqrt{A_{tot.min}} \tag{3.46}$$

$$b_{opt} = \sqrt{\frac{\beta_{f.min}}{3}} \cdot \sqrt{A_{tot.min}} \tag{3.47}$$

iv. The determined optimal dimensions of the web and flanges are then adapted to standardised dimensions such that:

$$b_{ad} \cdot t_{f.ad} \cdot \left(h_{w.ad} + t_{f.ad}\right) \geq b_{opt} \cdot t_{f.opt} \cdot \left(h_{w.opt} + t_{f.opt}\right) \tag{3.48}$$

$$\beta_{w.ad} = \frac{h_{w.ad}}{t_{w.ad}} \leq \beta_{w.max} = 800 \tag{3.49}$$

3.1.11. Conclusion

No reports or research could be found that are directly applicable to the exercise in this research although most of the research done is used to formulate the current problem statement. Researchers have simply overlooked and left a grey area on thicker webs being sometimes advantageous as they looked more at the possibilities of pushing the boundaries of maximum allowable web slenderness.

4. DESIGN CODE REQUIREMENTS

4.1. Introduction

This chapter intends to discuss the requirements/ recommendations of SANS10162-1 on the design of plate girders.

4.2. Requirements in SANS10162-1

4.2.1. Web slenderness

The limit for web slenderness to avoid flange induced buckling is based on equation (3.7) and is taken as:

$$\frac{h_w}{t_w} \leq \frac{83\,000}{f_y} \qquad (4.1)$$

It is important to note that, SANS10162-1 allows for waiving of this limit if analysis indicates that buckling of the compression flange into the web will not occur.

4.2.2. Web crippling and yielding

Unstiffened webs carrying loads concentrated normally over a short length of flange must resist the compressive stresses that arise in their plane. The ultimate strength of the unstiffened web subjected to such loading may be governed by either yielding of the web or by crippling of the web which is a localised out of plane buckling of the web adjacent to the loaded flange (Canadian Standards Association, 2001).

To ensure that the web is adequate to transversely carry loads the member is subjected to, the design code allows for a factored bearing resistance of the web B_r as:

i. For transverse loads applied at a distance from the member end greater than the member depth, the smaller of

a. $$B_r = \emptyset_{bi} \cdot t_w \cdot f_y\left(N + 10t_f\right) \qquad (4.2)$$

For relatively stocky webs or

b. $$B_r = 1.45\emptyset_{bi} \cdot t_w^2\sqrt{f_y \cdot E} \qquad (4.3)$$

For relatively thin webs that cripple before yielding.

ii. For transverse loads resulting from end reactions, the smaller of

a. $$B_r = \emptyset_{be} \cdot t_w \cdot f_y\left(N + 4t_f\right) \qquad (4.4)$$

For relatively stocky webs or

b. $$B_r = 0.6\emptyset_{be} \cdot t_w^2 \sqrt{f_y \cdot E} \quad (4.5)$$

For relatively thin webs that cripple before yielding.
Where:

N is the length of bearing flange [mm^2]

$\emptyset_{bi}$=0.80 and $\emptyset_{be}$=0.75.

Relatively stocky webs tend to yield before crippling and the vice versa is true for relatively thin web plates. The contribution of flanges is ignored in the expression of web crippling as it is argued that at interior load points, the normal stress in the flanges of efficiently designed girders would approach the yield strength at factored loads. Consequently, the flanges would not have significant plastic hinge capacity to develop a plastic hinge mechanism in the resistance of transverse loads (Canadian Standards Association, 2001). If the bearing resistance of the web is exceeded bearing stiffeners can be utilised however this is outside the parameters of this investigation.

4.2.3. Shear resistance and intermediate transverse stiffeners

For webs of plate girders with two flanges experiencing elastic deformations shear resistance is taken as:

$$V_r = \Phi \cdot A_v \cdot f_s \quad (4.6)$$

Where:

i. $A_v = t_w \cdot h_w$

ii. Calculation of f_s:

a. $f_s = 0.66 f_y$ when $\frac{h_w}{t_w} \leq 440\sqrt{\frac{k_v}{f_y}}$

Where:

$k_v = 4 + \frac{5.34}{\left({}^{s}/_{h_w}\right)^2}$ if ${}^{s}/_{h_w} < 1$ or $k_v = 5.34 + \frac{4}{\left({}^{s}/_{h_w}\right)^2}$ if ${}^{s}/_{h_w} \geq 1$ or

$k_v = 5.34$ if there are no stiffeners as s turns to ∞.

Where:

For $\frac{h_w}{t_w} \leq 150 : s \leq 3h_w$ and for $\frac{h_w}{t_w} < 150 : s \leq \frac{67500 h_w}{\left({}^{h_w}/_{t_w}\right)^2}$

b. When $440\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w} \leq 500\sqrt{\frac{k_v}{f_y}}$

$f_s = f_{cri} = 290\frac{\sqrt{f_y \times k_v}}{\left({}^{h_w}/_{t_w}\right)}$

With k_v, as defined in a.

c. When $500\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w} \leq 620\sqrt{\frac{k_v}{f_y}}$

$f_s = f_{cri} + f_t$

Where:

f_t is the tension-field post-buckling stress defined as

$f_t = k_a\left(0.50f_y - 0.866f_{cri}\right)$

Where:

k_a is the aspect coefficient defined as

$k_a = \frac{1}{\sqrt{1 + \left({}^{s}/_{h_w}\right)^2}}$

d. When $620\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w}$

$f_s = f_{cre} + f_t$

Where:

f_t is the tension field post buckling stress defined as

$f_t = k_a\left(0.50f_y - 0.866f_{cre}\right)$

Where:

$f_{cre} = \frac{180\ 000k_v}{\left({}^{h_w}/_{t_w}\right)^2}$

With k_v, as defined in a.

4.2.3.1. Stiffener spacing

At points of concentrated loads or reactions, bearing stiffeners can be utilised when the bearing resistance of the web is exceeded. For plate girders primarily resisting moments intermediate transverse stiffeners can be used to make the web less susceptible to buckling.

Where intermediate transverse stiffeners are required, the maximum spacing, s, between them is calculated as

for $$\frac{h_w}{t_w} \leq 150,\ s = 3h_w \quad (4.7)$$

or $$\frac{h_w}{t_w} > 150,\ s = \frac{67\ 500h_w}{\left({}^{h_w}/_{t_w}\right)^2} \quad (4.8)$$

The limits on intermediate transverse stiffener spacing are based on practical considerations. When s/h_w>3, the tension field contribution of the stiffeners is reduced and when h_w/t_w > 150 the maximum stiffener spacing is reduced for ease of fabrication and handling (Albert, 2003).

The stiffeners can be adopted singly or in pairs but the moment of inertia for the singular or parring of stiffeners shall not be less than $\left(\frac{h_w}{50}\right)^4$. The gross area of the stiffener(s), A_s cannot be less than the value from equation (4.9).

$$A_s \geq \frac{s \cdot t_w}{2} \cdot \left\{1 - \frac{{}^{s}/_{h_w}}{\sqrt{1 + \left({}^{s}/_{h_w}\right)^2}}\right\} \cdot C \cdot Y \cdot D \tag{4.9}$$

Where:

s is the centre-to-centre distance of adjacent stiffeners (mm)

$C = 1 - \frac{310\ 000 k_v}{f_y\left({}^{h_w}/_{t_w}\right)^2}$ but not less than 0.1

Y is the ratio of the specified minimum yield point of the web steel to that of the stiffener steel.

D is the stiffener factor which is:
1.0 for stiffeners furnished in pairs,
1.8 for single angle stiffeners or
2.4 for single plate stiffeners

The specified minimum moment of inertia ensures that the required stiffness is mobilised when web panels are behaving in an elastic manner. The minimum area requirements are to ensure that the stiffener can sustain the compression to which it is subjected when the web panel develops a tension field. As stiffeners subject to compression act as columns, providing a single stiffener on one side of the web means that it is loaded eccentrically and the factor D, as shown above is adopted to account for the resulting reduction in efficiency.

Stiffeners that are loaded through the girder top flange may have their lower ends cut short of the top of the bottom flange (Mahachi, 2004). The amount of cut back should be withing the limits shown in figure 4-1 . This is to prevent a local buckle in the girder web in the region of high strain whilst also providing a reasonable strain gradient. For stiffeners placed on both side of the web no weld is required between the flange and the stiffener whilst a nominal weld should be placed between the flange and the stiffener for stiffeners provided on one side of the web.

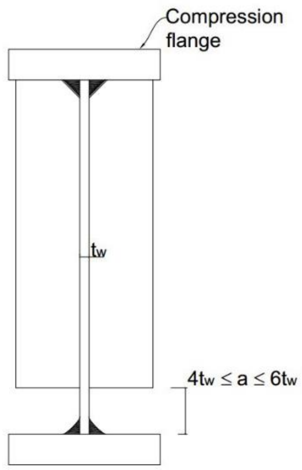


Figure 4-1: I-profile with stiffeners on both side of the web (Mahachi, 2004).

4.2.4. Moment resistance

To ensure that the plate girder exhibits a non-brittle like post critical behaviour, the yielding of the compression flange's outer fibre should be allowed (Abspoel, 2014). This means that in terms of cross-sectional classification which is based on local stability of elements the flange plate cannot be slender or exceed the limits of a class 3 section as defined in SANS10162-1.

The discussion will be looking at simply supported girders with maximum moments at mid-span which for class 3 sections the moment resistance, M_r is given by:

$$M_r = \Phi \cdot Z_e \cdot f_y \leq \Phi \cdot M_y \tag{4.10}$$

For plate girders with class 3 flanges but a web exceeding class 3 limits, a reduction factor as shown in equation (3.15) is applied to M_r. The girder's factored moment resistance becomes M'_r, given as:

$$M'_r = M_r \left[1 - 0.0005 \frac{A_w}{A_f} \cdot \left(\frac{h_w}{t_w} - \frac{1900}{\sqrt{M_u / (\Phi \cdot Z_e)}} \right) \right] \tag{4.11}$$

Where:

M_u is the actual moment in girder under ultimate load which can conservatively be taken as:

$$M_u = \Phi \cdot Z_e \cdot f_y \tag{4.12}$$

4.2.5. Combined Shear and Moment

This is a limit state of the web yielding by the combined action of flexural stress and the post buckling components of the tension field development in the web near the flange for transversely stiffened webs. This phenomenon is not of interest in this thesis.

4.2.6. Proportioning

The South African code does not go into much detail on proportioning and distributing the steel material between the flange and web areas, but guidance can be adopted from the SASCH (2016).

The SASCH (2016) states that for a given web depth-thickness ratio, the minimum mass cross-section for optimum moment capacity is that in which the area of the two flanges is approximately equal to the area of the web. This was proven by Abspoel (2015b) from experiments on plate girders carried out at Delft University of Technology.

For a given flange thickness and web thickness, a thicker flange plate gives a larger lever arm, $l_1 > l_2$ as shown in figure 4-2. This means that a more compact flange is preferred to increase moment carrying capacity.

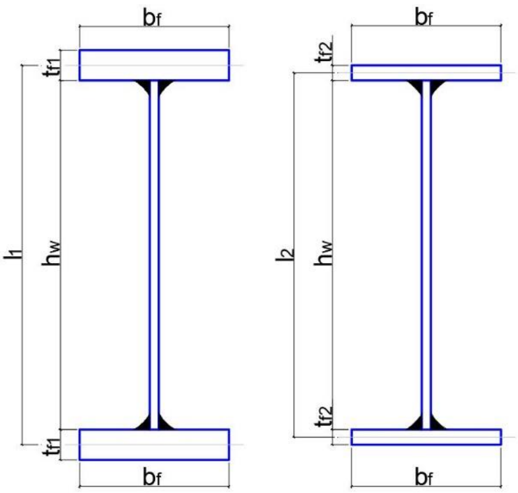


Figure 4-2: Lever arm for sections with different flange thicknesses only

Although making the flange more compact increases moment capacity, it was shown earlier that increasing the web height is more effective. Abspoel (2015b) in experiments carried out at the Delft University of Technology determined that the bending moment resistance deviated less than 2% in the case of flange slenderness β_f, varying just between just being class 2 and just being class 3 when the web is slenderness β_w, is just class 3. This percentage was found to decrease as β_w increased to around 0.3% at a maximum web slenderness β_w=800. Because of these considerations Abspoel (2015b) proposed a minimum flange slenderness, $\beta_f = 5$. All but one of the sections in table 2.20 of the SASCH (2016) have $\beta_f \geq 5$ so the recommendation by Abspoel (2015b) was adopted for this study.

From the experiments carried out by Abspoel (2015b) equations were derived by the author to estimate dimensions of the cross-section expressed in the total area A_{tot}, the ratio of areas $\rho = A_w/A_f$, the flange slenderness β_f, and the web slenderness β_w:

$$h_w = \sqrt{\frac{\rho}{\rho+2}} \cdot \sqrt{\beta_w} \cdot \sqrt{A_{tot}} \tag{4.13}$$

$$t_w = \sqrt{\frac{\rho}{\rho+2}} \cdot \sqrt{\frac{1}{\beta_w}} \cdot \sqrt{A_{tot}} \tag{4.14}$$

$$b = \sqrt{\frac{1}{\rho+2}} \cdot \sqrt{\beta_f} \cdot \sqrt{A_{tot}} \tag{4.15}$$

$$t_f = \sqrt{\frac{1}{\rho+2}} \cdot \sqrt{\frac{1}{\beta_f}} \cdot \sqrt{A_{tot}} \tag{4.16}$$

Where:

β_f ≥ 5
β_w ≤ 800 for steel grades S235 and S355
ρ $\cong 1$

4.2.7. Connections

The SAISC recommends that all welding be specified and executed in accordance with AWS D1.1 – American Welding Society Structural Welding Code – Steel except that the design for fatigue or resistance to static loading should be done in accordance with SANS10162-1 (De Clercq, et al., 2012).

A lower limit is set on the weld size of a fillet weld without preheating to avoid cracking because of rapid dissipation of heat into the parent metal. The weld size or leg length limit is in relation to the thicker connecting member which will normally be the flange in welded I-section girders and it does not have to exceed the thickness of the thinner connecting member. The limits laid down in AWS D1.1 are shown in table 4-1 extracted from the De Clercq et al. (2012).

Table 4-1: Minimum size of fillet welds without preheating (De Clercq, et al., 2012)

Thickness of thicker part (mm)	Minimum fillet weld size (mm)
Up to 12	5
Over 12 to 20	6
Over 20 to 40	8
Over 40 to 60	10
Over 60 to 150	12

Table 4-2 gives acceptable welding electrodes for Grade S355 steel and steels of lower yield stress.

Table 4-2: Electrodes to be used with Grade S275 and S355 steel.

Process	AWS Specification	Electrode	EN equivalent
SMAW	AWS A5.1	E70XX	E48XX (f_u = 480MPa)
SAW	AWS A5.17	F7XX-EXXX	
GMAW	AWS A5.18	ER70S-X	
FCAW	AWS A5.20	E7XT-X	

The meaning of each figure making up the full code for each electrode type and description of the welding processes can be found in De Clercq et al. (2012) and AWS D1.1.

According to Tredoux (2017) the maximum weld thickness a_v, that can be used without wasting material and not accounting for fabrication limitations is 0.7 times the thickness of the thinner plate as shown in equation (4.17).

$$a_v \approx 0.7 \cdot t_w \tag{4.17}$$

The ratio was derived with the assumption that the shear in the smaller web plate is equal to the shear in the two connecting welds, which will be the case when the girder section reaches its maximum shear resistance (Tredoux, 2017).

4.2.8. Other requirements

The ease of handling, fabrication and constructability of sections also plays a pivotal role in coming up with the optimum cross section.

Plate girders like other steel members work in a system of members making up a structure resisting different types of external loading. As such designers and contractors need girder members that can be attached to other steel members with relative ease. To that end, flange widths are usually limited to $b_f \geq$ 100mm. This ensures that drilling of holes for connections is not difficult and will not compromise performance of the girder (Tredoux, 2017).

The cross-sections should also be built up from available steel plates and there are three main groups that designers in South Africa can choose from. These are flat bars, hot rolled plates and standard steel plates.

4.2.8.1. Flat bars

Available in standard lengths up to 13m in 1m increments. Sections of widths up to 90mm and are typically only available in commercial quality steel grade according to the SASCH (2016). Flat bars of widths starting from 100mm can be convenient to adopt as flanges. Common flat bars dimensions and properties starting at a minimum width of 100mm are shown in table 4-3 extracted from the SASCH (2016).

Table 4-3: Flat bars dimensions and properties (Southern African Institute of Steel Construction, 2016)

Thickness	**Mass in kg/m for widths in mm**							
mm	**100**	**110**	**130**	**150**	**180**	**200**	**250**	**300**
6	4,71	5,18						
8	6,28	6,91	8,16	9,42				
10	7,85	8,64	10,2	11,8	14,1	15,7		
12	9,42	10,4	12,2	14,1	17,0	18,8		
16	12,6		16,3	18,8	22,6	25,1	31,4	37,7
20	15,7		20,4	23,6	28,3	31,4	39,2	47,1
25	19,6		25,5	29,4	35,3	39,2	49,1	58,9
30	23,6		31,6	35,3	42,4	47,1	58,9	70,6
40	31,4			47,1	56,5	62,8	78,5	94,2
45	39,2							106
50								118

4.2.8.2. Hot rolled plates

Due to the need to first straighten hot rolled plates (in coil form) before they can be used to fabricate required cross-sections which requires expensive equipment, they usually become more economical when doing mass production of sections. They hold a major advantage over flat bars and standard steel plates in that section span is not limited by the plate size (Tredoux, 2017).

Standard dimensions of hot rolled sheets are shown in table 4-4 extracted from the SASCH (2016). It is evident that they are available in small thicknesses making them ideal mostly for adaptation as web plates in plate girder fabrication.

Table 4-4: Available hot rolled sheets (1.2 mm < 4.5 mm) and hot rolled plates (4.5 mm -12 mm) standard dimensions (Southern African Institute of Steel Construction, 2016).

Standard thicknesses (mm)		
1,2 1,4 1,5 1,6 1,8 2,0 2,2 2,5 2,8 3,0 3,5 4,5 5,0 6,0 8,0 10 12		
Standard widths		
Thickness range (mm)	Widths (mm)	Increments (mm)
t < 2.5 t = 2.5 $2.8 \leq t \leq 12$	$600 \leq b \leq 1\,500$ $600 \leq b \leq 1\,500$ $600 \leq b \leq 1\,800$	25 25 25
Standard lengths		
1 000 to 6 000 mm in increments of 25mm subject to: Thickness $\leq$ 1.8 mm: maximum length = 3 650 mm Width > 1 225mm: minimum length = 1 800 mm		

4.2.8.3. Plates

Steel plates as shown in table 4-5 are readily available to be utilised easily either as flanges or webs for welded I-sections.

Table 4-5: Plates standard dimensions (Southern African Institute of Steel Construction, 2016).

Standard thicknesses (mm)	4.5	5	6	8	10	12	14	16	18
	20	22	25	28	30	32	35	38	40
	45	50	55	60	65	70	75	80	85
	90	100	110	115	120	125	130	140	150
Typical sizes (mm)	2 500 x 1 200		3 000 x 1500		4 000 x 2 000		8 000 x 2 000		
	10 000 x 2 400		13 000 x 2 400		12 000 x 3 000				

4.3. Moment resistance in international codes: EN1993-1-5 (Eurocode 3)

4.3.1. Web slenderness

To prevent flange induced buckling the Eurocode adopts an expression based on the equation by Basler and Thurlimann (1961) to limit web thickness as shown in equation (4.18).

$$\frac{h_w}{t_w} = k \cdot \frac{E}{f_{ytf}} \cdot \sqrt{\frac{A_w}{A_f}} \tag{4.18}$$

With the value of the dimensionless factor k, taken as:

- k=0.3 if plastic rotation is utilised,
- k=0.4 if plastic moment resistance is utilised and
- k=0.55 if elastic moment resistance is utilised.

4.3.2. Moment resistance

The steps to calculate moment resistance for class 4 sections are shown earlier with iterations culminating to the moment resistance equation (3.42).

For class 3 cross-sections

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min} \cdot f_y}{\gamma_{M0}} \tag{4.19}$$

Where:

$M_{c\,Rd}$ is design critical resistance moment.
$M_{el,Rd}$ is design elastic resistance moment.
$W_{el,min}$ is the elastic section modulus.
γ_{M0} is the partial factor for section resistance.

5. METHODOLOGY

5.1. Introduction

This chapter discusses the methodology or approach adopted in coming up with the various sections used in the experiment and the equations or requirements used.

5.2. Iteration procedure overview

Starting with a minimum cross-section area of 10 200 mm^2 which is the smallest cross-section area for plate girders in table 2.20 of the Southern African Steel Construction Handbook, two other plate girder cross-sections were computed. The sections were then compared for moment resistance and shear capacity efficiency calculated using equations from SANS10162-1. Each set of three sections started with the initial SASCH (2016) handbook was referred to as a single group of sections. The sections in the SASCH (2016) shown in appendix A will be referred to as 'S_0' sections.

In general. the additional two sections were computed based on;

a) Guidelines from SANS10162-1 and SASCH (2016) that, for a given cross-sectional area, a section with a class 4 slender web, and flanges limited to class 3 classification and flange area approximately equal to web area gives the highest moment capacity. In this text this section is referred to as 'S_1'.
b) A proposed section with both the web and flanges limited to class 3 section requirements herein referred to as 'S_2' and weight approximately equal to S_1.

S_0 sections were seen to sometimes carry different dimensions for the same cross-section area with different weights and hence different performance to weight ratios. The first iteration involved computing the dimensions for S_1 for a given cross section from S_0 for the group. S_1 was computed such that it gives the highest moment capacity allowable for the other restrictions adopted for it. Section S_2 was then computed for the group looking at achieving a comparable weight to the computed S_1 (weight optimisation).

As shown by Mela and Heinisuo (2014), optimising welded steel I-beams for weight is directly related to optimising for cost hence this study primarily looked at moment capacity optimisation for weight. The study optimised the sections for moment capacity first and then the respective shear properties of the sections were compared in terms of performance to weight ratios.

As indicated earlier, if the designer's freedom on the geometry of the elements making up the I-section is unlimited or unrestricted by other requirements like architectural or space requirements, the flange width, flange thickness, web height and web thickness can all be varied to come up with an optimum section. The major restrictions that were adopted in coming up with S_1 and S_2 are given in table 5-1.

Table 5-1: Main restrictions in determining S1 and S2 cross-section properties and dimensions.

Steel Grade	S355JR (common structural steel section is South Africa)
Minimum plate thickness	5mm (welding requirements)
Allowable plate thicknesses	As shown in tables (4-3), (4-4) and (4-5)

5.3. Iteration guidelines

Iterations to determine the optimum flange and web plate dimensions for ultimate moment carrying capacity were carried out with the intention of achieving the two additional sections S_1 and S_2.

Starting with a given cross-section area derived from S_0, section S_1 is computed. Section S_2 was then computed achieving a total mass of cross-section comparable to that of the just computed S_1 section. The following guidelines were adopted;

5.3.1. Class 4 web but flange not exceeding class 2, S_1.

i. $\rho = \frac{A_w}{A_f} \cong 1;$
ii. $t_f \geq t_w;$
iii. $b_f \geq 100\ mm;$
iv. $5 \leq \frac{0.5 \cdot b_f}{t_f} = \beta_f \leq \frac{200}{\sqrt{f_y}};$
v. $\frac{1900}{\sqrt{f_y}} < \beta_w = \frac{h_w}{t_w} \leq \frac{83\ 000}{f_y};$
vi. The study adopted the maximum possible web height for the class 4 web satisfying the above requirements to achieve a high moment carrying capacity.
vii. As shown in table 2-2, an increase in the flange breath is more effective than an increase in the flange thickness for moment capacity. Looking at the allowable dimensions, the flange width is more flexible to adjustments than the flange thickness. To that end, the iterations looked at adjusting the flange width more and exhausting possibilities before increasing or decreasing the flange thickness.

5.3.2. Class 3 sections, S_2.

i. $t_f \geq t_w;$
ii. $b_f \geq 100\ mm;$
iii. $5 \leq \frac{0.5 \cdot b_f}{t_f} = \beta_f \leq \frac{200}{\sqrt{f_y}};$
iv. $\beta_w = \frac{h_w}{t_w} \leq \frac{1\ 900}{\sqrt{f_y}};$
v. The study adopted the maximum possible web height for the class 3 web satisfying the above requirements to achieve a high moment carrying capacity.
vi. As shown in table 2-2, an increase in the flange breath is more effective than an increase in the flange thickness for moment capacity. Looking at the allowable dimensions, the flange width is more flexible to adjustments than the flange thickness.

To that end, the iterations looked at adjusting the flange width more and exhausting possibilities before increasing or decreasing the flange thickness.

Basically, section S_1 was guided towards having a slender, class 4 web section whilst a semi-compact, class 3 web is encouraged for section S_2. In both cases a large value for web height was targeted. Intermediate transverse stiffeners were assumed to be required for all slender webs.

5.4. Iteration procedures

All iterations followed the guidelines set out above and were stopped when any single one of them was violated.

5.4.1. Class 4 sections, S_1.

i. Starting with the allowable minimum web thickness as determined from the equations by Abspoel (2015b) and welding requirements, the maximum allowable web height was determined (Maximum web height for class 4 classification). This thickness was also adopted as the minimum flange thickness allowable.
ii. With the determined flange thickness from either step (i) or (iii), adjust the flange breath to give a section with the required cross-sectional area.
iii. Calculate and record the moment capacity, shear capacity and the mass of the cross-section at this stage.
iv. Increase the flange thickness to the next available plate size and repeat steps (ii) and (iii).
v. Adopt the section that gives the highest moment capacity as S_1 for the group.
vi. Should the recommended web thickness from step (i) fall between two allowable plate sizes, steps (ii), (iii), (iv) and (v) where carried out for both thicknesses.

5.4.2. Class 3 sections, S_2.

i. Starting with the allowable minimum web thickness as determined from the equations by Abspoel (2015b) and welding restrictions, determine the maximum allowable web height as per the guidelines above (Maximum web height for class 3 classification).
ii. With the determined flange thickness from either step (i) or (iii), adjust the flange breath to give a section with a mass approximately equal to that of S_1 above. If the exact mass could not be achieved a value as close as possible, just below the reference mass was preferred.
iii. Calculate and record the moment capacity, shear capacity and the mass of the cross-section at this stage.
iv. Increase the flange thickness to the next available plate size and repeat steps (ii) and (iii).
v. After exhausting the allowable sections in the steps above, increase the web thickness and repeat the steps.
vi. Adopt the section that gives the highest moment capacity as S_2 for the group.

Figure 5-1 shows comparable S_0, S_1 and S_2 girders from a cross-sectional for a given starting cross-sectional area of 33.8 x 10^3 mm^2 from S_0. The properties of each typical section are shown in the appendices. Stiffeners are indicated on the S_0 and S_1 cross -sections as they have class 4 webs and it assumed they will need stiffening.

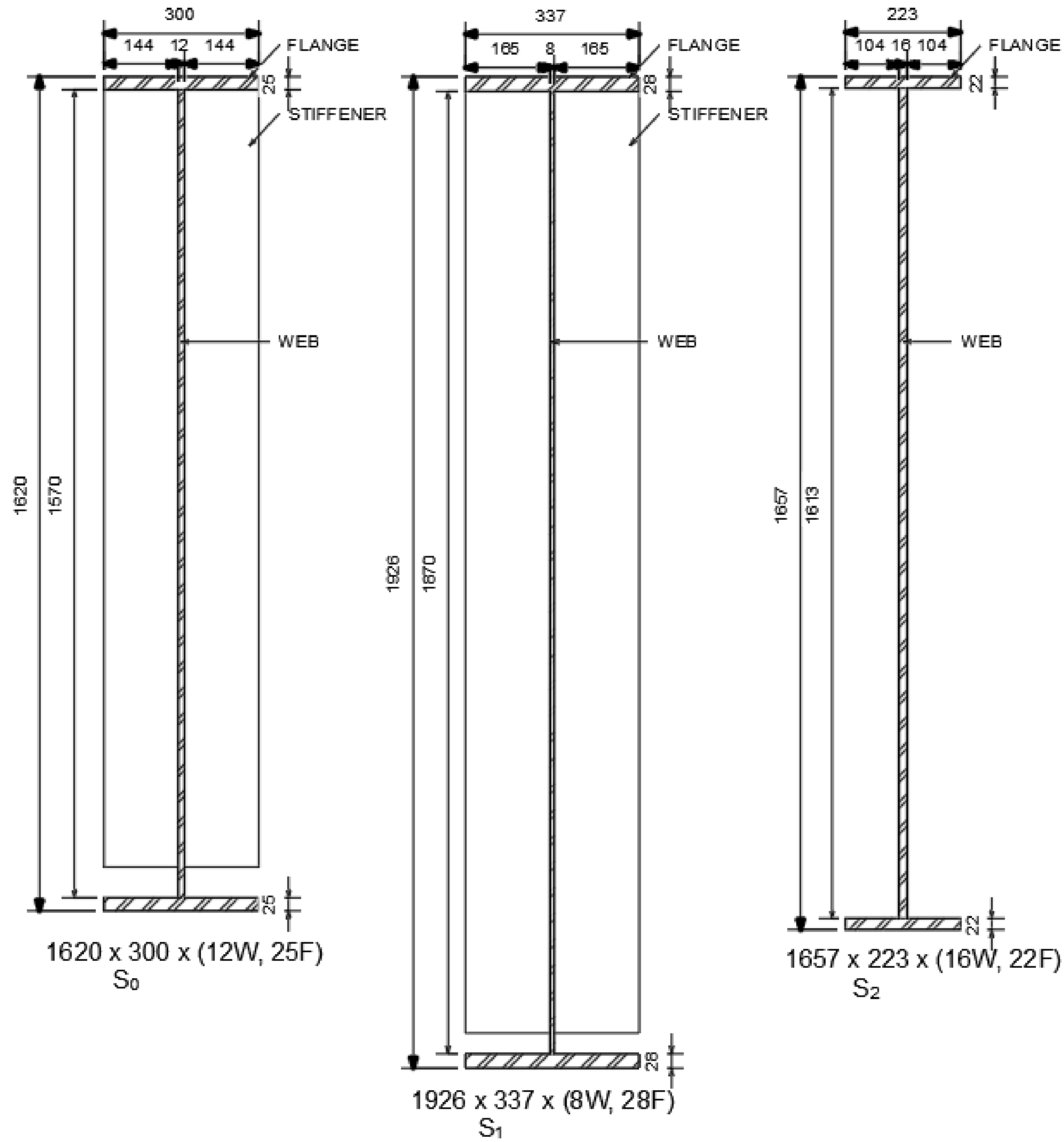


Figure 5-1: Comparable S_0, S_1 and S_2 cross-sections

5.5. Determining relevant section properties

Properties of the derived cross-sections important to the exercise were calculated from the following equations:

Cross-section area (mm^2)

$$A_{tot} = 2 \cdot \left(b_f \cdot t_f\right) + h_w \cdot t_w \quad (5.1)$$

Section mass per unit length (kg/m)

$$m = A_{tot} \cdot \rho_e \quad (5.2)$$

Where ρ_e = 7 850 kg/m^3.

Second moment of area about the principal section axes (mm^4)

$$I_x = \frac{1}{12} \cdot \left[b_f \cdot h^3 - \left(b_f - t_w\right) \cdot \left(h - 2t_f\right)^3\right] \quad (5.3)$$

$$I_y = \frac{1}{12} \cdot \left[2t_f \cdot b_f^3 + \left(h - 2t_f\right) \cdot t_w^3\right] \quad (5.4)$$

Elastic section modulus about the principal section axes (mm^3)

$$Z_{el,x} = \frac{2 \cdot I_x}{h} \quad (5.5)$$

$$Z_{el,y} = \frac{2 \cdot I_y}{b_f} \quad (5.6)$$

Plastic section modulus about the principal section axes (mm^3)

$$Z_{pl,x} = \frac{t_w \cdot h^2}{4} + (b - t_w) \cdot \left(h - t_f\right) \cdot t_f \quad (5.7)$$

$$Z_{pl,y} = \frac{b^2 \cdot t_f}{2} + \frac{h - 2t_f}{4} \cdot t_w^2 \quad (5.8)$$

6. RESULTS AND DISCUSSION

The numerical results from the analysis can be found in the appendices.

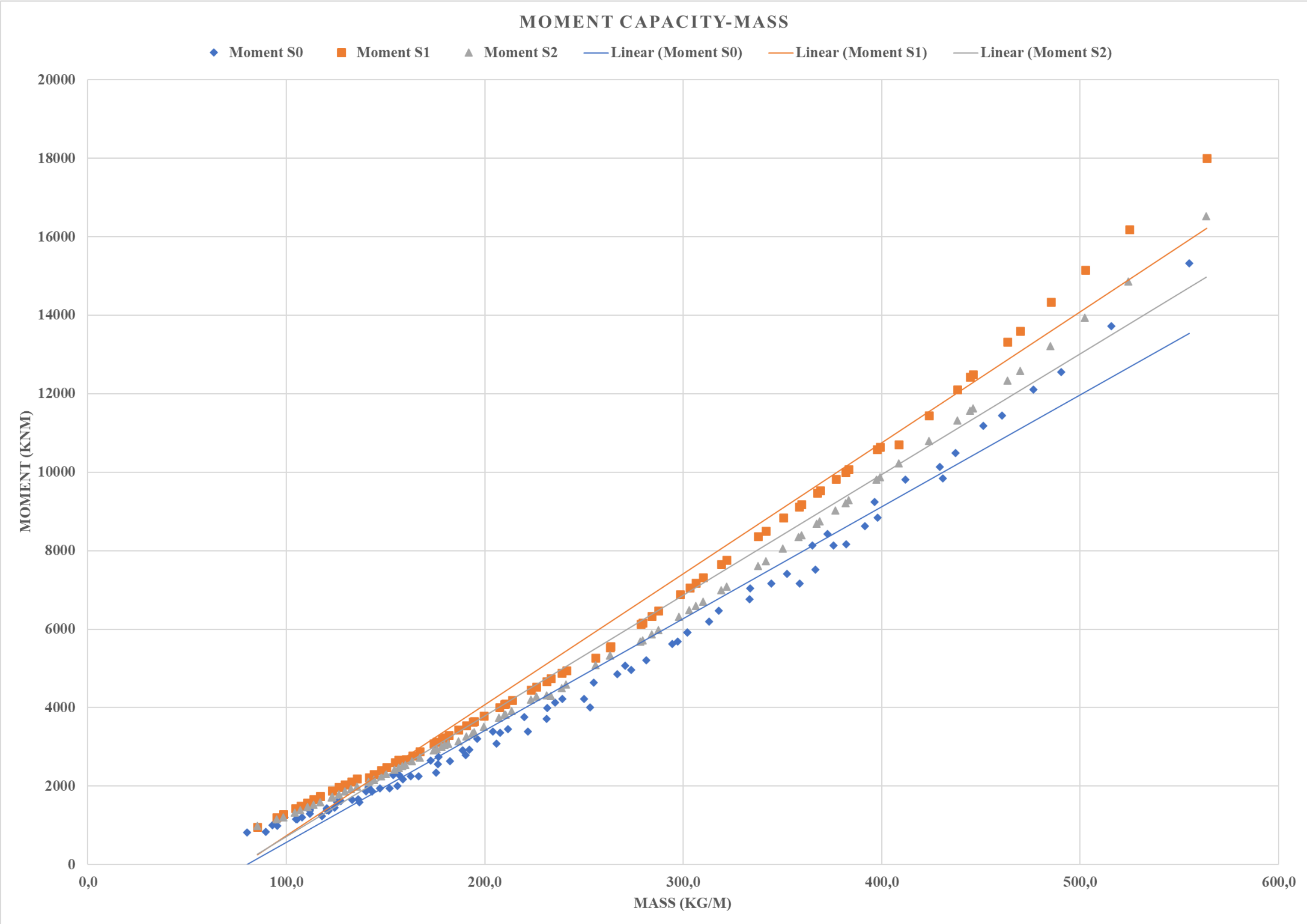


Figure 6-1: Moment capacity – Mass comparison for the 3 sections

As expected, the moment capacity values for both S_1 and S_2 are higher than the corresponding S_0 values as shown in figure 6-1. This is because S_1 borders around the maximum allowable class 4 web limit for the given cross-section area and S_2 was derived from the corresponding mass of S_1 as weight optimisation was carried out. The performance to weight ratios for S_1 and S_2 is also higher than those in S_0 as shown in figure 6-2. This shows that material in S_0 is not well utilised in comparison to the two.

The sample size consists of 92 sections and on average S_2 sections are 7% less efficient in moment capacity than S_1 sections with a small standard deviation of 1.8%. The difference in moment efficiency between S_2 to S_1 sections varies between 9.4% and -3.7%.

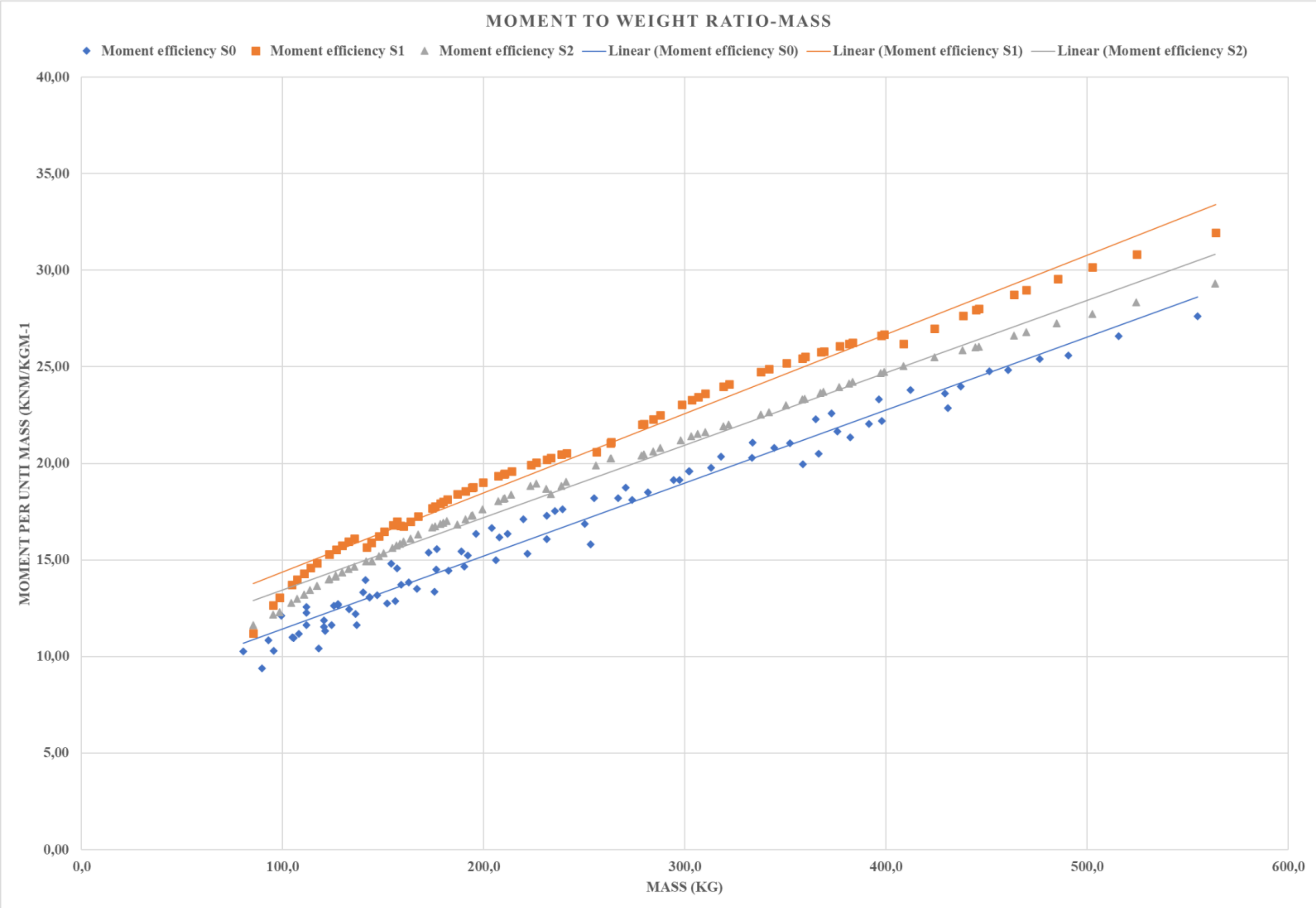


Figure 6-2: Moment performance per unit weight comparison for the 3 sections

An average of 7% reduction in efficiency can be concluded to be low but as the sections become bigger the equivalent reduction in moment capacity becomes significant. This is shown by the increasing gap between S_1 moment capacity and S_2 moment capacity in figure 6-1. For sections with mass less than or equal to 290kg/m the largest moment capacity difference is 489.76kNm which is for the heaviest pair of S_1 and S_2 sections within that group. The smallest difference is in turn found between the pair of the lightest sections in the group at -35.58kNm. This is also the only case where S_2 sections performed better than S_1 sections in terms of moment capacity. Taking the S_1 and S_2 sections of sizes 1197 x 336 x (5W, 16F) and 1 040 x 225 x (10W, 16F) respectively, the difference in moment carrying capacity is 195.4kNm. Using the recommended span to depth ratio of 17 from SASCH (2016) and the depth of the S_2 section, the girder span will be 17.68m. In terms of uniformly distributed loading, for a simply supported girder this translates to a maximum of 5kN/m difference in load capacity between the respective S_1 and S_2 sections. This is the biggest difference for the girders of mass below under 290kg/m under similar conditions. The average difference for the sections below 290kg/m is 3.4kN/m with an average span of 21.79m. In this group, S_2 sections perform about 6.7% less in moment capacity to weight efficiency than S_1 sections.

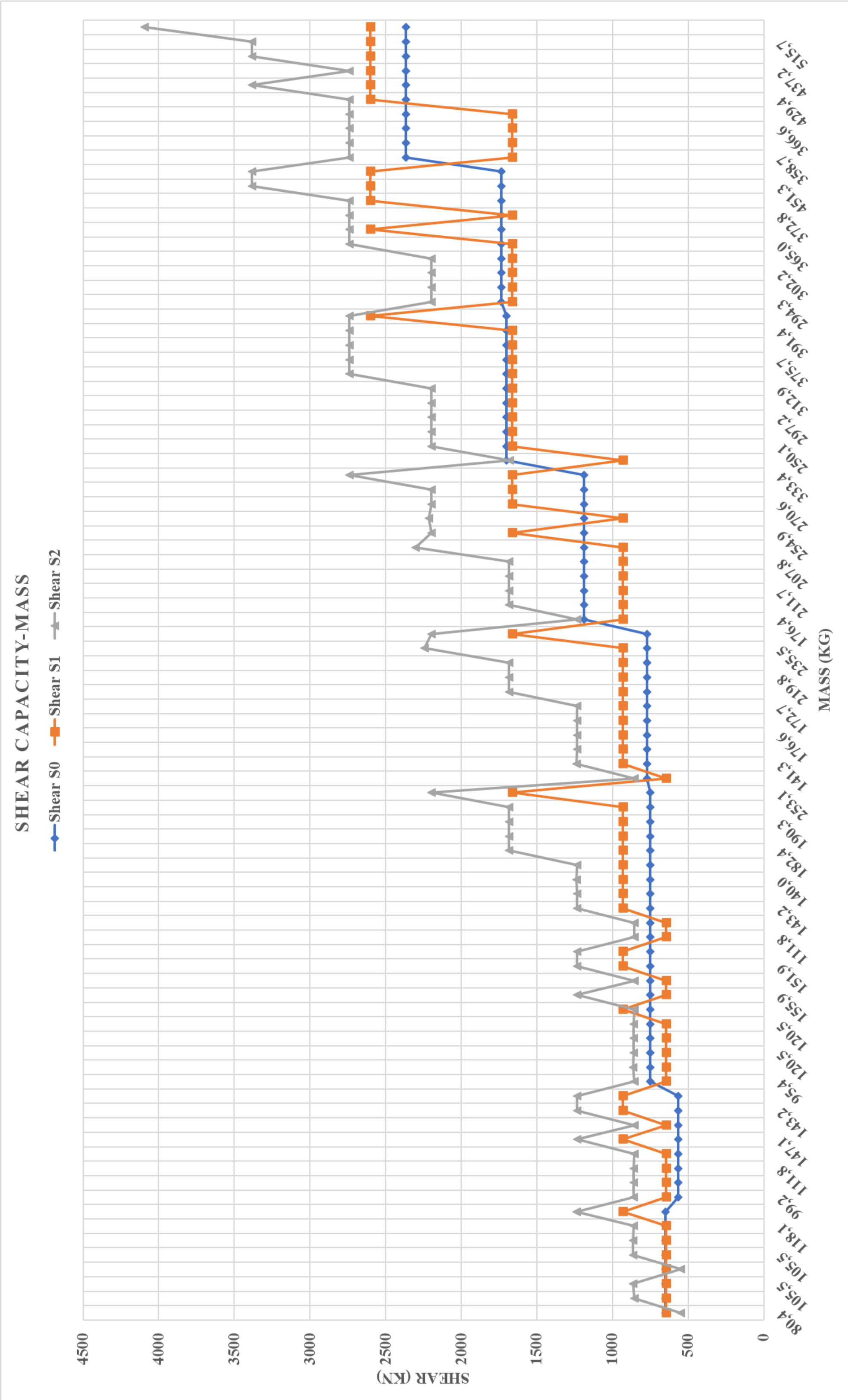


Figure 6-3: Shear capacity-mass comparison for the three sections

S_2 sections perform much better in shear than S_0 and S_1 sections as shown in figure 6-3. On average, S_1 sections were shown to be 43.8% less effective in a range of -147.0% to 14.9%. This high performance to weight ratio for S_2 sections is owing to the much thicker class 3 webs. S_1 sections outperformed their S_0 counterparts though with much thinner webs. This goes to prove the earlier observation that the available material was not configured to realise optimum resistance potential in SASCH (2016) sections.

On average S_2 sections can resist an extra 421.8kN of shear force for the sections below 290kg/m in mass mentioned above. The maximum is 1 309.59kN. The average increase in shear capacity to weight performance for sections in this group is 46.0% when moving from S_1 to S_2 sections. The S_2 Section 1 040 x 225 x (10W, 16F), which is the one with the lowest performance for moment in the group as mentioned above can resist an additional 212.01kN shear force. The largest cross-sectional area below the 290kg/m threshold for the S_1 section is 33.8x10^3 mm^2 and 35.6x10^3mm^2 for S_2 sections.

Figure 6-4: Shear performance to weight ratio comparison for the 3 sections

6.1. Cost comparison

For the group of sections with mass ≤ 290kg/m a section giving the largest span as recommended by the span to depth ratio in SASCH (2016) was selected and compared for cost with its corresponding S_1 section of the same span. The selected sections are shown in figure 6-5.

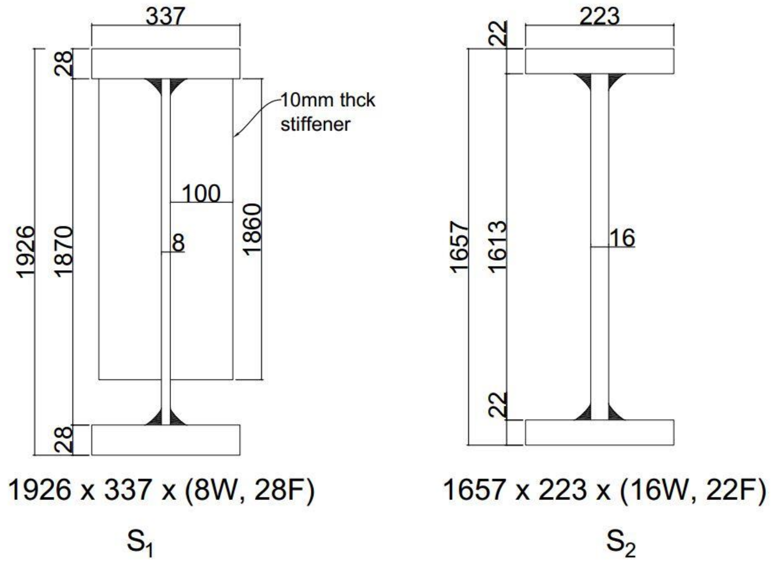


Figure 6-5: S_1 and S_2 sections for cost comparison.

Cutting of the lower end of the stiffener is as shown in figure 4-1 adopt 1 860mm long by 1 00mm wide stiffeners.

According to clause 11 in SANS10162-1, for stiffeners, $\frac{b}{t} \leq \frac{200}{\sqrt{f_y}}$. For b = 100mm adopt t = 10mm.

From the calculations done during analysis, the maximum allowable stiffener spacing is 2310.16 mm. Adopt 14 No. stiffeners at equal spacing on each side for a 28 170mm long span. From the same calculations, the required stiffener area, $A_{srequired}$ = 1 796 mm^2.

$A_{sprovided} = 200 * 10 = 2\,000\, mm^{2} > A_{srequired}$ ∴ ok.

$\left(\frac{h_w}{50}\right)^4 = 1.96E06 < \frac{hb^3}{12} = 1.24E09$ ∴ ok. (clause 14 SANS10162-1)

Messrs. Tarpet Steelworks (Pty) Ltd provided a broken-down cost comparison of the two girders as shown in table 6-1. The cost only covered material, fabrication of the sections as described above. Corrosion protection means hot deep galvanising to SANS 121:2011 with a minimum local coating thickness of 70µm and a minimum average coating thickness of 85µm. The costs are exclusive of value added tax.

Table 6-1: Cost comparison

	S_1	**S_2**
Material	R137 000.00	R133 000.00
Labour and consumables	R60 000.00	R54 000.00
Corrosion protection	R32 200.00	R31 520.00
Total	**R229 200.00**	**R218 520.00**

From the iterations the chosen S_1 section weighs 279.7kg/m with the S_2 section weighing 0.1kg/m less at 279.6kg/m. The weight for the S_1 section is inclusive of stiffener weight per unit length derived by multiplying the required stiffener area by steel density.

For the 28.17m span, S_1 gives a tonnage of 8.05t and S_2 weighs 7.88t. This is because even though the iterations came up with sections of almost the same mass per unit length, when the actual length is known, the number and total length of the stiffeners become exact and the mass of the two sections will differ as a result. The longer the span, the more the stiffeners and the heavier the S_2 section is compared to a corresponding S_1 section. This explains the cost differences on the material and corrosion protection between the two sections.

The cost of labour and consumables for the S_1 section is also higher than that of the S_2 section because of the presence of stiffeners in the former.

The total cost of production for the S_1 section is 5% higher than the S_2 section.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Conclusions

Taking a maximum mass per unit length of 290kg/m, on average, computed S_2 sections have a moment performance to weight ratio that is 6.7% lower than S_1 sections. In terms of shear resistance, they perform at an average that is 46% higher. The S_2 sections for the given mass threshold have proven to be more economical than their S_1 counterparts as the spans become realistic and bigger.

From the results above, it can be concluded that for a required cross-sectional area for moment resistance less than or equal to $35.6 \times 10^3 mm^2$ class 3 sections provide a competitive alternative to sections with class 4 webs and flanges below class 3 classification. For similar cross-sectional area requirements, they can perform better in terms of costs for large spans and operationally they are easier to fabricate with less elements to be welded together also resulting in reduced labour costs.

The study therefore shows that there exists a range of sections where class 3 webs/sections could offer an alternative to sections with class 4 webs, particularly where moment capacity difference is marginal or for cases where shear resistance is also a concern, with accompanying benefits.

7.2. Recommendations

1. Further study of the behaviour of the recommended girders performance under different loading conditions.
2. Further study of the behaviour of recommended girders on their performance under serviceability requirements like stiffness for deflections.

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9. APPENDICES

Appendix A: SASCH (2016) Sections, (S_0)

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3 mm^3	10^3 mm^3
704 x 200 (8W, 12F)	80,4	704	200	8	12	680	10,2	784	2 228	2 586
712 x 200 (8W, 16F)	92,9	712	200	8	16	680	11,8	985	2 766	3 152
720 x 200 (8W, 20F)	105,5	720	200	8	20	680	13,4	1 190	3 305	3 725
704 x 250 (8W, 12F)	89,8	704	250	8	12	680	11,4	928	2 636	3 001
712 x 250 (8W, 16F)	105,5	712	250	8	16	680	13,4	1 179	3 311	3 709
720 x 250 (8W, 20F)	121,2	720	250	8	20	680	15,4	1 435	3 986	4 425
712 x 300 x (8W, 16F)	118,1	712	300	8	16	680	15,0	1 372	3 855	4 266
720 x 300 x (8W, 20F)	136,9	720	300	8	20	680	17,4	1 680	4 667	5 125
812 x 200 x (8W, 16F)	99,2	812	200	8	16	780	12,6	1 330	3 277	3 764
820 x 200 x (8W, 20F)	111,8	820	200	8	20	780	14,2	1 597	3 894	4 417
812 x 250 x (8W, 16F)	111,8	812	250	8	16	780	14,2	1 584	3 901	4 401
820 x 250 x (8W, 20F)	127,5	820	250	8	20	780	16,2	1 917	4 675	5 217
830 x 250 x (8W, 25F)	147,1	830	250	8	25	780	18,7	2 342	5 644	6 248
812 x 300 x (8W, 16F)	124,3	812	300	8	16	780	15,8	1 837	4 525	5 038
820 x 300 x (8W, 20F)	143,2	820	300	8	20	780	18,2	2 237	5 456	6 017
830 x 300 x (8W, 25F)	166,7	830	300	8	25	780	21,2	2 747	6 620	7 254
904 x 200 x (8W, 12F)	92,9	904	200	8	12	880	11,8	1 409	3 118	3 690
912 x 200 x (8W, 16F)	105,5	912	200	8	16	880	13,4	1 739	3 814	4 416
920 x 200 x (8W, 20F)	118,1	920	200	8	20	880	15,0	2 075	4 510	5 149
904 x 250 x (8W, 12F)	102,4	904	250	8	12	880	13,0	1 648	3 646	4 225
912 x 250 x (8W, 16F)	118,1	912	250	8	16	880	15,0	2 060	4 518	5 133
920 x 250 x (8W, 20F)	133,8	920	250	8	20	880	17,0	2 480	5 391	6 049
930 x 250 x (8W, 25F)	153,4	930	250	8	25	880	19,5	3 014	6 483	7 205
904 x 300 x (8W, 12F)	111,8	904	300	8	12	880	14,2	1 887	4 174	4 760
912 x 300 x (8W, 16F)	130,6	912	300	8	16	880	16,6	2 381	5 222	5 850
920 x 300 x (8W, 20F)	149,5	920	300	8	20	880	19,0	2 885	6 271	6 949
930 x 300 x (8W, 25F)	173,0	930	300	8	25	880	22,0	3 526	7 584	8 336
1 004 x 250(8W, 12F)	108,6	1 004	250	8	12	980	13,8	2 104	4 190	4 897
1 012 x 250(8W, 16F)	124,3	1 012	250	8	16	980	15,8	2 612	5 161	5 905
1 020 x 250(8W, 20F)	140,0	1 020	250	8	20	980	17,8	3 128	6 133	6 921
1 030 x 250(8W, 25F)	159,7	1 030	250	8	25	980	20,3	3 784	7 348	8 202

SASCH (2016) Sections, (S_0)

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3 mm^3	10^3 mm^3
1 012 x 300 x (8W, 16F)	136,9	1 012	300	8	16	980	17,4	3 009	5 946	6 702
1 020 x 300 x (8W, 20F)	155,7	1 020	300	8	20	980	19,8	3 628	7 113	7 921
1 030 x 300 x (8W, 25F)	179,3	1 030	300	8	25	980	22,8	4 416	8 574	9 458
1 040 x 300 x (8W, 30F)	202,8	1 040	300	8	30	980	25,8	5 219	10 037	11 011
1 020 x 400 x (8W, 20F)	187,1	1 020	400	8	20	980	23,8	4 628	9 074	9 921
1 030 x 400 x (8W, 25F)	218,5	1 030	400	8	25	980	27,8	5 679	11 026	11 971
1 040 x 400 x (8W, 30F)	249,9	1 040	400	8	30	980	31,8	6 750	12 981	14 041
1 204 x 250 x (8W, 12F)	121,2	1 204	250	8	12	1180	15,4	3 227	5 360	6 361
1 212 x 250 x (8W, 16F)	136,9	1 212	250	8	16	1180	17,4	3 956	6 529	7 569
1 220 x 250 x (8W, 20F)	152,6	1 220	250	8	20	1180	19,4	4 696	7 698	8 785
1 230 x 250 x (8W, 25F)	172,2	1 230	250	8	25	1180	21,9	5 634	9 160	10 316
1 212 x 300 x (8W, 16F)	149,5	1 212	300	8	16	1180	19,0	4 529	7 473	8 526
1 220 x 300 x (8W, 20F)	168,3	1 220	300	8	20	1180	21,4	5 416	8 878	9 985
1 230 x 300 x (8W, 25F)	191,9	1 230	300	8	25	1180	24,4	6 541	10 636	11 822
1 240 x 300 x (8W, 30F)	215,4	1 240	300	8	30	1180	27,4	7 685	12 395	13 675
1 220 x 400 x (8W, 20F)	199,7	1 220	400	8	20	1180	25,4	6 856	11 239	12 385
1 230 x 400 x (8W, 25F)	231,1	1 230	400	8	25	1180	29,4	8 357	13 588	14 835
1 240 x 400 x (8W, 30F)	262,5	1 240	400	8	30	1180	33,4	9 882	15 938	17 305
1 402 x 250 x (10W, 16F)	170,3	1 402	250	10	16	1370	21,7	5 985	8 538	10 236
1 410 x 250 x (10W, 20F)	186,0	1 410	250	10	20	1370	23,7	6 973	9 891	11 642
1 420 x 250 x (10W, 25F)	205,7	1 420	250	10	25	1370	26,2	8 225	11 584	13 411
1 402 x 300 x (10W, 16F)	182,9	1 402	300	10	16	1370	23,3	6 753	9 634	11 345
1 410 x 300 x (10W, 20F)	201,7	1 410	300	10	20	1370	25,7	7 939	11 262	13 032
1 420 x 300 x (10W, 25F)	225,3	1 420	300	10	25	1370	28,7	9 441	13 297	15 155
1 430 x 300 x (10W, 30F)	248,8	1 430	300	10	30	1370	31,7	10 964	15 334	17 292
1 410 x 400 x (10W, 20F)	233,1	1 410	400	10	20	1370	29,7	9 872	14 002	15 812
1 420 x 400 x (10W, 25F)	264,5	1 420	400	10	25	1370	33,7	11 874	16 724	18 642
1 430 x 400 x (10W, 30F)	295,9	1 430	400	10	30	1370	37,7	13 905	19 447	21 492
1 440 x 400 x (10W, 35F)	327,3	1 440	400	10	35	1370	41,7	15 964	22 172	24 362
1 602 x 300 x (12W, 16F)	223,3	1 602	300	12	16	1570	28,4	9 907	12 368	15 008
1 610 x 300 x (12W, 20F)	242,1	1 610	300	12	20	1570	30,8	11 455	14 229	16 935
1 620 x 300 x (12W, 25F)	265,6	1 620	300	12	25	1570	33,8	13 411	16 557	19 357
1 630 x 300 x (12W, 30F)	289,2	1 630	300	12	30	1570	36,8	15 391	18 885	21 795

SASCH (2016) Sections, (S_0)

Designation	**m**	**h**	**b**	**t_w**	**t_f**	**h_w**	**A**	**About x-x**		
								I	**Z_e**	**Z_{pl}**
h x b x (t_w, t_f)	**kg/m**	**mm**	**mm**	**mm**	**mm**	**mm**	**10^3 mm^2**	**10^6 mm^4**	**10^3 mm^3**	**10^3 mm^3**
1 610 x 400 x (12W, 20F)	273,5	1 610	400	12	20	1570	34,8	13 983	17 370	20 115
1 620 x 400 x (12W, 25F)	304,9	1 620	400	12	25	1570	38,8	16 591	20 483	23 345
1 630 x 400 x (12W, 30F)	336,3	1 630	400	12	30	1570	42,8	19 232	23 597	26 595
1 640 x 400 x (12W, 35F)	367,7	1 640	400	12	35	1570	46,8	21 905	26 713	29 865
1 620 x 500 x (12W, 25F)	344,1	1 620	500	12	25	1570	43,8	19 771	24 409	27 332
1 630 x 500 x (12W, 30F)	383,4	1 630	500	12	30	1570	48,8	23 072	28 309	31 395
1 640 x 500 x (12W, 35F)	422,6	1 640	500	12	35	1570	53,8	26 414	32 212	35 482
1 820 x 300 x (12W, 25F)	284,5	1 820	300	12	25	1770	36,2	17 629	19 372	22 861
1 830 x 300 x (12W, 30F)	308,0	1 830	300	12	30	1770	39,2	20 127	21 996	25 599
1 810 x 400 x (12W, 20F)	292,3	1 810	400	12	20	1770	37,2	18 362	20 290	23 719
1 820 x 400 x (12W, 25F)	323,7	1 820	400	12	25	1770	41,2	21 656	23 798	27 349
1 830 x 400 x (12W, 30F)	355,1	1 830	400	12	30	1770	45,2	24 987	27 308	30 999
1 840 x 400 x (12W, 35F)	386,5	1 840	400	12	35	1770	49,2	28 354	30 820	34 669
1 820 x 500 x (12W, 25F)	363,0	1 820	500	12	25	1770	46,2	25 684	28 224	31 836
1 830 x 500 x (12W, 30F)	402,2	1 830	500	12	30	1770	51,2	29 847	32 620	36 399
1 840 x 500 x (12W, 35F)	441,5	1 840	500	12	35	1770	56,2	34 057	37 018	40 986
1 850 x 500 x (12W, 40F)	480,7	1 850	500	12	40	1770	61,2	38 312	41 418	45 599
2 120 x 300 x (14W, 25F)	345,2	2 120	300	14	25	2070	44,0	26 808	25 290	30 710
2 130 x 300 x (14W, 30F)	368,8	2 130	300	14	30	2070	47,0	30 194	28 352	33 897
2 140 x 300 x (14W, 35F)	392,3	2 140	300	14	35	2070	50,0	33 613	31 414	37 100
2 110 x 400 x (14W, 20F)	353,1	2 110	400	14	20	2070	45,0	27 821	26 371	31 717
2 120 x 400 x (14W, 25F)	384,5	2 120	400	14	25	2070	49,0	32 294	30 466	35 947
2 130 x 400 x (14W, 30F)	415,9	2 130	400	14	30	2070	53,0	36 810	34 563	40 197
2 140 x 400 x (14W, 35F)	447,3	2 140	400	14	35	2070	57,0	41 368	38 662	44 467
2 120 x 500 x (14W, 25F)	423,7	2 120	500	14	25	2070	54,0	37 781	35 642	41 185
2 130 x 500 x (14W, 30F)	463,0	2 130	500	14	30	2070	59,0	43 425	40 775	46 497
2 140 x 500 x (14W, 35F)	502,2	2 140	500	14	35	2070	64,0	49 123	45 909	51 835
2 150 x 500 x (14W, 40F)	541,5	2 150	500	14	40	2070	69,0	54 874	51 046	57 197

Appendix B: Computed S_1 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
1189 x 181 x (5W, 12F)	79,8	1 189	181	5	12	1165	10,2	2 163	3639	4253
1193 x 215 x (5W, 14F)	93,0	1 193	215	5	14	1165	11,8	2 751	4612	5245
1193 x 270 x (5W, 14F)	105,1	1 193	270	5	14	1165	13,4	3 286	5509	6153
1189 x 234 x (5W, 12F)	89,8	1 189	234	5	12	1165	11,4	2 604	4380	5002
1193 x 270 x (5W, 14F)	105,1	1 193	270	5	14	1165	13,4	3 286	5509	6153
1197 x 300 x (5W, 16F)	121,1	1 197	300	5	16	1165	15,4	4 006	6694	7365
1197 x 286 x (5W, 16F)	117,6	1 197	286	5	16	1165	15,0	3 850	6433	7101
1432 x 280 x (6W, 16F)	136,3	1 432	280	6	16	1400	17,4	5 864	8189	9284
1197 x 212 x (5W, 16F)	99,0	1 197	212	5	16	1165	12,6	3 024	5053	5702
1197 x 262 x (5W, 16F)	111,5	1 197	262	5	16	1165	14,2	3 582	5986	6647
1197 x 262 x (5W, 16F)	111,5	1 197	262	5	16	1165	14,2	3 582	5986	6647
1197 x 325 x (5W, 16F)	127,4	1 197	325	5	16	1165	16,2	4 285	7160	7838
1432 x 322 x (6W, 16F)	146,8	1 432	322	6	16	1400	18,7	6 537	9130	10235
1197 x 312 x (5W, 16F)	124,1	1 197	312	5	16	1165	15,8	4 140	6918	7592
1432 x 305 x (6W, 16F)	142,6	1 432	305	6	16	1400	18,2	6 265	8749	9850
1440 x 320 x (6W, 20F)	166,4	1 440	320	6	20	1400	21,2	7 825	10868	12028
1193 x 215 x (5W, 14F)	93,0	1 193	215	5	14	1165	11,8	2 751	4612	5245
1193 x 270 x (5W, 14F)	105,1	1 193	270	5	14	1165	13,4	3 286	5509	6153
1197 x 286 x (5W, 16F)	117,6	1 197	286	5	16	1165	15,0	3 850	6433	7101
1193 x 255 x (5W, 14F)	101,8	1 193	255	5	14	1165	13,0	3 140	5264	5906
1197 x 286 x (5W, 16F)	117,6	1 197	286	5	16	1165	15,0	3 850	6433	7101
1432 x 270 x (6W, 16F)	133,8	1 432	270	6	16	1400	17,0	5 703	7965	9057
1201 x 380 x (5W, 18F)	153,1	1 201	380	5	18	1165	19,5	5 445	9068	9788
1197 x 262 x (5W, 16F)	111,5	1 197	262	5	16	1165	14,2	3 582	5986	6647
1197 x 336 x (5W, 16F)	130,1	1 197	336	5	16	1165	16,6	4 408	7365	8046
1432 x 330 x (6W, 16F)	148,8	1 432	330	6	16	1400	19,0	6 666	9309	10416
1436 x 382 x (6W, 18F)	173,9	1 436	382	6	18	1400	22,2	8 285	11539	12690
1193 x 285 x (5W, 14F)	108,4	1 193	285	5	14	1165	13,8	3 432	5754	6401
1197 x 312 x (5W, 16F)	124,1	1 197	312	5	16	1165	15,8	4 140	6918	7592
1432 x 295 x (6W, 16F)	140,0	1 432	295	6	16	1400	17,8	6 104	8525	9624
1440 x 298 x (6W, 20F)	159,5	1 440	298	6	20	1400	20,3	7 381	10252	11403

Computed S_1 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
1432 x 280 x (6W, 16F)	136,3	1 432	280	6	16	1400	17,4	5 864	8189	9284
1444 x 260 x (6W, 22F)	155,7	1 444	260	6	22	1400	19,8	7 156	9911	11074
1436 x 400 x (6W, 18F)	179,0	1 436	400	6	18	1400	22,8	8 611	11993	13150
1450 x 348 x (6W, 25F)	202,5	1 450	348	6	25	1400	25,8	10 206	14077	15338
1440 x 385 x (6W, 20F)	186,8	1 440	385	6	20	1400	23,8	9 136	12688	13874
1450 x 388 x (6W, 25F)	218,2	1 450	388	6	25	1400	27,8	11 222	15478	16763
1910 x 420 x (8W, 20F)	249,3	1 910	420	8	20	1870	31,8	19 363	20275	22870
1197 x 300 x (5W, 16F)	121,1	1 197	300	5	16	1165	15,4	4 006	6694	7365
1432 x 280 x (6W, 16F)	136,3	1 432	280	6	16	1400	17,4	5 864	8189	9284
1440 x 275 x (6W, 20F)	152,3	1 440	275	6	20	1400	19,4	6 917	9608	10750
1440 x 338 x (6W, 20F)	172,1	1 440	338	6	20	1400	21,9	8 188	11372	12539
1432 x 330 x (6W, 16F)	148,8	1 432	330	6	16	1400	19,0	6 666	9309	10416
1440 x 325 x (6W, 20F)	168,0	1 440	325	6	20	1400	21,4	7 926	11008	12170
1444 x 364 x (6W, 22F)	191,7	1 444	364	6	22	1400	24,4	9 469	13115	14327
1456 x 340 x (6W, 28F)	215,4	1 456	340	6	28	1400	27,4	11 080	15219	16535
1440 x 425 x (6W, 20F)	199,4	1 440	425	6	20	1400	25,4	9 942	13809	15010
1450 x 420 x (6W, 25F)	230,8	1 450	420	6	25	1400	29,4	12 034	16598	17903
1930 x 324 x (8W, 30F)	270,0	1 930	324	8	30	1870	34,4	21 906	22700	25462
1436 x 370 x (6W, 18F)	170,5	1 436	370	6	18	1400	21,7	8 068	11237	12384
1444 x 348 x (6W, 22F)	186,1	1 444	348	6	22	1400	23,7	9 113	12622	13827
1444 x 405 x (6W, 22F)	205,8	1 444	405	6	22	1400	26,2	10 381	14378	15610
1450 x 298 x (6W, 25F)	182,9	1 450	298	6	25	1400	23,3	8 937	12327	13556
1444 x 394 x (6W, 22F)	202,0	1 444	394	6	22	1400	25,7	10 136	14039	15266
1444 x 462 x (6W, 22F)	225,5	1 444	462	6	22	1400	28,7	11 649	16134	17393
1920 x 335 x (8W, 25F)	248,9	1 920	335	8	25	1870	31,7	19 398	20206	22864
1464 x 333 x (6W, 32F)	233,2	1 464	333	6	32	1400	29,7	12 300	16803	18199
1920 x 375 x (8W, 25F)	264,6	1 920	375	8	25	1870	33,7	21 193	22076	24759
1920 x 455 x (8W, 25F)	296,0	1 920	455	8	25	1870	37,7	24 785	25817	28549
1926 x 478 x (8W, 28F)	327,6	1 926	478	8	28	1870	41,7	28 469	29562	32397
1456 x 358 x (6W, 28F)	223,3	1 456	358	6	28	1400	28,4	11 594	15925	17254
1914 x 360 x (8W, 22F)	241,8	1 914	360	8	22	1870	30,8	18 536	19368	21978
1926 x 337 x (8W, 28F)	265,6	1 926	337	8	28	1870	33,8	21 357	22177	24903
1934 x 342 x (8W, 32F)	289,3	1 934	342	8	32	1870	36,8	24 157	24981	27809

Computed S_1 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
1926 x 355 x (8W, 28F)	273,5	1 926	355	8	28	1870	34,8	22 265	23120	25860
1930 x 398 x (8W, 30F)	304,9	1 930	398	8	30	1870	38,8	25 913	26853	29680
1926 x 498 x (8W, 28F)	336,4	1 926	498	8	28	1870	42,8	29 477	30610	33460
1934 x 498 x (8W, 32F)	367,6	1 934	498	8	32	1870	46,8	33 187	34320	37304
1946 x 380 x (8W, 38F)	344,1	1 946	380	8	38	1870	43,8	30 647	31498	34545
1940 x 484 x (8W, 35F)	383,4	1 940	484	8	35	1870	48,8	35 101	36186	39265
2393 x 544 x (10W, 28F)	422,6	2 393	544	10	28	2337	53,8	53 236	44493	49678
1926 x 380 x (8W, 28F)	284,5	1 926	380	8	28	1870	36,2	23 526	24430	27189
1926 x 433 x (8W, 28F)	307,8	1 926	433	8	28	1870	39,2	26 199	27205	30005
1926 x 398 x (8W, 28F)	292,4	1 926	398	8	28	1870	37,2	24 434	25372	28145
1930 x 438 x (8W, 30F)	323,7	1 930	438	8	30	1870	41,2	28 079	29098	31960
1934 x 473 x (8W, 32F)	355,1	1 934	473	8	32	1870	45,2	31 740	32823	35782
2393 x 462 x (10W, 28F)	386,5	2 393	462	10	28	2337	49,2	46 815	39127	44248
1926 x 558 x (8W, 28F)	362,7	1 926	558	8	28	1870	46,2	32 503	33752	36648
2393 x 497 x (10W, 28F)	401,9	2 393	497	10	28	2337	51,2	49 556	41417	46565
2393 x 587 x (10W, 28F)	441,5	2 393	587	10	28	2337	56,2	56 604	47308	52525
2397 x 631 x (10W, 30F)	480,7	2 397	631	10	30	2337	61,2	63 669	53124	58461
1926 x 519 x (8W, 28F)	345,6	1 926	519	8	28	1870	44,0	30 536	31710	34576
1926 x 573 x (8W, 28F)	369,3	1 926	573	8	28	1870	47,0	33 260	34538	37445
2387 x 533 x (10W, 25F)	392,7	2 387	533	10	25	2337	50,0	47 808	40057	45128
1926 x 537 x (8W, 28F)	353,5	1 926	537	8	28	1870	45,0	31 444	32652	35532
1930 x 568 x (8W, 30F)	385,0	1 930	568	8	30	1870	49,0	35 119	36393	39370
2397 x 494 x (10W, 30F)	416,1	2 397	494	10	30	2337	53,0	52 155	43517	48733
2397 x 561 x (10W, 30F)	447,7	2 397	561	10	30	2337	57,0	57 786	48215	53491
2397 x 511 x (10W, 30F)	424,1	2 397	511	10	30	2337	54,0	53 583	44709	49940
2397 x 594 x (10W, 30F)	463,2	2 397	594	10	30	2337	59,0	60 559	50529	55834
2407 x 581 x (10W, 35F)	502,7	2 407	581	10	35	2337	64,0	67 847	56375	61889
2407 x 652 x (10W, 35F)	541,7	2 407	652	10	35	2337	69,0	74 838	62184	67783

Appendix C: Computed S_2 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
830 x 184 x (8W, 12F)	85,3	830	184	8	12	806	10,9	1 088	2 621	3 105
1028 x 123 x (10W, 10F)	98,4	1 028	123	10	10	1008	12,5	1 491	2 900	3 792
1030 x 145 x (10W, 14F)	110,5	1 030	145	10	14	1002	14,1	1 886	3 662	4 572
834 x 203 x (8W, 14F)	95,2	834	203	8	14	806	12,1	1 305	3 129	3 630
1030 x 145 x (10W, 14F)	110,5	1 030	145	10	14	1002	14,1	1 886	3 662	4 572
1028 x 218 x (10W, 14F)	126,4	1 028	218	10	14	1000	16,1	2 402	4 674	5 595
1037 x 175 x (10W, 16F)	122,9	1 037	175	10	16	1005	15,7	2 305	4 446	5 384
1233 x 140 x (12W, 14F)	144,3	1 233	140	12	14	1205	18,4	3 206	5 200	6 745
1029 x 135 x (10W, 12F)	104,3	1 029	135	10	12	1005	13,3	1 684	3 273	4 173
1033 x 173 x (10W, 14F)	116,9	1 033	173	10	14	1005	14,9	2 103	4 072	4 993
1033 x 173 x (10W, 14F)	116,9	1 033	173	10	14	1005	14,9	2 103	4 072	4 993
1040 x 213 x (10W, 16F)	132,6	1 040	213	10	16	1008	16,9	2 640	5 078	6 030
1238 x 185 x (12W, 14F)	154,6	1 238	185	12	14	1210	19,7	3 712	5 996	7 562
1036 x 230 x (10W, 14F)	129,7	1 036	230	10	14	1008	16,5	2 535	4 894	5 831
1238 x 165 x (12W, 14F)	150,2	1 238	165	12	14	1210	19,1	3 502	5 658	7 220
1242 x 240 x (12W, 16F)	174,3	1 242	240	12	16	1210	22,2	4 658	7 500	9 100
1028 x 123 x (10W, 10F)	98,4	1 028	123	10	10	1008	12,5	1 491	2 900	3 792
1030 x 145 x (10W, 14F)	110,5	1 030	145	10	14	1002	14,1	1 886	3 662	4 572
1039 x 175 x (10W, 16F)	123,0	1 039	175	10	16	1007	15,7	2 316	4 459	5 400
1031 x 149 x (10W, 12F)	107,1	1 031	149	10	12	1007	13,6	1 779	3 452	4 357
1039 x 175 x (10W, 16F)	123,0	1 039	175	10	16	1007	15,7	2 316	4 459	5 400
1040 x 248 x (10W, 16F)	141,4	1 040	248	10	16	1008	18,0	2 934	5 642	6 603
1238 x 202 x (12W, 14F)	158,4	1 238	202	12	14	1210	20,2	3 890	6 284	7 854
1033 x 173 x (10W, 14F)	116,9	1 033	173	10	14	1005	14,9	2 103	4 072	4 993
1040 x 225 x (10W, 16F)	135,6	1 040	225	10	16	1008	17,3	2 741	5 271	6 227
1242 x 170 x (12W, 16F)	156,7	1 242	170	12	16	1210	20,0	3 816	6 145	7 727
1242 x 269 x (12W, 16F)	181,6	1 242	269	12	16	1210	23,1	5 006	8 062	9 669
1036 x 157 x (10W, 14F)	113,6	1 036	157	10	14	1008	14,5	2 001	3 864	4 787
1036 x 230 x (10W, 14F)	129,7	1 036	230	10	14	1008	16,5	2 535	4 894	5 831
1238 x 154 x (12W, 14F)	147,8	1 238	154	12	14	1210	18,8	3 387	5 471	7 031
1238 x 243 x (12W, 14F)	167,4	1 238	243	12	14	1210	21,3	4 320	6 979	8 556

Computed S_2 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
1233 x 140 x (12W, 14F)	144,3	1 233	140	12	14	1205	18,4	3 206	5 200	6 745
1238 x 225 x (12W, 14F)	163,4	1 238	225	12	14	1210	20,8	4 131	6 674	8 248
1438 x 145 x (14W, 14F)	186,8	1 438	145	14	14	1410	23,8	5 329	7 411	9 849
1438 x 252 x (14W, 14F)	210,3	1 438	252	14	14	1410	26,8	6 848	9 524	11 982
1438 x 180 x (14W, 14F)	194,5	1 438	180	14	14	1410	24,8	5 826	8 102	10 547
1442 x 283 x (14W, 16F)	226,0	1 442	283	14	16	1410	28,8	7 874	10 922	13 415
1644 x 242 x (16W, 16F)	263,3	1 644	242	16	16	1612	33,5	10 716	13 037	16 698
1036 x 215 x (10W, 14F)	126,4	1 036	215	10	14	1008	16,1	2 426	4 683	5 616
1233 x 140 x (12W, 14F)	144,3	1 233	140	12	14	1205	18,4	3 206	5 200	6 745
1242 x 183 x (12W, 16F)	160,0	1 242	183	12	16	1210	20,4	3 972	6 396	7 982
1242 x 262 x (12W, 16F)	179,8	1 242	262	12	16	1210	22,9	4 922	7 926	9 532
1242 x 170 x (12W, 16F)	156,7	1 242	170	12	16	1210	20,0	3 816	6 145	7 727
1242 x 246 x (12W, 16F)	175,8	1 242	246	12	16	1210	22,4	4 730	7 616	9 218
1442 x 177 x (14W, 16F)	199,4	1 442	177	14	16	1410	25,4	6 150	8 530	10 997
1442 x 272 x (14W, 16F)	223,3	1 442	272	14	16	1410	28,4	7 695	10 673	13 164
1442 x 208 x (14W, 16F)	207,2	1 442	208	14	16	1410	26,4	6 654	9 229	11 704
1612 x 160 x (16W, 16F)	238,6	1 612	160	16	16	1580	30,4	8 520	10 570	14 071
1652 x 260 x (16W, 20F)	284,1	1 652	260	16	20	1612	36,2	12 510	15 146	18 881
1242 x 256 x (12W, 16F)	178,3	1 242	256	12	16	1210	22,7	4 850	7 810	9 414
1438 x 177 x (14W, 14F)	193,9	1 438	177	14	14	1410	24,7	5 783	8 043	10 487
1442 x 233 x (14W, 16F)	213,5	1 442	233	14	16	1410	27,2	7 061	9 793	12 274
1438 x 163 x (14W, 14F)	190,8	1 438	163	14	14	1410	24,3	5 584	7 767	10 208
1438 x 250 x (14W, 14F)	209,9	1 438	250	14	14	1410	26,7	6 819	9 484	11 942
1570 x 160 x (16W, 16F)	233,4	1 570	160	16	16	1538	29,7	7 942	10 117	13 440
1644 x 241 x (16W, 16F)	263,0	1 644	241	16	16	1612	33,5	10 695	13 011	16 672
1631 x 160 x (16W, 16F)	241,0	1 631	160	16	16	1599	30,7	8 790	10 778	14 362
1656 x 220 x (16W, 22F)	278,5	1 656	220	16	22	1612	35,5	12 047	14 549	18 303
1657 x 311 x (16W, 22F)	310,0	1 657	311	16	22	1613	39,5	14 741	17 793	21 594
1885 x 236 x (18W, 22F)	341,6	1 885	236	18	22	1841	43,5	18 370	19 491	24 924
1455 x 220 x (14W, 22F)	231,1	1 455	220	14	22	1411	29,4	8 247	11 336	13 904
1645 x 212 x (16W, 16F)	255,8	1 645	212	16	16	1613	32,6	10 096	12 275	15 933
1657 x 223 x (16W, 22F)	279,6	1 657	223	16	22	1613	35,6	12 153	14 669	18 428
1663 x 256 x (16W, 25F)	303,1	1 663	256	16	25	1613	38,6	14 182	17 056	20 890

Computed S_2 sections

Designation	m	h	b	t_w	t_f	h_w	A	About x-x		
								I	Z_e	Z_{pl}
h x b x (t_w, t_f)	kg/m	mm	mm	mm	mm	mm	10^3 mm^2	10^6 mm^4	10^3mm^3	10^3mm^3
1657 x 246 x (16W, 22F)	287,6	1 657	246	16	22	1613	36,6	12 830	15 485	19 256
1657 x 337 x (16W, 22F)	319,0	1 657	337	16	22	1613	40,6	15 506	18 715	22 529
1885 x 261 x (18W, 22F)	350,3	1 885	261	18	22	1841	44,6	19 325	20 503	25 949
1885 x 352 x (18W, 22F)	381,7	1 885	352	18	22	1841	48,6	22 799	24 190	29 679
1885 x 284 x (18W, 22F)	358,2	1 885	284	18	22	1841	45,6	20 203	21 435	26 892
1897 x 312 x (18W, 28F)	397,3	1 897	312	18	28	1841	50,6	24 619	25 955	31 579
1905 x 367 x (18W, 32F)	444,5	1 905	367	18	32	1841	56,6	29 961	31 455	37 248
1657 x 276 x (16W, 22F)	297,9	1 657	276	16	22	1613	38,0	13 712	16 550	20 335
1657 x 345 x (16W, 22F)	321,8	1 657	345	16	22	1613	41,0	15 741	18 999	22 817
1663 x 264 x (16W, 25F)	306,2	1 663	264	16	25	1613	39,0	14 450	17 379	21 218
1663 x 344 x (16W, 25F)	337,6	1 663	344	16	25	1613	43,0	17 134	20 606	24 494
1891 x 277 x (18W, 25F)	368,9	1 891	277	18	25	1841	47,0	21 416	22 651	28 174
1901 x 315 x (18W, 30F)	408,5	1 901	315	18	30	1841	52,0	25 901	27 250	32 933
1885 x 337 x (18W, 22F)	376,5	1 885	337	18	22	1841	48,0	22 226	23 582	29 064
1905 x 326 x (18W, 32F)	423,9	1 905	326	18	32	1841	54,0	27 660	29 039	34 791
2101 x 324 x (20W, 28F)	463,5	2 101	324	20	28	2045	59,0	33 748	32 125	39 716
2109 x 361 x (20W, 32F)	502,4	2 109	361	20	32	2045	64,0	39 173	37 148	44 904
1885 x 288 x (18W, 22F)	359,6	1 885	288	18	22	1841	45,8	20 355	21 597	27 056
1897 x 280 x (18W, 28F)	383,2	1 897	280	18	28	1841	48,8	23 054	24 305	29 905
1901 x 328 x (18W, 30F)	414,6	1 901	328	18	30	1841	52,8	26 584	27 969	33 662
1891 x 273 x (18W, 25F)	367,3	1 891	273	18	25	1841	46,8	21 242	22 467	27 987
1897 x 316 x (18W, 28F)	399,0	1 897	316	18	28	1841	50,8	24 814	26 162	31 789
1905 x 354 x (18W, 32F)	438,0	1 905	354	18	32	1841	55,8	29 231	30 689	36 469
2101 x 338 x (20W, 28F)	469,6	2 101	338	20	28	2045	59,8	34 590	32 927	40 529
1905 x 370 x (18W, 32F)	446,0	1 905	370	18	32	1841	56,8	30 130	31 632	37 428
2109 x 326 x (20W, 32F)	484,8	2 109	326	20	32	2045	61,8	36 757	34 857	42 577
2115 x 370 x (20W, 35F)	524,4	2 115	370	20	35	2045	66,8	42 270	39 971	47 846
2300 x 446 x (22W, 25F)	563,6	2 300	446	22	25	2250	71,8	49 738	43 251	53 210

Appendix D: Computed S_0 section properties and capacities

Designation	Flange		Web		M_r or M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2		kNm/ kgm^{-1}	kN/ kgm^{-1}
704 x 200 x (8W, 12F)	8,3	2	85	2	826	651	0,0	80,4	10,3	8,1
712 x 200 x (8W, 16F)	6,3	1	85	2	1007	651	0,0	92,9	10,8	7,0
720 x 200 x (8W, 20F)	5,0	1	85	2	1157	651	0,0	105,5	11,0	6,2
704 x 250 x (8W, 12F)	10,4	3	85	2	842	651	0,0	89,8	9,4	7,3
720 x 200 x (8W, 20F)	5,0	1	85	2	1157	651	0,0	105,5	11,0	6,2
720 x 250 x (8W, 20F)	6,3	1	85	2	1374	651	0,0	121,2	11,3	5,4
712 x 300 x (8W, 16F)	9,4	3	85	2	1232	651	0,0	118,1	10,4	5,5
720 x 300 x (8W, 20F)	7,5	1	85	2	1591	651	0,0	136,9	11,6	4,8
812 x 200 x (8W, 16F)	6,3	1	98	3	1203	568	0,0	99,2	12,1	5,7
820 x 200 x (8W, 20F)	5,0	1	98	3	1371	568	0,0	111,8	12,3	5,1
812 x 250 x (8W, 16F)	7,8	2	98	3	1406	568	0,0	111,8	12,6	5,1
820 x 250 x (8W, 20F)	6,3	1	98	3	1620	568	0,0	127,5	12,7	4,5
830 x 250 x (8W, 25F)	5,0	1	98	3	1940	568	0,0	147,1	13,2	3,9
812 x 300 x (8W, 16F)	9,4	3	98	3	1446	568	0,0	124,3	11,6	4,6
820 x 300 x (8W, 20F)	7,5	1	98	3	1868	568	0,0	143,2	13,0	4,0
830 x 300 x (8W, 25F)	6,0	1	98	3	2252	568	0,0	166,7	13,5	3,4
904 x 200 x (8W, 12F)	8,3	2	110	4	983	752	315,7	95,4	10,3	7,9
912 x 200 x (8W, 16F)	6,3	1	110	4	1206	752	315,7	108,0	11,2	7,0
920 x 200 x (8W, 20F)	5,0	1	110	4	1391	752	315,7	120,5	11,5	6,2
904 x 250 x (8W, 12F)	10,4	3	110	4	1152	752	315,7	104,8	11,0	7,2
912 x 250 x (8W, 16F)	7,8	2	110	4	1432	752	315,7	120,5	11,9	6,2
920 x 250 x (8W, 20F)	6,3	1	110	4	1665	752	315,7	136,2	12,2	5,5
930 x 250 x (8W, 25F)	5,0	1	110	4	2004	752	315,7	155,9	12,9	4,8
904 x 300 x (8W, 12F)*	12,5	4	110	4	1322	752	315,7	114,3	11,6	6,6
912 x 300 x (8W, 16F)	9,4	3	110	4	1657	752	315,7	133,1	12,5	5,6
920 x 300 x (8W, 20F)	7,5	1	110	4	1938	752	315,7	151,9	12,8	4,9
930 x 300 x (8W, 25F)	6,0	1	110	4	2346	752	315,7	175,5	13,4	4,3
1 004 x 250 x (8W, 12F)	10,4	3	123	4	1301	752	400,3	111,8	11,6	6,7
1 012 x 250 x (8W, 16F)	7,8	2	123	4	1614	752	400,3	127,5	12,7	5,9
1 020 x 250 x (8W, 20F)	6,3	1	123	4	1874	752	400,3	143,2	13,1	5,2
1 030 x 250 x (8W, 25F)	5,0	1	123	4	2253	752	400,3	162,8	13,8	4,6

Computed S_0 section properties and capacities

Designation	Flange		Web		M_r or M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm²		kNm/kgm⁻¹	kN/kgm⁻¹
1 012 x 300 x (8W, 16F)	9,4	3	123	4	1866	752	400,3	140,0	13,3	5,4
1 020 x 300 x (8W, 20F)	7,5	1	123	4	2180	752	400,3	158,9	13,7	4,7
1 030 x 300 x (8W, 25F)	6,0	1	123	4	2634	752	400,3	182,4	14,4	4,1
1 040 x 300 x (8W, 30F)	5,0	1	123	4	3089	752	400,3	206,0	15,0	3,6
1 020 x 400 x (8W, 20F)	10,0	3	123	4	2790	752	400,3	190,3	14,7	3,9
1 030 x 400 x (8W, 25F)	8,0	2	123	4	3397	752	400,3	221,7	15,3	3,4
1 040 x 400 x (8W, 30F)	6,7	1	123	4	4004	752	400,3	253,1	15,8	3,0
1 204 x 250 x (8W, 12F)	10,4	3	148	4	1587	772	557,9	125,6	12,6	6,1
1 212 x 250 x (8W, 16F)	7,8	2	148	4	1971	772	557,9	141,3	14,0	5,5
1 220 x 250 x (8W, 20F)	6,3	1	148	4	2288	772	557,9	157,0	14,6	4,9
1 230 x 250 x (8W, 25F)	5,0	1	148	4	2747	772	557,9	176,6	15,6	4,4
1 212 x 300 x (8W, 16F)	9,4	3	148	4	2278	772	557,9	153,8	14,8	5,0
1 220 x 300 x (8W, 20F)	7,5	1	148	4	2659	772	557,9	172,7	15,4	4,5
1 230 x 300 x (8W, 25F)	6,0	1	148	4	3209	772	557,9	196,2	16,4	3,9
1 240 x 300 x (8W, 30F)	5,0	1	148	4	3758	772	557,9	219,8	17,1	3,5
1 220 x 400 x (8W, 20F)	10,0	3	148	4	3397	772	557,9	204,1	16,6	3,8
1 230 x 400 x (8W, 25F)	8,0	2	148	4	4129	772	557,9	235,5	17,5	3,3
1 240 x 400 x (8W, 30F)	6,7	1	148	4	4861	772	557,9	266,9	18,2	2,9
1 402 x 250 x (10W, 16F)	7,8	2	137	4	2559	1189	770,8	176,4	14,5	6,7
1 410 x 250 x (10W, 20F)	6,3	1	137	4	2925	1189	770,8	192,1	15,2	6,2
1 420 x 250 x (10W, 25F)	5,0	1	137	4	3460	1189	770,8	211,7	16,3	5,6
1 402 x 300 x (10W, 16F)	9,4	3	137	4	2919	1189	770,8	189,0	15,4	6,3
1 410 x 300 x (10W, 20F)	7,5	1	137	4	3358	1189	770,8	207,8	16,2	5,7
1 420 x 300 x (10W, 25F)	6,0	1	137	4	3998	1189	770,8	231,3	17,3	5,1
1 430 x 300 x (10W, 30F)	5,0	1	137	4	4636	1189	770,8	254,9	18,2	4,7
1 410 x 400 x (10W, 20F)	10,0	3	137	4	4219	1189	770,8	239,2	17,6	5,0
1 420 x 400 x (10W, 25F)	8,0	2	137	4	5069	1189	770,8	270,6	18,7	4,4
1 430 x 400 x (10W, 30F)	6,7	1	137	4	5919	1189	770,8	302,0	19,6	3,9
1 440 x 400 x (10W, 35F)	5,7	1	137	4	6767	1189	770,8	333,4	20,3	3,6
1 602 x 300 x (12W, 16F)	9,4	3	131	4	3719	1701	1022,3	231,3	16,1	7,4
1 610 x 300 x (12W, 20F)	7,5	1	131	4	4220	1701	1022,3	250,1	16,9	6,8
1 620 x 300 x (12W, 25F)	6,0	1	131	4	4957	1701	1022,3	273,7	18,1	6,2
1 630 x 300 x (12W, 30F)	5,0	1	131	4	5689	1701	1022,3	297,2	19,1	5,7

Computed S_0 section properties and capacities

Designation	Flange		Web		M_r or M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm²		kNm/ kgm^{-1}	kN/ kgm^{-1}
1 610 x 400 x (12W, 20F)	10,0	3	131	4	5212	1701	1022,3	281,5	18,5	6,0
1 620 x 400 x (12W, 25F)	8,0	2	131	4	6189	1701	1022,3	312,9	19,8	5,4
1 630 x 400 x (12W, 30F)	6,7	1	131	4	7163	1701	1022,3	344,3	20,8	4,9
1 640 x 400 x (12W, 35F)	5,7	1	131	4	8135	1701	1022,3	375,7	21,7	4,5
1 620 x 500 x (12W, 25F)	10,0	3	131	4	7416	1701	1022,3	352,2	21,1	4,8
1 630 x 500 x (12W, 30F)	8,3	2	131	4	8633	1701	1022,3	391,4	22,1	4,3
1 640 x 500 x (12W, 35F)	7,1	1	131	4	9848	1701	1022,3	430,7	22,9	3,9
1 820 x 300 x (12W, 25F)	6,0	1	148	4	5630	1737	1255,4	294,3	19,1	5,9
1 830 x 300 x (12W, 30F)	5,0	1	148	4	6466	1737	1255,4	317,9	20,3	5,5
1 810 x 400 x (12W, 20F)	10,0	3	148	4	5922	1737	1255,4	302,2	19,6	5,7
1 820 x 400 x (12W, 25F)	8,0	2	148	4	7035	1737	1255,4	333,6	21,1	5,2
1 830 x 400 x (12W, 30F)	6,7	1	148	4	8140	1737	1255,4	365,0	22,3	4,8
1 840 x 400 x (12W, 35F)	5,7	1	148	4	9241	1737	1255,4	396,4	23,3	4,4
1 820 x 500 x (12W, 25F)	10,0	3	148	4	8427	1737	1255,4	372,8	22,6	4,7
1 830 x 500 x (12W, 30F)	8,3	2	148	4	9804	1737	1255,4	412,1	23,8	4,2
1 840 x 500 x (12W, 35F)	7,1	1	148	4	11179	1737	1255,4	451,3	24,8	3,8
1 850 x 500 x (12W, 40F)	6,3	1	148	4	12552	1737	1255,4	490,6	25,6	3,5
2 120 x 300 x (14W, 25F)	6,0	1	148	4	7161	2366	1715,3	358,7	20,0	6,6
2 130 x 300 x (14W, 30F)	5,0	1	148	4	8157	2366	1715,3	382,3	21,3	6,2
2140 x 300x(14W, 35F)**	4,3	1	148	4	9141	2366	1715,3	405,8	22,5	5,8
2 110 x 400 x (14W, 20F)	10,0	3	148	4	7512	2366	1715,3	366,6	20,5	6,5
2 120 x 400 x (14W, 25F)	8,0	2	148	4	8835	2366	1715,3	398,0	22,2	5,9
2 130 x 400 x (14W, 30F)	6,7	1	148	4	10141	2366	1715,3	429,4	23,6	5,5
2 140 x 400 x (14W, 35F)	5,7	1	148	4	11438	2366	1715,3	460,8	24,8	5,1
2 120 x 500 x (14W, 25F)	10,0	3	148	4	10482	2366	1715,3	437,2	24,0	5,4
2 130 x 500 x (14W, 30F)	8,3	2	148	4	12103	2366	1715,3	476,5	25,4	5,0
2 140 x 500 x (14W, 35F)	7,1	1	148	4	13717	2366	1715,3	515,7	26,6	4,6
2 150 x 500 x (14W, 40F)	6,3	1	148	4	15327	2366	1715,3	555,0	27,6	4,3

*Section has a class 4 flange.
**Section has $\beta_f < 5$.

These two sections fall outside the recommendations in this thesis and where not included in the comparison of the results.

Appendix E: Computed S_1 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	V_r per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2	kg/m	kNm/kgm^{-1}	kN/kgm^{-1}
1189 x 181 x (5W, 12F)	7,5	1	233	4	957	646	697,5	85,3	11,21	7,58
1193 x 215 x (5W, 14F)	7,7	1	233	4	1285	646	697,5	98,5	13,05	6,56
1193 x 270 x (5W, 14F)	9,6	3	233	4	1581	646	697,5	110,5	14,30	5,85
1189 x 234 x (5W, 12F)	9,8	3	233	4	1208	646	697,5	95,3	12,67	6,78
1193 x 270 x (5W, 14F)	9,6	3	233	4	1581	646	697,5	110,5	14,30	5,85
1 197 x 300 x (5W, 16F)	9,4	3	233	4	1967	646	697,5	126,6	15,54	5,11
1 197 x 286 x (5W, 16F)	8,9	2	233	4	1882	646	697,5	123,0	15,30	5,25
1 432 x 280 x (6W, 16F)	8,8	2	233	4	2291	933	1007,1	144,2	15,89	6,47
1 197 x 212 x (5W, 14F)	6,6	1	233	4	1431	646	697,5	104,5	13,70	6,19
1 197 x 262 x (5W, 16F)	8,2	2	233	4	1737	646	697,5	117,0	14,84	5,52
1 197 x 262 x (5W, 16F)	8,2	2	233	4	1737	646	697,5	117,0	14,84	5,52
1 197 x 325 x (5W, 16F)	10,2	3	233	4	2118	646	697,5	132,8	15,95	4,86
1 432 x 322 x (6W, 16F)	10,1	3	233	4	2602	933	1007,1	154,7	16,82	6,03
1 197 x 312 x (5W, 16F)	9,8	3	233	4	2040	646	697,5	129,6	15,74	4,99
1 432 x 305 x (6W, 16F)	9,5	3	233	4	2477	933	1007,1	150,5	16,46	6,20
1 440 x 320 x (6W, 20F)	8,0	2	233	4	3084	933	1007,1	174,3	17,69	5,35
1 193 x 215 x (5W, 14F)	7,7	1	233	4	1285	646	697,5	98,5	13,05	6,56
1193 x 270 x (5W, 14F)	9,6	3	233	4	1581	646	697,5	110,5	14,30	5,85
1197 x 286 x (5W, 16F)	8,9	2	233	4	1882	646	697,5	123,0	15,30	5,25
1193 x 255 x (5W, 14F)	9,1	3	233	4	1501	646	697,5	107,3	13,99	6,03
1197 x 286 x (5W, 16F)	8,9	2	233	4	1882	646	697,5	123,0	15,30	5,25
1432 x 270 x (6W, 16F)	8,4	2	233	4	2217	933	1007,1	141,7	15,65	6,59
1201 x 380 x (5W, 18F)	10,6	3	233	4	2659	646	697,5	158,6	16,77	4,07
1197 x 262 x (5W, 16F)	8,2	2	233	4	1737	646	697,5	117,0	14,84	5,52
1197 x 336 x (5W, 16F)	10,5	3	233	4	2185	646	697,5	135,6	16,11	4,77
1432 x 330 x (6W, 16F)	10,3	3	233	4	2661	933	1007,1	156,7	16,98	5,95
1436 x 382 x (6W, 18F)	10,6	3	233	4	3296	933	1007,1	181,8	18,13	5,13
1193 x 285 x (5W, 14F)	10,2	3	233	4	1661	646	697,5	113,8	14,59	5,68
1197 x 312 x (5W, 16F)	9,8	3	233	4	2040	646	697,5	129,6	15,74	4,99
1432 x 295 x (6W, 16F)	9,2	3	233	4	2403	933	1007,1	147,9	16,24	6,31
1440 x 298 x (6W, 20F)	7,5	1	233	4	2889	933	1007,1	167,4	17,26	5,57

Computed S_1 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	V_r per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2	kg/m	kNm/ kgm^{-1}	kN/ kgm^{-1}
1432 x 280 x (6W, 16F)	8,8	2	233	4	2291	933	1007,1	144,2	15,89	6,47
1444 x 260 x (6W, 22F)	5,9	1	233	4	2781	933	1007,1	163,6	16,99	5,70
1436 x 400 x (6W, 18F)	11,1	4	233	4	3439	933	1007,1	186,9	18,40	4,99
1450 x 348 x (6W, 25F)	7,0	1	233	4	4095	933	1007,1	210,4	19,46	4,43
1440 x 385 x (6W, 20F)	9,6	3	233	4	3658	933	1007,1	194,7	18,79	4,79
1450 x 388 x (6W, 25F)	7,8	1	233	4	4533	933	1007,1	226,1	20,05	4,13
1910 x 420 x (8W, 20F)	10,5	3	234	4	5559	1664	1796,3	263,4	21,10	6,32
1197 x 300 x (5W, 16F)	9,4	3	233	4	1967	646	697,5	126,6	15,54	5,11
1432 x 280 x (6W, 16F)	8,8	2	233	4	2291	933	1007,1	144,2	15,89	6,47
1440 x 275 x (6W, 20F)	6,9	1	233	4	2685	933	1007,1	160,2	16,76	5,82
1440 x 338 x (6W, 20F)	8,5	2	233	4	3244	933	1007,1	180,0	18,02	5,18
1432 x 330 x (6W, 16F)	10,3	3	233	4	2661	933	1007,1	156,7	16,98	5,95
1440 x 325 x (6W, 20F)	8,1	2	233	4	3129	933	1007,1	175,9	17,79	5,30
1444 x 364 x (6W, 22F)	8,3	2	233	4	3792	933	1007,1	199,6	19,00	4,68
1456 x 340 x (6W, 28F)	6,1	1	233	4	4452	933	1007,1	223,3	19,94	4,18
1440 x 425 x (6W, 20F)	10,6	3	233	4	4010	933	1007,1	207,3	19,34	4,50
1450 x 420 x (6W, 25F)	8,4	2	233	4	4884	933	1007,1	238,7	20,46	3,91
1930 x 324 x (8W, 30F)	5,4	1	234	4	6335	1664	1796,3	284,1	22,30	5,86
1436 x 370 x (6W, 18F)	10,3	3	233	4	3201	933	1007,1	178,4	17,94	5,23
1444 x 348 x (6W, 22F)	7,9	2	233	4	3637	933	1007,1	194,0	18,75	4,81
1444 x 405 x (6W, 22F)	9,2	3	233	4	4189	933	1007,1	213,7	19,60	4,37
1450 x 298 x (6W, 25F)	6,0	1	233	4	3545	933	1007,1	190,8	18,58	4,89
1444 x 394 x (6W, 22F)	9,0	2	233	4	4082	933	1007,1	209,9	19,45	4,44
1444 x 462 x (6W, 22F)	10,5	3	233	4	4738	933	1007,1	233,4	20,30	4,00
1920 x 335 x (8W, 25F)	6,7	1	234	4	5537	1664	1796,3	263,0	21,05	6,33
1464 x 333 x (6W, 32F)	5,2	1	233	4	4948	933	1007,1	241,1	20,52	3,87
1920 x 375 x (8W, 25F)	7,5	1	234	4	6136	1664	1796,3	278,7	22,01	5,97
1920 x 455 x (8W, 25F)	9,1	2	234	4	7323	1664	1796,3	310,1	23,61	5,37
1926 x 478 x (8W, 28F)	8,5	2	234	4	8505	1664	1796,3	341,7	24,89	4,87
1456 x 358 x (6W, 28F)	6,4	1	233	4	4673	933	1007,1	231,2	20,21	4,04
1914 x 360 x (8W, 22F)	8,2	2	234	4	5267	1664	1796,3	255,9	20,58	6,50
1926 x 337 x (8W, 28F)	6,0	1	234	4	6169	1664	1796,3	279,7	22,06	5,95
1934 x 342 x (8W, 32F)	5,3	1	234	4	7060	1664	1796,3	303,4	23,27	5,49

Computed S_1 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	V_r per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2	kg/m	kNm/ kgm^{-1}	kN/ kgm^{-1}
1926 x 355 x (8W, 28F)	6,3	1	234	4	6469	1664	1796,3	287,6	22,49	5,79
1930 x 398 x (8W, 30F)	6,6	1	234	4	7651	1664	1796,3	319,0	23,99	5,22
1926 x 498 x (8W, 28F)	8,9	2	234	4	8834	1664	1796,3	350,5	25,21	4,75
1934 x 498 x (8W, 32F)	7,8	1	234	4	9999	1664	1796,3	381,7	26,19	4,36
1946 x 380 x (8W, 38F)	5,0	1	234	4	9114	1664	1796,3	358,2	25,44	4,65
1940 x 484 x (8W, 35F)	6,9	1	234	4	10584	1664	1796,3	397,5	26,63	4,19
2393 x 544 x (10W, 28F)	9,7	3	234	4	12423	2599	2805,7	444,6	27,94	5,85
1926 x 380 x (8W, 28F)	6,8	1	234	4	6884	1664	1796,3	298,6	23,06	5,57
1926 x 433 x (8W, 28F)	7,7	1	234	4	7762	1664	1796,3	321,9	24,11	5,17
1926 x 398 x (8W, 28F)	7,1	1	234	4	7183	1664	1796,3	306,5	23,44	5,43
1930 x 438 x (8W, 30F)	7,3	1	234	4	8359	1664	1796,3	337,8	24,74	4,93
1934 x 473 x (8W, 32F)	7,4	1	234	4	9530	1664	1796,3	369,2	25,81	4,51
2393 x 462 x (10W, 28F)	8,3	2	234	4	10707	2599	2805,7	408,6	26,21	6,36
1926 x 558 x (8W, 28F)	10,0	3	234	4	9821	1664	1796,3	376,8	26,06	4,42
2393 x 497 x (10W, 28F)	8,9	2	234	4	11441	2599	2805,7	424,0	26,99	6,13
2393 x 587 x (10W, 28F)	10,5	3	234	4	13317	2599	2805,7	463,5	28,73	5,61
2397 x 631 x (10W, 30F)	10,5	3	234	4	15157	2599	2805,7	502,7	30,15	5,17
1926 x 519 x (8W, 28F)	9,3	3	234	4	9180	1664	1796,3	359,7	25,52	4,63
1926 x 573 x (8W, 28F)	10,2	3	234	4	10067	1664	1796,3	383,4	26,25	4,34
2387 x 533 x (10W, 25F)	10,7	3	234	4	11004	2599	2805,7	414,7	26,54	6,27
1926 x 537 x (8W, 28F)	9,6	3	234	4	9476	1664	1796,3	367,6	25,78	4,53
1930 x 568 x (8W, 30F)	9,5	3	234	4	10648	1664	1796,3	399,1	26,68	4,17
2397 x 494 x (10W, 30F)	8,2	2	234	4	12112	2599	2805,7	438,2	27,64	5,93
2397 x 561 x (10W, 30F)	9,4	3	234	4	13605	2599	2805,7	469,7	28,96	5,53
2397 x 511 x (10W, 30F)	8,5	2	234	4	12492	2599	2805,7	446,2	28,00	5,83
2397 x 594 x (10W, 30F)	9,9	3	234	4	14337	2599	2805,7	485,3	29,55	5,36
2407 x 581 x (10W, 35F)	8,3	2	234	4	16183	2599	2805,7	524,7	30,84	4,95
2407 x 652 x (10W, 35F)	9,3	3	234	4	18009	2599	2805,7	563,8	31,94	4,61

Appendix F: Computed S_2 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2		kNm/kgm^{-1}	kN/kgm^{-1}
830 x 184 x (8W, 12F)	8	1	101	3	992	550	0	85,3	11,63	6,44
1028 x 123 x (10W, 10F)	6	1	101	3	1212	858	0	98,4	12,31	8,72
1030 x 145 x (10W, 14F)	5	1	100	3	1461	863	0	110,5	13,22	7,81
834 x 203 x (8W, 14F)	7	1	101	3	1160	550	0	95,2	12,18	5,77
1030 x 145 x (10W, 14F)	5	1	100	3	1461	863	0	110,5	13,22	7,81
1 028x 218 x (10W, 14F)	8	2	100	3	1788	865	0	126,4	14,14	6,84
1 037 x 175 x (10W, 16F)	5	1	101	3	1720	861	0	122,9	14,00	7,01
1 223 x 140 x (12W, 14F)	5	1	100	3	2155	1241	0	144,3	14,94	8,60
1 029 x 135 x (10W, 12F)	6	1	101	3	1333	861	0	104,3	12,78	8,25
1 033 x 173 x (10W, 14F)	6	1	101	3	1595	861	0	116,9	13,64	7,36
1 033 x 173 x (10W, 14F)	6	1	101	3	1595	861	0	116,9	13,64	7,36
1 008 x 213 x (10W, 16F)	7	1	101	3	1927	858	0	132,6	14,53	6,47
1 238 x 185 x (12W, 14F)	7	1	101	3	2416	1235	0	154,6	15,62	7,99
1 036 x 230 x (10W, 14F)	8	2	101	3	1863	858	0	129,7	14,37	6,62
1 238 x 165 x (12W, 14F)	6	1	101	3	2307	1235	0	150,2	15,35	8,22
1242 x 240 x (12W, 16F)	8	1	101	3	2907	1235	0	174,3	16,68	7,09
1 028 x 123 x (10W, 10F)	6	1	101	3	1212	858	0	98,4	12,31	8,72
1030 x 145 x (10W, 14F)	5	1	100	3	1461	863	0	110,5	13,22	7,81
1039 x 175 x (10W, 16F)	5	1	101	3	1725	859	0	123,0	14,02	6,98
1031 x 149 x (10W, 12F)	6	1	101	3	1392	859	0	107,1	13,00	8,02
1039 x 175 x (10W, 16F)	5	1	101	3	1725	859	0	123,0	14,02	6,98
1040 x 248 x (10W, 16F)	8	2	101	3	2110	858	0	141,4	14,92	6,07
1238 x 202 x (12W, 14F)	7	1	101	3	2509	1235	0	158,4	15,84	7,80
1033 x 173 x (10W, 14F)	6	1	101	3	1595	861	0	116,9	13,64	7,36
1040 x 225 x (10W, 16F)	7	1	101	3	1989	858	0	135,6	14,67	6,33
1242 x 170 x (12W, 16F)	5	1	101	3	2469	1235	0	156,7	15,76	7,88
1242 x 269 x (12W, 16F)	8	2	101	3	3089	1235	0	181,6	17,02	6,80
1036 x 157 x (10W, 14F)	6	1	101	3	1529	858	0	113,6	13,46	7,55
1036 x 230 x (10W, 14F)	8	2	101	3	1863	858	0	129,7	14,37	6,62
1238 x 154 x (12W, 14F)	6	1	101	3	2246	1235	0	147,8	15,20	8,36
1238 x 243 x (12W, 14F)	9	2	101	3	2734	1235	0	167,4	16,33	7,38

Computed S_2 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2		kNm/kgm^{-1}	kN/kgm^{-1}
1233 x 140 x (12W, 14F)	5	1	100	3	2155	1241	0	144,3	14,94	8,60
1238 x 225 x (12W, 14F)	8	2	101	3	2635	1235	0	163,4	16,12	7,56
1438 x 145 x (14W, 14F)	5	1	101	3	3147	1684	0	186,8	16,84	9,01
1438 x 252 x (14W, 14F)	9	2	101	3	3828	1684	0	210,3	18,20	8,00
1438 x 180 x (14W, 14F)	6	1	101	3	3370	1684	0	194,5	17,32	8,65
1442 x 283 x (14W, 16F)	9	2	101	3	4286	1684	0	226,0	18,96	7,45
1644 x 242 x (16W, 16F)	8	1	101	3	5335	2198	0	263,3	20,27	8,35
1036 x 215 x (10W, 14F)	8	1	101	3	1794	858	0	126,4	14,20	6,79
1233 x 140 x (12W, 14F)	5	1	100	3	2155	1241	0	144,3	14,94	8,60
1242 x 183 x (12W, 16F)	6	1	101	3	2550	1235	0	160,0	15,94	7,72
1242 x 262 x (12W, 16F)	8	2	101	3	3045	1235	0	179,8	16,94	6,87
1242 x 170 x (12W, 16F)	5	1	101	3	2469	1235	0	156,7	15,76	7,88
1242 x 246 x (12W, 16F)	8	1	101	3	2945	1235	0	175,8	16,75	7,03
1442 x 177 x (14W, 16F)	6	1	101	3	3513	1684	0	199,4	17,62	8,44
1442 x 272 x (14W, 16F)	9	2	101	3	4206	1684	0	223,3	18,84	7,54
1442 x 208 x (14W, 16F)	7	1	101	3	3739	1684	0	207,2	18,05	8,12
1612 x 160 x (16W, 16F)	5	1	99	3	4496	2243	0	238,6	18,84	9,40
1652 x 260 x (16W, 20F)	7	1	101	3	5862	2198	0	284,1	20,63	7,74
1242 x 256 x (12W, 16F)	8	2	101	3	3008	1235	0	178,3	16,87	6,93
1438 x 177 x (14W, 14F)	6	1	101	3	3351	1684	0	193,9	17,28	8,68
1442 x 233 x (14W, 16F)	7	1	101	3	3922	1684	0	213,5	18,37	7,89
1438 x 163 x (14W, 14F)	6	1	101	3	3261	1684	0	190,8	17,09	8,82
1438 x 250 x (14W, 14F)	9	2	101	3	3816	1684	0	209,9	18,18	8,02
1570 x 160 x (16W, 16F)	5	1	96	3	4294	2304	0	233,4	18,40	9,87
1644 x 241 x (16W, 16F)	8	1	101	3	5327	2198	0	263,0	20,25	8,36
1631 x 160 x (16W, 16F)	5	1	100	3	4589	2216	0	241,0	19,04	9,19
1656 x 220 x (16W, 22F)	5	1	101	3	5683	2198	0	278,5	20,41	7,89
1657 x 311 x (16W, 22F)	7	1	101	3	6705	2197	0	310,0	21,63	7,09
1885 x 236 x (18W, 22F)	5	1	102	3	7739	2740	0	341,6	22,65	8,02
1455 x 220 x (14W, 22F)	5	1	101	3	4317	1682	0	231,1	18,68	7,28
1645 x 212 x (16W, 16F)	7	1	101	3	5090	2197	0	255,8	19,90	8,59
1657 x 223 x (16W, 22F)	5	1	101	3	5722	2197	0	279,6	20,46	7,86
1663 x 256 x (16W, 25F)	5	1	101	3	6486	2197	0	303,1	21,40	7,25

Computed S_2 section properties and capacities

Designation	Flange		Web		M_r / M'_r	V_r	A_s (stiffener)	m_{tot}	Mr or M'r per kg/m	Vr per kg/m
h x b x (t_w, t_f)	β_f	Class	β_w	Class	kNm	kN	mm^2		kNm/kgm^{-1}	kN/kgm^{-1}
1657 x 246 x (16W, 22F)	6	1	101	3	5979	2197	0	287,6	20,79	7,64
1657 x 337 x (16W, 22F)	8	1	101	3	6995	2197	0	319,0	21,93	6,89
1885 x 261 x (18W, 22F)	6	1	102	3	8057	2740	0	350,3	23,00	7,82
1885 x 352 x (18W, 22F)	8	2	102	3	9215	2740	0	381,7	24,14	7,18
1885 x 284 x (18W, 22F)	6	1	102	3	8350	2740	0	358,2	23,31	7,65
1897 x 312 x (18W, 28F)	6	1	102	3	9805	2740	0	397,3	24,68	6,90
1905 x 367 x (18W, 32F)	6	1	102	3	11566	2740	0	444,5	26,02	6,17
1657 x 276 x (16W, 22F)	6	1	101	3	6314	2197	0	297,9	21,19	7,37
1657 x 345 x (16W, 22F)	8	2	101	3	7085	2197	0	321,8	22,02	6,83
1663 x 264 x (16W, 25F)	5	1	101	3	6588	2197	0	306,2	21,51	7,17
1663 x 344 x (16W, 25F)	7	1	101	3	7605	2197	0	337,6	22,53	6,51
1891 x 277 x (18W, 25F)	6	1	102	3	8748	2740	0	368,9	23,72	7,43
1901 x 315 x (18W, 30F)	5	1	102	3	10226	2740	0	408,5	25,03	6,71
1885 x 337 x (18W, 22F)	8	1	102	3	9024	2740	0	376,5	23,97	7,28
1905 x 326 x (18W, 32F)	5	1	102	3	10803	2740	0	423,9	25,48	6,46
2101 x 324 x (20W, 28F)	6	1	102	3	12332	3384	0	463,5	26,61	7,30
2109 x 361 x (20W, 32F)	6	1	102	3	13943	3384	0	502,4	27,75	6,74
1885 x 288 x (18W, 22F)	7	1	102	3	8401	2740	0	359,6	23,36	7,62
1897 x 280 x (18W, 28F)	5	1	102	3	9285	2740	0	383,2	24,23	7,15
1901 x 328 x (18W, 30F)	5	1	102	3	10452	2740	0	414,6	25,21	6,61
1891 x 273 x (18W, 25F)	5	1	102	3	8690	2740	0	367,3	23,66	7,46
1897 x 316 x (18W, 28F)	6	1	102	3	9870	2740	0	399,0	24,73	6,87
1905 x 354 x (18W, 32F)	6	1	102	3	11324	2740	0	438,0	25,85	6,26
2101 x 338 x (20W, 28F)	6	1	102	3	12584	3384	0	469,6	26,79	7,21
1905 x 370 x (18W, 32F)	6	1	102	3	11621	2740	0	446,0	26,06	6,14
2109 x 326 x (20W, 32F)	5	1	102	3	13220	3384	0	484,8	27,27	6,98
2115 x 370 x (20W, 35F)	5	1	102	3	14856	3384	0	524,4	28,33	6,45
2300 x 446 x (22W, 25F)	9	2	102	3	16522	4094	0	563,6	29,31	7,26

Appendix G: Moment, shear capacities and performance to weight ratio comparisons

S_1	S_2						
Designation	**Designation**	**M_r / M'_r**	**V_r**	**Mr or M'r efficiency**		**V_r efficiency**	
h x b x (t_w, t_f)	**h x b x (t_w, t_f)**	**(S_1-S_2) kNm**	**(S_1-S_2) kN**	**S_2/S_1**	**Δ (1– S_2/S_1)**	**S_2/S_1**	**Δ (1– S_2/S_1)**
1189 x 181 x (5W, 12F)	830 x 184 x (8W, 12F)	-35,6	96,7	103,7%	-3,7%	85,1%	14,9%
1193 x 215 x (5W, 14F)	1028 x 123 x (10W, 10F)	73,4	-212,0	94,3%	5,7%	132,8%	-32,8%
1193 x 270 x (5W, 14F)	1030 x 145 x (10W, 14F)	120,0	-217,1	92,4%	7,6%	133,6%	-33,6%
1189 x 234 x (5W, 12F)	834 x 203 x (8W, 14F)	47,9	96,7	96,1%	3,9%	85,1%	14,9%
1193 x 270 x (5W, 14F)	1030 x 145 x (10W, 14F)	120,0	-217,1	92,4%	7,6%	133,6%	-33,6%
1 197 x 300 x (5W, 16F)	1 028x 218 x (10W, 14F)	179,8	-218,9	91,0%	9,0%	134,0%	-34,0%
1 197 x 286 x (5W, 16F)	1 037 x 175 x (10W, 16F)	162,4	-214,6	91,5%	8,5%	133,4%	-33,4%
1 432 x 280 x (6W, 16F)	1 223 x 140 x (12W, 14F)	136,3	-307,5	94,0%	6,0%	132,9%	-32,9%
1 197 x 212 x (5W, 14F)	1 029 x 135 x (10W, 12F)	98,2	-214,6	93,3%	6,7%	133,4%	-33,4%
1 197 x 262 x (5W, 16F)	1 033 x 173 x (10W, 14F)	141,5	-214,6	91,9%	8,1%	133,3%	-33,3%
1 197 x 262 x (5W, 16F)	1 033 x 173 x (10W, 14F)	141,5	-214,6	91,9%	8,1%	133,3%	-33,3%
1 197 x 325 x (5W, 16F)	1 008 x 213 x (10W, 16F)	191,8	-212,0	91,1%	8,9%	133,0%	-33,0%
1 432 x 322 x (6W, 16F)	1 238 x 185 x (12W, 14F)	185,8	-302,4	92,9%	7,1%	132,5%	-32,5%
1 197 x 312 x (5W, 16F)	1 036 x 230 x (10W, 14F)	176,8	-212,0	91,3%	8,7%	132,7%	-32,7%
1 432 x 305 x (6W, 16F)	1 238 x 165 x (12W, 14F)	170,0	-302,4	93,3%	6,7%	132,6%	-32,6%
1 440 x 320 x (6W, 20F)	1242 x 240 x (12W, 16F)	176,8	-302,4	94,3%	5,7%	132,4%	-32,4%
1 193 x 215 x (5W, 14F)	1 028 x 123 x (10W, 10F)	73,4	-212,0	94,3%	5,7%	132,8%	-32,8%
1193 x 270 x (5W, 14F)	1030 x 145 x (10W, 14F)	120,0	-217,1	92,4%	7,6%	133,6%	-33,6%
1197 x 286 x (5W, 16F)	1039 x 175 x (10W, 16F)	157,3	-212,9	91,7%	8,3%	133,0%	-33,0%
1193 x 255 x (5W, 14F)	1031 x 149 x (10W, 12F)	108,5	-212,9	92,9%	7,1%	133,1%	-33,1%
1197 x 286 x (5W, 16F)	1039 x 175 x (10W, 16F)	157,3	-212,9	91,7%	8,3%	133,0%	-33,0%
1432 x 270 x (6W, 16F)	1040 x 248 x (10W, 16F)	107,3	74,8	95,3%	4,7%	92,1%	7,9%
1201 x 380 x (5W, 18F)	1238 x 202 x (12W, 14F)	149,7	-589,2	94,5%	5,5%	191,4%	-91,4%
1197 x 262 x (5W, 16F)	1033 x 173 x (10W, 14F)	141,5	-214,6	91,9%	8,1%	133,3%	-33,3%
1197 x 336 x (5W, 16F)	1040 x 225 x (10W, 16F)	195,4	-212,0	91,0%	9,0%	132,8%	-32,8%
1432 x 330 x (6W, 16F)	1242 x 170 x (12W, 16F)	192,1	-302,4	92,8%	7,2%	132,5%	-32,5%
1436 x 382 x (6W, 18F)	1242 x 269 x (12W, 16F)	206,9	-302,4	93,8%	6,2%	132,6%	-32,6%
1193 x 285 x (5W, 14F)	1036 x 157 x (10W, 14F)	131,7	-212,0	92,2%	7,8%	133,1%	-33,1%
1197 x 312 x (5W, 16F)	1036 x 230 x (10W, 14F)	176,8	-212,0	91,3%	8,7%	132,7%	-32,7%
1432 x 295 x (6W, 16F)	1238 x 154 x (12W, 14F)	156,2	-302,4	93,6%	6,4%	132,5%	-32,5%
1440 x 298 x (6W, 20F)	1238 x 243 x (12W, 14F)	155,5	-302,4	94,6%	5,4%	132,4%	-32,4%

Moment, shear capacities and performance to weight ratio comparisons

S_1	S_2						
Designation	**Designation**	M_r / M'_r	V_r	**Mr or M'r efficiency**		V_r **efficiency**	
h x b x (t_w, t_f)	h x b x (t_w, t_f)	(S_1-S_2) kNm	(S_1-S_2) kN	S_2/S_1	Δ (1– S_2/S_1)	S_2/S_1	Δ (1– S_2/S_1)
1432 x 280 x (6W, 16F)	1233 x 140 x (12W, 14F)	136,3	-307,5	94,0%	6,0%	132,9%	-32,9%
1444 x 260 x (6W, 22F)	1238 x 225 x (12W, 14F)	146,0	-302,4	94,9%	5,1%	132,6%	-32,6%
1436 x 400 x (6W, 18F)	1438 x 145 x (14W, 14F)	292,4	-750,5	91,5%	8,5%	180,5%	-80,5%
1450 x 348 x (6W, 25F)	1438 x 252 x (14W, 14F)	266,2	-750,5	93,5%	6,5%	180,5%	-80,5%
1440 x 385 x (6W, 20F)	1438 x 180 x (14W, 14F)	288,4	-750,5	92,2%	7,8%	180,6%	-80,6%
1450 x 388 x (6W, 25F)	1442 x 283 x (14W, 16F)	247,0	-750,5	94,6%	5,4%	180,5%	-80,5%
1910 x 420 x (8W, 20F)	1644 x 242 x (16W, 16F)	223,6	-533,8	96,0%	4,0%	132,2%	-32,2%
1197 x 300 x (5W, 16F)	1036 x 215 x (10W, 14F)	172,8	-212,0	91,3%	8,7%	133,0%	-33,0%
1432 x 280 x (6W, 16F)	1233 x 140 x (12W, 14F)	136,3	-307,5	94,0%	6,0%	132,9%	-32,9%
1440 x 275 x (6W, 20F)	1242 x 183 x (12W, 16F)	134,4	-302,4	95,1%	4,9%	132,6%	-32,6%
1440 x 338 x (6W, 20F)	1242 x 262 x (12W, 16F)	198,2	-302,4	94,0%	6,0%	132,5%	-32,5%
1432 x 330 x (6W, 16F)	1242 x 170 x (12W, 16F)	192,1	-302,4	92,8%	7,2%	132,5%	-32,5%
1440 x 325 x (6W, 20F)	1242 x 246 x (12W, 16F)	183,5	-302,4	94,2%	5,8%	132,5%	-32,5%
1444 x 364 x (6W, 22F)	1442 x 177 x (14W, 16F)	278,9	-750,5	92,7%	7,3%	180,6%	-80,6%
1456 x 340 x (6W, 28F)	1442 x 272 x (14W, 16F)	246,4	-750,5	94,5%	5,5%	180,5%	-80,5%
1440 x 425 x (6W, 20F)	1442 x 208 x (14W, 16F)	270,5	-750,5	93,3%	6,7%	180,5%	-80,5%
1450 x 420 x (6W, 25F)	1612 x 160 x (16W, 16F)	387,9	-1309,6	92,1%	7,9%	240,4%	-140,4%
1930 x 324 x (8W, 30F)	1652 x 260 x (16W, 20F)	472,9	-533,8	92,5%	7,5%	132,1%	-32,1%
1436 x 370 x (6W, 18F)	1242 x 256 x (12W, 16F)	192,9	-302,4	94,0%	6,0%	132,5%	-32,5%
1444 x 348 x (6W, 22F)	1438 x 177 x (14W, 14F)	286,8	-750,5	92,2%	7,8%	180,6%	-80,6%
1444 x 405 x (6W, 22F)	1442 x 233 x (14W, 16F)	267,0	-750,5	93,7%	6,3%	180,6%	-80,6%
1450 x 298 x (6W, 25F)	1438 x 163 x (14W, 14F)	283,3	-750,5	92,0%	8,0%	180,5%	-80,5%
1444 x 394 x (6W, 22F)	1438 x 250 x (14W, 14F)	266,9	-750,5	93,5%	6,5%	180,5%	-80,5%
1444 x 462 x (6W, 22F)	1570 x 160 x (16W, 16F)	444,4	-1370,8	90,6%	9,4%	247,0%	-147,0%
1920 x 335 x (8W, 25F)	1644 x 241 x (16W, 16F)	210,7	-533,8	96,2%	3,8%	132,1%	-32,1%
1464 x 333 x (6W, 32F)	1631 x 160 x (16W, 16F)	359,2	-1282,9	92,8%	7,2%	237,6%	-137,6%
1920 x 375 x (8W, 25F)	1656 x 220 x (16W, 22F)	452,8	-533,8	92,7%	7,3%	132,2%	-32,2%
1920 x 455 x (8W, 25F)	1657 x 311 x (16W, 22F)	618,5	-532,4	91,6%	8,4%	132,0%	-32,0%
1926 x 478 x (8W, 28F)	1885 x 236 x (18W, 22F)	765,7	-1076,1	91,0%	9,0%	164,7%	-64,7%
1456 x 358 x (6W, 28F)	1455 x 220 x (14W, 22F)	356,2	-749,3	92,4%	7,6%	180,4%	-80,4%
1914 x 360 x (8W, 22F)	1645 x 212 x (16W, 16F)	176,8	-532,4	96,7%	3,3%	132,0%	-32,0%
1926 x 337 x (8W, 28F)	1657 x 223 x (16W, 22F)	446,5	-532,4	92,8%	7,2%	132,0%	-32,0%
1934 x 342 x (8W, 32F)	1663 x 256 x (16W, 25F)	573,3	-532,4	92,0%	8,0%	132,1%	-32,1%

Moment, shear capacities and performance to weight ratio comparisons

S_1	S_2						
Designation	Designation	M_r / M'_r	V_r	Mr or M'r efficiency		V_r efficiency	
h x b x (t_w, t_f)	h x b x (t_w, t_f)	(S_1-S_2) kNm	(S_1-S_2) kN	S_2/S_1	Δ (1−S_2/S_1)	S_2/S_1	Δ (1−S_2/S_1)
1926 x 355 x (8W, 28F)	1657 x 246 x (16W, 22F)	489,8	-532,4	92,4%	7,6%	132,0%	-32,0%
1930 x 398 x (8W, 30F)	1657 x 337 x (16W, 22F)	655,9	-532,4	91,4%	8,6%	132,0%	-32,0%
1926 x 498 x (8W, 28F)	1885 x 261 x (18W, 22F)	776,9	-1076,1	91,3%	8,7%	164,7%	-64,7%
1934 x 498 x (8W, 32F)	1885 x 352 x (18W, 22F)	783,5	-1076,1	92,2%	7,8%	164,7%	-64,7%
1946 x 380 x (8W, 38F)	1885 x 284 x (18W, 22F)	764,1	-1076,1	91,6%	8,4%	164,7%	-64,7%
1940 x 484 x (8W, 35F)	1897 x 312 x (18W, 28F)	778,3	-1076,1	92,7%	7,3%	164,7%	-64,7%
2393 x 544 x (10W, 28F)	1905 x 367 x (18W, 32F)	856,9	-140,9	93,1%	6,9%	105,4%	-5,4%
1926 x 380 x (8W, 28F)	1657 x 276 x (16W, 22F)	570,4	-532,4	91,9%	8,1%	132,3%	-32,3%
1926 x 433 x (8W, 28F)	1657 x 345 x (16W, 22F)	677,6	-532,4	91,3%	8,7%	132,0%	-32,0%
1926 x 398 x (8W, 28F)	1663 x 264 x (16W, 25F)	594,8	-532,4	91,8%	8,2%	132,1%	-32,1%
1930 x 438 x (8W, 30F)	1663 x 344 x (16W, 25F)	753,3	-532,4	91,0%	9,0%	132,1%	-32,1%
1934 x 473 x (8W, 32F)	1891 x 277 x (18W, 25F)	781,6	-1076,1	91,9%	8,1%	164,8%	-64,8%
2393 x 462 x (10W, 28F)	1901 x 315 x (18W, 30F)	481,2	-140,9	95,5%	4,5%	105,4%	-5,4%
1926 x 558 x (8W, 28F)	1885 x 337 x (18W, 22F)	796,1	-1076,1	92,0%	8,0%	164,8%	-64,8%
2393 x 497 x (10W, 28F)	1905 x 326 x (18W, 32F)	638,5	-140,9	94,4%	5,6%	105,4%	-5,4%
2393 x 587 x (10W, 28F)	2101 x 324 x (20W, 28F)	984,8	-784,7	92,6%	7,4%	130,2%	-30,2%
2397 x 631 x (10W, 30F)	2109 x 361 x (20W, 32F)	1214,3	-784,7	92,0%	8,0%	130,3%	-30,3%
1926 x 519 x (8W, 28F)	1885 x 288 x (18W, 22F)	778,8	-1076,1	91,5%	8,5%	164,7%	-64,7%
1926 x 573 x (8W, 28F)	1897 x 280 x (18W, 28F)	781,4	-1076,1	92,3%	7,7%	164,7%	-64,7%
2387 x 533 x (10W, 25F)	1901 x 328 x (18W, 30F)	552,3	-140,9	95,0%	5,0%	105,4%	-5,4%
1926 x 537 x (8W, 28F)	1891 x 273 x (18W, 25F)	785,5	-1076,1	91,8%	8,2%	164,8%	-64,8%
1930 x 568 x (8W, 30F)	1897 x 316 x (18W, 28F)	777,6	-1076,1	92,7%	7,3%	164,7%	-64,7%
2397 x 494 x (10W, 30F)	1905 x 354 x (18W, 32F)	788,3	-140,9	93,5%	6,5%	105,5%	-5,5%
2397 x 561 x (10W, 30F)	2101 x 338 x (20W, 28F)	1020,6	-784,7	92,5%	7,5%	130,2%	-30,2%
2397 x 511 x (10W, 30F)	1905 x 370 x (18W, 32F)	870,2	-140,9	93,1%	6,9%	105,5%	-5,5%
2397 x 594 x (10W, 30F)	2109 x 326 x (20W, 32F)	1117,1	-784,7	92,3%	7,7%	130,3%	-30,3%
2407 x 581 x (10W, 35F)	2115 x 370 x (20W, 35F)	1326,3	-784,7	91,9%	8,1%	130,3%	-30,3%
2407 x 652 x (10W, 35F)	2300 x 446 x (22W, 25F)	1487,2	-1494,5	91,8%	8,2%	157,5%	-57,5%