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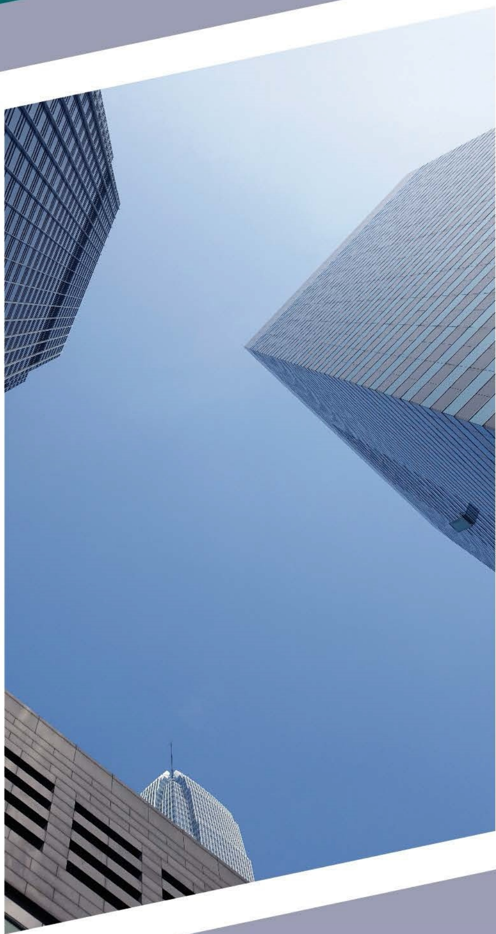
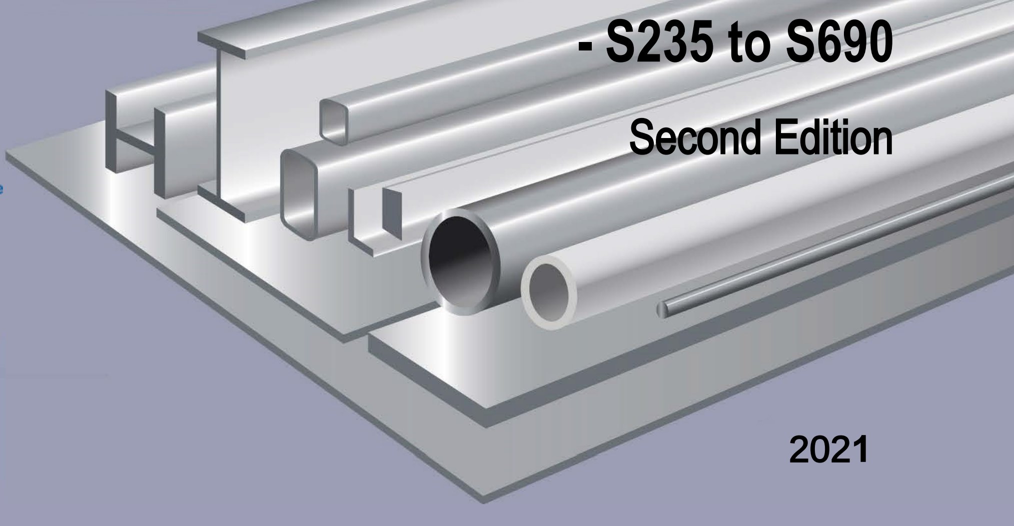


TECHNICAL GUIDE

EFFECTIVE DESIGN AND CONSTRUCTION TO STRUCTURAL EUROCODES: EN 1993-1 DESIGN OF STEEL STRUCTURES - S235 to S690

Second Edition



THE HONG KONG POLYTECHNIC UNIVERSITY
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Chinese National Engineering Research Centre For Steel Construction (Hong Kong Branch)

HONG KONG CMSA
Constructional Metal Structures Association
香港建築金屬結構協會

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2021

Technical Guide on Effective Design and Construction to Structural Eurocodes:
EN 1993-1 Design of Steel Structures – S235 to S690

Second Edition

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Publisher:
Construction Industry Council, Hong Kong SAR

Supporting Organisation:
Hong Kong Constructional Metal Structures Association

ISBN: 978-988-79983-0-3

Foreword

The **Construction Industry Council** (*www.hkcic.org*) (CIC) was formed on 1 February 2007 in accordance with the Construction Industry Council Ordinance (Cap. 587) in Hong Kong. The main functions of the CIC are to forge consensus on long-term strategic issues, to convey the industry's needs and aspirations to the Government as well as to provide a communication channel for the Government to solicit advice on all construction-related matters.

The **CIC Research Fund** was established in September 2012 to enhance efficiency and competitiveness of the local construction industry. The CIC Research Fund encourages research and development activities as well as applications of innovative techniques that directly meet the needs of the industry. Moreover, it promotes establishment of standards and good practices for the construction industry now and into the future.

The project leading to the publication of this document is the first project funded by the CIC Research Fund announced in January 2013. It aims to facilitate technological upgrading of structural engineers and related construction professionals in Hong Kong to work effectively and efficiently in full accordance with the Structural Eurocodes, in particular, in structural steel design. Owing to the wide adoption of the Structural Eurocodes in many parts of the world beyond Member States of the European Union, the use of Structural Eurocodes presents huge opportunities for Hong Kong structural engineers and construction professionals to work on large scale infrastructure projects overseas.

According to the World Steel Association (*www.worldsteel.org*), China has been the largest steel producer in the world since early 2000s. In 2020, China produces about 1053 million metric tons of steel materials, representing 56.5% of the world production. With the support of the Chinese Steel Construction Industry, Hong Kong construction professionals will be able to export their professional services to the international construction markets with quality structural steelwork through their international operation and practice. Hence, this will facilitate Hong Kong as a whole to develop into *the International Engineering Centre for Design and Construction of Infrastructure* for Asia and beyond.

Foreword

The Construction Industry Council (CIC} is a statutory body established in February 2007 with the mission to forge consensus on long-term strategic issues concerning the Hong Kong construction industry, and to convey the needs and aspirations of the industry to the Government. Since its establishment, the CIC has been driving R&D activities and the application of innovative techniques to enhance the performance of the construction industry in terms of safety, productivity, quality and sustainability. In order to support on-going socio-economic development of Hong Kong, we are devoted to preparing the construction professionals for new challenges ahead by equipping them with skills and knowledge on emerging technologies and standards.

In order to increase construction productivity and reduce manpower dependency, an effective use of structural steel for construction has been proven to be highly successful in many countries, including China, Japan, Singapore and Australia. As China is the largest steel producer in the world since 2000 with many high-quality steel fabricators are found in the Greater Bay Area, Hong Kong has the absolute advantage of adopting steel structures for delivering high-quality construction facilities.

The publication of the Second Edition of this Technical Guide is timely and important to facilitate engineers in the Construction Industry to design effectively in accordance with the Structural Eurocodes using steel materials manufactured in either Europe or China. The introduction of high strength S690 steel materials for construction is warmly welcome because of their increased strength to self-weight ratios. As the steel tonnages required are significantly decreased, the corresponding carbon footprints are thereby reduced.

Last but not the least, I would like to congratulate Ir Prof. K.F. Chung, Director of Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch) in conducting the comprehensive research and development programme on the high strength S690 steel materials and members. He has also implemented research findings skilfully into efficient design methods and compile design data to facilitate engineers to adopt high strength S690 steel materials in their construction projects.

Ir Thomas O.S. Ho JP
Chairman
Construction Industry Council
Hong Kong SAR

Foreword

First Edition published in 2015

Since their official release in 2010, the Structural Eurocodes have been widely adopted in construction projects throughout the Member States of the European Community as well as a number of countries and cities in Southeast Asia such as Singapore, Malaysia and Hong Kong. Through effective design and construction using the Structural Eurocodes, designers, contractors and building materials suppliers are able to contribute to the international construction market with minimal technical barriers in the Region and beyond. The ability to produce steel materials to precise specifications and the associated quality control systems in addition to advanced skills in engineering design and construction will be essential.

Jointly published by the Construction Industry Council, Hong Kong SAR, the Hong Kong Constructional Metal Structures Association and the Hong Kong Polytechnic University, the Technical Guide entitled "Effective Design and Construction to Structural Eurocodes: EN 1993-1-1 Design of Steel Structures" is considered to be highly relevant to the current needs of many design and construction engineers in Hong Kong as well as in many major cities in the Region. First published in June 2015, this Technical Guide provides detailed guidance on the design and construction of structural steelwork using European steel materials and products. More importantly, it also provides specific guidance on the use of Chinese steel materials, allowing engineers to select suitable steel materials and products according to generic project requirements on time and on budgets in meeting various specific project requirements.

Consequently, design and construction engineers in Hong Kong and the Region will find the Technical Guide very helpful in providing practical advice on the selection of steel materials and products as well as technical guidance on the engineering design of structural steelwork conforming to Structural Eurocodes. It is expected that the Technical Guide will enable engineers to exploit new opportunities in international construction markets, striving for enhanced economic development of the construction industry in Hong Kong as well as in the Region.

Prof. Qing-Rui YUE
President
Chinese National Engineering Research Centre for Steel Construction
China Steel Construction Society
Beijing, China

Foreword

First Edition published in 2015

Roll forming is an established manufacturing process which has been developed for the mass production of profiles and sections over the past 100 years. In recent years, China has become the largest producer of roll formed profiles and sections in the world. Its annual production is estimated to be 127 million metric tons in 2013, i.e., over 50% of world production. The majority of the production includes thick gauge circular, rectangular and square hollow sections and thin gauge profiles in various sizes and thicknesses with different steel materials. The products are widely used as pipes and ducts in petroleum and chemical refineries, structural members in offshore structures and building frames as well as deckings, wall claddings and roof panels in buildings. Comprehensive design rules for applications of cold-formed sections and profiles in construction are now available in the Structural Eurocodes.

The Technical Guide "Effective Design and Construction to Structural Eurocodes – EN 1993-1-1 Design of Steel Structures" jointly published by the Construction Industry Council, Hong Kong SAR, the Hong Kong Constructional Metal Structures Association and the Hong Kong Polytechnic University is highly commendable. The Technical Guide is a major contribution to the Hong Kong Construction Industry, enabling its design and construction skills in structural steelwork to conform also to the Structural Eurocodes. In particular, the use of Chinese cold formed hollow sections is clearly illustrated in the document, and Design Tables are provided to facilitate adoption of Chinese cold formed hollow sections in construction projects.

We believe that the Technical Guide will promote effective design and construction of structural steelwork using both European and Chinese steel materials and products. The Technical Guide will soon be regarded as the definitive reference for engineering design of cold formed hollow sections conforming to the Structural Eurocodes in many parts of the world, making a positive impact to the export of Chinese steel materials for overseas construction projects.

Prof. Dr.-Ing. Jing-Tao Han
President
Chinese Confederation of Roll Forming Industry
Professor
University of Science and Technology Beijing
Beijing, China

Preface

This document is compiled by Ir Professor K.F. Chung, Ir Dr. H.C. Ho, Dr. Y.F. Hu and Ir Professor Michael C.H. Yam of the Hong Kong Polytechnic University. The publication of this document is supported by a research and development project funded by the State Ministry of Science and Technology of People's Republic of China, and undertaken by the Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch) (www.polyu.edu.hk/cnerc-steel) at the Hong Kong Polytechnic University. The project aims to promote effective use of quality Chinese steel materials in construction projects designed to international codes and construction practice. It is also supported by the Hong Kong Constructional Metal Structures Association (*www.cmsa.org.hk*). It should be noted that the project leading to the publication of the first edition of this document is fully funded by the CIC Research Fund of the Construction Industry Council (*www.hkcic.org*) in Hong Kong.

This document aims to facilitate the technological upgrading of structural engineers and related construction professionals in Hong Kong to work effectively and efficiently in full accordance with the Structural Eurocodes. Moreover, steel materials manufactured to selected European and Chinese steel materials specifications with steel grades ranging from S235 to S690 are covered in various chapters of the document. This provides a level playing field for both European and Chinese steel materials in the technical context of modern structural steel design.

In order to promote effective use of high strength S690 steels in construction, a series of scientific investigations and structural engineering research have been conducted by the Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch) since 2015. Based on these research findings, design recommendations on material requirements, section classifications as well as section and member resistances under compression and bending have been developed and verified. Moreover, section properties tables of high strength S690 steel welded sections and hollow sections are provided to facilitate practical application of high strength steels.

The project is also supported by the following professional associations:

- the Steel Construction Institute (*www.steel-sci.org*), the U.K.
- the Institution of Structural Engineers (*www.istructe.org*), the U.K., and
- the Institution of Civil Engineers, Hong Kong Association (*www.ice.org.hk*).

An International Advisory Committee has been established to provide technical guidance for the project, and a member list of the Committee is as follows:

The U.K.

Dr. Graham Couchman	The Steel Construction Institute
Professor Leroy Gardner	Imperial College London
Professor Dennis S.H. Lam	Bradford University
Professor David A. Nethercot	Imperial College London
Mr. Y. K. Cheng	The Institution of Structural Engineers, U.K.
Mr. C.M. Lee	The Institution of Civil Engineers – Hong Kong Association

Singapore

Professor Richard J.Y. Liew	National University of Singapore
Mr. Melvin So	Singapore Structural Steel Society
Mr. K. Thanabal	Building and Construction Authority
Professor S.P. Chiew	Singapore Institute of Technology

Hong Kong

Ir Professor Francis T.K. Au	The University of Hong Kong
Dr. C.M. Chan	The Hong Kong University of Science and Technology
Dr. T.M. Chan	The Hong Kong Polytechnic University
Ir Dr. Goman W.M. Ho	Ove Arup & Partners Hong Kong Ltd.
Ir Dr. Julian S.H. Kwan	Civil Engineering and Development Department, the Government of Hong Kong SAR
Ir John K.S. Kwong	Development Bureau, the Government of Hong Kong SAR
Dr. Paul H.F. Lam	The City University of Hong Kong
Ir H.Y. Lee	Hong Kong Constructional Metal Structures Association
Ir Alan H.N. Yau	AECOM Building Engineering Co. Ltd.
Ir H.P. Au Yeung	Buildings Department, the Government of Hong Kong SAR

The manuscript of the document was prepared by Ir Professor K.F. Chung, Ir Dr. H.C. Ho and Ir Professor Michael C.H. Yam assisted by Dr. Y.F. Hu, Dr. K. Wang and Dr. T.Y. Ma. The worked examples were compiled by Ir Professor K.F. Chung and Ir Dr. H.C. Ho, and checked by Ir Professor Michael C.H. Yam, Dr. T.M. Chan and Dr. Y.F. Hu. All the Design Tables were compiled by Ms. W. Feng and Dr. Y.F. Hu under the supervision of Ir Professor K.F. Chung.

During the compilation of the document, various drafts have been critically reviewed by the Engineering Technology Committee of the Hong Kong Constructional Metal Structures Association as well as various senior engineers and experts on steel construction. Hence, the final version of the document has been revised according to all of these technical comments, after rigorous consideration to attain a balanced view taking into account international trends, local practice, levels of structural accuracy and adequacy as well as user-friendliness in practical design.

K.F. Chung, H.C. Ho, Y.F. Hu and M.C.H. Yam
Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch)
The Hong Kong Polytechnic University
Hong Kong Constructional Metal Structures Association

EXECUTIVE SUMMARY

This document provides technical guidance on the key structural steel design rules for both rolled and welded sections given in the Structural Eurocode *EN 1993-1 Design of Steel Structures* and the associated *UK National Annex* together with relevant non-contradictory complementary information (NCCI).

This document is compiled to assist structural engineers and related construction professionals in Hong Kong and the neighbouring areas to perform modern structural steel design to EN 1993-1 in an effective and efficient manner. Technical information is presented in the context of the local construction industry, and references to prevailing regulations and codes of practice are made whenever necessary. In addition to European steel materials, selected Chinese steel materials are also included as equivalent steel materials which are readily accepted for construction projects designed to EN 1993-1. This provides a level playing field for both European and Chinese steel materials in the technical context of modern structural steel design.

In general, all the key design rules given in EN 1993-1 are described and supplemented with explanatory notes in the same sequence as that found in the Eurocode:

- General
- Basis of design
- Materials
- Durability
- Structural analysis
- Ultimate limit states
- Serviceability limit states

In order to illustrate various structural design procedures, a total of 8 worked examples with different cross-section properties and resistances as well as different member buckling resistances are provided. Comprehensive design procedures for the following structural members are also presented in a rational manner:

i) column members undergoing flexural buckling,
ii) beam members undergoing lateral torsional buckling, and
iii) beam-column members undergoing buckling under combined compression and bending.

Detailed design information and parameters are also presented in a tabulated format for easy reference.

A complete chapter together with a total of 51 Design Tables is compiled to facilitate practical design of the following:

- Rolled sections of S275 and S355 steel materials
  - rolled I- and H-sections
  - hot-finished circular, rectangular and square hollow sections

- Equivalent welded sections of Q235, Q275, Q355, Q460 and Q690 steel materials
  - welded I- and H-sections
  - cold-formed circular, rectangular and square hollow sections

Hence, rolled sections complying to European steel materials specifications and equivalent welded sections with selected Chinese steel materials have been included for structural engineers and related construction professionals to use in large scale construction projects in Hong Kong and neighbouring cities whenever necessary.

It should be noted that a chapter on design development of high strength S690 steel is introduced, and various experimental and numerical investigations into structural behaviour of high strength S690 steels and their welded sections are concisely presented. These investigations generate both measured and predicted data to verify applicability of various design rules given in EN 1993-1-1 (2005) and 1-12 (2007). Moreover, section properties tables of high strength S690 steel welded sections and hollow sections are provided to facilitate practical application of high strength steels.

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Section 1 Adopting Structural Eurocodes

(1) The Structural Eurocodes are a new set of European design codes for building and civil engineering works. Conceived and developed over the past 40 years with the combined expertise of the member states of the European Union, they are arguably the most advanced structural codes in the world. The Structural Eurocodes are intended to be mandatory for European public works and likely to become the de-facto standard for the private sector – both in Europe and world-wide. The Eurocodes had been available as European pre-standards (ENVs) for several years, and all of them were published as full European Standards (ENs) in 2007.

(2) Owing to the withdrawal of various British structural design standards in March 2010, the Works Department of the Government of Hong Kong SAR has been migrating to the Eurocodes in stages, for the design of public works and civil engineering structures. Mandatory adoption of the Eurocodes will commence in 2015. Since a number of countries outside the European Union, in particular some Asian countries, have already adopted the structural Eurocodes for design and construction of building structures, there is a growing need for design and construction engineers in Hong Kong to acquire the new skills.

1.1 Organization of Eurocodes

(1) A total of 58 parts of the Eurocodes are published under 10 area headings:

- Eurocode 0 – EN 1990: Basis of Structural Design
- Eurocode 1 – EN 1991: Actions on Structures
- Eurocode 2 – EN 1992: Design of Concrete Structures
- Eurocode 3 – EN 1993: Design of Steel Structures
- Eurocode 4 – EN 1994: Design of Composite Steel and Concrete Structures
- Eurocode 5 – EN 1995: Design of Timber Structures
- Eurocode 6 – EN 1996: Design of Masonry Structures
- Eurocode 7 – EN 1997: Geotechnical Design
- Eurocode 8 – EN 1998: Design of Structures for Earthquake Resistance
- Eurocode 9 – EN 1999: Design of Aluminium Structures

(2) It should be noted that

i) the first two areas, namely, EN 1990 and EN 1991, are common to all designs – basis and actions;
ii) the other six areas, namely, from EN 1992 to EN 1996 and EN 1999, are material-specific – concrete, steel, composite steel and concrete, timber, masonry, aluminum; and
iii) the other two areas, namely, EN 1997 and EN 1998, cover geotechnical and seismic aspects.

(3) In order to avoid duplication of design rules as well as problems in updating various parts at different times, one of the prevailing regulations in drafting the Eurocodes is

that no design rule should be presented twice within the entire set of the Eurocodes. As a consequence, there is extensive cross-referencing.

1.2 Composition of EN 1993

(1) Various parts of EN 1993 are listed follows:

Part	1-1:	General rules and rules for buildings
	1-2:	General – Structural fire design
	1-3:	General – Cold formed thin gauge members and sheeting
	1-4:	General – Structures in stainless steel
	1-5:	General – Strength and stability of planar plated structures without transverse loading
	1-6:	General – Strength and stability of shell structures
	1-7:	General – Design values for plated structures subjected to out of plane loading
	1-8:	General – Design of joints
	1-9:	General – Fatigue strength
	1-10:	General – Material toughness and through thickness assessment
	1-11:	General – Design of structures with tension components
	1-12:	General – Supplementary rules for high strength steels
Part	2-1:	Bridges
Part	3-1:	Towers, masts and chimneys – Towers and masts
	3-2:	Towers, masts and chimneys – Chimneys
Part	4-1:	Silos, tanks and pipelines – Silos
	4-2:	Silos, tanks and pipelines – Tanks
	4-3:	Silos, tanks and pipelines – Pipelines
Part	5:	Piling
Part	6:	Crane supporting structures

(2) As indicated by the name, Part 1.1 provides the general rules for structural steel design which are formulated for direct application in building design while the other 11 sections in Part 1 are supplementary to Part 1.1 for application to various steel structures. Owing to the importance of these sections within the Eurocodes, design and construction engineers in Hong Kong need a good understanding of EN 1993-1-1 to make the most of the advantages offered by the Eurocodes.

1.3 Aims and Scope

(1) This document provides technical guidance on key design rules for structural steel design for both the rolled and the welded sections given in the Structural Eurocode EN 1993-1-1 Design of Steel Structures (2005) and the associated UK National Annex together with relevant non-contradictory complementary information. Technical

information is presented in the context of the local construction industry, and references to prevailing regulations and codes of practice are made whenever necessary. Figure 1.1 illustrates various cross-sections of typical welded and rolled sections covered in this document.

(2) All the Nationally Determined Parameters (NDPs) recommended by the Works Bureau of the Government of Hong Kong SAR and provided in the updated design manuals of various government departments have been adopted. These items include load factors, loads, and methods for calculating certain loads, partial safety factors and advice where a choice of design approach is allowed.

(3) In general, all the key design rules given in EN 1993-1-1 are described and supplemented with explanatory notes in the same sequence as found in the Eurocode:

- General
- Basis of design
- Materials
  - yield strengths
- Durability
- Structural analysis
- Ultimate limit states
  - resistances of cross-sections under single actions
  - resistances of cross-sections under combined actions
  - buckling resistances of members under single actions
  - buckling resistances of members under combined actions
- Serviceability limit states

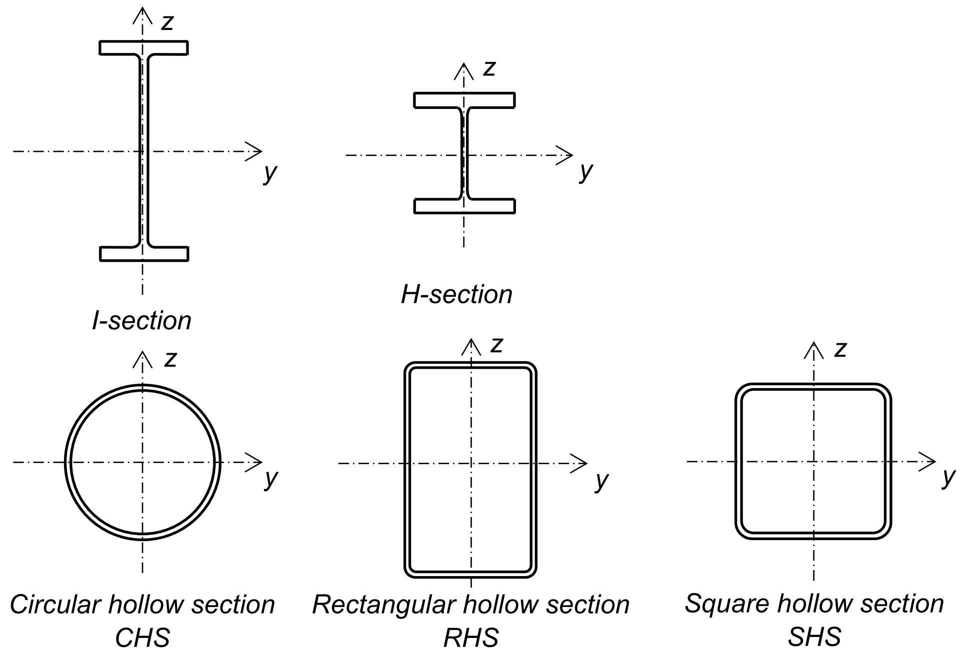
(4) In order to illustrate various design procedures for structural design, a total of 8 worked examples with different cross-section properties and resistances as well as different member buckling resistances are provided. Comprehensive design procedures for the following buckling failure criteria are also provided:

i) column members undergoing flexural buckling,
ii) beam members undergoing lateral torsional buckling, and
iii) beam-column members undergoing buckling under combined compression and bending

Detailed design information and parameters are also presented in tabulated format for easy reference.

A complete section together with a total of 51 Design Tables has been compiled to facilitate the practical design of both rolled and welded sections assuming steel materials of different yield strengths.

Rolled sections:



Welded sections:

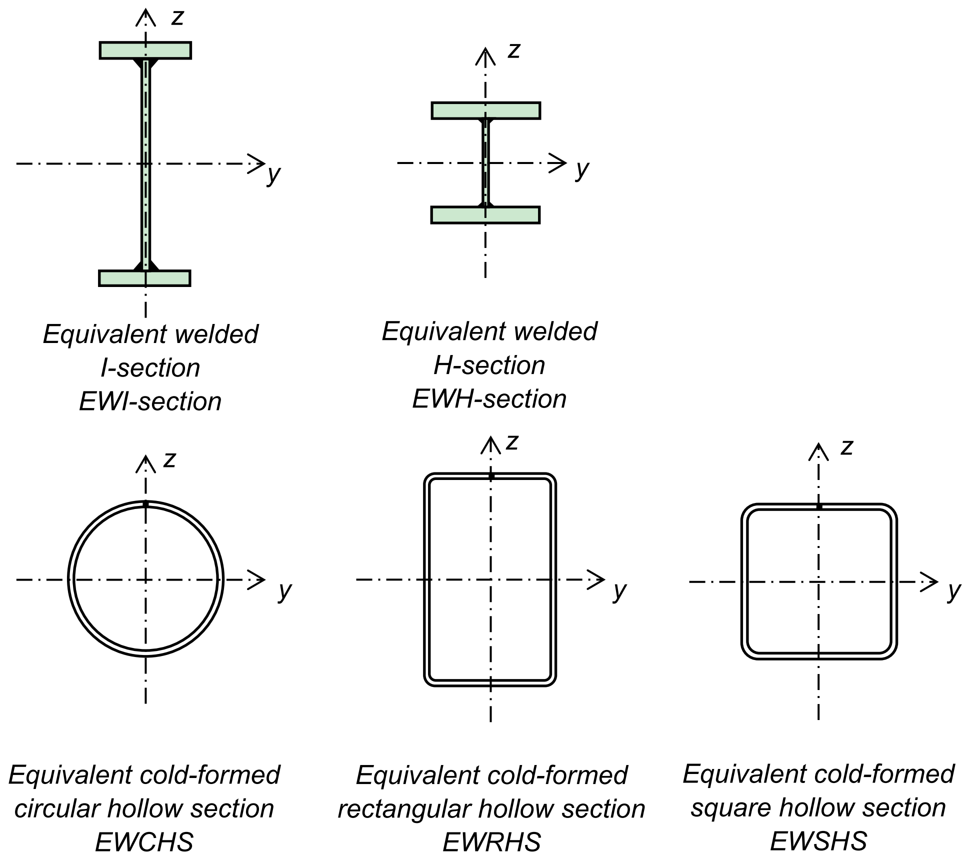


Figure 1.1 Cross-sections of typical rolled and welded sections

(5) A complete section is compiled to facilitate practical design of the following:

- Rolled sections of S275 and S355 steel materials
  - rolled I- and H-sections
  - hot-finished circular, rectangular and square hollow sections
- Welded sections of Q235, Q275, Q355, Q460 and Q690 steel materials
  - welded I- and H-sections
  - cold-formed circular, rectangular and square hollow sections

(6) Hence, rolled sections complying to European steel materials specifications, and welded sections of selected Chinese steel materials have been included for design and construction engineers to use on large scale construction projects in Hong Kong and neighbouring cities.

1.4 Modern Structural Design Codes

(1) Traditionally, a design code is expected to provide all key design requirements and considerations enabling a structural engineer to perform structural design. Proven lower bound design methods are also provided to assist the structural engineer to justify the structural adequacy of a structure in a prescriptive manner, i.e. if a structure is designed and confirmed to satisfy all the design rules, structural adequacy of the structure is deemed to be achieved. However, there is an overriding implicit assumption behind this, i.e. the structure being designed is assumed to behave in an essentially similar fashion to those structures for which the design methods have been developed and derived. While the extreme situation of structural failure would have been prevented, there is little information on how the structure is actually going to behave in relation to some specific requirements, in particular, during serviceability limit states.

(2) A review of the organization of many modern structural design codes reveals a typical layout as follows:

a) Materials
   - Material types and manufacturing processes
   - Physical, chemical and mechanical properties
   - Requirements on structural performance

b) Sections and dimensions
   - Typical shapes and sizes, limiting dimensions and scope of applications

c) Cross-section resistances
   - Cross-section resistances under single actions
   - Cross-section resistances under combined actions

d) Member resistances
   - Member resistances under single actions
   - Member resistances under combined actions

e) System behaviour

f) Connection design
   - Force analysis methods
   - Basic resistances of fasteners, fixings and connectors
   - Resistances and deformations of joints
   - Detailing rules

(3) All these topics are considered to be essential for effective control of the design of a structure, and the given layout is considered to be a simple, effective, and structured arrangement to assist a structural engineer to perform his design in a straight forward manner.

(4) In practice, the design code is often considered to be a legal document enabling a structural engineer to perform his statutory duty to his client as well as to the regulatory authority. Consequently, the design clauses in the code are often written and compiled adopting a prescriptive approach, i.e. everything is spelled out with every use cautioned and every limit defined. However, while most of the design clauses are well controlled, there are occasions when the design becomes grossly conservative or things become unnecessarily complicated when interpretation between the lines of the design clauses is required, or the design lies outside the intended use of the design clauses. Hence, the prescriptive approach is generally considered to be restrictive, and little information is provided once the limits of the design clauses are crossed. Moreover, it is generally difficult to know how efficient the design is.

1.4.1 Modern design approach

(1) With recent advances in development of structural design codes, the performance-based approach should be considered a major advance which enables the rational design and analysis of structural behaviour against well-defined requirements at specific levels of acceptability. This approach is commonly adopted in seismic design as well as in fire resistant design of building structures and bridges whilst the levels of structural responses and acceptability are explicitly defined for specific structures. It is obvious that adopting effective performance-based design requires a high level of understanding of the structural behaviour and the responses of structures. Hence, the structural examination of selected critical members is, in general, insufficient, and it is necessary to perform a numerical simulation of the structural behaviour of the entire structure under specific performance requirements. Supplementary member checks may be carried out, whenever necessary.

(2) Ideally, a design method in a modern design code should be formulated in such a way that a structural engineer is able to perform the design while understanding the underlying principles when working through the design procedures. Moreover, the design procedures should be complied with in a fashion that enables the structural engineer to compromise on the calculation efforts he is prepared to make against the structural accuracy and economy of the structure. He should be able to decide

whether it is sufficient to adopt simple and yet conservative data, or if it is necessary to evaluate specific design parameters precisely, depending on the situation he is dealing with. When the structural engineer is making choices and decisions as the design proceeds, he is able to control the design rationally, i.e. to engineer not just the final product, but also the design process.

1.5 Harmonized Design Rules

(1) It is interesting to review the development of a number of national steel codes, and to examine some of the design methods and clauses which have evolved over the years; an illustration based on the checking of member buckling is given below. It concerns the use of the 'slenderness' parameter of a member, which is derived from elastic buckling theory, to facilitate simple and direct evaluation of member resistances for steel columns and beams as well as steel-concrete composite columns.

1.5.1 Member buckling check for hot-rolled steel sections

(1) Consider the member buckling check in the British Steel Code BS5950 published by the British Standards Institution (2000) and the "Code of Practice for the Structural Use of Steel" published by the Buildings Department of the Government of Hong Kong SAR (2011). For a column susceptible to axial buckling, the slenderness of the column, λ, has been established for many years, and is defined as follows:

$$\lambda = \frac{L_E}{r_y} \tag{1.1}$$

where

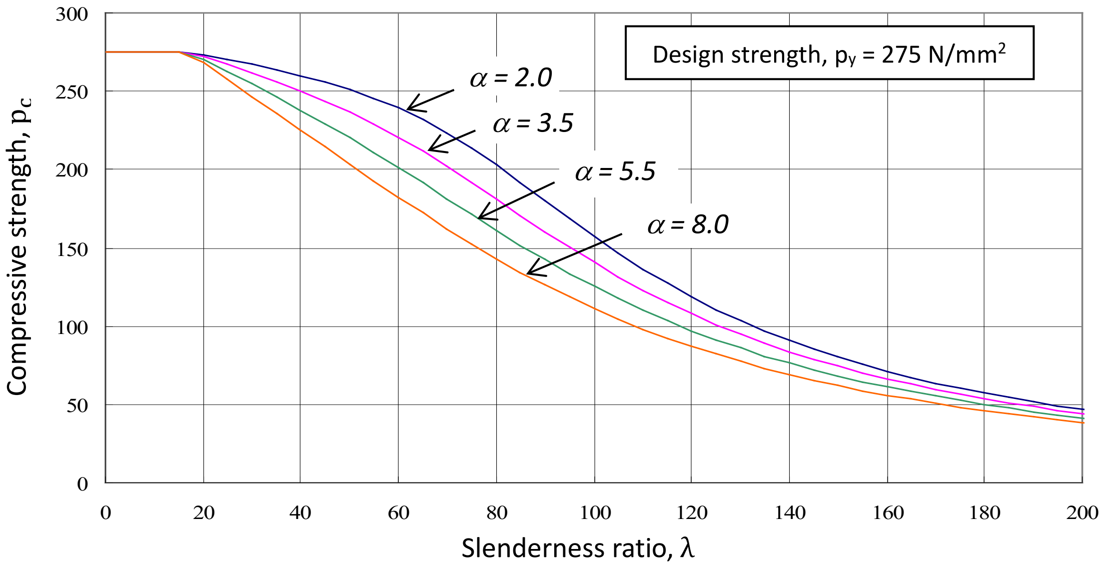
L_E is the effective length of the column, depending on its boundary conditions; and

r_y is the radius of gyration of the cross-section of the column, a function of its cross-section geometry.

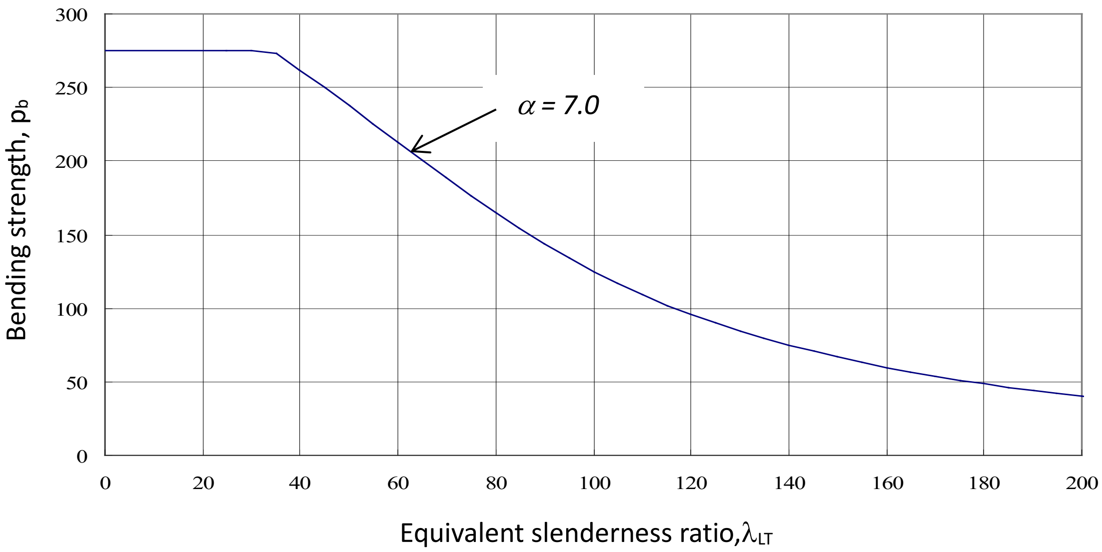
(2) It should be noted that λ is an important structural parameter of a column and is a direct measure of the tendency of the column to undergo elastic buckling. Through a non-linear interaction curve, which is commonly referred as the Perry-Robertson formula, the effect of axial buckling in a real column is expressed as a reduction in its design strength from its yield value, i.e. its compressive strength.

(3) The compressive strength of a real column with material and geometrical initial imperfections is readily obtained using a specific column buckling curve after considering material yielding and geometrical instability. It should be noted that based on section shapes and sizes as well as bending axes during buckling, the value of the imperfection parameter, α , is determined after careful calibration against test data. Thus, a total of four column buckling curves are established, and they are plotted onto the same graph as shown in Figure 1.2a). For columns with welded sections made of thick steel plates, the design methodology is the same although the design yield strengths of the columns should be reduced by 20 N/mm^2 to allow for the presence of

high residual stresses due to welding.



a) Column buckling curves



b) Beam buckling curve

Figure 1.2 Member buckling curves to BS5950 Part 1

(4) For a beam susceptible to lateral buckling, an equivalent slenderness of the beam, λ_{LT}, is devised and defined as follows:

$$\lambda_{LT} = u\,v\,\lambda \tag{1.2}$$

where

u and v are secondary section properties of the beam related to lateral bending and torsion.

(5) The adoption of the equivalent slenderness beam parameter is a good example of harmonized codification, and both design parameters, u and v, may be considered as correction factors which enable the lateral buckling check of a beam to be performed in a way similar to the axial buckling check of a column. Hence, the effect of lateral buckling in a real beam is expressed as a reduction in its design strength from its yield value, i.e. its bending strength. The bending strength of a real beam with material and geometrical initial imperfections is readily obtained after considering material yielding and geometrical instability, as shown in Figure 1.2b).

(6) It should be noted that in BS5950, there is only one beam buckling curve while different design coefficients are adopted for rolled and welded beam sections in calculating various parameters. For standardized steel sections, tabulated values of u and v are readily found in section dimensions and properties tables.

(7) Hence, it is demonstrated that in buckling checks of both columns and beams, the design methods are considered to be highly structured and rational, and all design parameters and coefficients are derived explicitly with analytical formulation. However, it should be noted that the structural adequacy and economy of the design methods often hinge on one single value, the effective length of the member. Up to the very present, there is still little or no effective means of examining the buckling behaviour of a particular member in a structure except through advanced finite element modelling, and the determination of the effective length of the member, and hence, the member slenderness, remains, otherwise, largely empirical.

1.5.2 Member buckling check using normalized slenderness

(1) It is interesting to note that the harmonized design checks for both axial and lateral buckling of steel members given in BS5950 have been adopted in EN 1993-1-1 (2005) with a different formulation. The design rules are re-formulated in such a way that the effect of member buckling in real steel columns and beams are expressed as a reduction to the resistances of the cross-sections, i.e. a strength reduction factor, χ multiplied by the axial compression resistances of the cross-sections of the column members, and a strength reduction factor, χ_b multiplied by the moment resistances of the cross-sections of the beam members respectively.

(2) Moreover, modified slenderness ratios are adopted, which are defined as follows:

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} \text{ or } \sqrt{\frac{N_{c,Rd}}{N_{cr}}} \quad \text{for axial or flexural buckling of columns} \qquad (1.3)$$

and

$$\bar{\lambda}_{LT} = \frac{\lambda_{LT}}{\lambda_1} \text{ or } \sqrt{\frac{M_{c,Rd}}{M_{cr}}} \quad \text{for lateral buckling of beams} \qquad (1.4)$$

where

λ_1 is a material parameter given by:

$= \pi \sqrt{\frac{E}{f_y}}$

E is the elastic modulus of steel;

f_y is the yield strength of steel;

$N_{c,Rd}$ is the design axial resistance of the column;

N_{cr} is the elastic critical buckling resistance of the column;

$= \pi^2 \frac{EI}{L_{cr}^2}$

I is the second moment of area of the cross-section of the column;

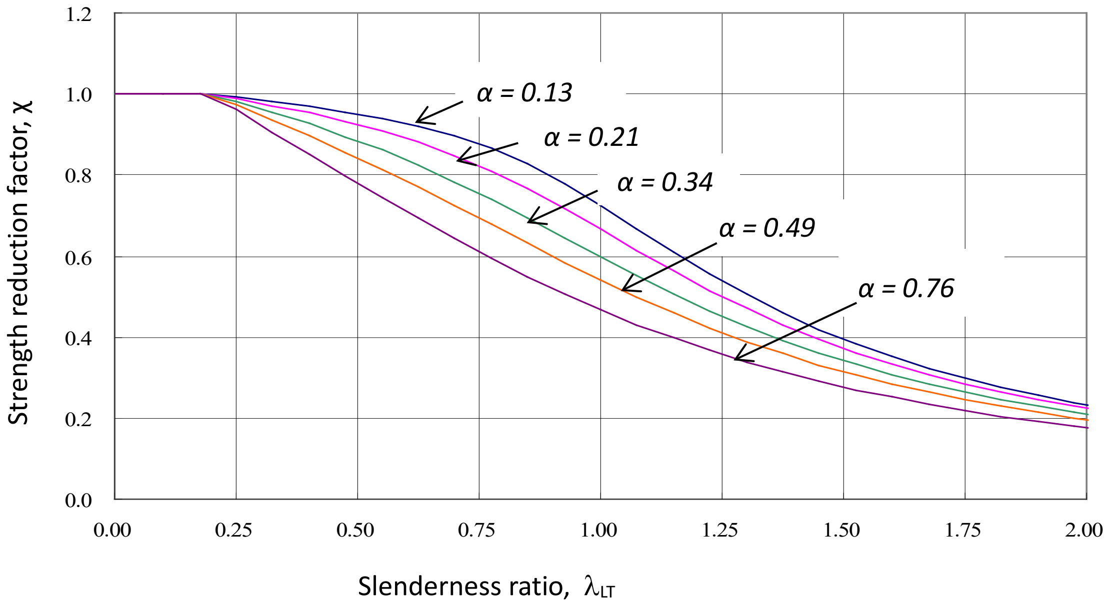
L_{cr} is the buckling length;

$M_{c,Rd}$ is the design moment resistance of the beam; and

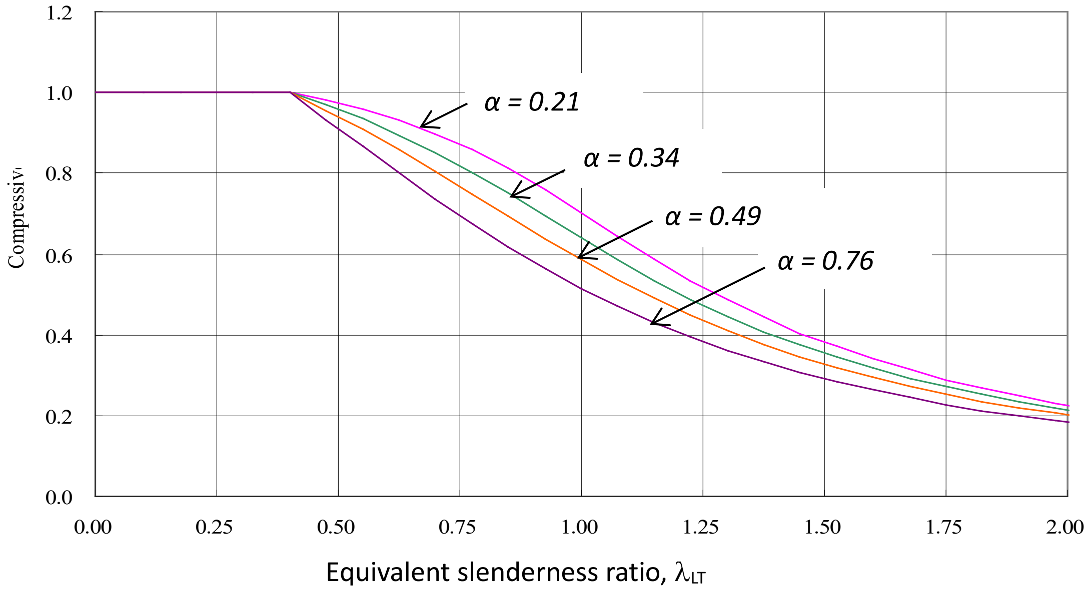
M_{cr} is the elastic critical buckling moment resistance of the beam

(3) It should be noted that the modified slenderness ratio, $\bar{\lambda}$, is defined either as a ratio of the geometrical slenderness to the material parameter of the member, or a ratio of the square root of the ratio of the cross-sectional axial resistance of the member to its corresponding elastic critical buckling resistance. Hence, the design methods are "normalized" against the mechanical properties of the members, and they are equally applicable to other materials, such as other metal and timber members, provided that calibration against geometrical and mechanical initial imperfections has been performed.

(4) As shown in Figure 1.3, there are five different buckling curves for columns and four for beams. The selection on the imperfection parameter, α, depends on section types and sizes as well as bending axes, if applicable.



a) Column buckling curves



b) Beam buckling curves

Figure 1.3 Member buckling curves to EN 1993-1-1

1.5.3 Member buckling check for composite columns

(1) For composite columns of concrete encased H sections or concrete in-filled hollow sections, the same design methodology has been adopted in EN 1994-1-1 (2004), and the axial buckling resistances of the composite columns are based on the modified slenderness ratio which is defined as follows:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rd}}{N_{cr}}} \quad \text{for axial or flexural buckling of columns} \quad (1.5)$$

where

$N_{pl,Rd}$ is the design plastic resistance of the composite column, which is equal to the sum of the section capacities of the individual components: concrete core, steel section and steel reinforcement;

N_{cr} is the elastic axial buckling resistance of the composite column;

$= \pi^2 \dfrac{(EI)_{eff}}{L_{cr}^2}$

$(EI)_{eff}$ is the effective flexural rigidity of the composite column, which is equal to the sum of the effective flexural rigidities of the individual components: concrete core, steel section and steel reinforcement; and

L_{cr} is the buckling length.

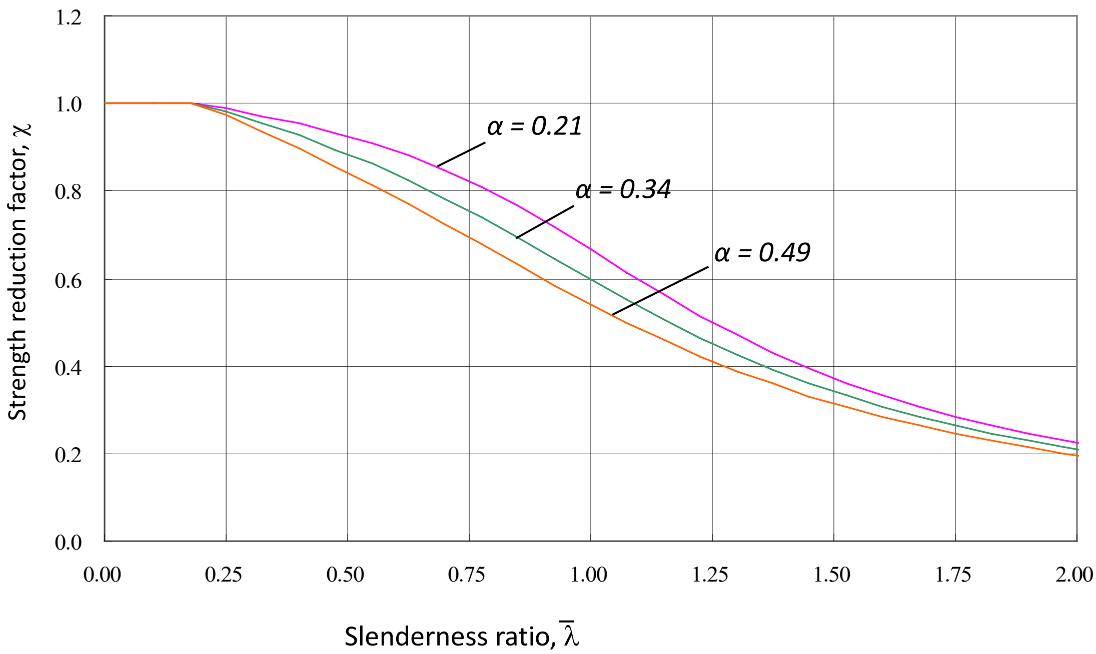


Figure 1.4 Member buckling curves to EN 1994-1-1

(2) Hence, the effect of axial buckling in real composite columns is expressed as a strength reduction to the resistances of the cross-sections of the column members, i.e. a strength reduction factor, χ_c, multiplied by the compression resistances of the cross-sections of the composite columns. As shown in Figure 1.4, there are three different column buckling curves. The selection depends on section types as well as bending axes, if applicable.

(3) Consequently, it is demonstrated that by adopting the same design methodology, i.e. the slenderness ratio of a member or its associated resistance ratio, the effect of buckling is readily expressed as a strength reduction factor multiplied by the resistance of the cross-section of the member. The same methodology is shown to be highly

satisfactory in steel beams and columns as well as composite columns. Moreover, the adoption of different buckling curves enables wide coverage of the many cross-sections of different shapes and sizes as well as bending axes.

1.5.4 Member buckling check for steel and composite columns at elevated temperatures

(1) It should be noted that based on rigorous material tests of a number of constructional materials at elevated temperatures, various sets of strength reduction factors are given in EN 1993-1-2 (2005) and EN1994-1-2 (2005) for general use. Figure 1.5 plots these factors for different constructional materials for easy reference. It is interesting to note that all of these materials retain only 50% of their original strengths when their temperatures reach 500 to 600 ^{o}C.

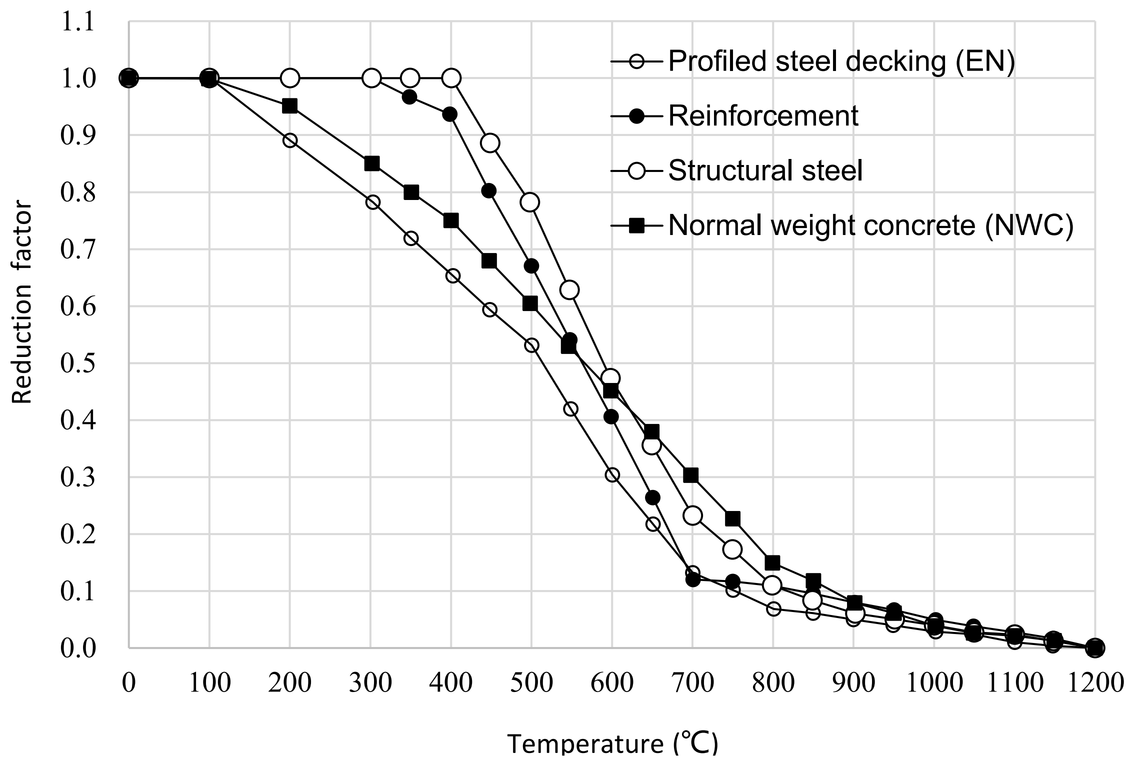


Figure 1.5 Strength reduction factors at elevated temperatures

(2) Based on a known temperature distribution within a structural member obtained either from fire tests or numerical heat transfer analyses, the resistance of the member at elevated temperatures may be readily evaluated according to EN 1993-1-2 and EN 1994-1-2. A flow chart of various design procedures on steel beams and columns as well as composite columns at both normal and elevated temperatures is provided in Figure 1.6 to facilitate the use of these design procedures in practical design.

(3) Owing to the effective design development of member buckling in the Structural Eurocodes, the normalized slenderness ratios of steel beams and columns as well as composite columns are shown to be effective in determining corresponding strength reduction factors due to member buckling, as shown in Figure 1.6. Moreover, the

same design formulation for member buckling design of various types of structural members is readily used at both normal and elevated temperatures with parameters having different values according to the materials of the members.

Design procedures		**Key design parameters**	
		Normal temperatures	*Elevated temperatures*
1. Evaluate both the design and the characteristic resistances.	a. Steel column:	N_{cr} & $N_{pl,Rd}$	$N_{fi,\theta,cr}$ & $N_{fi,\theta,Rd}$
	b. Steel beam:	*Not applicable*	*Not applicable*
	c. Composite column:	N_{cr} & $N_{pl,Rd}$	$N_{fi,\theta,cr}$ & $N_{fi,\theta,Rd}$
2. Evaluate various structural parameters.	a. Steel column:	-	$k_{y,\theta}$, $k_{E,\theta}$
	b. Steel beam:	-	$k_{y,\theta}$, $k_{E,\theta}$
	c. Composite column:	$(EI)_{eff}$	$(EI)_{fi,eff}$
3. Evaluate the non-dimensional slenderness.	a. Steel column:	$\bar{\lambda}$	$\bar{\lambda}_{\theta}$
	b. Steel beam:	$\bar{\lambda}_{LT}$	$\bar{\lambda}_{LT,\theta}$
	c. Composite column:	$\bar{\lambda}$	$\bar{\lambda}_{\theta}$
4. Determine the imperfection factor and the reduction factor.	a. Steel column:	ϕ & χ	ϕ_{θ} & χ_{θ}
	b. Steel beam:	ϕ_{LT} & χ_{LT}	$\phi_{LT,\theta}$ & $\chi_{LT,\theta}$
	c. Composite column:	ϕ & χ	ϕ_{θ} & χ_{θ}
5. Evaluate the buckling resistance.	a. Steel column:	$N_{b,Rd}$	$N_{b,fi,\theta,Rd}$
	b. Steel beam:	$M_{b,Rd}$	$M_{b,fi,\theta,Rd}$
	c. Composite column:	$N_{b,Rd}$	$N_{b,fi,\theta,Rd}$

Figure 1.6
Harmonized design of member buckling at both normal and elevated temperatures

1.6 Symbols and Terminology

(1) The Eurocode system for symbols generally adopts a common notation for the principal variables. Differentiation between related variables, such as axial force and compression resistance, is achieved by the use of subscripts. Multiple subscripts are used where necessary, for example to distinguish between design bending resistances about the y-y and the z-z axes; each component is separated by a comma.

In general, the Eurocode system for symbols is particular and precise, being effective in providing clarity and avoiding ambiguity.

(2) In general, symbols are defined where they are used within the text. A list of the most common symbols used is given in Clause 1.6 of EN 1993-1-1 for easy reference.

Table 1.1 presents a comparison of some of the key symbols adopted in the U.K. and Hong Kong to those adopted in EN 1993-1-1.

Table 1.1 Comparison of key symbols

U.K. and Hong Kong	EN 1993-1-1	U.K. and Hong Kong	EN 1993-1-1
A	A	P	N
Z	W_{el}	M_x	M_y
S	W_{pl}	V	V
I_x	I_y	H	I_w
I_y	I_z	J	I_t

U.K. and Hong Kong	EN 1993-1-1
p_y	f_y
p_b	$\chi_{LT} f_y$
p_c	χf_y
r	i
λ	$\bar{\lambda}$

(3) In this document, a dot is used as the decimal separator, in line with the existing U.K. and Hong Kong practice. However, it should be noted that the Eurocodes themselves use a comma as the separator.

(4) The Eurocodes contain alternative terms to those familiar to the U.K. and Hong Kong

designers, and some important changes are summarized in Table 1.2.

Table 1.2 Important changes on terminology

U.K. and Hong Kong terms	Eurocode terms
Loads	Actions
Dead load	Permanent action
Imposed or live load; wind load	Variable action
Ultimate loads	Design value of actions
Check	Verification
Internal forces and bending moments which result from the application of the actions	Effects of actions
Capacity, or Resistance	Resistance
Second-order effects	Effects of deformed geometry

1.7 Conventions for Member Axes

(1) The convention for member axes is:

x – x axis along a member
y – y axis major axis of a cross-section
z – z axis minor axis of a cross-section

(2) For typical I- and H-sections and structural hollow sections, the convention used for cross-section axes are:

y – y axis major axis of the cross-section which is parallel to the flanges
z – z axis minor axis of the cross-section which is perpendicular to the flanges

The cross-section axes of typical sections are illustrated in Figure 1.1. Table 1.3 summarizes the differences in the notation of the axes in both members and cross-sections.

Table 1.3 Difference in the notation of axes

	U.K. and Hong Kong	Eurocodes
Longitudinal axis along the member	X (?)	X
Major axis of a cross-section	X	Y
Minor axis of a cross-section	Y	Z

1.8 Format

(1) All the clauses and paragraphs in this document are numbered consecutively.

(2) In the Eurocodes, a distinction is made between Principles and Application Rules:

i. Principles are identified by the letter P following the paragraph number.

ii. Application Rules are generally recognized rules which comply with the Principles and satisfy their requirements.

This distinction is retained in this document.

1.9 Equivalent Steel Materials

(1) For many years, almost all steel structures in Hong Kong were designed to the British structural steel design code, BS5950, and all the steel materials were specified correspondingly to the British steel materials specifications such as BS4360. However, as early as the 1990s, non-British steel materials found their way to Hong Kong as well as Singapore and other neighbouring cities in Southeast Asia. Occasionally, contractors wanted to use non-British steel materials, such as Japanese, Australian and Chinese steel materials. The proposed changes ranged from merely adopting such materials for some members of temporary structures to their use for complete beam-column frames of building structures. Over the years, many successful projects were reported in Hong Kong which benefited from good quality non-British steel materials, timely supply and delivery as well as improved structural economy. However, there were also a few bad examples of the use of non-British steel materials having inconsistent chemical compositions, inadequate mechanical properties and lack of traceability.

(2) In the 2000s, owing to large fluctuations in the costs of steel materials on the global markets, Chinese steel materials became practical alternatives to British steel materials in a number of construction projects in Asia, in particular, in Hong Kong, Macau and Singapore. During the drafting of the "Code of Practice for the Structural Use of Steel" for the Buildings Department of the Government of Hong Kong SAR from February 2003 to August 2005, it was decided necessary to devise a means to allow, or more accurately, to formalize the use of Chinese steel materials as equivalent steel materials for structures which were originally designed to BS5950. Various parts of Section 3 of the Hong Kong Steel Code provide basic principles and considerations for accepting, as well as qualifying, steel materials manufactured to the following national materials specifications:

- Australian / New Zealand standards,
- Chinese standards,
- Japanese standards, and
- American standards.

A practical classification system for non-British steel materials is introduced in the Code in which the design strengths of these non-British steel materials depend on a newly defined factor, namely, the material class factor, γ_{Mc} .

(3) Similar use of non-British steel materials was also formally adopted in Singapore with the issue of a technical guide entitled "Design Guide on Use of Alternative Steel Materials to BS5950" in 2008, and then its revised version entitled "Design Guide on Use of Alternative Structural Steel to BS5950 and Eurocode 3" by the Building and Construction Authority of the Ministry of National Development. These Design Guides aimed to provide technical guidelines and design information on the use of non-British steel materials, and the classification system for various steel materials given in the "Code of Practice for the Structural Use of Steel" was adopted after modification. Under the provisions of these Design Guides, alternative steel materials not manufactured to British and European steel materials standards may be allowed in structural design based on the Structural Eurocodes for construction projects in Singapore.

(4) In 2014, the use of non-British steel materials in Hong Kong, Singapore and other neighbouring cities in Asia was further promoted through the publication of a Professional Guide on "Selection of Equivalent Steel Materials to European Steel Materials Specifications" (Publication CMSA-PG01). The Professional Guide is jointly published by the Hong Kong Constructional Metal Structures, Macau Society of Metal Structures and Chinese National Engineering Research Centre for Steel Construction. It presents essential technical guidance to design and construction engineers as well as engineers from regulatory authorities on the selection of steel materials equivalent to material requirements specified in the European steel materials specifications.

Through the use of the Professional Guide, selected steel materials manufactured to the modern materials specifications of Australia/New Zealand, China, Japan, and the United States of America are fully endorsed to be equivalent to steel materials manufactured to the European steel materials specifications, provided that all of these steel materials have been demonstrated to be in full compliance with the requirements of both material performance and quality control as detailed in the Professional Guide. Consequently, these equivalent steel materials can be readily employed on construction projects for which the structural steelwork is designed to EN 1993 and EN 1994.

(5) Given a satisfactory demonstration of both the material performance and the quality assurance procedures adopted during their manufacturing processes, steel materials with yield strengths from 235 to 690 N/mm^2 are classified as follows:

- **Class E1 Steel Materials with γ_{Mc} = 1.0**

  Steel materials which are

  i) manufactured in accordance with one of the **Acceptable Materials Specifications** listed in Appendix A of the Professional Guide with a fully demonstrated compliance on their material performance, and

  ii) manufactured in accordance with **an Acceptable Quality Assurance System** with full demonstration of effective implementation.

  Thus, compliance with all the material requirements has been demonstrated through **intensive routine testing** conducted during the effective implementation of a certificated **Factory Production Control** system which accords with European steel materials specifications. The Factory Production Control System must be certified by an independent qualified certification body.

- **Class E2 Steel Materials with γ_{Mc} = 1.1**

  Steel materials which are

  i) manufactured in accordance with one of the **Acceptable Materials Specifications** listed in Appendix A of the Professional Guide with a fully demonstrated compliance on their material performance, and

  ii) manufactured in accordance with an effectively implemented quality assurance system which is different to a Factory Control Production System.

Thus, the steel materials are manufactured in accordance with all the material requirements given in one of the Acceptable Materials Specifications, but without a certified Factory Production Control System which accords with European steel materials specifications.

In general, although many steel manufacturers will have already established a form of quality assurance during the manufacturing processes, the high level of consistency in the material performance of the steel materials required in European steel materials specifications cannot be verified in the absence of a certified Factory Production Control System. Hence, a demonstration of the conformity of the steel materials is required, and additional material tests with sufficient sampling should be conducted for various batches of supply to demonstrate full compliance with both the material performance and the quality assurance requirements.

- **Class E3 Steel Materials**

Steel materials for which they cannot be demonstrated they were

  i) manufactured in accordance with any of the **Acceptable Materials Specifications** listed in Appendix A; nor

  ii) manufactured in accordance with an **Acceptable Quality Assurance System**.

Hence, any steel material which cannot be demonstrated to be either Class E1 Steel Material or Class E2 Steel Material will be classified as Class E3 Steel Material, and the nominal value of yield strength of the steel material is limited to 170 N/mm^2 for structural design; no additional material test is needed in general. However, the design yield strength of the steel material may be increased if additional material tests with sufficient sampling have been conducted for various batches of supply before use.

For details of specific requirements on material performance and quality assurance, refer to the Professional Guide. Also refer to Section 3.2.3 of the Professional Guide for details of additional materials tests.

(6) Table 1.4 summarizes the classification system applying to the various classes of steel materials.

Table 1.4 Classification system for various classes of steel materials

Nominal yield strength (N/mm^2)	**Class**	**Compliance with material performance requirements**	**Compliance with quality assurance requirements**	**Additional material tests**	**Material class factor, γ_{MC} for**	
					minimum yield strength, R_{eH}	**ultimate tensile strength, R_m**
≥ 235 and ≤ 690	**E1**	Y	Y	N	1.0	1.0
	E2	Y	N	Y	1.1	1.1
	E3	N	N	N	---	---

(7) A newly defined factor, namely, the material class factor, γ_{MC} , is adopted as a result of the classification, and hence, the nominal values of the yield strength and of the ultimate tensile strength of the equivalent steel materials are given as follows:

- Nominal value of yield strength

$$f_y = R_{eH} / \gamma_{MC} \tag{6a}$$

- Nominal value of ultimate tensile strength

$$f_u = R_m / \gamma_{MC} \tag{6b}$$

where R_{eH} is the minimum yield strength to product standards;
R_m is the ultimate tensile strength to product standards; and
γ_{MC} is the material class factor given in Table 1.4.

It should be noted that

a) Plastic analysis and design is permitted for Classes E1 and E2 Steel Materials assuming yield strengths not larger than 460 N/mm^2.

b) For Classes E1 and E2 Steel Materials with yield strengths larger than 460 N/mm^2 but smaller than or equal to 690 N/mm^2, design rules given in EN 1993-1-12 should be used.

c) Only elastic analysis and design should be used for Class E3 Steel Materials.

Section 2 Basis of Structural Design

This Section presents the key principles as well as the relevant application rules in EN 1990 that relate to the design of steel structures together with specific requirements given in EN 1993-1-1. These include specific rules on basic requirements, reliability management, principles of limit state design, partial factor method as well as combinations of action. It is important to be familiar with the various terminologies and mathematical formats of the expressions, formulae and equations adopted in the Eurocodes.

2.1 General Requirements

Design of a structure requires the demonstration of structural adequacy under various effects of actions in extreme events, i.e. the ultimate limit state, and of full compliance against various requirements in deformation, vibration and durability during its intended life, i.e. serviceability limit states.

2.1.1 Basic requirements

(1)P A structure shall be designed and executed in such a way that during its intended life, with appropriate degrees of reliability and in an economical way, it will sustain all actions likely to occur during execution and use, and meet specified serviceability requirements.

(2)P A structure shall be designed to have adequate structural resistance, serviceability and durability.

(3)P In the case of fire, the structural resistance shall be adequate for the required period of time.

(4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as explosion, impact, and consequences of human errors, to an extent disproportionate to the original cause.

(5)P Potential damage shall be avoided or limited by appropriate choice of one or more of the following:

- avoiding, eliminating or reducing the hazards to which the structure can be subjected;
- selecting a structural form which has low sensitivity to the hazards considered;
- selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;
- avoiding structural systems that can collapse without warning;
- tying structural members together.

(6) The basic requirements should be met by the use of appropriate materials, design and detailing, and quality control.

2.1.2 Reliability management

(1)P The reliability required for structures within the scope of EN 1990 shall be achieved by:
a) design in accordance with EN 1990 to EN 1999, and
b) appropriate execution and quality management measures.

(2) Different levels of reliability may be adopted, among other things:
- for structural resistance;
- for serviceability.

(3) The choice of the levels of reliability for a particular structure should take account of various relevant factors, including:
- possible cause and mode of attaining a limit state;
- possible consequences of failure in terms of risk to life, injury, potential economical losses;
- public aversion to failure;
- expenses and procedures necessary to reduce the risk of failure.

(4) The levels of reliability that apply to a particular structure may be specified in one or both of the following ways:
- by classification of the whole structure;
- by classification of its individual components.

(5) The levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of:
a) preventative protective measures;
b) measures relating to design calculations:
   - representative values of actions;
   - choice of partial factors;
c) measures relating to quality management;
d) measures aimed to reduce errors in design and execution of the structure, and gross human errors
e) other measures relating to the following design matters:
   - basic requirements;
   - degree of robustness (structural integrity)
   - durability, including the choice of the design working life;
   - extent and quality of preliminary investigations of soils and possible environmental influences
   - accuracy of mechanical models;
   - detailing

f) efficient execution, e.g. in accordance with the execution standards referred to in EN 1991 to EN 1999.

g) adequate inspection and maintenance according procedures specified in the project documentation.

(6) The measures to prevent potential causes of failure and to reduce their consequences may, in appropriate circumstances, be interchanged to a limited extent provided that the required reliability levels are maintained.

(7) The level of reliability should be achieved by the use of appropriate quality management in design and execution.

(8) In general, execution should be performed in accordance with EN 1090-2, and execution class EXC2 should be specified.

> EN 1090-2 gives 4 classes of requirements for execution of the structure as a whole or for components of a structure, namely, Classes EXC1 to EXC4, with increasing strictness requirements. For common buildings and structures, Class EXC2 for the whole structure is normally considered to be sufficient.

2.1.3 Design working life

(1) Common building structures should be designed for a working life of at least 50 years.

> In general, 50 years is the normal design working life for building structures, and this is implicitly adopted in the usual characteristic values of actions selected together with the various associated partial factors of safety.

2.2 Principles of Limit State Design

(1) The resistances of cross-sections and members specified in this document for the ultimate limit states as defined in Section 3.3 of EN 1991-1-3 are based on tests in which the steel materials exhibited sufficient ductility to allow to application of simplified design methods.

> Various design situations are introduced which should be considered for design against both ultimate and serviceability limit states.

2.2.1 Design situations

(1)P The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfill its function.

(2)P Design situations shall be classified as follows:

Persistent design situations	-	normal conditions of use
Transient design situations	-	temporary conditions applicable to the structure
Accidental design situations	-	exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact or the consequences of localised failure
Seismic design situations	-	conditions applicable to the structure when subjected to seismic events

> In general, the persistent design situation is the most common in practice while transient design situations occur during the construction stages as well as during renovation and refurbishment.

(3)P The selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution as well as the use of the structure.

2.2.2 Ultimate limit states

(1)P The limit states that concern the safety of people and the safety of the structure shall be classified as ultimate limit states.

(2) In some circumstances, the limit states that concern the protection of the contents should be classified as ultimate limit states.

(3) States prior to structural collapse, which, for simplicity, are considered in place of the collapse itself, may be treated as ultimate limit states.

(4)P The following ultimate limit states shall be verified where they are relevant:
- loss of equilibrium of the structure or any part of it, considered as a rigid body;
- failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure, or any part of it, including supports and foundations;
- failure caused by fatigue or other time-dependent effects.

Different sets of partial factors are associated with the various ultimate limit states.

2.2.3 Serviceability limit states

(1)P The limit states that concern the functioning of a structure or its structural members under normal use, comfort of people, and deformation of construction works (leading to extensive cracking) shall be classified as serviceability limit states.

(2)P A distinction shall be made between reversible and irreversible serviceability limit states.

(3) Verification of serviceability limit states should be based on criteria concerning the following aspects:

a) deformations that affect

- appearance,
- comfort of users, or
- functioning of the structure (including functioning of machines or services), or that cause damages to finishes or non-structural members;

b) vibrations that adversely affect

- comfort to people, or
- functional effectiveness of the structure;

c) damages that are likely to adversely affect

- appearance,
- durability, or
- functioning of the structure.

2.3 Basic Variables and Limit State Design

2.3.1 Actions and environmental influences

(1) Actions for the design of steel structures should be taken from EN 1991. For the combination of actions and partial factors of actions, refer to Annex A to EN 1990.

(2) The actions to be considered in the erection stage should be obtained from EN 1991-1-6.

(3) Where the effects of predicted absolute and differential settlements need to be considered, best estimates of imposed deformations should be used.

2.3.2 Material and product properties

(1) Material properties for steels and other construction products and the geometrical data to be used for design should be those specified in the relevant ENs, ETAGs or ETAs unless otherwise indicated in this document.

2.3.3 Limit state design

(1)P Design for limit states shall be based on the use of structural and load models for relevant limit states.

(2)P It shall be verified that no limit state is exceeded when relevant design values for
– actions,
– material properties, or
– product properties, and
– geometrical data
are used in these models.

(3)P Verifications shall be carried out for all relevant design situations and load cases.

(4) The requirements of Clause (1)P above should be achieved by the partial factor method described in Clause 2.4 Verification by Partial Factor Method.

(5) As an alternative, a design directly based on probabilistic methods may be used.

(6)P The selected design situations shall be considered and critical load cases identified.

(7) For a particular verification, load cases should be selected, identifying compatible load arrangements, sets of deformations and imperfections that should be considered simultaneously with fixed variable and permanent actions.

(8)P Possible deviations from assumed directions or positions of actions shall be taken into account.

(9) Structural and load models can be either physical models or mathematical models.

2.4 Verification by Partial Factor Method

2.4.1 Design values

(1) The design value F_d of an action F is expressed as:

$$F_d = \gamma_F \psi F_k \tag{2.1}$$

where

γ_F is a partial factor for the action F;

ψ is the combination factor and is equal to 1.0 for permanent actions, or to ψ_0, ψ_1, or ψ_2 for variable actions; and

F_k is the characteristic value of the action, F.

In general, the design value of an action is usually expressed as $\gamma_F \psi F_k$ rather than F_d for clarity. Moreover, permanent and variable actions are distinguished symbolically by the use of G_k for permanent actions and Q_k for variable actions, i.e. $\gamma_G \psi G_k$ and $\gamma_Q \psi Q_k$ respectively.

(2) The design value X_d of a material property is expressed as:

$$X_d = \frac{X_k}{\gamma_M} \tag{2.2}$$

where

X_k is a characteristic value of the material; and

γ_M is a partial factor for a material property.

In general, the design value of a material property is usually expressed as $\frac{X_k}{\gamma_M}$ rather than X_d for clarity.

(3) Geometrical data for cross-sections and systems may be taken from product standards hEN or drawings for the execution to EN 1090 and treated as nominal values.

Design values of geometrical imperfections specified in this document are equivalent geometric imperfections that take into account the effects of:

- geometrical imperfections of members as governed by geometrical tolerances in product standards or the execution standard;
- structural imperfections due to fabrication and erection;
- residual stresses; and
- variation of yield strengths

(4) The design value of resistance is expressed as a function of the design value of a material property and a geometrical data:

$$R_d = R\left\{\frac{X_k}{\gamma_M}; a\right\} \tag{2.3a}$$

where

a is the geometric parameter.

Alternatively, the design resistance may be obtained directly from the characteristic value of a material by:

$$R_d = \frac{R_k}{\gamma_M} \tag{2.3b}$$

where

R_k is the characteristic value of the particular resistance determined with characteristic or nominal values for the material properties and dimensions; and

γ_M is the global partial factor for the particular resistance.

2.4.2 Ultimate limit states

(1)P The following ultimate limit states of a structure shall be verified:

EQU Loss of static equilibrium of the structure or any part of it considered as a rigid body.

STR Failure or excessive deformation of the structure or its structural members including supports where the strength of the structural material governs.

GEO Failure or excessive deformation of the ground where the strengths of soils or rocks are significant in providing resistances.

FAT Fatigue failure of the structure or its structural members.

In general, the STR limit state is the only limit state that needs to be considered.

(2)P When considering a limit state of rupture or excessive deformation of a section, a member or a connection, i.e. STR limit state, it shall be verified that:

$$E_d \leq R_d \tag{2.4}$$

where

E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments; and

R_d is the design value of the corresponding resistance.

2.4.3 Combination of actions at ULS

2.4.3.1 General

(1) For each design situation, the design values of the effects of the actions should be determined from the combination of the actions that may occur simultaneously.

(2) Each combination of actions should include a leading or main variable action, or an accidental action.

2.4.3.2 Persistent or transient design situations

(1) The combination of effects of actions to be considered should be based on:

- the design value of the leading variable action, and
- the design combination values of the accompanying variable actions.

(2) The combinations of actions may either be expressed as

$$\sum_{j\geq1}\gamma_{G,j}G_{k,j} \text{ "+" } \gamma_P P \text{ "+" } \gamma_{Q,1}Q_{k,1} \text{ "+" } \sum_{i\geq1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \text{(2.5) [Eqn. 6.10 of EN 1990]}$$

or alternatively, for the STR limit state, the less favourable of the two following expressions:

$$\sum_{j\geq1}\gamma_{G,j}G_{k,j} \text{ "+" } \gamma_P P \text{ "+" } \gamma_{Q,1}\psi_{0,1}Q_{k,1} \text{ "+" } \sum_{i\geq1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \text{(2.5a) [Eqn. 6.10a of EN 1990]}$$

$$\sum_{j\geq1}\xi_j\gamma_{G,j}G_{k,j} \text{ "+" } \gamma_P P \text{ "+" }\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i\geq1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \text{(2.5b) [Eqn. 6.10b of EN 1990]}$$

where

"+" implies "to be combined with";

$\sum$ implies "the combined effect of";

$G_{k,j}$ are the characteristic values of the permanent actions;

$Q_{k,1}$ is the characteristic value of one of the variable actions;

$Q_{k,i}$ are the characteristic values of the other variable actions;

$\gamma_{G,j}$ is the partial factor for the permanent action $G_{k,j}$;

$\gamma_{Q,i}$ is the partial factor for the variable action $Q_{k,i}$;

$\psi_{0,i}$ is the ψ_0 factor for the combination value of the variable action $Q_{k,i}$;

ξ_j is a reduction factor applied to unfavorable permanent actions (in Expression 6.10b of EN 1990);
= 0.925 according to NA of EN 1990.

According to the Eurocodes approach, it is necessary to apply all variable actions to the structure under consideration to examine the effects of actions on the structure. It should be noted that each variable action is in turn considered as the "leading" variable action while all the other variable actions are applied correspondingly with each of them multiplied by a relevant factor. It is thought that Expression (6.10) of EN 1990 gives a quick, but conservative approach when compared to Expressions (6.10a) and (6.10b) of EN 1990, which are slightly more involved.

In general, it is expected that Expression (6.10b) of EN 1990 will normally be the governing case.

(3) The partial factors to be used in the combination of actions and the factors on accompanying actions are given in Table 2.1 which are extracted from Tables N.A.A1.2(a) and N.A.A1.2(b) of UK NA to EN 1990 and modified accordingly to local practice. The corresponding partial factors for buildings and bridges are also presented in Tables 2.2 and 2.3 for easy reference.

Table 2.1 Partial factors for actions, γ_F

Buildings				
Ultimate Limit State	Permanent Actions $\gamma_{G,j}$		Leading or Main Variable Action $\gamma_{Q,1}$	Accompanying Variable Action $\gamma_{Q,i}$
	Unfavorable	Favorable		
EQU	1.40	1.00	1.60	1.60
STR	1.40	1.00	1.60	1.60

Civil engineering works					
Ultimate Limit State	Permanent Actions $\gamma_{G,j}$	Leading or Main Variable Action $\gamma_{Q,1}$	Traffic Actions (gr1a, gr1b, gr2, gr3, gr4, gr5, gr6)	Rail Traffic Actions	Wind Actions
	Unfavorable	Favorable			
EQU	1.05	0.95	1.35	To be agreed	2.10
STR	1.35	0.95	1.35	To be agreed	2.10

Note: When variable actions are favourable, Q_k should be taken as zero.

For building structures, reference should be made to "Code of Practices for the Structural Use of Steel 2011" for the detailed design values of actions.

For civil engineering works, reference should be made to "Structures Design Manual for Highways and Railways 2013" for the detailed design values of actions.

Table 2.2 Values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Permanent actions + General variable actions	0.875	0.75	0.75
Permanent actions + Equivalent horizontal actions	0.875	0.75	0.75
Permanent actions + Wind actions + General variable actions	0.75	0.75	0.75
Temperature (non-fire) in buildings	0.75	0.75	0.75

[a]On roofs, imposed loads should not be combined with wind loads.

Table 2.3 Values of ψ factors for bridges

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see "Structures Design Manual for Highways and Railways")			
Traffic loads gr1a: TS, UDL	0.75	0.75	0.0
Traffic loads gr1b: Single axle	0.00	0.75	0.0
Traffic loads gr2: Horizontal forces	0.00	0.00	0.0
Traffic loads gr3: Pedestrain loads	0.00	0.40	0.0
Traffic loads gr4: Crowd loading	0.00	-	0.0
Traffic loads gr5: Vertical forces from SV and SOV vehicles	0.00	-	0.0
Traffic loads gr6: Horizontal forces from SV and SOV vehicles	0.00	0.00	0.0
Wind loads: Permanent design situation	0.50	0.20	0.0
Wind loads: During erection	1.00	-	0.0
Thermal actions	0.60	0.60	0.50

2.4.4 Serviceability limit states

(1)P It shall be verified that:

$$E_d \leq C_d \tag{2.6}$$

where

E_d is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination; and

C_d is the limiting design value of the relevant combination.

As the partial factors for actions γ_F are implicitly taken as 1.0, they are therefore not shown in the expressions for the effects of actions for clarity.

2.4.5 Combination of actions for SLS

(1) The combinations of actions for serviceability limit states are:

- Characteristic applicable for irreversible limit states;
- Frequent applicable for reversible limit states; and
- Quasi-permanent applicable for long-term effects and the appearance of the structure.

(2) The expressions for the effects due to the combinations of actions are:

Characteristic combination

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{2.7}$$

Frequent combination

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{2.8}$$

Quasi-permanent combination

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{2.9}$$

where

$\psi_{1,1}$ is the factor for the frequent value of the variable action $Q_{k,i}$ (see Table 2.2)

$\psi_{2,1}$ is the factor for the quasi-permanent value of the variable action $Q_{k,i}$ (see Table 2.2).

Advice on which combination to use is given in EN 1993-1-1 and its National Annex. The National Annex to EN 1993-1-1 states that serviceability deflections should be based on the unfactored variable actions, and that permanent actions need not be included. Refer to Section 7 of this document for further information.

Section 3 Materials

3.1 General

(1) The nominal values of material properties given in this Section should be adopted as characteristic values in design calculations.

(2) This Part of EN 1993 covers the design of steel structures fabricated from steel materials conforming to the steel grades listed in Table 3.1.

(3) In general, EN 1993-1-1 covers steel materials conforming to EN 10025 Parts 2, 3, 4, 5 and 6, EN 10210-1 and EN 10219-1 in grades S235 to S690.

However, for quality steel materials which are manufactured to other materials specifications but satisfy both material performance and quality assurance requirements, they are readily considered to be equivalent steel materials.

(4) Depending on the supply sources of these steel materials, if it can be demonstrated that these steel materials satisfy both material performance and quality control requirements as described in Section 1.9, they are then considered as Class E1 Steel Materials, and the corresponding material class factor, γ_{Mc}, is taken to be 1.0.

However, if these steel materials are demonstrated to satisfy only the material performance requirements but not the quality control requirements as described in Section 1.9, they are then considered as Class E2 Steel Materials, and the corresponding material class factor, γ_{Mc}, is taken to be 1.1.

(5) Table 3.1 presents all the steel grades given in Table 3.1 of EN 1993-1-1:

Table 3.1 European Steel Materials:

EN 10025 – 2	EN 10025 – 3	EN 10025 – 4	EN 10025 – 5	EN 10025 – 6
• S235 • S275 • S355 • S450	• S275 N/NL • S355 N/NL • S420 N/NL • S460 N/NL	• S275 M/ML • S355 M/ML • S420 M/ML • S460 M/ML	• S235 W • S355 W	• S460 Q/QL/QL1 • S690 Q/QL/QL1

EN 10210 – 1	EN 10219 – 1	
• S235 H • S275 H • S355 H • S275 NH/NLH • S355 NH/NLH • S420 NH/NHL • S460 NH/NLH	• S235 H • S275 H • S355 H • S275 NH/NLH • S355 NH/NLH • S460 NH/NLH	• S275MH/MLH • S355 MH/MLH • S420 MH/MLH • S460 MH/MLH

(6) Table 3.2 presents commonly used Chinese steel grades which are considered to be equivalent steel materials for adoption in structural design to EN 1993:

Table 3.2 Chinese steel materials:

GB/T 700-2006	GB/T 1591-2018	GB/T 4171-2008	GB/T 19879-2015
• Q195 • Q215 • Q235 • Q275	• Q355, Q355N/M • Q390, Q390N/M • Q420, Q420N/M • Q460, Q460N/M • Q500M • Q550M • Q620M • Q690M	• Q235NH • Q265GNH • Q310GNH • Q295NH/GNH • Q355NH/GNH • Q415NH • Q460NH • Q500NH • Q550NH	• Q235GJ • Q345GJ • Q390GJ • Q420GJ • Q460GJ • Q500GJ • Q550GJ • Q620GJ • Q690GJ

GB/T 6725-2017		GB/T 8162-2018	
• Q195 • Q215 • Q235 • Q345 • Q390 • Q420	• Q460 • Q500 • Q550 • Q620 • Q690 • Q750	• Q345 • Q390 • Q420 • Q460	• Q500 • Q550 • Q620 • Q690

Refer to the Professional Guide entitled “Selection of Equivalent Steel Materials to European Steel Materials Specifications” for further details on equivalent steel materials manufactured to different materials specifications.

3.2 Structural Steel

3.2.1 Material properties

(1) The nominal values of the yield strength f_y and of the tensile strength f_u for structural steel should be obtained either by

a) adopting the values of $f_y = R_{eH}$ and $f_u = R_m$ directly from the product standard, or
b) using the values given in Table 3.1 of EN 1993-1-1.

In general, both f_y and f_u are material strengths of the steel materials measured either along the longitudinal or in the transverse directions with respect to the rolling direction during manufacturing. Both f_y and f_u are determined in standard tensile tests to EN 10002 which specifies details of testing procedures (material sampling, dimensions of coupon sizes, straining rates) and data analyses.

3.2.2 Ductility requirements

(1) For steel materials for which a minimum ductility is required, the following three requirements should all be satisfied:

- the ratio of tensile strength to yield strength:

  $f_u / f_y \geq 1.1$ for S235~S460, $\geq$ 1.05 for S690 (3.1a)

  where

  f_u is the tensile strength, and
  f_y is the yield strength.

- the elongation at failure :

  elongation at failure $\geq$ 15% for S235~S460, $\geq$ 10% for S690 (3.1b)

  which is based on a standard gauge length of 5.65 $\sqrt{A_0}$ where A_0 is the original cross-sectional area of the coupon.

- the ultimate strain ε_u:

  $\varepsilon_u \geq 15\varepsilon_y$ (3.1c)

  where
  ε_u is the strain corresponding to the tensile strength f_u, and
  ε_y is the yield strain, i.e. $\varepsilon_y = f_y / E$.

Ductility is one of the most important mechanical properties of modern steel materials which allow steel structures to undergo large deformations without fracture, especially in highly stressed parts of members or joints. Moreover, ductility facilitates mobilization of cross-sectional resistances, and simplifies the determination of cross-sectional resistances without the need to examine the actual stress distribution within a cross-section. Hence, these three limits on ductility requirements are effective measures in providing a safety margin for steel structures against failure by plastic collapse through large or even excessive deformations in the strain-hardening range of the steel materials.

3.2.3 Fracture toughness

(1) The material should have sufficient fracture toughness to avoid brittle fracture of tension elements at the lowest service temperature expected to occur within the intended design life of the structure.

The lowest service temperature for building and civil engineering structures in Hong Kong is 0 °C. Refer to the Code of Practice for the Structural Use of Steel (2011) for further details.

3.2.4 Through-thickness properties

(1) Where steel materials with improved through-thickness properties are necessary according to EN 1993-1-10, steel materials according to the required quality class in EN 10164 should be used.

Table 3.3 Choice of quality class according to EN 10164

Target value of Z_{Ed} according to EN 1993-1-10	Required value of Z_{Rd} expressed in terms of design Z-values according to EN 10164
$Z_{Ed} \leq 10$	-
$10 < Z_{Ed} \leq 20$	Z15
$20 < Z_{Ed} \leq 30$	Z25
$Z_{Ed} \geq 30$	Z35

The through-thickness property is a measure of the ability of steel plates to ensure integrity against lamination (or separation) when they are subject to high tensile stresses acting in the through-thickness direction.

For those welded steel plates with high tensile residual stresses induced in the through-thickness direction, lamination within the plate thickness may occur leading to extensive local cracks in the welded zones. Hence, it is necessary to specify an appropriate target value for the permissible reduction in cross-sectional area of the steel material in the through-thickness direction, Z_{Ed}. Particular care should be given to welded beam-to-column connections, and welded end plates where there is tension in the through-thickness direction.

3.2.5 Tolerances

(1) The dimensional and mass tolerances of plates, rolled sections, and hollow sections should conform to the relevant product standards unless more severe tolerances are specified.

(2) For welded components, the tolerances given in EN 1090 should be applied.

Refer to the Code of Practice for the Structural Use of Steel (2011) for further details.

(3) For structural analysis and design, the nominal values of dimensions should be used.

3.2.6 Design values of material coefficients

Modulus of elasticity	E = 210,000 N/mm^2
Shear modulus	G = $E/[2(1+\nu)]$
	= 81,000 N/mm^2
Poisson's ratio	ν = 0.3
Coefficient of linear thermal expansion	α = 14 $\times 10^{-6}$ °C
Density	7850 kg/m^3

3.3 Connecting Devices

3.3.1 Fasteners

Requirements for fasteners are given in EN 1993-1-8.

3.3.2 Welding consumables

Requirements for welding consumables are given in EN 1993-1-8.

4 Durability

(1) The basic requirements for durability are set out in EN 1090.

> The durability of a structure is its ability to remain fit for use during its design working life given appropriate maintenance
>
> According to EN 1990, a structure should be so designed that deterioration over its design working life does not impair the performance of the structure. Moreover, it is essential for a designer to identify various requirements that need to be allowed for during the design stage to achieve a high level of durability according to the expected design working life of the structure.
>
> A structure should be designed in such a way, and provided with protection as necessary, so that no significant deterioration is likely to occur within the period between successive inspections. Critical parts of the structure need to be available for inspection, without complicated dismantling.
>
> Other interrelated factors that need to be taken into account to ensure an adequately durable structure are given below:
>
> - intended and future use of the structure
> - required performance criteria
> - expected environmental influences
> - composition, properties and performance of materials
> - choice of structural system
> - shape of members, structural detailing, and buildability
> - quality of workmanship and level of control
> - particular protective measures
> - maintenance during the intended life

(2) The means of executing the protective treatment undertaken off-site and on-site should be in accordance with EN 1090.

(3) Parts susceptible to corrosion, mechanical wear or fatigue should be designed such that inspection, maintenance and reconstruction can be carried out satisfactorily and access is available for in-service inspection and maintenance.

(4)B For building structures, no fatigue assessment is normally required except as follows:

a) members supporting lifting appliances or rolling loads
b) members subject to repeated stress cycles from vibrating machinery
c) members subject to wind-induced vibrations
d) members subject to crowd-induced oscillations.

(5) For elements that cannot be inspected, an appropriate corrosion allowance should be included.

(6)B Corrosion protection does not need to be applied to internal building structures if the internal relative humidity does not exceed 80%.

The following factors should be taken into account in design of corrosion protective systems for a structure in order to ensure its durability under conditions relevant both to its intended use and to its design working life.

- The environment of the structure, whether bimetallic corrosion is possible and the degree of exposure of the structure.
- Accessibility of the structure for inspection and maintenance, (i.e. easy, difficult or impossible). Access, safety and member shapes, and structural detailing are relevant.
- The relationship between corrosion protection and fire protection systems.

Typical examples of commonly occurring exposure conditions are given below.

Table 4.1 Exposure conditions

Exposure Class	**Type of Exposure**	**Examples**
1	Non-corrosive	Steelwork in an internal controlled (i.e. dry) environment. Steel piles driven into undisturbed and non-corrosive ground.
2	Mild (typically internal)	Steelwork in an internal humid environment.
3	Moderate (internal or external)	Steelwork built into perimeter cladding. External steelwork in a dry climate.
4	Severe	External steelwork exposed to rain and humidity. Internal steelwork over a swimming pool, kitchen or water tank.
5	Extreme	External steelwork in a marine environment. Steel piles driven into corrosive ground. Steelwork exposed to salt water.

Refer to the Code of Practice for the Structural Use of Steel for further details.

Section 5 Structural Analysis

5.1 Structural Modeling for Analysis

5.1.1 Structural Modeling and basic assumptions

(1) Analysis should be based upon calculation models of the structure that are appropriate for the limit state under consideration.

> Generally, a structural model is established in accordance with the geometry and the member configuration of a structure. An allowance for inevitable imperfections present within a structure is also made. It should be noted that no member imperfection is incorporated into the structural model since these are implicitly allowed for during structural design in accordance with Section 6.

(2) The calculation model and the basic assumptions for the calculations should reflect the structural behaviour at the relevant limit state with appropriate accuracy, and reflect the anticipated type of behaviour of the cross-sections, members, joints and bearings.

(3) The method used for the analysis should be consistent with the design assumptions.

> When a designer considers connections in a steel structure to be either pinned joints or rigid joints during structural analysis, he needs to design these connections correspondingly. For a nominally pinned base of a structure, a 10% of the column stiffness $\left(\frac{EI}{L}\right)$ is often assumed in structural analysis at ultimate limit state, in particular, in assessing frame stability; and 20% at a serviceability limit state.

5.2 Global Analysis

5.2.1 Effects of deformed geometry of a structure

(1) The internal forces and moments within a structure may generally be determined using either:

- first order analysis, using the initial geometry of the structure or
- second order analysis, taking into account the influence of the deformation of the structure.

(2) The effects of deformed geometry (or the second-order effects) should be considered if they increase the action effects significantly or modify significantly the structural behaviour.

> In general, the effects of deformed geometry of a structure are considered to be non-advantageous owing to large reduction in the resistances of the members.

(3) First order analysis may be used for the structure if the increase of the relevant internal forces or moments or any other change in the structural behaviour caused by deformations can be neglected.

This condition may be assumed to be fulfilled if the following criterion is satisfied:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10 \quad \text{for elastic analysis} \tag{5.1a}$$

or

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 15 \quad \text{for plastic analysis} \tag{5.1b}$$

where

- α_{cr} is the factor by which the loading would have to be increased to cause elastic instability in a global mode;
- F_{Ed} is the design load acting on the structure; and
- F_{cr} is the elastic critical buckling load for the global instability model based on initial elastic stiffnesses.

(4)B Portal frames with shallow roof slopes and regular beam-column plane frames in buildings may be checked for sway mode failure with first order analysis if Expression (5.1) is satisfied for each storey.

In these structures, α_{cr} may be calculated using the following approximate formula, provided that the axial compression in the beams or rafters is not significant:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) \tag{5.2}$$

where:

- H_{Ed} is the design value of the horizontal reaction at the bottom of the storey to the horizontal loads and fictitious horizontal forces (which are applied to produce the effects of sway imperfections to the structure as given in Clause 5.3.2);
- V_{Ed} is the total design vertical load on the structure acting at the bottom of the storey;
- $\delta_{H,Ed}$ is the horizontal displacement at the top of the storey, relative to the bottom of the storey, when the frame is loaded with horizontal loads (e.g. wind) and fictitious horizontal forces which are applied at each floor level; and
- h is the storey height.

> In the U.K., Expression 5.2 above is considered to be inappropriate for portal frames. A modified expression for portal frames $\alpha_{cr,est}$ should be calculated following the recommended approach given in the paper entitled "Eurocode 3 and the in-plane stability of portal frames" which was published in the November 2005 issue of *The Structural Engineer*.

5.2.2 Structural stability of frames

(1) If, according to Clause 5.2.1, the influence of the deformation of the structure has to be taken into account, (2) to (6) should be applied to consider these effects and to verify its structural stability.

(2) Verification of the structural stability of frames or their parts should be carried out considering: i) imperfections, and ii) second order effects.

(3) According to the type of the frame and of the global analysis, imperfections and second order effects may be accounted for by one of the following methods:

a) both totally by global analysis;
b) partially by global analysis and partially through individual stability checks of members according to Clause 6.3; and
c) for basic cases by individual stability checks of equivalent members according to Clause 6.3 using appropriate buckling lengths according to the global buckling mode of the structure.

(4) Second order effects may be calculated by using an analysis appropriate to the structure (including step-by-step or other iterative procedures). For frames where the first sway buckling mode is predominant, first order elastic analysis should be carried out with subsequent amplification of relevant action effects (e.g. additional bending moments) by appropriate factors.

(5)B For single storey frames designed on the basis of elastic global analysis, second order sway effects due to vertical loads may be calculated by increasing the horizontal loads H_{Ed} (e.g. wind) and equivalent loads $V_{Ed}\ \phi$ due to imperfections (see Clause 5.3.2(7)), and other possible sway effects according to first order theory by the factor:

$$\frac{1}{1-\frac{1}{\alpha_{cr}}} \qquad \text{provided that } \alpha_{cr} \geq 3.0, \qquad (5.3)$$

where α_{cr} may be calculated according to Expression (5.2) in Clause 5.2.1(4)B, provided that the roof slope is shallow and that the axial compression in the beams or rafters is not significant as defined in Clause 5.2.1(4)B.

5.3 Imperfections

5.3.1 Basis

(1) Appropriate allowances should be incorporated in the structural model to cover the effects of imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of flatness, lack of fit and any minor eccentricities present in joints of the unloaded structure.

(2) Equivalent geometric imperfections should be used with values which reflect the

possible effects of all types of imperfections unless these effects are included in the resistance formula for member design.

(3) The following imperfections should be taken into account:

a) global imperfections for frames and bracing systems

b) local imperfections for individual members

> It is essential to incorporate imperfections in the structural model of a structure. Global imperfections may be taken into account by modelling the frame out-of-plumb, or by a series of equivalent horizontal forces applied to a frame modelled vertically. In general, the latter approach is recommended.
>
> It should be noted that i) imperfections in individual members may be modelled, or ii) members may be modelled as straight whilst imperfections are implicitly allowed for by verifying member resistances in accordance with Section 6.

5.4 Methods of Analysis Allowing for Material Non-linearities

5.4.1 General

(1) The internal forces and moments in a structure may be determined using either

a) elastic global analysis, or
b) plastic global analysis.

(2) Elastic global analysis may be used in all cases.

(3) Plastic global analysis may be used only where the structure has sufficient rotation capacity at the actual locations of the plastic hinges, whether this is in the members or in the joints.

Where a plastic hinge occurs in a member, the member cross-section should be doubly symmetric or singly symmetric with a plane of symmetry in the same plane as the rotation of the plastic hinge, and it should satisfy the requirements specified in 5.6. Where a plastic hinge occurs in a joint, the joint should have sufficient strength to ensure the hinge remains in the member, i.e. it should be able to sustain the plastic resistance of the member for a sufficient rotation.

(4)B As a simplified method for a limited plastic re-distribution of moments in continuous beams where following an elastic analysis, some peak moments exceed the plastic bending resistances by a maximum of 15%, the parts in excess of these peak moments may be re-distributed in any member, provided that:

a) the internal forces and moments in the frame remain in equilibrium with the applied loads,
b) all the members in which the moments are reduced have Class 1 or Class 2 cross

sections, and

c) lateral torsional buckling of the members in prevented.

5.4.2 Elastic global analysis

(1) Elastic global analysis should be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level is.

(2) Internal forces and moments may be calculated according to elastic global analysis even if the resistance of a cross-section is based on its plastic resistance.

(3) Elastic global analysis may also be used for cross-sections of which the resistances are limited by local buckling.

5.4.3 Plastic global analysis

(1) Plastic global analysis allows for the effects of material non-linearity in calculating the action effects of a structural system. The behaviour should be modelled by one of the following methods:

- by elastic-plastic analysis with plastified sections and joints as plastic hinges,
- by non-linear plastic analysis considering the partial plastification of members in plastic zones, or
- by rigid plastic analysis neglecting the elastic behaviour between hinges.

(2) Plastic global analysis may be used where the members have sufficient rotation capacity to enable the required re-distributions of bending moments to develop.

(3) Plastic global analysis should only be used where stability of the members at plastic hinges can be assured.

(4) A bi-linear stress-strain relationship may be used for the grades of structural steel specified in Section 3.

(5) Rigid plastic analysis may be applied if no effects of the deformed geometry (e.g. second-order effects) have to be considered. In this case, joints are classified only by strengths.

(6) The effects of deformed geometry of the structures and the corresponding structural stability of the frame should be verified according to the principles in 5.2.

5.5 Classification of Cross-sections

5.5.1 Basis

(1) The role of cross section classification is to identify the extent to which the moment resistances and the rotation capacities of the cross-sections are limited by their local buckling resistances.

5.5.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- **Class 1** cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis without reduction of the resistance.
- **Class 2** cross-sections are those which can develop their plastic moment resistance, but which have limited rotation capacity because of occurrence of local buckling.
- **Class 3** cross-sections are those in which the stresses in the extreme compression parts of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- **Class 4** cross-sections are those in which local buckling will occur before the attainment of yield strength in one or more parts of the cross-sections.

Class 4 cross-sections are outside the scope of this document.

(2) Compression parts include every part of a cross-section which is either totally or partially in compression under the load combination considered.

(3) A cross-section is classified according to the highest (least favourable) class of its compression parts.

(4) The limiting proportions for Class 1, 2, and 3 compression parts should be obtained from Table 5.1.

(5) A part which fails to satisfy the limits for Class 3 should be taken as Class 4.

5.6 Cross-section Requirements for Plastic Global Analysis

(1) At plastic hinge locations, the cross-section of the member which contains the plastic hinge should have a rotation capacity of not less than that required at the plastic hinge location.

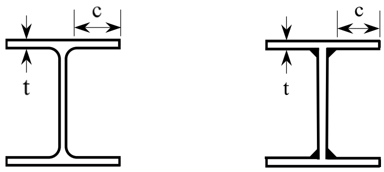
(2) In a uniform member, sufficient rotation capacity may be assumed at a plastic hinge if both the following requirements are satisfied:

a) The member has a Class 1 cross-section at the plastic hinge location; and

b) Where a transverse force that exceeds 10% of the shear resistance of the cross-section is applied to the web at the plastic hinge location, web stiffeners should be provided within a distance along the member of h/2 from the plastic hinge location, where h is the height of the cross-section at this location.

Table 5.1a Maximum c/t ratios of compression parts

Outstand flanges



Rolled sections Welded sections

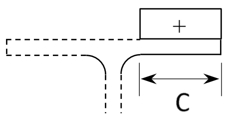
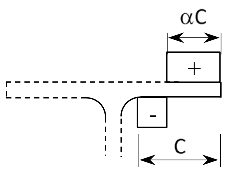
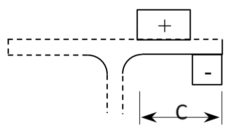
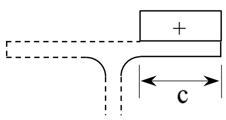
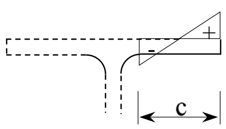
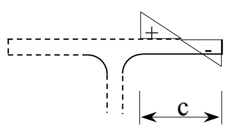
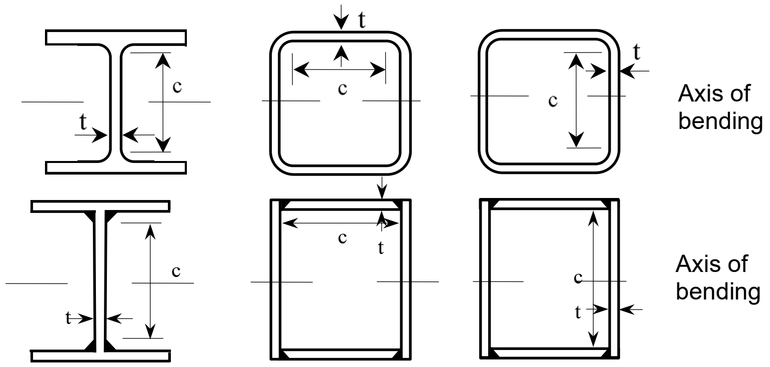
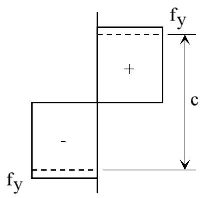
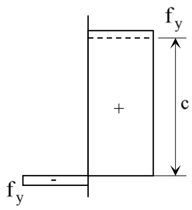
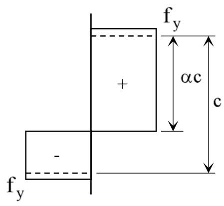
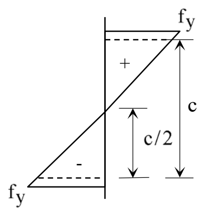
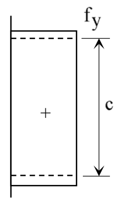
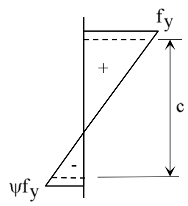
Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$\frac{c}{t} \leq 9\varepsilon$	$\frac{c}{t} \leq \frac{9\varepsilon}{\alpha}$	$\frac{c}{t} \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$\frac{c}{t} \leq 10\varepsilon$	$\frac{c}{t} \leq \frac{10\varepsilon}{\alpha}$	$\frac{c}{t} \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$\frac{c}{t} \leq 14\varepsilon$	$\frac{c}{t} \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

Table 5.1b Maximum c/t ratios of compression parts

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression	
Stress distribution in parts (compression positive)				
1	$\frac{c}{t} \le 72\,\varepsilon$	$\frac{c}{t} \le 33\,\varepsilon$	when $\alpha > 0.5$	$\frac{c}{t} \le \frac{396\varepsilon}{13\alpha - 1}$
			when $\alpha \le 0.5$	$\frac{c}{t} \le \frac{36\varepsilon}{\alpha}$
2	$\frac{c}{t} \le 83\,\varepsilon$	$\frac{c}{t} \le 38\,\varepsilon$	when $\alpha > 0.5$	$\frac{c}{t} \le \frac{456\varepsilon}{13\alpha - 1}$
			when $\alpha \le 0.5$	$\frac{c}{t} \le \frac{41.5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)				
3	$\frac{c}{t} \le 124\,\varepsilon$	$\frac{c}{t} \le 42\,\varepsilon$	when $\psi > -1$	$\frac{c}{t} \le \frac{42\varepsilon}{0.67 + 0.33\psi}$
			when $\psi \le -1$	$\frac{c}{t} \le 62\varepsilon(1-\psi)\sqrt{-\psi}$

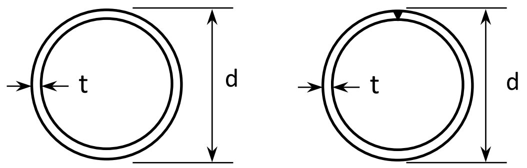
The values of α and ψ are given by

(1) $\alpha = \frac{1}{2}\left(1 + \frac{N_{Ed}}{f_y\, c\, t_w}\right)$

(2) $\psi = \frac{N_{Ed}}{A\, f_y} - 1$

where N_{Ed} is positive in compression.

Table 5.1c Maximum c/t ratios of compression parts



Class	Section in bending and/or compression
1	$\frac{d}{t} \leq 50\,\varepsilon^2$
2	$\frac{d}{t} \leq 70\,\varepsilon^2$
3	$\frac{d}{t} \leq 90\,\varepsilon^2$

Section 6 Ultimate Limit States

6.1 Partial Factors for Resistances

(1) The partial factors γ_M should be applied to the various characteristic values of resistances in this section as follows:

Resistances	EC3	UK-NA	Hong Kong
Resistances of cross-section in tension, compression, shear, and bending, and any of their combination, γ_{M0}	1.0	1.0	1.0
Resistances of members to instability, γ_{M1}	1.0	1.0	1.0
Resistances of cross-sections in tension to fracture, γ_{M2}	1.25	1.1	1.1

γ_{M2} is used with ultimate material strengths, for example when verifying net areas subject to tension (see Clause 6.2.3(3)(b)) and when verifying net areas subject to a shear force in connection design. A different value of γ_{M2} is used when calculating the resistance of connection components.

6.2 Resistances of Cross-Sections

6.2.1 General

(1)P The design value of an action effect in each cross-section, E_d, should not exceed the corresponding resistance, R_d.

$$E_d \leq R_d \quad \text{or} \quad \frac{E_d}{R_d} \leq 1 \qquad (6.1)$$

Design checking against a cross-section resistance rather than a limiting stress within the cross-section allows economical design as the post-yielding strength or even the plastic resistance of the cross-section is mobilized. Moreover, the formulation is consistent for various degrees of strength mobilization including i) elastic, ii) elasto-plastic, and iii) plastic stress blocks.

If several action effects act simultaneously, the combined effect should not exceed the resistance for that combination.

(2) Shear lag effects and local buckling effects should be included by an effective width according to EN 1993-1-5. Shear buckling effects should also be considered according to EN 1993-1-5.

(3) The design values of resistances should depend on the classification of the cross-section.

(4) Elastic verification according to the elastic resistance may be carried out for all cross sectional classes provided the effective cross sectional properties are used for the verification of Class 4 cross sections.

(5) The plastic resistance of cross sections should be verified by finding a stress distribution which is in equilibrium with the internal forces and moments without exceeding the yield strength. This stress distribution should be compatible with the associated plastic deformations.

6.2.2 Section properties

6.2.2.1 Gross cross-section

(1) The properties of the gross cross-section should be determined using the nominal dimensions.

> Section analysis should be performed to determine various section properties using the nominal dimensions of the gross cross-section, and typically these include:
>
> - Cross-sectional area, A;
> - Second moment of area, I;
> - Radius of gyration, i;
> - Section modulus – elastic, W_{el} and plastic, W_{pl}
>
> Refer to Appendix D for a worked example on section analysis of a rolled I-section.

Holes for fasteners need not be deducted, but allowance should be made for larger openings. Splice materials should not be included.

6.2.2.2 Net section

(1) The net area of a cross-section should be taken as its gross area less appropriate deductions for all holes and other openings.

(2) For calculating net section properties, the deduction for a single fastener hole should be the gross cross-sectional area of the hole in the plane of its axis. For countersunk holes, appropriate allowance should be made for the countersunk portion.

For deductions where the holes are staggered, refer to BS EN 1993-1-1 Clause 6.2.2.2(4).

6.2.3 Tension force

(1)P The design value of the tension force N_{Ed} at each cross-section should satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \qquad (6.2)$$

(2) For a section without holes, the design tension resistance $N_{t,Rd}$ should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section, $N_{pl,Rd}$, which should be determined as follows:

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \qquad (6.3a)$$

b) the design ultimate resistance of the net cross-section at holes for fasteners, $N_{u,Rd}$, which should be determined as follows:

$$N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}} \qquad (6.3b)$$

(3) For an angle connected through one leg, see BS EN 1993-1-8 Clause 3.10.3. Similar consideration should also be given to other types of sections connected through outstands.

> As the action is applied at an eccentricity to the centroid of the cross-section, additional moment is induced. For simplicity, instead of designing the cross-section under combined axial force and bending, the cross-sectional area is reduced instead.

6.2.4 Compression force

(1)P The design value of the compression force N_{Ed} at each cross-section should satisfy:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \qquad (6.4)$$

(2) The design resistance of the cross-section for uniform compression $N_{c,Rd}$ should be determined as follows:

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = N_{pl,Rd} \quad \text{for Class 1, 2 or 3 cross-sections} \qquad (6.5)$$

where $N_{pl,Rd}$ is the design plastic resistance of the cross-section for compression.

(3) Fastener holes, except for oversize and slotted holes as defined in EN 1090, need not be allowed for in compression members, provided that they are filled by fasteners.

6.2.5 Bending moment

(1)P The design value of the bending moment M_{Ed} at each cross-section should satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \qquad (6.6)$$

where

$M_{c,Rd}$ is determined considering fastener holes, see (3) to (5).

(2) The design resistance of the cross-section for bending about one principal axis of the cross-section, $M_{c,Rd}$, should be determined as follows:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \quad \text{for Class 1 or 2 cross-sections} \tag{6.7a}$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min} f_y}{\gamma_{M0}} \quad \text{for Class 3 cross-sections} \tag{6.7b}$$

where

$W_{el,min}$ is the minimum elastic section modulus which corresponds with the maximum elastic stress;

$M_{pl,Rd}$ is the design plastic resistance for bending; and

$M_{el,Rd}$ is the design elastic resistance for bending.

(3) For bending about both axes, the methods given in Clause 6.2.9 should be used.

(4) Fastener holes in the tension flange may be ignored in determining the bending resistance provided that for the tension flange:

$$\frac{A_{f,net} 0.9 f_u}{\gamma_{M2}} \geq \frac{A_f f_y}{\gamma_{M0}} \tag{6.8}$$

where

A_f is the area of the tension flange; and

$A_{f,net}$ is the net area of the tension flange.

(5) Fastener holes in the tension zone of the web need not be allowed for, provided that the limit given in (4) is satisfied for the complete tension zone comprising the tension flange plus the tension zone of the web.

(6) Fastener holes, except for oversize and slotted holes, in the compression zone of the cross-section need not be allowed for, provided they are filled by fasteners.

6.2.6 Shear force

(1)P The design value of the shear force V_{Ed} at each cross-section should satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \tag{6.9}$$

where:

$V_{c,Rd}$ is the design shear resistance.

For plastic design, $V_{c,Rd}$ is the design plastic shear resistance, $V_{pl,Rd}$, as given in (2).

For elastic design, $V_{c,Rd}$ is the design elastic shear resistance calculated using (4) and (5).

(2) In the absence of torsion, the design plastic resistance for shear, $V_{pl,Rd}$, is given by:

$$V_{pl,Rd} = \frac{A_v \left(f_y / \sqrt{3}\right)}{\gamma_{M0}} \tag{6.10}$$

where

A_v is the shear area.

(3) The shear area A_v should be determined as shown in Figure 6.1.

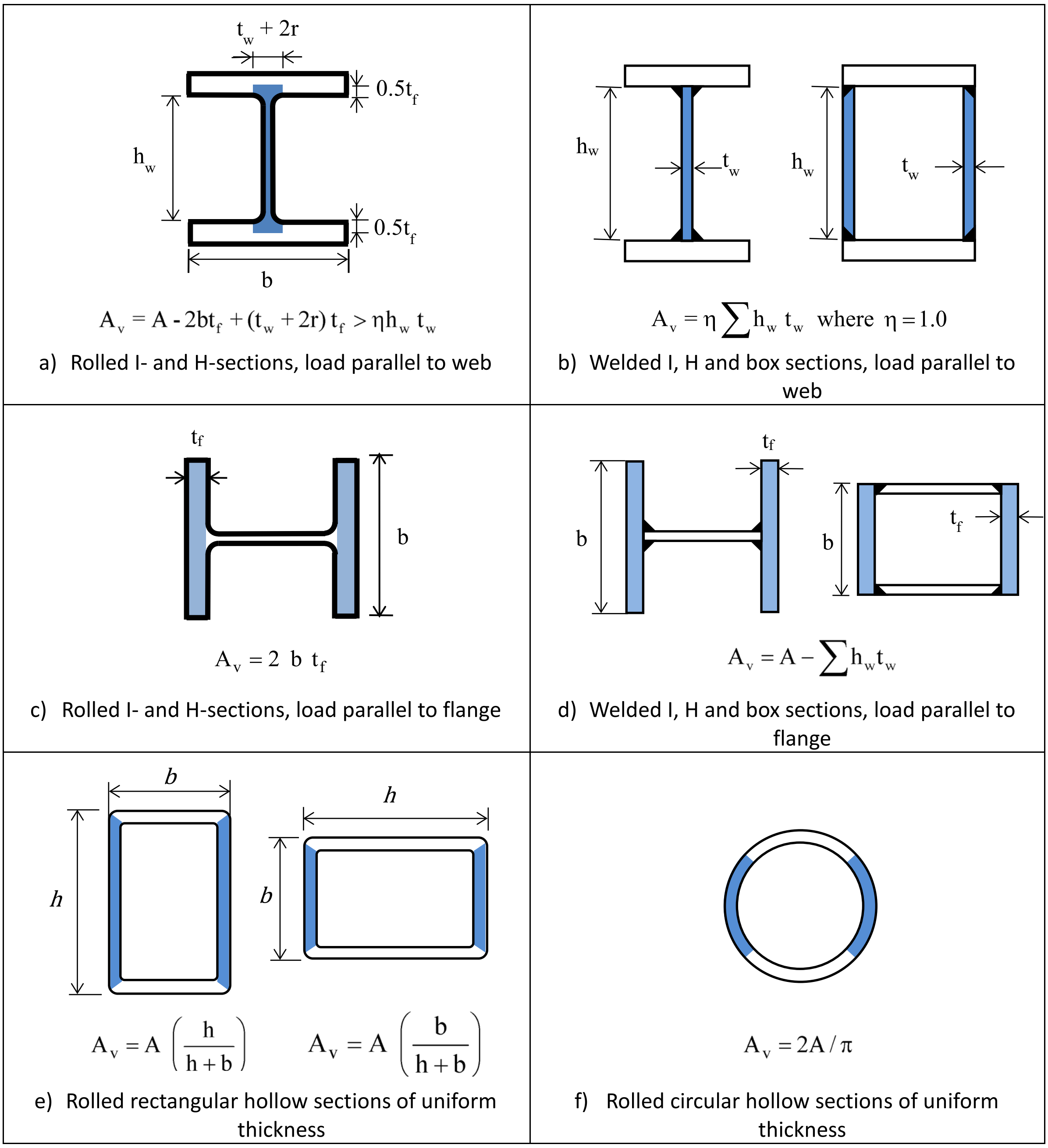


Figure 6.1 Shear areas for various rolled and welded sections [Cl. 6.2.6.(3)]

(4) Shear buckling in webs without intermediate stiffeners is avoided if:

$$\frac{h_w}{t_w} \leq \frac{72\varepsilon}{\eta} \quad \text{where } \eta \text{ may be taken conservatively as 1.} \tag{6.11}$$

Otherwise, the shear buckling resistance must be verified in accordance with EN 1993-1-5.

(5) Fastener holes need not be allowed for in shear verification except in verifying the design shear resistance at connection zones as given in EN 1991-1-8.

A deduction for fastener holes is made when checking block tearing in accordance with EN 1991-1-8 Clause 3.10.2.

Refer to Worked Example I-1 Determination of section resistances of Part I of Appendix D for details. Also refer to Worked Example II-1 Design of a fully restrained steel beam of Part II of Appendix D for details.

6.2.7 Torsion

(1) For members subject to torsion for which distortional deformations may be disregarded, the design value of the torsional moment, T_{Ed}, at each cross-section should satisfy:

$$\frac{T_{Ed}}{T_{Rd}} \leq 1.0 \tag{6.12}$$

where

T_{Rd} is the design torsional resistance of the cross-section.

(2) As a simplification, in the case of a member with a closed hollow cross-section, such as a structural hollow section, it may be assumed that the effects of torsional warping can be neglected.

Also as a simplification, in the case of a member with open cross-section, such as a I- or a H-section, it may be assumed that the effects of St. Venant torsion can be neglected.

6.2.8 Bending and shear force

(1) For a cross-section under a shear force, allowance should be made for its effect on the bending resistance of the cross-section.

(2) When $V_{Ed} < 0.5V_{pl,Rd}$ (see Clause 6.2.6(2)), the effect of the shear force on the bending resistance may be neglected, except where shear buckling reduces the section resistance. See EN 1993-1-5.

(3) When $V_{Ed} \geq 0.5V_{pl,Rd}$, the reduced moment resistance, $M_{y,V,Rd}$ should be taken as the design resistance of the cross-section, calculated using a reduced yield strength, $(1-\rho)f_y$, for the shear area, where $\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$, and $V_{pl,Rd}$ is obtained from Clause 6.2.6 (2).

(4) When torsion is present ρ should be obtained from $\rho = \left(\frac{2V_{Ed}}{V_{pl,T,Rd}} - 1\right)^2$, see Clause 6.2.7, but should be taken as 0 for $V_{Ed} \leq 0.5V_{pl,T,Rd}$.

(5) The reduced plastic moment resistance allowing for the effect of the shear force may be obtained for I-sections with equal flanges and bending about the major axis as follows:

$$M_{y,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w}\right] f_y}{\gamma_{M0}} \quad \text{but } M_{y,V,Rd} \leq M_{y,c,Rd} \qquad (6.13)$$

where:

$M_{y,c,Rd}$ is obtained from Clause 6.2.5(2)

$A_w = h_w t_w$

For I- and H-sections as well as rectangular and square hollow sections, the interaction of bending moments and shear forces is not severe as the induced stresses do not act along the same direction. Hence,

- when $V_{Ed} < 0.5V_{pl,Rd}$, the webs are fully effective to resist both the shear forces and the bending moments. The flanges are fully effective to resist the bending moments.
- when $V_{Ed} \geq 0.5V_{pl,Rd}$, the webs are primarily assigned to resist the shear forces although they may also contribute to resist the bending moments together with the flanges.
- when $V_{Ed} = V_{pl,Rd}$, the webs are fully utilized to resist the shear forces with no contribution to resist the bending moment. The flanges remain to be fully effective to resist the bending moments.

Refer to Worked Example I-2 Cross section resistance under combined bending and shear of Part I of Appendix D for details.

6.2.9 Bending and axial force

6.2.9.1 Class 1 and 2 cross-sections

(1) When an axial force is present, allowance should be made for its effect on the plastic moment resistance.

(2)P For Class 1 and 2 cross-sections, the following criteria should be satisfied:

$$\frac{M_{Ed}}{M_{N,Rd}} \leq 1 \qquad (6.14)$$

where

$M_{N,Rd}$ is the reduced design plastic moment resistance under the axial force N_{Ed}.

(3) For a rectangular solid section without fastener holes, $M_{N,Rd}$, should be taken as:

$$M_{N,Rd} = M_{pl,Rd}\left[1-\left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right] \qquad (6.15)$$

(4) For doubly symmetric I- and H-sections or other flanged sections, allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis when both the following are satisfied:

$$N_{Ed} \leq 0.25 N_{pl,Rd} \text{ and} \qquad (6.16a)$$

$$N_{Ed} \leq \frac{0.5 h_w t_w f_y}{\gamma_{M0}} \qquad (6.16b)$$

For doubly symmetrical I- and H-sections, allowance need not be made for the effect of the axial force on the plastic resistance moment about the z-z axis when:

$$N_{Ed} \leq \frac{h_w t_w f_y}{\gamma_{M0}} \qquad (6.16c)$$

(5) The following approximations may be used for standard rolled I- or H-sections and for welded I- or H-sections with equal flanges:

$$M_{N,y,Rd} = M_{pl,y,Rd}\frac{(1-n)}{(1-0.5a)} \text{ but } M_{N,y,Rd} \leq M_{pl,y,Rd} \qquad (6.17)$$

for $n \leq a$: $\quad M_{N,z,Rd} = M_{pl,z,Rd} \qquad (6.17a)$

for $n > a$: $\quad M_{N,z,Rd} = M_{pl,z,Rd}\left[1-\left(\frac{n-a}{1-a}\right)^2\right] \qquad (6.17b)$

where

$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

$$a = \frac{(A-2bt_f)}{A} \text{ but } a \leq 0.5$$

The following approximations may be used for rectangular structural hollow sections of uniform thickness and for welded box sections with equal flanges and equal webs:

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5a_w} \text{ but } M_{N,y,Rd} \leq M_{pl,y,Rd} \qquad (6.17c)$$

$$M_{N,z,Rd} = M_{pl,z,Rd} \frac{1-n}{1-0.5a_f} \text{ but } M_{N,z,Rd} \leq M_{pl,z,Rd} \qquad (6.17d)$$

where

$$a_w = \frac{A - 2bt_f}{A} \qquad \text{but } a_w \leq 0.5$$

$$a_f = \frac{A - 2ht_w}{A} \quad \text{but } a_f \leq 0.5$$

In general, the effect of combined bending and axial force is more pronounced than that of the effect of combined bending and shear as the induced stresses act along the same (longitudinal) direction. The design formulation using plastic stress blocks utilizes the cross-section resistance more effectively.

(6) For biaxial bending, the following criterion may be used:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1 \qquad (6.18)$$

in which α and β are constants and they may be taken as follows:

	α	β
I- and H- sections	2	5n; but ≥ 1
Circular hollow sections	2	2
Rectangular hollow sections	1.66 / (1-1.13n2)	1.66 / (1-1.13n2)

where

$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

Refer to Worked Example I-3 Cross section resistance under combined bending and axial force of Part I of Appendix D for details.

6.2.9.2 Class 3 cross-sections

(1) For Class 3 cross-sections, the maximum longitudinal stress due to moment and axial force, taking account of fastener holes where relevant, should not exceed f_y / γ_{M0}.

For Class 3 cross-sections, linear elastic interaction of the bending moment with the axial force should be used to determine the maximum longitudinal stress, which should not exceed the design yield strength, f_y / γ_{M0}, i.e. elastic design.

6.2.10 Bending, shear and axial forces

(1) Where shear and axial force are present in a cross-section, allowance should be made for the effect of both shear force and axial force on the moment resistance of the cross-section.

(2) Where $V_{Ed} < 0.5V_{pl,Rd}$, no reduction of the resistances defined for bending and axial force in 6.2.9 need be made, except where shear buckling reduces the section resistance, see EN 1993-1-5.

(3) Where $V_{Ed} \geq 0.5V_{pl,Rd}$, the design resistance of the cross-section to combinations of moment and axial force should be calculated using a reduced yield strength, $(1-\rho)\,f_y$, for the shear area, where $\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$, and $V_{pl,Rd}$ is obtained from 6.2.6 (2).

> Instead of reducing the yield strength, it is also possible to reduce the plate thickness of the relevant part of the cross-section. In general, this approach requires more calculation, but gives smaller reductions, when compared with a reduction in yield strength.

6.3 Buckling Resistances of Members

6.3.1 Uniform members in compression

EN 1993-1-1 covers three modes of buckling when subject to axial compression:

- flexural buckling which may be critical in I- and H-sections, and hollow sections
- torsional buckling which may be critical for cruciform sections with wide outstands
- torsional-flexural buckling which may be critical for asymmetric sections

In general, torsional buckling and torsional flexural buckling are not the critical buckling modes for doubly symmetric I- or H-sections or hollow sections of practical cross-section dimensions and member lengths. Flexural buckling is also commonly known as axial buckling or Euler buckling.

6.3.1.1 Buckling resistance

(1) A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{6.19}$$

where:

N_{Ed} is the design value of the compression force

$N_{b,Rd}$ is the design buckling resistance of the compression member

(2) The design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \tag{6.20}$$

where:

χ is the reduction factor for the relevant buckling mode

6.3.1.2 Buckling curves

(1) For axial compression in members, the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1.0 \tag{6.21}$$

where

$\phi \quad = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$

α is an imperfection factor

$\bar{\lambda}$ is the non-dimensional slenderness

$\bar{\lambda} \quad = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda}$ for Class 1, 2 and 3 cross sections

L_{cr} is the buckling length in the buckling plane considered

i is the radius of gyration about the relevant axis

N_{cr} is the elastic critical force for the relevant buckling mode

$N_{cr} \quad = \frac{\pi^2 EI}{L^2}$ for Class 1, 2 and 3 cross sections

> For rolled or welded I- and H-sections, torsional and torsional-flexural buckling modes are not critical in practical cases.

(2) The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Tables 6.1 and 6.2, and Figure 6.2.

Table 6.1 Imperfection factors for flexural buckling curves

Buckling curve	a_0	a, a*	b, b*	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

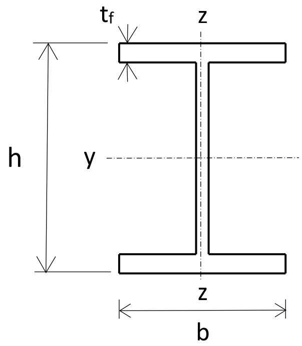
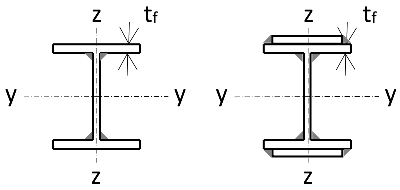
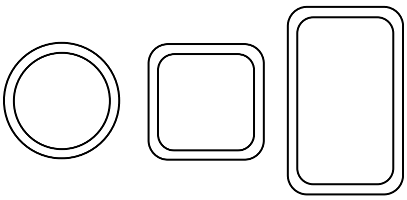
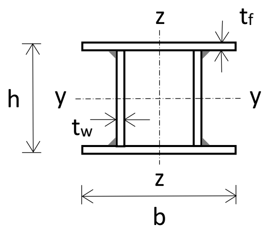
(3) The values of χ corresponding to the non-dimensional slenderness $\bar{\lambda}$ may be obtained from Appendix A.

> The values of χ are tabulated in Table A1 of Appendix A for direct determination of the buckling curves for various steel materials.

(4) For slenderness $\bar{\lambda} \leq 0.2$ or for $N_{Ed} / N_{cr} \leq 0.04$, the buckling effects may be ignored and only cross-sectional checks apply.

> For a column member with a slenderness $\bar{\lambda} \leq 0.2$, the column behaves essentially as a short column. Hence, axial buckling is not critical.
>
> Refer to Worked Example II-3 Design of a steel column under axial compression of Part II of Appendix D for details.

Table 6.2 Selection of flexural buckling curve for a cross-section

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S460	Buckling curve S690
Rolled sections		$h/b > 1.2$	$t_f \leq 40$ mm	y-y z-z	a b	a_0 a_0	- -
			$40 \leq t_f \leq 100$ mm	y-y z-z	b c	a a	- -
		$h/b \leq 1.2$	$t_f \leq 100$ mm	y-y z-z	b c	a a	- -
			$t_f > 100$ mm	y-y z-z	d d	c c	- -
Welded I-sections		$t_f \leq 40$ mm		y-y z-z	b c	b c	a* a*
		$t_f > 40$ mm		y-y z-z	c d	c d	c d
Hollow sections		hot-finished		any	a	a_0	-
		cold-formed		any	c	c	b*
Welded box sections		generally applicable except as below		any	b	b	b
		thick welds: $a > 0.5\ t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c	c

Notes:

a* denotes the use of buckling curve a with the corresponding value of $\bar{\lambda}_0$ at 0.1.

b* denotes the use of buckling curve b with the corresponding value of $\bar{\lambda}_0$ at 0.1.

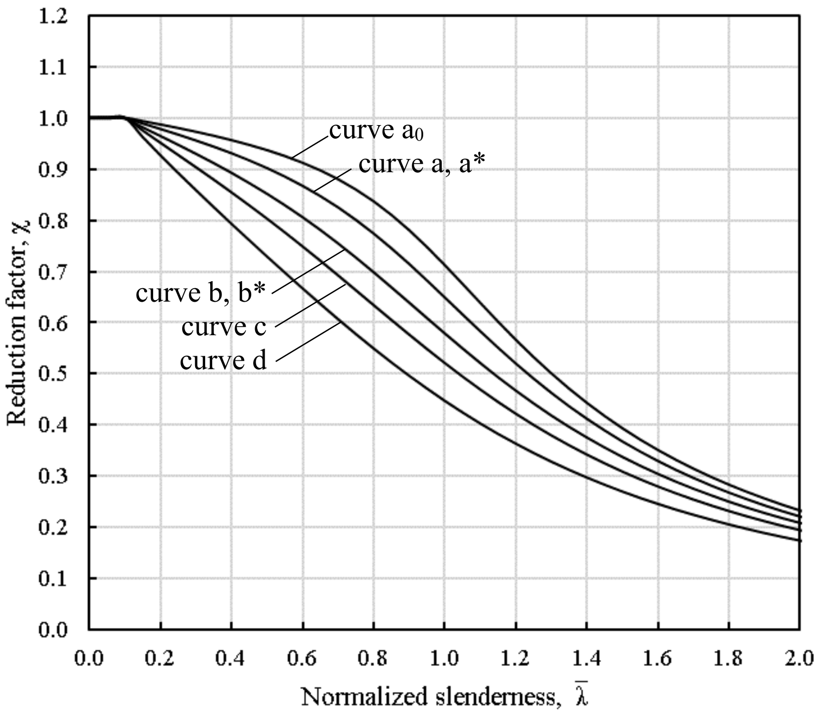


Figure 6.2 Buckling curves for axial compression in members

6.3.2 Uniform members in bending

6.3.2.1 Buckling resistance

(1) A laterally unrestrained member subject to major axis bending should be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0 \qquad (6.22)$$

where

M_{Ed} is the design value of the moment

$M_{b,Rd}$ is the design buckling resistance moment.

(2) Beams with sufficient restraint to the compression flange are not susceptible to lateral-torsional buckling. In addition, beams with cross-sections of circular or square hollow section`s, fabricated circular tubes or square box sections are not susceptible to lateral-torsional buckling.

(3) The design buckling moment resistance of a laterally unrestrained beam should be taken as

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \qquad (6.23)$$

where

W_y is the appropriate section modulus as follows:

$W_y = W_{pl,y}$ for Class 1 and 2 cross-sections

$W_y = W_{el,y}$ for Class 3cross-sections

χ_{LT} is the reduction factor for lateral-torsional buckling.

> The following three different design procedures for members in bending are presented:
>
> Procedure B1- Clause 6.3.2.2 Lateral torsional buckling curves – General case
>
> Procedure B2- Clause 6.3.2.3 Lateral torsional buckling curves for rolled sections or equivalent welded sections
>
> Procedure B3- An alternative procedure recommended by the Steel Designers' Manual

6.3.2.2 Lateral torsional buckling curves - general case

(1) For members of constant cross-sections under bending, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$, should be determined from:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but} \quad \chi_{LT} \le 1.0 \qquad (6.24)$$

where

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

α_{LT} is an imperfection factor

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

M_{cr} is the elastic critical moment for lateral-torsional buckling

Table 6.3 Buckling curves for lateral torsional buckling

<table>
<tr><th>Cross-section</th><th>Limits</th><th colspan="2">Buckling curve</th></tr>
<tr><td>Rolled I-sections</td><td>h / b ≤ 2
h / b > 2</td><td colspan="2">a
b</td></tr>
<tr><td rowspan="2">Welded sections</td><td rowspan="2">h / b ≤ 2
h / b > 2</td><td>S235 ~ S460</td><td>S690</td></tr>
<tr><td>c
d</td><td>b
b</td></tr>
<tr><td>Other cross-sections</td><td>-</td><td colspan="2">d</td></tr>
</table>

Table 6.4 Imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0.21	0.34	0.49	0.76

(2) M_{cr} is based on the gross cross-sectional properties, and takes into account the loading conditions, the real moment distribution and the lateral restraints.

An expression to evaluate M_{cr} is not given in EN 1993-1-1. Refer to Appendix B1 for determination of M_{cr}. Alternatively, the value of M_{cr} may be determined using standard software with eigenvalue analysis.

(3) For slenderness $\bar{\lambda}_{LT} \le \bar{\lambda}_{LT0}$ or for $\frac{M_{Ed}}{M_{cr}} \le \bar{\lambda}_{LT0}^2$, the lateral torsional buckling effects may be ignored, and only cross-sectional checks apply.

For a beam member with a slenderness $\bar{\lambda}_{LT} \le \bar{\lambda}_{LT0}$, the beam behaves essentially as a short beam. Hence, lateral torsional buckling is not critical. A similar conclusion may be drawn for a beam with $\frac{M_{Ed}}{M_{cr}} \le \bar{\lambda}_{LT0}^2$.

Refer to Worked Example II-2 Design of an unrestrained steel beam against lateral torsional buckling of Part II of Appendix D for details.

6.3.2.3 Lateral torsional buckling curves for rolled sections or equivalent welded sections

(1) For rolled or equivalent welded sections in bending, the value of χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ should be determined from:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\ \bar{\lambda}_{LT}^2}} \quad \text{but} \quad \chi_{LT} \leq 1.0 \quad \text{and} \quad \chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^2} \qquad (6.25)$$

where

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}\right) + \beta\ \bar{\lambda}_{LT}^2\right]$$

$\bar{\lambda}_{LT0} = 0.40$ (maximum value)

$\beta = 0.75$ (minimum value)

Table 6.5
Selection of buckling curves for rolled sections and equivalent welded sections

Cross-section	Limits	Buckling curve	
Rolled I-sections	$h / b \leq 2$ $h / b > 2$	b c	
Welded I-sections		S235 ~ S460	S690
	$h / b \leq 2$ $h / b > 2$	c d	b b

Values of χ_{LT} may be obtained from Figure 6.3.

(2) When taking into account the moment distribution between the lateral restraints of members, the reduction factor $\bar{\lambda}_{LT}$ may be modified as follows:

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \quad \text{but} \quad \chi_{LT,mod} \leq 1.0 \qquad (6.26)$$

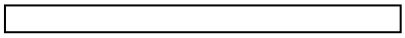
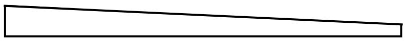
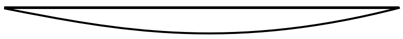
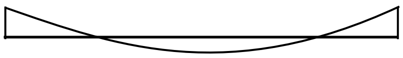
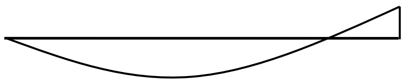
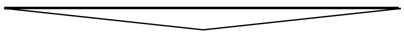
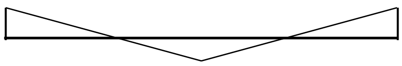
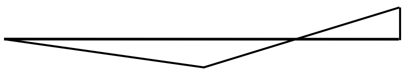
where f is the correction factor for the moment distribution

f $1 - 0.5\left(1 - k_c\right)\left[1 - 2.0\left(\bar{\lambda}_{LT} - 0.8\right)^2\right]$

k_c is a correction factor according to Table 6.6

Refer to Worked Example II-2 Design of an unrestrained steel beam against lateral torsional buckling of Part II of Appendix D for details.

Table 6.6 Correction factors k_c

Moment distribution	k_c
 $\psi = 1$	1.0
 $-1 \leq \psi \leq 1$	$\dfrac{1}{1.33 - 0.33\psi}$
	0.94
	0.90
	0.91
	0.86
	0.77
	0.82

6.3.2.4 An alternative procedure recommended by the Steel Designers' Manual

(1) As an alternative to calculate M_{cr} and hence $\bar{\lambda}_{LT}$, the value of $\bar{\lambda}_{LT}$ may be calculated directly from the expression given below.

(2) Where loads are not destabilising, for simply supported rolled I-, H-sections and channel sections, the non-dimensional slenderness $\bar{\lambda}_{LT}$ is given by:

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UVD\bar{\lambda}_z \sqrt{\beta_w} \qquad (6.27)$$

where:

$\frac{1}{\sqrt{C_1}}$ is a parameter dependant on the shape of the bending moment diagram, which may conservatively be taken as 1.0, or otherwise given in Table 6.7 for loads which are not destabilizing;

U is a section property which is given in section property tables, or may conservatively be taken as 0.9;

V is a parameter related to slenderness, and for symmetric rolled sections where the loads are not destabilising, may be conservatively taken as 1.0 or as

$$V = \frac{1}{\sqrt[4]{1 + \frac{1}{20}\left(\frac{\lambda_z}{h/t_f}\right)^2}}$$

Conservatively, the product of U and V may be taken as 0.9.

$\lambda_z = \frac{kL}{i_z}$, in which k may conservatively be taken as 1.0 for beams supported and restrained against twist at both ends. With certain additional restraint conditions, k may be less than 1.0;

$\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1}$

L is the distance between points of lateral restraint;

λ_1 is a material parameter;

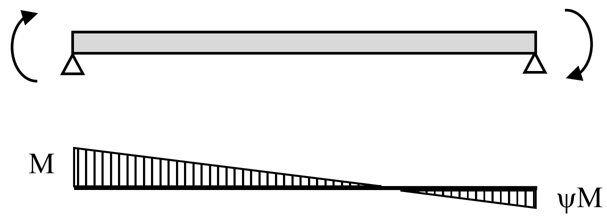
$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\,\varepsilon$

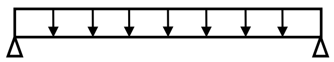
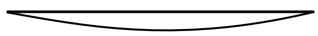
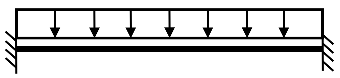
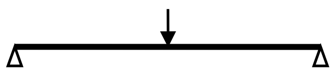
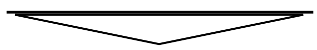
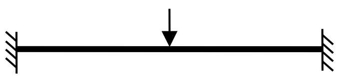
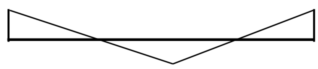
$\beta_w = \frac{W_y}{W_{pl,y}}$

(3) It is conservative to assume that the product $UV = 0.9$ and that $\beta_w = 1.0$.

Where loads are destabilizing, a parameter D should be introduced in the expression for $\bar{\lambda}_{LT}$. The value of D should be taken as 1.2 for simply supported beams. For cantilever beams, the value of D may range from 1.7 to 2.5, depending on the restraints provided at supports. Refer to NCCI SN002 for details.

Table 6.7 Values of $\frac{1}{\sqrt{C_1}}$ and C_1 for various moment conditions (load is not destabilizing)

End Moment Loading	ψ	$\frac{1}{\sqrt{C_1}}$	C_1
 $-1 \leq \psi \leq +1$	+1.00	1.00	1.00
	+0.75	0.92	1.17
	+0.50	0.86	1.36
	+0.25	0.80	1.56
	0.00	0.75	1.77
	-0.25	0.71	2.00
	-0.50	0.67	2.24
	-0.75	0.63	2.49
	-1.00	0.60	2.76

Intermediate Transverse Loading			
		0.94	1.13
		0.62	2.60
		0.86	1.35
		0.77	1.69

The value of the imperfection parameter α_{LT} corresponding to the appropriate buckling curve is given by Table 6.8.

Table 6.8 Imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0.21	0.34	0.49	0.76

(4) Recommendations for the buckling curves are given in Table 6.9.

Table 6.9 Recommendations for the selection of lateral torsional buckling curve

Cross-section	**Limits**	**Buckling curve**
Rolled doubly symmetric I and H sections, and hot-finished hollow sections	$h/b \le 2$	b
	$2 \le h/b \le 3.1$	c
	$h/b > 3.1$	d
Angles (for moments in the major principal plane)		d
All other hot-rolled sections		d
Cold-formed hollow sections	$h/b \le 2$	c
	$h/b > 2$	d

Values of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from Figures 6.3 and 6.4.

(5) Refer to Worked Example II-2 Design of an unrestrained steel beam against lateral torsional buckling of Part II of Appendix D for details.

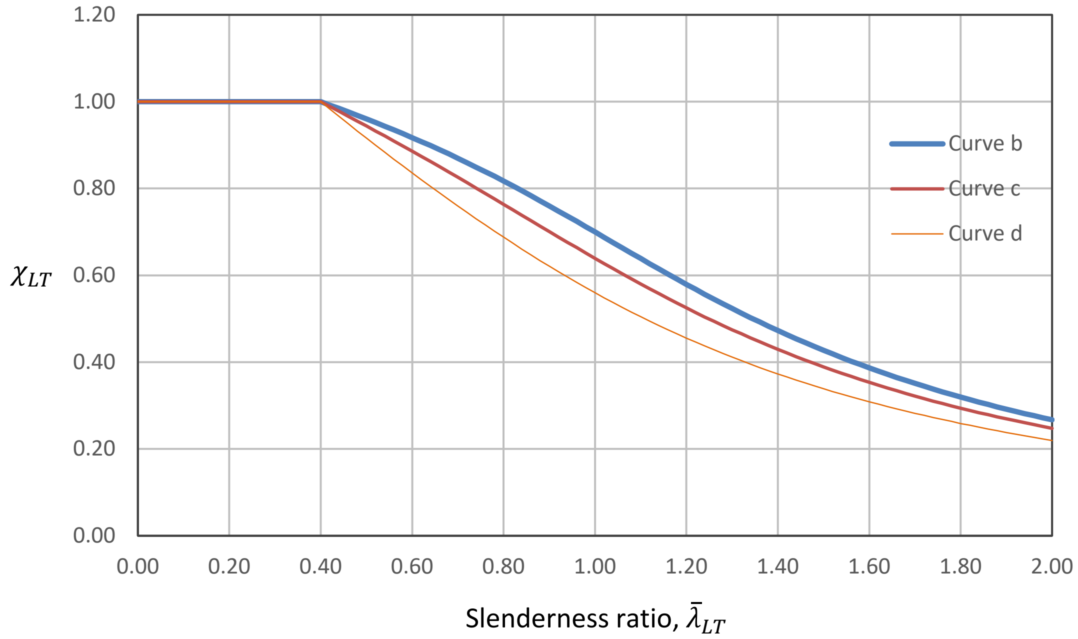


Figure 6.3 Lateral torsional buckling curves for rolled sections

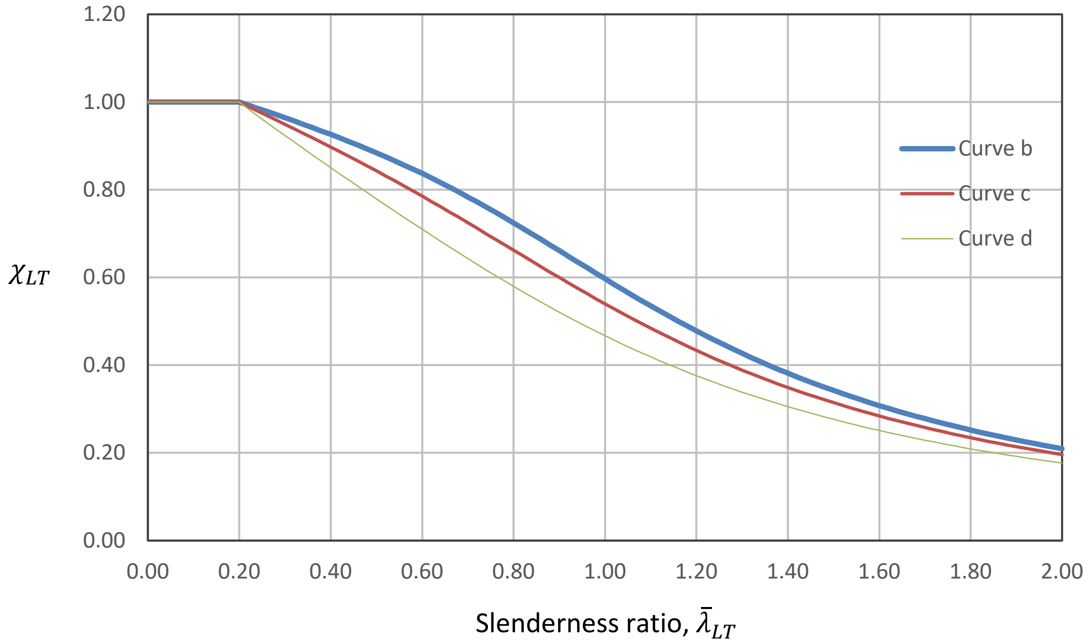


Figure 6.4 Lateral torsional buckling curves for welded sections

Table 6.10 Comparison and design procedure of an unrestrained beam to EN 1993-1-1

<table>
<tr><th rowspan="2">Step</th><th>Procedure B1</th><th>Procedure B2</th><th>Procedure B3</th></tr>
<tr><td>Cl. 6.3.2.2
General case</td><td>Cl. 6.3.2.3
Rolled sections or
equivalent welded sections</td><td>Steel Designers' Manual</td></tr>
<tr><td>1</td><td colspan="3">L_{cr}</td></tr>
<tr><td>2</td><td colspan="2">$M_{cr}, W_y f_y$
M_{cr} is based on gross sectional properties and taken into account loading conditions, moment distributions and lateral restraints.</td><td>$C_1, U, V, \bar{\lambda}_z, \beta_w$
C_1 is based on the shape of the bending moment diagram.</td></tr>
<tr><td>3</td><td colspan="2">$\bar{\lambda}_{LT} = \sqrt{\dfrac{W_y f_y}{M_{cr}}}$</td><td>$\bar{\lambda}_{LT} = \dfrac{1}{\sqrt{C_1}} UV\bar{\lambda}_z \sqrt{\beta_w}$</td></tr>
<tr><td>4</td><td>Buckling curves<table><tr><td rowspan="2">Rolled I-sections</td><td>h/b ≤ 2</td><td>a</td></tr><tr><td>h/b > 2</td><td>b</td></tr><tr><td rowspan="2">Welded sections</td><td>h/b ≤ 2</td><td>c (b)</td></tr><tr><td>h/b > 2</td><td>d (b)</td></tr></table>* (b) for S690 steel</td><td>Buckling curves<table><tr><td rowspan="2">Rolled I-sections</td><td>h/b ≤ 2</td><td>b</td></tr><tr><td>h/b > 2</td><td>c</td></tr><tr><td rowspan="2">Welded sections</td><td>h/b ≤ 2</td><td>c (b)</td></tr><tr><td>h/b > 2</td><td>d (b)</td></tr></table>* (b) for S690 steel</td><td>Buckling curves<table><tr><td rowspan="3">Rolled I-sections</td><td>h/b ≤ 2</td><td>b</td></tr><tr><td>2 < h/b ≤ 3.1</td><td>c</td></tr><tr><td>h/b > 3.1</td><td>d</td></tr><tr><td rowspan="2">Welded sections</td><td>h/b ≤ 2</td><td>c (b)</td></tr><tr><td>h/b > 2</td><td>d (b)</td></tr></table>* (b) for S690 steel</td></tr>
<tr><td>5</td><td colspan="3"><table><tr><td>Buckling curve</td><td>a</td><td>b</td><td>c</td><td>d</td></tr><tr><td>α_{LT}</td><td>0.21</td><td>0.34</td><td>0.49</td><td>0.76</td></tr></table></td></tr>
<tr><td>6</td><td>For all sections,
$\bar{\lambda}_{LT,0} = 0.20$
$\beta = 1.00$</td><td>For rolled and equivalent welded sections,
$\bar{\lambda}_{LT,0} = 0.40$ (max.)
β = 0.75 (min.)</td><td>For rolled sections, hot-finished and cold-formed hollow sections,
$\bar{\lambda}_{LT,0} = 0.40$
β = 0.75 (min.)
For welded sections,
$\bar{\lambda}_{LT,0} = 0.20$
$\beta = 1.00$</td></tr>
<tr><td>7</td><td colspan="3">$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}\right) + \beta\ \bar{\lambda}_{LT}^{2}\right]$</td></tr>
<tr><td>8</td><td colspan="3">$\chi_{LT} = \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{2} - \beta\ \bar{\lambda}_{LT}^{2}}}$</td></tr>
<tr><td>9</td><td>$M_{b,Rd} = \chi_{LT} \dfrac{W_y f_y}{\gamma_{M1}}$</td><td>$\chi_{LT,mod} = \dfrac{\chi_{LT}}{f}$
where
f is based on the moment distribution between lateral restraints of the member.</td><td>$M_{b,Rd} = \chi_{LT} \dfrac{W_y f_y}{\gamma_{M1}}$</td></tr>
<tr><td>10</td><td>-</td><td>$M_{b,Rd} = \chi_{LT,mod} \dfrac{W_y f_y}{\gamma_{M1}}$</td><td>-</td></tr>
</table>

6.3.3 Uniform members in bending and axial compression

(1) For members of structural systems, verification of buckling resistance of doubly symmetric cross-sections may be carried out on the basis of the individual single span members regarded as cut out of the system. Second order effects of the sway system ($P-\Delta$ effects) should be taken into account, either by considering the end moments of the member or by means of appropriate buckling lengths about each axis for the global buckling mode.

(2) Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy}\frac{M_{y,Ed}}{M_{b,Rd}} + k_{yz}\frac{M_{z,Ed}}{M_{cb,z,Rd}} \le 1$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy}\frac{M_{y,Ed}}{M_{b,Rd}} + k_{zz}\frac{M_{z,Ed}}{M_{cb,z,Rd}} \le 1$$

where:

N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the maximum moments about the *y-y* and the *z-z* axes along the member, respectively

$N_{b,y,Rd}$ and $N_{b,z,Rd}$ are the design buckling resistances of the member about the major and the minor axes respectively from Clause 6.3.1.1 (2)

$M_{b,Rd}$ is the design buckling resistance moment from Clause 6.3.2.1(3)

$M_{cb,z,Rd}$ $= \frac{W_{pl,z} f_y}{\gamma_{M1}}$ for Class 1 and 2 sections

$= \frac{W_{el,z} f_y}{\gamma_{M1}}$ for Class 3 sections

$k_{yy}, k_{yz}, k_{zy}, k_{zz}$ are interaction factors, which may be determined from Annex A or B of BS EN 1993-1-1.

The above criteria are based on the expressions in Clause 6.3.3(4) of EN 1993-1-1, interpreted in accordance with ECCS TC8 Rules for Member Stability in EN 1993-1-1 Background documentation and design guidelines.

Annex B is recommended as the simpler approach for manual calculations. Use of either Annex is permitted by the U.K. National Annex.

In some cases, conservative values of the k factors may be sufficient for initial design. The following table gives maximum values, based on Annex B of EN 1993-1-1, and assuming the sections are susceptible to torsional deformations (i.e. not hollow sections).

Interaction factor	Maximum values	
	Class 1 and 2	Class 3
k_{yy}	$C_{my} \times 1.8$	$C_{my} \times 1.6$
k_{yz}	$0.6 \times k_{zz}$	k_{zz}
k_{zy}	1.0	1.0
k_{zz}	$C_{my} \times 2.4$	$C_{mz} \times 2.4$

Appendix D summarizes all the equations necessary to calculate the interaction factors. Alternatively, the values of the interaction factors may simply be read off from various graphs.

Refer to Worked Example II-4 Design of a beam-column under combined compression and major axis bending of Part II of Appendix D for details.

6.3.4 Columns in simple construction

The rules in this clause are based on the NCCI in Access Steel Document SN048 (available from www.access-steel.com) with some different symbols following modifications to the design value given in Clause 6.3.3.

(1) When the criteria given in Clause 6.3.4(2) are satisfied, a column in simple construction subject to combined bending and axial compression may be verified against buckling failure as follows:

$$\frac{N_{Ed}}{N_{min,b,Rd}} + \frac{M_{y,Ed}}{M_{b,Rd}} + 1.5\frac{M_{z,Ed}}{M_{cb,z,Rd}} \leq 1$$

where:

N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the maximum design bending moments about the y-y and the z-z axes along the member.

$N_{min,b,Ed}$ is the lesser of $\dfrac{\chi_y A f_y}{\gamma_{M1}}$ and $\dfrac{\chi_z A f_y}{\gamma_{M1}}$

$M_{b,Rd} = \chi_{LT}\dfrac{W_{pl,y} f_y}{\gamma_{M1}}$

$M_{cb,z,Rd} = \dfrac{W_{pl,z} f_y}{\gamma_{M1}}$

(2) The following criteria must be satisfied to use the verification given in (1):

- The column is a rolled H-section, or equivalent welded sections.
- The cross-section is Class 1, 2 or 3 under compression.
- The bending moment diagram about each axis is linear.
- The column is restrained laterally in both the y-y and the z-z directions at each floor, but it is unrestrained between floors.
- $\psi \leq -0.11$ where ψ is the ratio of the moments at the two ends.
- For a pin ended column ($\psi = 0$), the following alternative criterion must be satisfied to use the simplified interaction expression:

$$\frac{N_{Ed}}{N_{b,y,Rd}} \leq 0.83 \text{ in which } N_{b,y,Rd} = \frac{\chi_y A f_y}{\gamma_{M1}} \text{ (the resistance in the major axis)}$$

Note: $\psi = 0$ *if there is a true pin at one end of the column (such as a base). In this case the simplified interaction expression is only valid if the axial force in the column is less than 83% of its resistance in the major axis.*

(3) Where the criteria in Clause 6.3.4(2) are not satisfied, the method given in Clause 6.3.3 should be used.

(4) The design bending moments should be determined by considering the vertical beam reactions to act at a distance of 100 mm from the face of the column (web or flange).

Section 7 Serviceability Limit States

7.1 General

(1) A steel structure should be designed and constructed such that all relevant serviceability criteria are satisfied.

> Serviceability limit states consider service requirements for a structure or a structural member under normally applied actions. Examples are deflection, human induced vibration, wind induced oscillation and durability.

(2) The basic requirements for serviceability limit states are given in Clause 3.4 of EN 1990.

(3) Any serviceability limit state and the associated loading model as well as the associated analysis model should be specified for a structure.

(4) Where plastic global analysis is used for ultimate limit state design, plastic redistribution of forces and moments at the serviceability limit state should be considered accordingly.

> The serviceability actions should be taken as the characteristic values of the actions, i.e. unfactored.

7.2 Serviceability Limit States for Buildings

7.2.1 Vertical deflections

(1) With reference to EN 1990 – Annex 1.4 limits for vertical deflection according to Figure A1.1 should be specified for each structure and agreed with the client.

7.2.2 Horizontal deflections

(1) With reference to EN 1990 – Annex 1.4 limits for horizontal deflection according to Figure A1.2 should be specified for each structure and agreed with the client.

7.2.3 Dynamic effects

(1) With reference to EN 1990 – Annex 1.4.4, vibrations of structures which are accessible to the public should be limited to avoid significant discomfort to users, and limits should be specified for each structure and agreed with the client.

> Deflections or deformations under all actions should not impair the resistance or the effective functioning of a structure, a structural member, a supporting member or its components, nor cause damage to finishes. For typical structures, the deflection limits given in the following table are recommended.

Table 7.1 Suggested limits for vertical deflection due to characteristic combination (variable actions only)

a)	*Deflection of profiled steel sheeting*	
	Vertical deflection during construction when the effects of ponding are not taken into account	Span/180 (but ≤ 20 mm)
	Vertical deflection during construction when the effects of ponding are taken into account	Span/130 (but ≤ 30 mm)
	Vertical deflection of roof cladding under self-weight and wind action	Span/90 (but ≤ 30 mm)
	Lateral deflection of wall cladding under wind action	Span/120 (but ≤ 30 mm)
b)	*Vertical deflection of composite slab*	
	Due to imposed actions	Span/350 (but ≤ 20 mm)
	Due to the total actions plus due to prop removal (if any) less due to self-weight of the slab	Span/250
c)	*Vertical deflection of beams* *- due to imposed actions*	
	Cantilevers	Length/180
	Beams carrying plasters or other brittle finishes	Span/360
	Other beams except purlins and sheeting rails	Span/200
	Purlins and sheeting rails	To suit cladding
d)	*Horizontal deflection of columns* *- due to imposed actions and wind actions*	
	Horizontal drift at topmost storey of buildings	Height/500
	Horizontal drift at top of a single storey portal not supporting human	To suit cladding
	Relative inter-storey drift	Storey height/400
	Columns in portal frame buildings	To suit cladding
	Columns supporting crane runways	To suit crane runway
e)	*Crane girders*	
	Vertical deflection due to static vertical wheel actions from overhead traveling cranes	Span/600
	Horizontal deflection (calculated on the top flange properties alone) due to horizontal crane actions	Span/500
f)	*Trusses*	
	Typical trusses not carrying brittle panels	Span/200

Note: Pre-camber in an unloaded structural member may be used to reduce the calculated deflection of that member under the loading conditions.

7.3 Wind-induced Oscillation

Vibration and oscillation of a structure should be limited to avoid discomfort to users and damage to contents. For special structures, including long-span bridges, large stadium roofs and chimneys, wind tunnel model tests are recommended to provide data for wind resistant design to meet serviceability limits.

7.4 Wind Sensitive Buildings and Structures

A design procedure which incorporates dynamic analysis in addition to static analysis should be undertaken for wind sensitive buildings and structures. Structures with low natural frequencies or large height-to-least dimension ratios should receive special checking. Reference should be made to the Code of Practice on Wind Effects in Hong Kong (2019).

For slender, flexible and lightly damped tall buildings and structures, those with a long afterbody or complex geometry, and those with an eccentricity between mass and stiffness centres, aeroelastic instabilities such as lock-in, galloping and flutter may cause large amplitude crosswind responses. Specialist advice and wind tunnel model test are recommended to provide data for wind resistant design to meet serviceability limits.

Refer to the Code of Practice for the Structural Use of Steel for details.

Section 8 Design Data for Rolled and Welded Sections

8.1 General

Tabulated design data are essential for practicing engineers to perform structural design. For structural steel design, section dimensions are the basic data, and rational use of these data gives important structural quantities, i.e. section properties and resistances of both rolled and welded sections, enabling designers to establish structural adequacy against strength requirements in ultimate limit states as well as structural performance against deformation or vibration in serviceability limit states.

In this Section, design data on section dimensions and properties as well as section resistances of both rolled and welded sections with practical steel materials are provided to assist structural engineers to perform effective structural steel design. Table 8.1 presents the types of rolled and welded sections covered in the present Section. Typical cross-sections of these rolled and welded sections are illustrated in Figure 8.1.

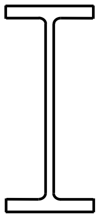
Table 8.1 Ranges of rolled and welded sections

Rolled sections	Welded sections
Rolled I-section: ***I-section***	Equivalent welded I-section: ***EWI-section***
Rolled H-section: ***H-section***	Equivalent welded H-section: ***EWH-section***
Hot-finished circular hollow section: ***CHS***	Equivalent cold-formed circular hollow section: ***EWCHS***
Hot-finished rectangular hollow section: ***RHS***	Equivalent cold-formed rectangular hollow section: ***EWRHS***
Hot-finished square hollow section: ***SHS***	Equivalent cold-formed square hollow section: ***EWRHS***

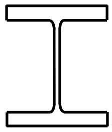
Rolled Sections

It should be noted that both I- and H-sections are manufactured to BS 4-1 while all the hot-finished hollow sections are manufactured to EN 10210-2. All rolled I- and H-sections given in BS 4-1 have been included in the present Section, but only selected hot-finished hollow sections specified in EN 10210-2 with a dimension larger than 100 mm are considered. All of these rolled sections are assumed to be manufactured to EN 10025 and EN10210-1, and hence, they are commonly considered as steel Class E1 Steel Materials with a material class factor, γ_{Mc} = 1.0 as discussed in Section 1.9. Resistances of all these sections with common steel grades, i.e. S275 and S355 steel materials, are tabulated in a systematic manner for practical design. Table 8.2 summarizes various design information provided for the rolled sections covered in this Section.

Rolled sections:



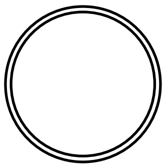
I-section



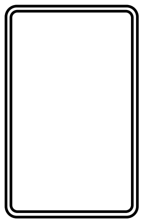
H-section

I- and H-sections to:
- EN 10025 on materials
- BS4-1 on dimensions

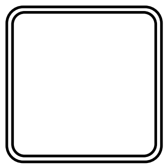
CHS, RHS and SHS to:
- EN 10210-1 on materials
- EN 10210-2 on dimensions



Circular hollow section
CHS

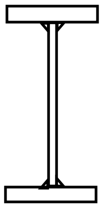


Rectangular hollow section
RHS

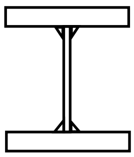


Square hollow section
SHS

Welded sections:



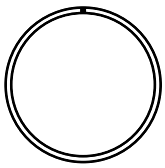
Equivalent welded
I-section
EWI-section



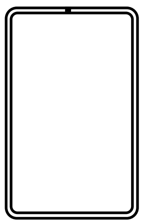
Equivalent welded
H-section
EWH-section

EWIS and EWHS to:
- GB/T 700 & GB/T 1591 on materials
- design methods in Section 8.4

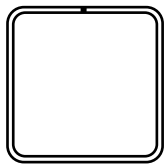
EWCHS, EWRHS and EWSHS to:
- GB/T 6725 & GB/T 8162 on materials
- design methods in Section 8.4 as well as GB/T 6728 & GB/T 17395 on dimensions.



Equivalent cold-formed
circular
hollow section
EWCHS



Equivalent cold-formed
rectangular
hollow section
EWRHS



Equivalent cold-formed
square
hollow section
EWSHS

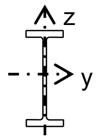
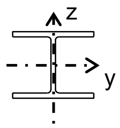
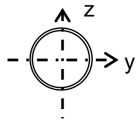
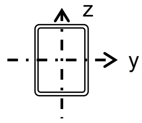
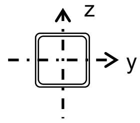
Figure 8.1 Cross-sections of typical rolled and welded sections

Detailing rules of welding for I- and H-sections

(1) The height of the weld root, r, is assumed to be equal to the thickness of the web, t_w, or at least 0.7 times the flange thickness, i.e. 0.7 t_f, whichever is smaller.

(2) To ensure welding quality, r should not be smaller than 8.0 mm nor larger than 16.0 mm.

Table 8.2 Summary of design information for rolled sections

	I-section 	***H-section*** 	***CHS*** 	***RHS*** 	***SHS*** 
Dimensions and properties	72 sections	31 sections	52 sections	44 sections	32 sections
Resistances for S275 steel	✓	✓	✓	✓	✓
Resistances for S355 steel	✓	✓	✓	✓	✓

Note: All these rolled sections are assumed to be Class E1 Steel Materials with a material class factor $\gamma_{Mc} = 1.0$ as discussed in Section 1.9.

Welded sections

Equivalent welded I- and H-sections

All the equivalent welded I- and H-sections are fabricated with steel plates to GB/T 700 and GB/T 1591 with standard thicknesses. For simplicity, the following plate thicknesses are assumed:

6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm
25.0 mm	30.0 mm	40.0 mm
50.0 mm	60.0 mm	80.0 mm

It is envisaged that with a rational combination of these plate thicknesses in the flanges and the webs of the sections, a series of welded sections with similar section depths and flange widths are readily manufactured covering a wide range of section properties and resistances for practical design. These section properties and resistances are similar to those rolled sections in the same series of section designations. Moreover, resistances of all these sections with common steel grades, i.e. Q235, Q275, Q355, Q460 and Q690 steel materials, are tabulated in a systematic manner for practical design.

Equivalent cold formed hollow sections

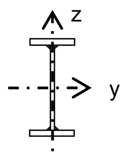
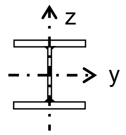
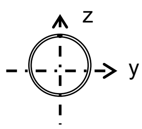
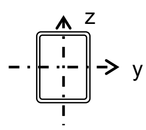
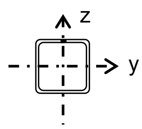
All the equivalent cold-formed hollow sections are manufactured with steel plates to GB/T 6725 and GB/T 8162 while their dimensions are manufactured to GB/T 6728 and GB/T 17395. The following plate thicknesses are assumed:

6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm

Depending on the performance of material properties as well as the demonstration of quality assurance system during manufacturing, Chinese Steel Materials may be classified as Class E1 or E2 Steel Materials with a material class factor, $\gamma_{Mc} = 1.0$ or 1.1 respectively as discussed in Section 1.9. Resistances of all these sections with common steel grades, i.e. Q275, Q355, Q460 and Q690 steel materials, are tabulated in a systematic manner for practical design.

All of these welded sections are proposed as equivalent welded sections to those rolled sections based on various structural requirements, such as compression and bending resistances. Standard welding procedures are assumed to be applied effectively during their fabrication. Table 8.3 summarizes various items of design information provided for the equivalent welded sections covered in this Section.

Table 8.3 Summary of design information for equivalent welded sections

	EWI-section 	***EWH-section*** 	***EWCHS*** 	***EWRHS*** 	***EWSHS*** 
Dimensions and properties	72 sections	31 sections	68 sections	44 sections	32 sections
Resistances for Q235 steel	✓	✓	---	---	---
Resistances for Q275 steel	✓	✓	✓	✓	✓
Resistances for Q355 steel	✓	✓	✓	✓	✓
Resistances for Q460 steel	✓	✓	✓	✓	✓
Resistances for Q690 steel	✓	✓	✓	✓	✓

Note: Depending on the performance of material properties as well as the demonstration of quality assurance system during manufacturing, Chinese Steel Materials may be classified as Class E1 or E2 Steel Materials with a material class factor, γ_{Mc} = 1.0 or 1.1 respectively.

For ease of presentation, all welded sections presented in Design Tables 19 to 51 are assumed to be made of Class E2 Steel Materials with γ_{Mc} = 1.1 for Q235, Q275, Q355 and Q460 steel, and of Class E1 Steel Materials with γ_{Mc} = 1.0 for Q690 Steel.

8.2 Design strengths

For rolled sections with S275 and S355 Steel Materials, the design strengths of the steel sections with steel plates of various thicknesses are presented in Table 8.4.

Table 8.4 Design strengths of different steel grades of rolled sections
Class E1 Steel Materials with $\gamma_{Mc} = 1.0$

Steel grade	Thickness, t (mm)	Yield strength, R_{eH} (N/mm²)	Design strength, f_y (N/mm²)
S275	t ≤ 16	275	275
	16 < t ≤ 40	265	265
	40 < t ≤ 63	255	255
	63 < t ≤ 80	245	245
S355	t ≤ 16	355	355
	16 < t ≤ 40	345	345
	40 < t ≤ 63	335	335
	63 < t ≤ 80	325	325

For welded sections with Q235, Q275, Q355, Q460 and Q690 Steel Materials, the design strengths of the steel sections with steel plates of different thicknesses are presented in Table 8.5.

Table 8.5 Design strengths of different steel grades of welded sections
a) Class E2 Steel Materials with $\gamma_{Mc} = 1.1$

Steel grade	Thickness, t (mm)	Yield strength, R_{eH} (N/mm²)	Design strength, f (N/mm²)
Q235	t ⩽ 16	235	214
	16 < t ⩽40	225	205
	40 < t ⩽100	215	195
Q275	t ⩽ 16	275	250
	16 < t ⩽ 40	265	241
	40 < t ⩽ 60	255	232
	60 < t ⩽ 100	245	223
Q355	t ⩽ 16	355	323
	16 < t ⩽ 40	345	314
	40 < t ⩽ 63	335	305
	63 < t ⩽ 80	325	295
	80 < t ⩽ 100	315	286
Q460	t ⩽ 16	460	418
	16 < t ⩽ 40	450	409
	40 < t ⩽ 63	430	391
	63 < t ⩽ 80	410	373
	80 < t ⩽ 100	400	364

b) Class E1 Steel Materials with $\gamma_{Mc} = 1.0$

Steel grade	Thickness, t (mm)	Yield strength, R_{eH} (N/mm²)	Design strength, f (N/mm²)
Q690	t ⩽ 16	690	690
	16 < t ⩽ 40	680	680
	40 < t ⩽ 63	670	670
	63 < t ⩽ 80	650	650

Note: For high quality Chinese Q690 steel, their design strengths are assigned to be the same as their yield strengths, i.e., $\gamma_{Mc} = 1.0$.

8.3 Section Classification

Section classification of all the rolled and the welded sections are performed according to Clause 5.5 of EN 1993-1-1. Depending on the susceptibility of various plate elements of the sections against local buckling under compression, plastic or elastic cross-section resistances may be readily mobilized for Class 1, 2 or 3 sections. For Class 4 sections, elastic properties are not applicable, and provisions given in EN 1993-1-8 should be considered.

Table 8.6 presents the section classification rules given in Table 5.2 of EN 1993-1-1 for I- and H-sections while Table 8.7 presents various limiting ratios of the geometric parameters of the sections, namely, c_f / t_f and d / t_w for section classification under i) compression, ii) bending about the major axis, and iii) bending about the minor axis.

Table 8.6 Section classification rules for I- and H-sections

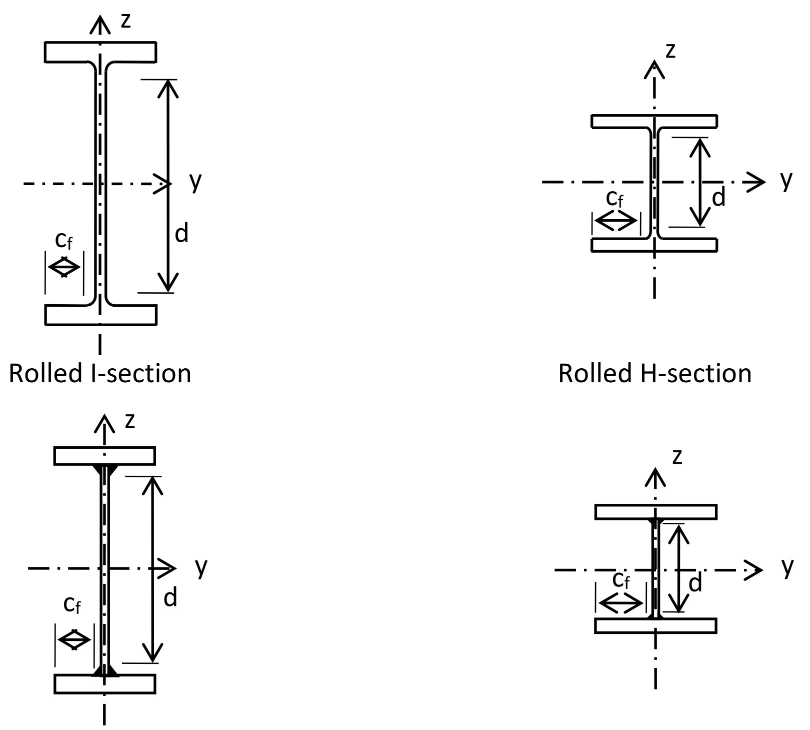


Plate element	Class 1	Class 2	Class 3	Class 4
Internal part under compression, d/t_w	≤ 33 ε	≤ 38 ε	≤ 42 ε	> 42 ε
Internal part under bending, d/t_w	≤ 72 ε	≤ 83 ε	≤ 124 ε	> 124 ε
Outstanding part under compression, c_f/t_f	≤ 9 ε	≤ 10 ε	≤ 14 ε	>14 ε

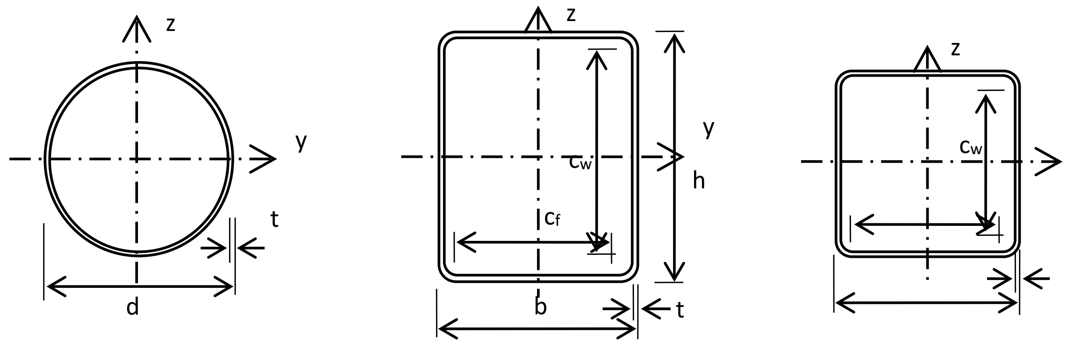
$\varepsilon = \sqrt{235/f_y}$	f_y (N/mm^2)	235	275	355	460	690
	ε	1.00	0.92	0.81	0.71	0.58

Table 8.7 Limiting ratios of section classification for I- and H-sections

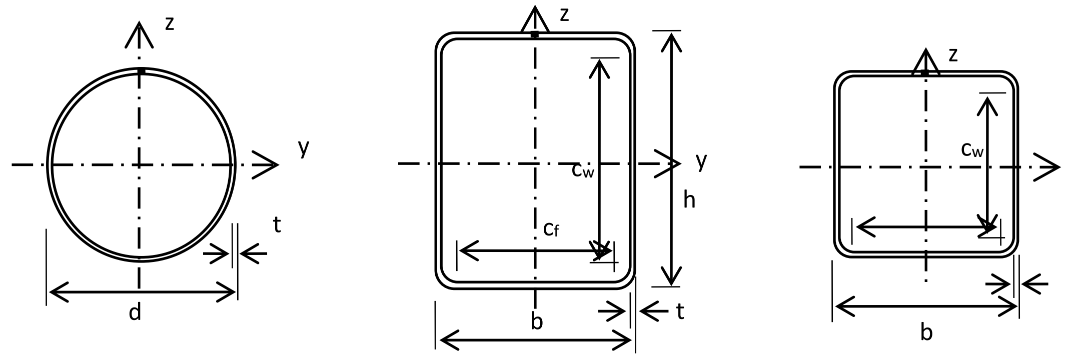
I- and H-sections under compression							
Plate element		Flange			Web		
Geometrical parameter		c_f / t_f			d / t_w		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class3**
Rolled section	S275	8.3	9.2	12.9	30.5	35.1	38.8
	S355	7.3	8.1	11.4	26.8	30.9	34.2
Welded section	Q235	9.0	10.0	14.0	33.0	38.0	42.0
	Q275	8.3	9.2	12.9	30.5	35.1	38.8
	Q355	7.3	8.1	11.4	26.8	30.9	34.2
	Q460	6.4	7.1	10.0	23.6	27.2	30.0
	Q690	5.3	5.8	8.2	19.3	22.2	24.5
I- and H-sections under bending about the major axis							
Plate element		Flange			Web		
Geometrical parameter		c_f / t_f			d / t_w		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class3**
Rolled section	S275	8.3	9.2	12.9	66.6	76.7	114.6
	S355	7.3	8.1	11.4	58.6	67.5	100.9
Welded section	Q235	9.0	10.0	14.0	72.0	83.0	124.0
	Q275	8.3	9.2	12.9	66.6	76.7	114.6
	Q355	7.3	8.1	11.4	58.6	67.5	100.9
	Q460	6.4	7.1	10.0	51.5	59.3	88.6
	Q690	5.3	5.8	8.2	42.0	48.4	72.4
I- and H-sections under bending about the minor axis							
Plate element		Flange			Web		
Geometrical parameter		c_f / t_f			d / t_w		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class3**
Rolled section	S275	8.3	9.2	12.8	*Not applicable*		
	S355	7.3	8.1	11.2			
Welded section	Q235	9.0	10.0	13.8	*Not applicable*		
	Q275	8.3	9.2	12.8			
	Q355	7.3	8.1	11.2			
	Q460	6.4	7.1	9.9			
	Q690	6.4	7.1	8.1			

Table 8.8 presents the section classification rules given in Table 5.2 of EN 1993-1-1 for hot-finished and cold-formed hollow sections while Table 8.9 presents various limiting ratios of the geometric parameters of the hollow sections, namely, c_f / t and c_w / t for section classification under i) compression, ii) bending about the major axis, and iii) bending about the minor axis.

Table 8.8 Section classification of hollow sections



Hot-finished circular hollow section ***CHS***	Hot-finished rectangular hollow section ***RHS***	Hot-finished square hollow section ***SHS***



Equivalent cold-formed circular hollow section ***EWCHS***	Equivalent cold-formed rectangular hollow section ***EWRHS***	Equivalent cold-formed square hollow section ***EWSHS***

Plate element	Class 1	Class 2	Class 3	Class 4
Internal parts under compression, c_f / t	≤ 33 ε	≤ 38 ε	≤ 42 ε	> 42 ε
Internal parts under bending, c_w / t	≤ 72 ε	≤ 83 ε	≤ 124 ε	> 124 ε

Circular section under compression and / or bending, d / t	≤ $50\varepsilon^2$	≤ $70\varepsilon^2$	≤ $90\varepsilon^2$	> $90\varepsilon^2$

$\varepsilon = \sqrt{235/f_y}$	f_y (N/mm^2)	235	275	355	460	690
	ε	1.00	0.92	0.81	0.71	0.58

Table 8.9 Limiting ratios of section classification for hollow sections

a) Rectangular and square hollow sections

Rectangular and square hollow sections under compression							
Plate element		Flange			Web		
Geometrical parameter		c_f / t			c_w / t		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class 3**
Rolled section	S275	30.5	35.1	38.8	30.5	35.1	38.8
	S355	26.8	30.9	34.2	26.8	30.9	34.2
Welded section	Q235	33.0	38.0	42.0	33.0	38.0	42.0
	Q275	30.5	35.1	38.8	30.5	35.1	38.8
	Q355	26.8	30.9	34.2	26.8	30.9	34.2
	Q460	23.6	27.2	30.0	23.6	27.2	30.0
	Q690	19.3	22.2	24.5	19.3	22.2	24.5
Rectangular and square hollow sections under bending about the major axis							
Plate element		Flange			Web		
Geometrical parameter		c_f / t			c_w / t		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class 3**
Rolled section	S275	30.5	35.1	38.8	66.6	76.7	114.6
	S355	26.8	30.9	34.2	58.6	67.5	100.9
Welded section	Q235	33.0	38.0	42.0	72.0	83.0	124.0
	Q275	30.5	35.1	38.8	66.6	76.7	114.6
	Q355	26.8	30.9	34.2	58.6	67.5	100.9
	Q460	23.6	27.2	30.0	51.5	59.3	88.6
	Q690	19.3	22.2	24.5	42.0	48.4	72.4
Rectangular and square hollow sections under bending about the minor axis							
Plate element		Web			Flange		
Geometrical parameter		c_w / t			c_f / t		
Section classification		**Class 1**	**Class 2**	**Class 3**	**Class 1**	**Class 2**	**Class3**
Rolled section	S275	30.5	35.1	38.8	66.6	76.7	114.6
	S355	26.8	30.9	34.2	58.6	67.5	100.9
Welded section	Q235	33.0	38.0	42.0	72.0	83.0	124.0
	Q275	30.5	35.1	38.8	66.6	76.7	114.6
	Q355	26.8	30.9	34.2	58.6	67.5	100.9
	Q460	23.6	27.2	30.0	51.5	59.3	88.6
	Q690	19.3	22.2	24.5	42.0	48.4	72.4

b) Circular hollow sections

Circular hollow sections under i) compression, and ii) bending				
Plate element		Circular section		
Geometrical parameter		d / t		
Section classification		**Class 1**	**Class 2**	**Class 3**
Rolled section	S275	42.7	59.8	76.9
	S355	33.1	46.3	59.6
Welded section	Q235	50.0	70.0	90.0
	Q275	42.7	59.8	76.9
	Q355	33.1	46.3	59.6
	Q460	25.5	35.8	46.0
	Q690	17.0	23.8	30.7

8.4 Rolled Sections

A wide range of rolled sections covered in this Section for application are summarized in Table 8.10. It should be noted that

i) all rolled I-sections available in BS 4-1 are included in the Design Tables, i.e. I-sections from 127 x 76 x 13 kg/m to 914 x 419 x 388 kg/m with a total of 72 sections.

ii) all rolled H-sections available in BS 4-1 are included in the Design Tables, i.e. H-sections from 152 x152 x 23 kg/m to 356 x 406 x 634 kg/m with a total of 31 sections.

iii) selected hot-finished circular hollow sections with standard plate thicknesses available in EN 10210-2 are included in the Design Tables, i.e. CHS from 139.7 x 6.3 mm to 813.0 x 20.0 mm with a total of 55 sections.

iv) selected hot-finished rectangular hollow sections with standard plate thicknesses available in EN 10210-2 are included in the Design Tables, i.e. RHS from 120 x 80 x 6.3 mm to 500 x 300 x 20.0 mm with a total of 44 sections.

v) selected hot-finished square hollow sections with standard plate thicknesses available in EN 10210-2 are included in the Design Tables, i.e. SHS from 100 x 100 x 6.3 mm to 400 x 400 x 20.0 mm with a total of 32 sections.

Table 8.10 Full ranges of typical rolled sections available for application

I-section		H-section	Hot-finished circular hollow section	Hot-finished rectangular hollow section	Hot-finished square hollow section
914x419x388#	457x191x98	356x406x634#	139.7x6.3	120x80x6.3	100x100x6.3
x343#	x89	x551#	x8.0	x8.0	x8.0
914x305x289#	x82	x467#	168.3x6.3	160x80x6.3	150x150x6.3
x253#	x74	x393#	x8.0	x8.0	x8.0
x224#	x67	x340#	x10.0	x10.0	x10.0
x201#	457x152x82	x287#	x12.5	200x100x6.3	200x200x6.3
838x292x226#	x74	x235#	219.1x6.3	x8.0	x8.0
x194#	x60	356x368x202#	x8.0	x10.0	x10.0
x176#	x60	x177#	x10.0	200x150x6.3	x12.5
762x267x197	x52	x153#	x12.5	x8.0	220x220x6.3
x173	406x178x74	x129#	273.0x6.3	x10.0	x8.0
x147	x67	305x305x283	x8.0	250x150x6.3	x10.0
x134	x60	x240	x10.0	x8.0	x12.5
686x254x170	x54	x198	x12.5	x10.0	250x250x6.3
x152	406x140x46	x158	323.9x6.3	x12.5	x8.0
x140	x39	x137	x8.0	260x180x6.3	x10.0
x125	356x171x67	x118	x10.0	x8.0	x12.5
610x305x238	x57	x97	x12.5	x10.0	x16.0
x179	x51	254x254x167	x16.0	x12.5	300x300x6.3
x149	x45	x132	355.6x6.3	x16.0	x8.0
610x229x140	356x127x39	x107	x8.0	300x200x6.3	x10.0
x125	x33	x89	x10.0	x8.0	x12.5
x113	305x165x54	x73	x12.5	x10.0	x16.0
x101	x46	203x203x86	x16.0	x12.5	350x350x8.0
533x210x122	x40	x71	406.4x8.0	x16.0	x10.0
x109	305x127x48	x60	x10.0	350x250x6.3	x12.5
x101	x42	x52	x12.5	x8.0	x16.0
x92	x37	x46	x16.0	x10.0	400x400x8.0
x82	305x102x33	152x152x37	x20.0	x12.5	x10.0
	x28	x30	457.0x8.0	x16.0	x12.5
	x25	x23	x10.0	400x200x6.3	x16.0
	254x146x43		x12.5	x8.0	x20.0
	x37		x16.0	x10.0	
	x31		x20.0	x12.5	
	254x102x28		508.0x8.0	x16.0	
	x25		x10.0	450x250x8.0	
	x22		x12.5	x10.0	
	203x133x30		x16.0	x12.5	
	x26		x20.0	x16.0	
	203x102x23		610.0x8.0	500x300x8.0	
	178x102x19		x10.0	x10.0	
	152x89x16		x12.5	x12.5	
	127x76x13		x16.0	x16.0	
			x20.0	x20.0	
			711.0x10.0		
			x12.5		
			x16.0		
			x20.0		
			813.0x10.0		
			x12.5		
			x16.0		
			x20.0		
Number of sections:	72	31	52	44	32
Total:					231

\# Limited availability.

Full series of I- and H-sections have been provided while only selected hot-finished circular, rectangular and square hollow sections are included. Section resistances for S275 and S355 steel materials are tabulated separately.

8.5 Equivalent Welded Sections

Design data on equivalent welded sections are provided to assist structural engineers to use welded sections readily whenever necessary. The design methods for equivalent welded sections are described in the following sections.

8.5.1 Equivalent welded I-Sections

(1) The section depth h of the welded I-sections is selected to be equal to that of the rolled I-sections under consideration plus a maximum of 5 mm.

(2) The plate thicknesses of the flanges and the webs of the welded I-sections are:

6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm
25.0 mm	30.0 mm	40.0 mm

(3) In most cases, both the web thickness and the flange thickness of the equivalent welded I-sections are taken to be larger than those of the rolled I-sections as far as rational, as shown in Figure 8.2. Moreover, the flange width of the welded I-sections is selected in such a way as to achieve a value of cross-sectional area which is at least 10% larger than that of the rolled I-sections.

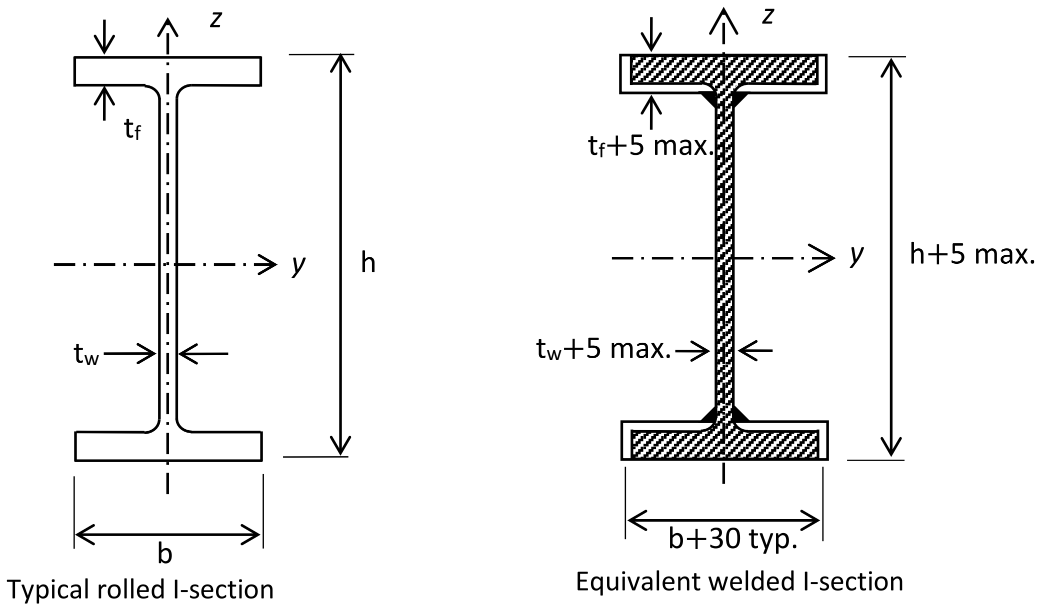


Figure 8.2 Design method of equivalent welded I-sections

(4) For a rolled I-section with a web thickness or a flange thickness of odd values, for example, t_w = 8.7 mm or t_f = 13.2 mm, the web thickness and the flange thickness of the welded I-section are then selected to be 8.0 mm and 12.0 mm (rather than 10.0 mm and 16.0 mm) respectively, i.e. of thinner plates. In order to achieve equivalency, the flange width of the welded I-section will then be increased significantly, when compared with that of the rolled I-section, in order to acquire a larger moment resistance of that of the rolled I-section.

8.5.2 Equivalent welded H-sections

(1) The section depth h of the welded H-sections is selected to be equal to that of the rolled H-sections under consideration plus a maximum of 5 mm.

(2) The plate thicknesses of the flanges and the webs of the welded H-sections are:

6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm
25.0 mm	30.0 mm	40.0 mm
50.0 mm	60.0 mm	80.0 mm

(3) In most cases, both the web thickness and the flange thickness of the welded H-sections are taken to be larger than those of the rolled H-sections as far as rational, as shown in Figure 8.3. Moreover, the flange width of the welded H-sections is selected in such a way as to achieve a value of cross-sectional area which is at least 10% larger than that of the rolled H-sections.

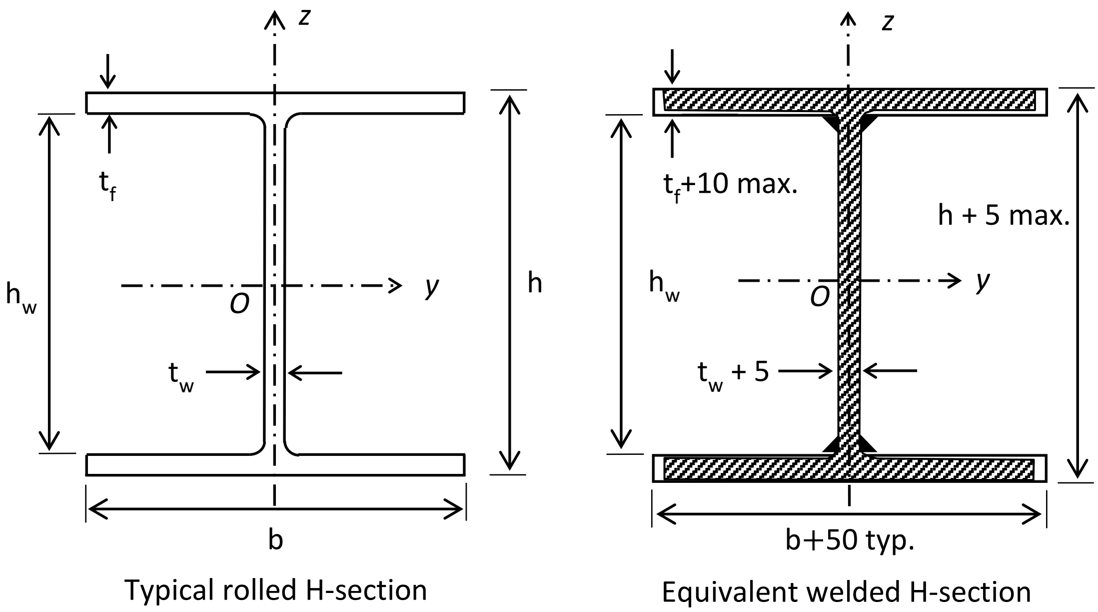


Figure 8.3 Design method of equivalent welded H-sections

(4) As there is a significant reduction in the yield strengths of thick steel plates, especially when $t_f \geq 40$ mm, the flange thicknesses of the proposed welded H-sections may be significantly larger than those of the rolled H-sections. Nevertheless, the maximum increase in the flange thickness is limited to 10 mm.

8.5.3 Equivalent cold-formed circular hollow sections

(1) The external diameter d of the EWCHS is selected to be equal to that of the hot-finished CHS under consideration plus a maximum of 5 mm.

(2) The plate thicknesses of the EWCHS are:

6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm
25 mm	30 mm	36 mm

It should be noted that the plate thickness of the EWCHS is selected to be equal to that of the hot-finished CHS under consideration ± 0.5 mm, as shown in Figure 8.4.

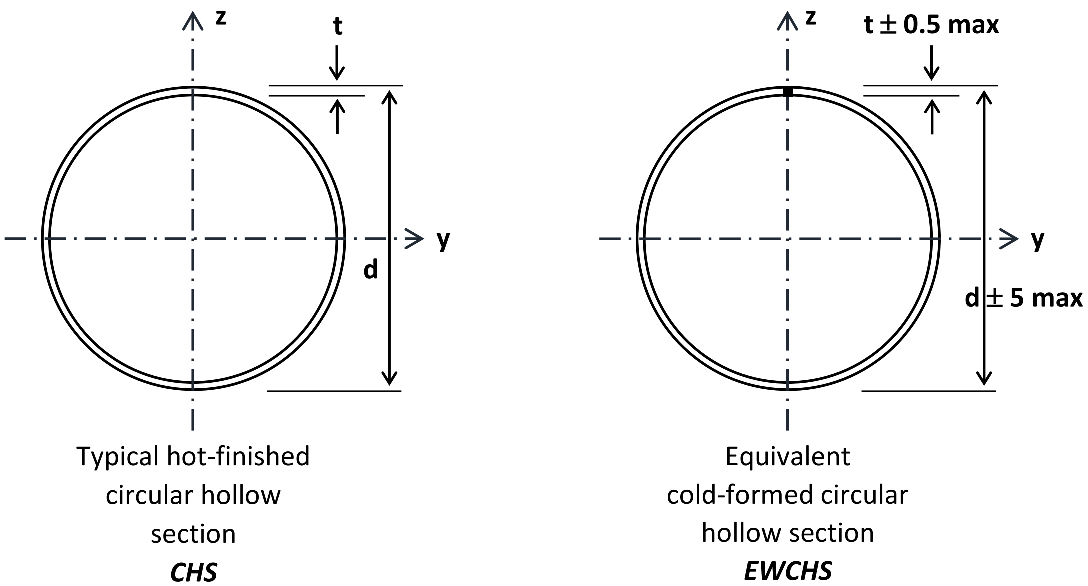


Figure 8.4 Design method for equivalent cold-formed circular hollow sections

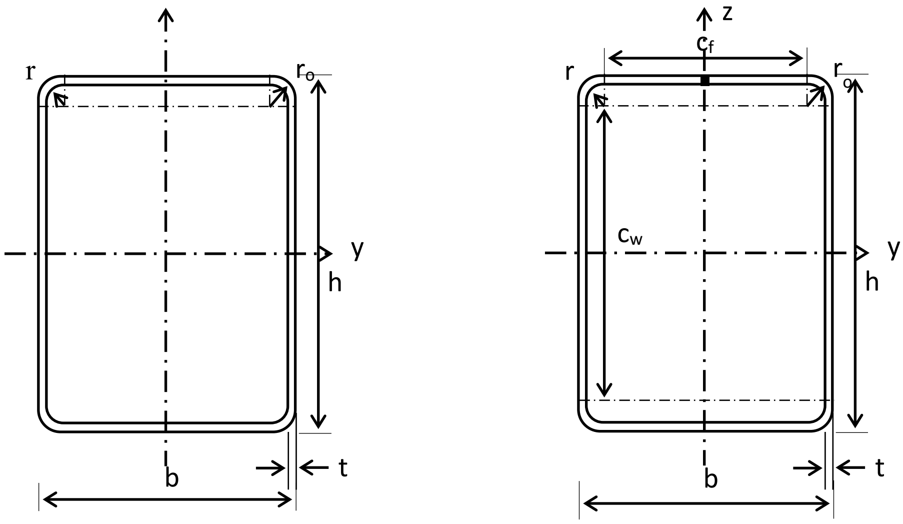
(3) It should be noted that the largest EWCHS covered in GB/T 6728 has an external diameter equal to 610.0 mm. For those EWCHS with external diameters equal to 711.0 and 813.0 mm, refer to GB/T 21835 for details.

8.5.4 Equivalent cold-formed rectangular and square hollow sections

(1) The external dimensions, h and b, of the EWRHS and the EWSHS are selected to be equal to those of the hot-finished sections under consideration, as shown in Figures 8.5 and 8.6.

(2) The plate thicknesses of the EWRHS and the EWSHS are:

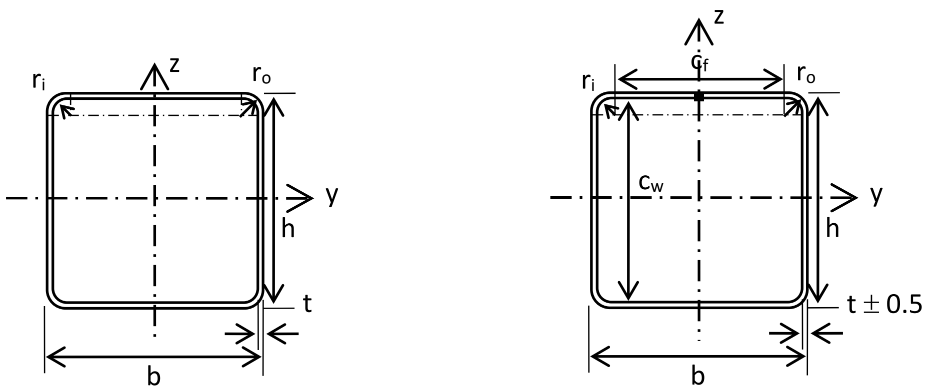
6.0 mm	8.0 mm	10.0 mm
12.0 mm	16.0 mm	20.0 mm



Typical hot-finished rectangular hollow section ***RHS***

Equivalent cold-formed rectangular hollow section ***EWRHS***

Figure 8.5 Design method for equivalent cold-formed rectangular hollow sections



Typical Hot-finished square hollow section ***SHS***

Equivalent cold-formed square hollow section ***EWSHS***

Figure 8.6 Design method for equivalent cold-formed square hollow sections

(3) It should be noted that both the inner and the outer corner radii, r_i and r_o , for cold-formed RHS and SHS given in EN 10219-2 are considered to be very stringent, as shown in Table 8.11. In some cases, these limiting values are even smaller than those given in EN 10210-2 for hot-finished RHS and SHS.

Table 8.11 Allowable corner radii of hot-finished and cold-formed RHS and SHS

EN 10210-2: Hot-finished structural hollow sections of non-alloy and fine grain steels				
Hot finished RHS, SHS	Thickness	All range		
	r_i	2.0 t		
	r_o	3.0 t		
EN 10219-2: Cold-formed welded structural hollow sections of non-alloy and fine grain steels				
Cold-formed RHS, SHS	Thickness	t = 6 mm	t = 8, 10 mm	t = 12, 16, 20 mm
	r_i	0.6 t ~ 1.4 t	1.0 t ~ 2.0 t	1.4 t ~ 2.6 t
	r_o	1.6 t ~ 2.4 t	2.0 t ~ 3.0 t	2.4 t ~ 3.6 t
GB/T 6728: Cold-formed steel hollow sections of general structures				
	Thickness	t = 6, 8, 10 mm		t = 12, 16, 20 mm
Q235 Q275	r_i	1.0 t ~ 2.0 t		1.0 t ~ 2.5 t
	r_o	2.0 t ~ 3.0 t		2.0 t ~ 3.5 t
Q345 Q460	r_i	1.0 t ~ 2.5 t		1.5 t ~ 3.0 t
	r_o	2.0 t ~ 3.5 t		2.5 t ~ 4.0 t

Cold-formed RHS, SHS	Thickness	t = 6, 8, 10 mm	t = 12, 16, 20 mm
Q690	r_i	2.5 t ~ 3.5 t	2.5 t ~ 3.5 t
	r_o	3.5 t ~ 4.5 t	3.5 t ~ 4.5 t

Notes: These reference values are recommended by modern fabricators

Moreover, according to Table 8.12, large local strains are always induced in the corners of the cold-formed RHS and SHS with small corner radii. Hence, welding in the immediate vicinity of the corners requires caution, otherwise significant cracking may be induced.

Table 8.12 Corner radii and local residual strains in cold-formed zones

BS EN 1993-1-8: Design of steel structures: Design of joints				
r_i / t	Residual strain	Maximum thickness (mm)		
		Static load control	Fatigue control	Killed steel
≥ 25	≤ 2%	Any	Any	Any
≥ 10	≤ 5%	Any	16	Any
≥ 3.0	≤ 14%	24	12	24
≥ 2.0	≤ 20%	12	10	12
≥ 1.5	≤ 25%	8	8	8
≥ 1.0	≤ 33%	4	4	4
Note: Conflict with EN 10219 will be assumed satisfied if t ≤ 12.5 mm.				

(4) Table 8.13 presents the proposed corner radii of EWRHS and EWSHS for various steel grades. It should be noted that these corner radii are less stringent when compared to those given in Table 8.11 for both hot-finished and cold-formed RHS and SHS to EN10210-2 and 10219-2 respectively. Nevertheless, these corner radii are permitted according to GB/T 6728.

Table 8.13 Proposed corner radii of EWRHS and EWSHS

Equivalent cold-formed hollow sections	Corner radii	t = 6, 8, 10 mm	t = 12, 16, 20 mm
EWRHS EWSHS	r_i	2.5 t	3.0 t
	r_o	3.5 t	4.0 t

(5) For further details on the dimensions of cold-formed rectangular and square hollow sections, refer to GB/T 6728.

(6) A full list of the rolled and of the welded sections are presented in Tables 8.10 and 8.14 respectively.

8.6 Design Tables on Section Dimensions, Properties and Resistances

According to the comprehensive design rules given in EN 1993-1-1, a total of 51 Design Tables are compiled to assist structural engineers to use both rolled and welded sections whenever appropriate in practical design. These Design Tables include:

- 12 Design Tables on section dimensions and properties;
- 12 Design Tables on section resistances of rolled sections; and
- 27 Design Tables on section resistances of welded sections.

Section resistances for a total of 468 rolled and welded sections are calculated and tabulated for a wide range of section types and dimensions as well as a wide range of steel materials with different yield strengths.

Table 8.15 summarizes various key design parameters of structural steel design of the Design Tables.

8.6.1 Section dimensions and properties

For details on the selection of section dimensions for various rolled and welded sections, refer to Sections 8.4 and 8.5 respectively.

Expressions for the calculations of various section properties are fully presented in Steel Building Design: Design Data (2013).

Table 8.14 Full ranges of proposed equivalent welded sections for application

Welded I-section		Welded H-section	Cold-formed circular hollow section	Cold-formed rectangular hollow section	Cold-formed square hollow section
920x450x420	460x220x112	420x480x716	*140x6.0*	120x80x6.0	100x100x6.0
x353	x104	x563	*x8.0*	x8.0	x8.0
920x360x312	x90	x532	*x10.0*	160x80x6.0	150x150x6.0
x282	x83	x456	*170x6.0*	x8.0	x8.0
x249	x70	x368	*x8.0*	x10.0	x10.0
x218	460x180x91	x300	*x10.0*	200x100x6.0	200x200x6.0
840x350x246	x80	x275	*x12.0*	x8.0	x8.0
x214	x73	360x440x218	*220x6.0*	x10.0	x10.0
x184	x62	x217	*x8.0*	200x150x6.0	x12.0
760x320x220	x56	x172	*x10.0*	x8.0	220x220x6.0
x194	410x210x85	x167	*x12.0*	x10.0	x8.0
x167	x78	370x330x316	*270x6.0*	250x150x6.0	x10.0
x147	x65	x260	*x8.0*	x8.0	x12.0
690x280x198	x59	x213	*x10.0*	x10.0	250x250x6.0
x173	400x150x49	x177	*x12.0*	x12.0	x8.0
x151	x43	x152	*x16.0*	260x180x6.0	x10.0
x133	355x180x73	x142	*320x6.0*	x8.0	x12.0
620x330x258	x62	x114	*x8.0*	x10.0	x16.0
x186	x54	270x310x184	*x10.0*	x12.0	300x300x6.0
x160	x49	x151	*x12.0*	x16.0	x8.0
610x260x158	355x170x44	x121	*x16.0*	300x200x6.0	x10.0
x138	x38	x116	*x20.0*	x8.0	x12.0
x122	310x160x59	x93	*360x6.0*	x10.0	x16.0
x112	x45	210x230x101	*x8.0*	x12.0	350x350x8.0
540x250x148	x39	x85	*x10.0*	x16.0	x10.0
x128	310x140x54	x73	*x12.0*	350x250x6.0	x12.0
x113	x45	x58	*x16.0*	x8.0	x16.0
x104	x41	x50	*x20.0*	x10.0	400x400x8.0
x88	310x110x37	170x170x42	*400x8.0*	x12.0	x10.0
	x33	x34	*x10.0*	x16.0	x12.0
	x28	x29	*x12.0*	400x200x6.0	x16.0
	260x170x48		*x16.0*	x8.0	x20.0
	x43		*x20.0*	x10.0	
	x33		*x25.0*	x12.0	
	260x130x33		*460x8.0*	x16.0	
	x29		*x10.0*	450x250x8.0	
	x25		*x12.0*	x10.0	
	210x150x34		*x16.0*	x12.0	
	x29		*x20.0*	x16.0	
	200x110x27		*x25.0*	500x300x8.0	
	180x100x19		*500x8.0*	x10.0	
	150x100x18		*x10.0*	x12.0	
	130x80x15		*x12.0*	x16.0	
			x16.0	x20.0	
			x20.0		
			x25.0		
			x30.0		
			610x8.0		
			x10.0		
			x12.0		
			x16.0		
			x20.0		
			x25.0		
			x30.0		
			710x10.0		
			x12.0		
			x16.0		
			x20.0		
			x25.0		
			x30.0		
			x36.0		
			810x10.0		
			x12.0		
			x16.0		
			x20.0		
			x25.0		
			x30.0		
			x36.0		
Number of sections	72	31	68	44	32
Total					247

Notes: (1) All equivalent welded sections are proposed to match the structural performance of those rolled sections given in Table 8.10.

8.6.2 Section resistances

The resistances of the cross-sections of various rolled and welded sections against bending moments, shear forces and axial compression forces have been calculated and tabulated.

It should be noted that for all the Design Tables, Class E1 Steel Materials are assumed in all rolled sections and all welded sections.

8.6.2.1 Moment resistances

All rolled and welded sections are doubly symmetrical, and most of them have two distinctive moment resistances, namely

i) $M_{y,Rd}$ about the major y-y axis, and

ii) $M_{z,Rd}$ about the minor z-z axis.

However, only a moment resistance, M_{Rd}, is provided for both circular and square hollow sections.

The corresponding flexural rigidities as well as the section classifications of the sections for bending about the major and the minor axes are also given as appropriate. However, it should be noted that no resistance is given for any Class 4 section owing to the occurrence of local buckling in plate elements of the section, leading to low structural efficiency.

8.6.2.2 Shear resistances

The shear resistances of the sections are calculated conservatively with the factor for shear area, η , being taken to 1.0 as recommended in Clause 6.2.6(3) of EN1993-1-1. Hence, there is no need to check against shear buckling in the web plate elements when the following conditions apply:

i) $\frac{d}{t} \leq 72\varepsilon$ for rolled sections

ii) $\frac{h_w}{t} \leq 72\varepsilon$ for welded sections

It should be noted that only the shear resistances of the sections acting along the direction of the webs of the sections are provided.

8.6.2.3 Axial compression resistances

In most sections, the gross areas of the sections are fully effective owing to the stocky nature of the plate elements. Hence, full compression resistances of these sections are readily mobilized.

However, for both rolled and welded I-sections, RHS and CHS with large d / t values under high compressive stress levels, local buckling in the plate elements of these cross-sections is critical. Hence, they are taken as Class 4 sections, and effective areas should be used, instead of their gross areas, in the calculation of the cross-section resistances against axial compression forces. These resistances are printed in italics in the Design Tables. Refer to Section 4.4 of EN 1993-1-5 for details of the design rule for evaluation of effective areas using the reduction factor for plate buckling, ρ .

As a whole, the Design Tables provide practical design data for structural engineers to assess the structural performance of various sections against material yielding as well as member buckling during practical design.

Table 8.15 Summary of Design Tables

Rolled sections	Section type	Design Table	
Dimensions and properties	***I-section***	01A / 01B 02A / 02B	
	H-section	03A / 03B	
	CHS	04	
	RHS	05	
	SHS	06	
Section resistances γ_{Mc} = 1.0	Steel materials	S275	S355
	I-section	07 08	13 14
	H-section	09	15
	CHS	10	16
	RHS	11	17
	SHS	12	18

Welded sections	Section type	Design Table				
Dimensions and properties	EWI-section	19A / 19B 20A / 20B				
	EWH-section	21A / 21B				
	EWCHS	22				
	EWRHS	23				
	EWSHS	24				
Section resistances γ_{Mc} = 1.1 for Q235~Q460; γ_{Mc} =1.0 for Q690	Steel materials	Q235	Q275	Q355	Q460	Q690
	EWI-section	25 26	28 29	34 35	40 41	46 47
	EWH-section	27	30	36	42	48
	EWCHS	*Not applicable*	31	37	43	49
	EWRHS		32	38	44	50
	EWSHS		33	39	45	51

Section 9 Structural Design of High Strength S690 Steels and Their Welded Sections

9.1 General

Owing to advances in steel-making technology in the past decades, high strength steels are modern steel products with excellent strength-to-self-weight ratios, and most of them are produced using a quenching-and-tempering (QT) process. High strength steels are able to offer effective structural solutions with a significant reduction in tonnages when compared with those of S355 steels. Since 2000, they have been used in large lifting-equipment, machinery and offshore structures. Many design and construction engineers believe that wide adoption of high strength steels in construction will have a huge impact on the construction industry worldwide. Significant savings in steel materials and reduction in self-weights of buildings and bridges can be readily achieved, making a tremendous contribution to sustainable infrastructure development globally.

However, high strength steels have not been widely adopted in construction so far owing to a lack of efficient design methods for structural applications. Moreover, possible deterioration in mechanical properties of heat affected zones of their welded joints is often a concern to many design and construction engineers. Consequently, the following experimental and numerical investigations into structural performance of high strength S690 steels and their welded sections have been conducted at the Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch):

i. Mechanical properties of S690 steels;
ii. Residual stresses of S690 welded H-sections and I-sections;
iii. Stocky columns of welded S690 H-sections;
iv. Slender columns of welded S690 H-sections;
v. Steel beams of welded S690 I-sections;
vi. Strength reduction and softening of S690 welded joints after welding;
vii. Stocky columns of S690 welded H-sections with splices (butt-welded joints); and
vii. Steel plies of S690 welded H-sections with splices (butt-welded joints).

This chapter presents various experimental and numerical investigations into structural behaviour of high strength S690 steels and their welded sections. These investigations generate both measured and predicted data to verify applicability of various design rules given in EN 1993-1-1 (2005) and 1-12 (2007). Details of the verification on applicability of these design rules are presented in the following sections.

9.2 Mechanical Properties of S690 Steels

According to modern structural design standards, such as EN 1993 -1-1 (2005) and -12 (2007), ductility requirements for S690 steels, which are quantified primarily from monotonic tests, are stipulated as follows:

i) the tensile to yield strength ratio, $f_u/f_y \geq 1.05$; (9.1)

ii) the strain at fracture, $\varepsilon_L \geq 10\%$; and (9.2)

iii) the strain corresponding to tensile strength, $\varepsilon_u \geq 15f_y/E_s$. (9.3)

These requirements provide specific guidance to qualify high strength S690 steels for structural application. In order to facilitate their wide adoption in construction, a large number of standard tensile tests of various batches of high strength S690 steels to BS EN 10025-6 and GB 1591 in different sizes and cross-sectional shapes were conducted to investigate their basic mechanic properties. Figure 9.1 illustrates a typical stress-strain curve obtained from a standard tensile test on a cylindrical coupon extracted from a S690 steel plate. The yield strength, f_y, of the S690 steel is found to be 787 N/mm^2 while its tensile strength, f_u, is 822 N/mm^2. Thus, the tensile to yield strength ratio, f_u / f_y, is 1.05. The strain at fracture, ε_L, is found to be 17.8%, which is larger than the minimum value of 10%. Moreover, the strain corresponding to tensile strength, ε_u, is larger than 15 f_y / E_s. Consequently, the S690 steel plate is demonstrated to meet all the ductility requirements stipulated in EN 1993 -1-12.

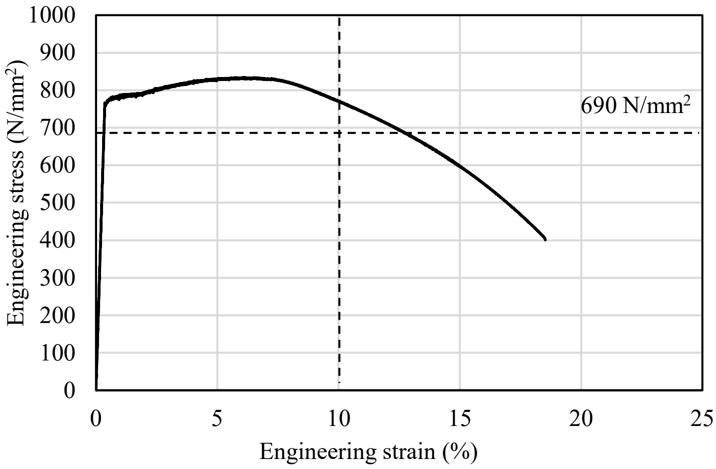
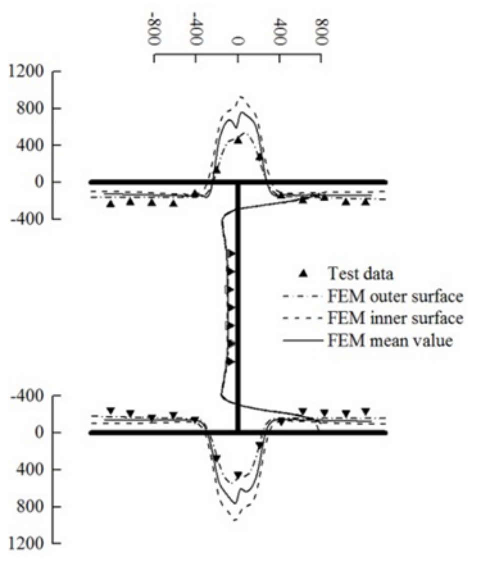


Figure 9.1 Typical stress-strain curve of high strength S690 steel

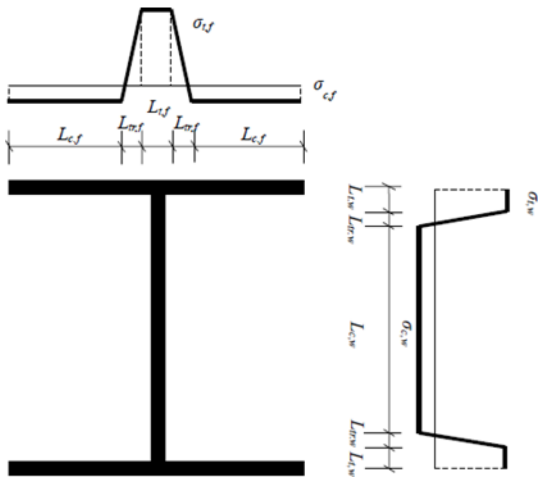
9.3 Residual Stress Patterns S690 Welded Sections

Residual stresses are inevitably induced in welded sections during fabrication. These stresses are always in self-equilibrium, and they tend to cause early yielding, and hence, member buckling in these welded sections. While sectioning is commonly employed to evaluate residual stresses in these sections, accuracy of the results depends heavily on how a welded section is cut into narrow strips. In general, it is very difficult to obtain residual stresses at the proximity of the flange-web junctions, especially those areas in direct contact of fillet welds.

In the present study, an integrated experimental and numerical investigation was undertaken, and a highly non-linear heat transfer finite element model with solid elements was established to carry out coupled thermomechanical analyses to simulate an arc welding process (Liu and Chung, 2016a; Liu and Chung, 2016b; Liu and Chung, 2018; Chung et al., 2022). After a careful calibration against both measured surface temperatures and surface residual strains at selected locations of welded sections, the model was able to predict satisfactorily residual strains and stresses throughout the entire welded sections due to differential heating and cooling in the flange-web junctions. It should be noted that by summing up the stresses throughout the thicknesses, a simplified residual stress pattern of the welded sections is readily obtained, as shown in Figure 9.2.

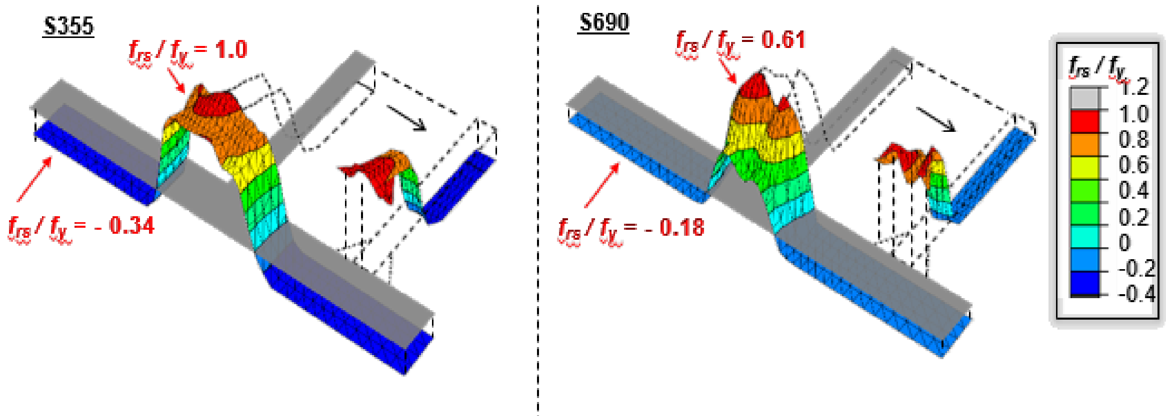


a) Measured and predicted residual stress patterns



$$\begin{cases} \sigma_{c,f} = (0.001 \cdot B - 0.35) \cdot f_{y,f} \\ \sigma_{c,w} = (0.001 \cdot h - 0.35) \cdot f_{y,w} = \sigma_{c,f} \end{cases} \qquad \begin{matrix} L_{c,f} = 0.0005 \cdot B + 0.31 \\ L_{c,w} = 0.001 \cdot h + 0.63 = 2L_{c,f} \end{matrix}$$

b) Simplified residual stress pattern



c) Comparison on predicted residual stresses in typical welded H-sections with different yield strengths

Figure 9.2 Residual stress patterns for fabricated S690 H-sections

As shown in Figure 9.2, the residual stresses in these S690 welded sections are typically found to be proportionally less pronounced, when compared with those S355 welded sections of similar dimensions, primarily due to an increase in the yield strengths of the steels. The residual stress fields fully compatible with solid elements are readily adopted in subsequent numerical investigations into various structural behaviour of S690 welded H -sections (as columns) as well as welded I-sections (as beams). Moreover, the residual stresses in S690 cold-formed circular hollow sections (CFCHS) and cold-formed square hollow sections (CFSHS) due to transverse bending and longitudinal welding were investigated experimentally and numerically (Hu et al., 2020;

Xiao et al., 2022). By adopting an integrated numerical approach using three coordinated finite element models, both magnitudes and distributions of residual stresses in S690 CFCHS and CFSHS were examined. The predicted results of i) two-dimensional plane strain analyses, ii) three-dimensional heat transfer analyses, and iii) three-dimensional thermomechanical analyses have been carefully calibrated against various experimental data, including surface temperatures and surface residual strains.

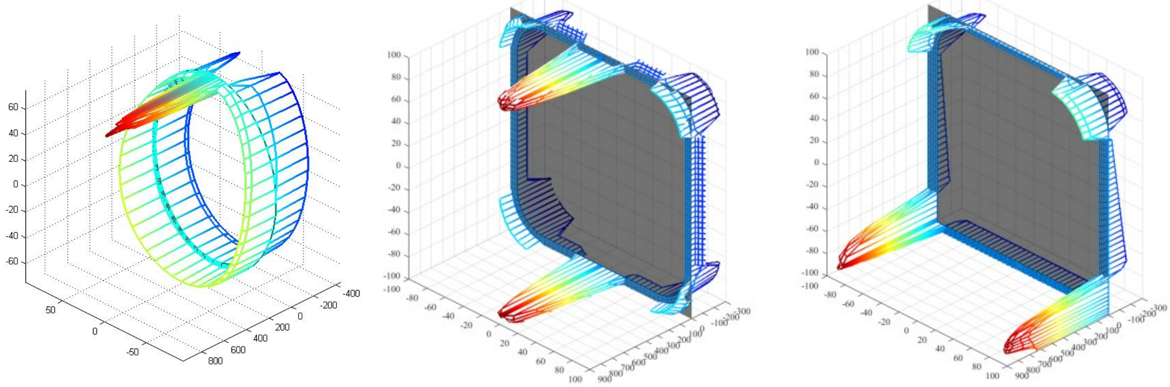
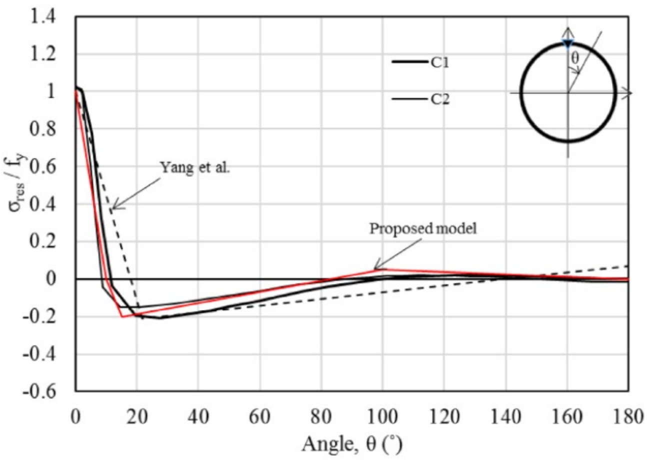
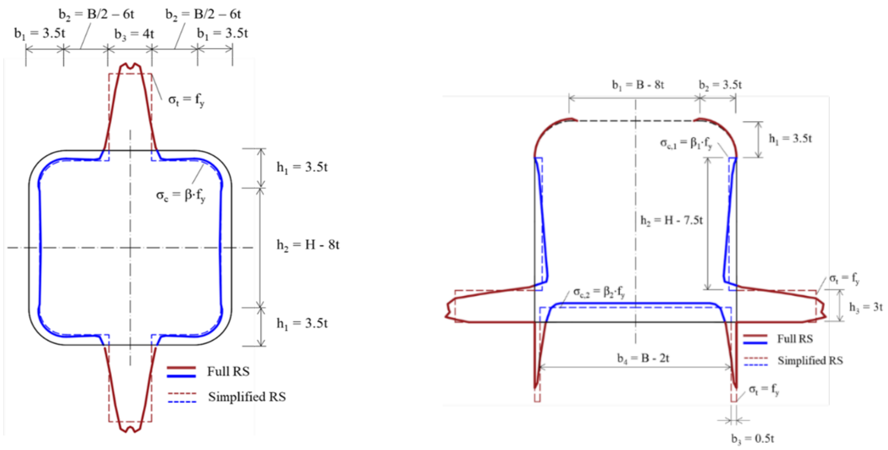


Figure 9.3 Three-dimensional plots of residual stresses of S690 CFCHS and CFSHS



$$\frac{\sigma_{res,m}}{f_y} = \begin{cases} 1.0 - 0.1\cdot\theta & 0 \leq \theta \leq 10^\circ \\ 0.40 - 0.04\cdot\theta & 10^\circ < \theta \leq 15^\circ \\ -0.244 + 0.00294\cdot\theta & 15^\circ < \theta \leq 100^\circ \\ 0.113 - 0.00063\cdot\theta & 100^\circ < \theta \leq 180^\circ \end{cases}$$

a) Residual stresses for S690 CFCHS



b) Residual stresses for S690 CFSHS

Figure 9.4 Simplified residual stress patterns of S690 CFCHS and CFSHS

Figure 9.3 shows typical three-dimensional plots of residual stresses in S690 CFCHS and CFSHS. It should be noted while both transverse bending and longitudinal welding are able to induce longitudinal residual stresses in these sections, the welding-induced residual stresses are significantly larger than those induced during transverse bending. Simplified residual stress patterns of S690 CFCHS and CFSHS are shown in Figure 9.4.

The proposed modelling technique for transverse bending and longitudinal welding with compatible meshes of two-dimensional and three-dimensional models is demonstrated to be highly effective. The technique is readily applicable to simulate residual stresses of all fabricated sections manufactured with transverse bending and longitudinal welding, and hence, the predicted residual stresses are readily incorporated into subsequent structural analyses of all fabricated members.

9.4 Stocky Columns of S690 Welded H-Sections

A total of 12 stocky columns of S690 welded H-sections with various width-to-thickness ratios were tested under axial compression to investigate their structural behaviour (Wang et al., 2016). The nominal dimension of four typical cross-sections of S690 welded H-sections adopted in the current investigation are illustrated in Figure 9.5.

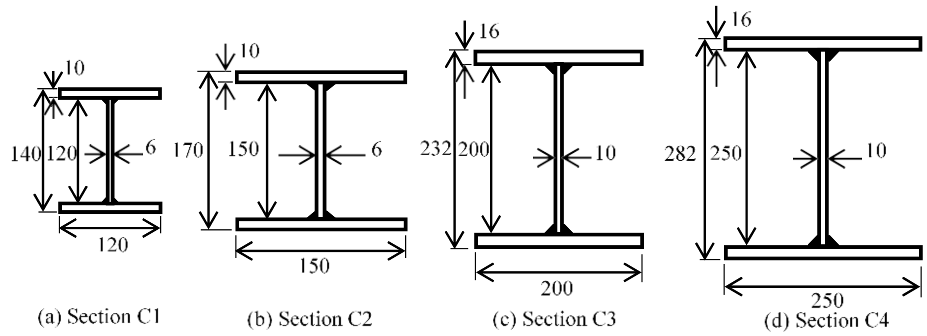
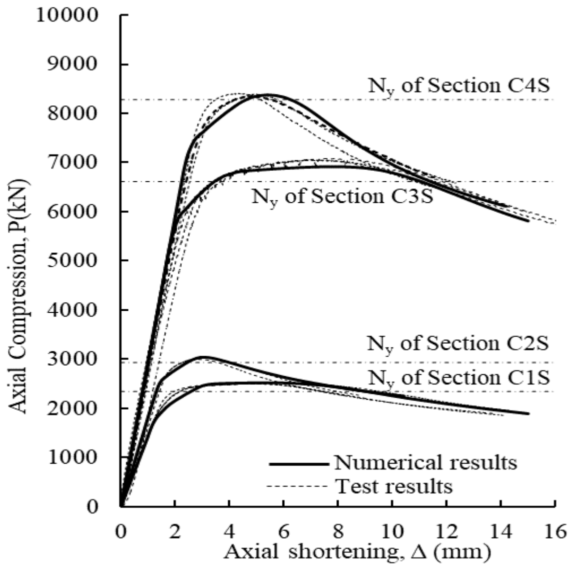


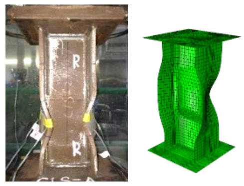
Figure 9.5 Nominal cross-sectional dimensions of S690 welded H-sections

Figure 9.6 plots the measured load-shortening curves of these stocky welded H-sections while typical failure modes of these H-sections are also illustrated. It is evident that plastic local plate buckling in both the flange outstands and the webs of these welded H-sections are also observed at large deformations. Moreover, a total of four finite element models with solid elements have been developed to perform material and geometrical non-linear analyses after incorporation of those predicted residual stress fields described in Section 9.3. Good agreement between the measured and the predicted load-shortening curves of these welded sections are achieved. The finite element models are then employed to perform various parametric studies on flange outstands and webs of typical S690 welded H-sections with different width-to-thickness ratios. The information are able to facilitate development of suitable design rules for section classification of welded H-sections under i) compression, and ii) combined compression and bending.

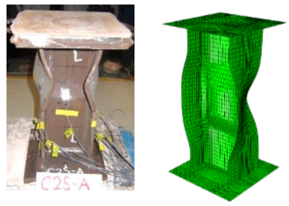
Modern design rules such as EN 1993-1-1 and -12 are found to be readily applicable to these stocky columns of S690 welded H-sections under compression, and Figure 9.6 illustrates good comparison between the measured and the design data.



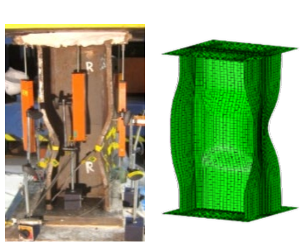
a) Measured load shortening curves of stocky columns



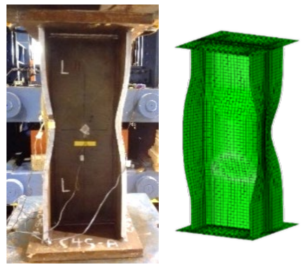
b) Section C1S-a



c) Section C2S-a



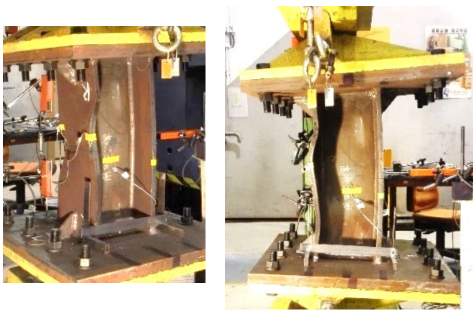
d) Section C3S-a



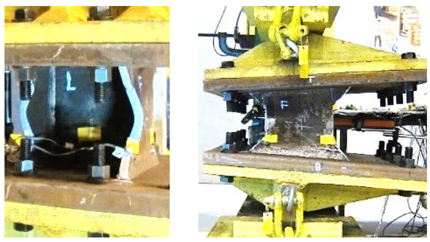
e) Section C4S-a

Figure 9.6 Measured load-shortening curves and typical failure modes of S690 stocky columns

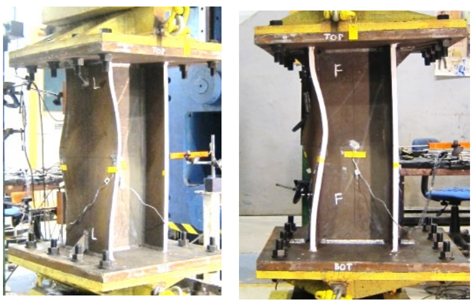
Another eight stocky columns of Sections C3 and C4 were also tested under combined compression and bending, and Figure 9.7 illustrates typical failure modes of these columns. It is evident that plastic local plate buckling in both the flange outstands and the webs of these welded H-sections are apparent at large deformations.



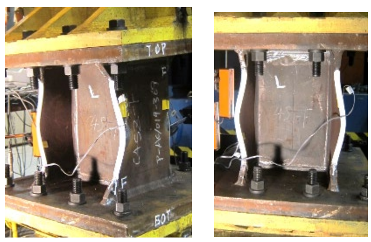
Section C3S-Y
Bending about Y axis



Section C3S-Z
Bending about Z axis



Section C4S-Y
Bending about Y axis



Section C4S-Z
Bending about Z axis

Figure 9.7 Typical failure modes of S690 stocky columns under combined compression and bending

Modern design rules such as EN 1993-1-1 and -12 are found to be readily applicable to these stocky columns of S690 welded H-sections under combined compression and bending, and Figure 9.8 illustrates a good comparison between the measured and the design data.

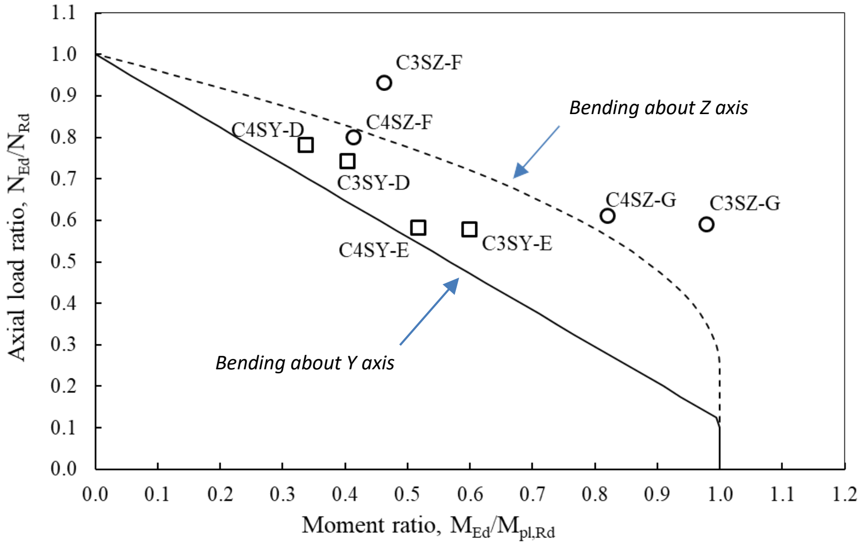


Figure 9.8 Comparison on measured and design data for S690 stocky columns under combined compression and bending

9.5 Slender Columns of S690 Welded H-Sections

A total of 7 slender columns of S690 welded H-sections with four different cross-sections, namely, Sections C1 to C4 as shown in Figure 9.5, and two different effective lengths were tested under compression to investigate their structural behaviour (Wang et al., 2017; Ma et al., 2018; Chung et al., 2022). Typical set-up of the slender column tests under a concentric axial load is illustrated in Figure 9.9. It should be noted that all the columns were loaded through concentric attachments which were pinned at both the top and the bottom ends. As expected, all of them were found to fail in an overall buckling mode while plastic local plate buckling in flange outstands of the welded H-sections were also observed at large deformations.

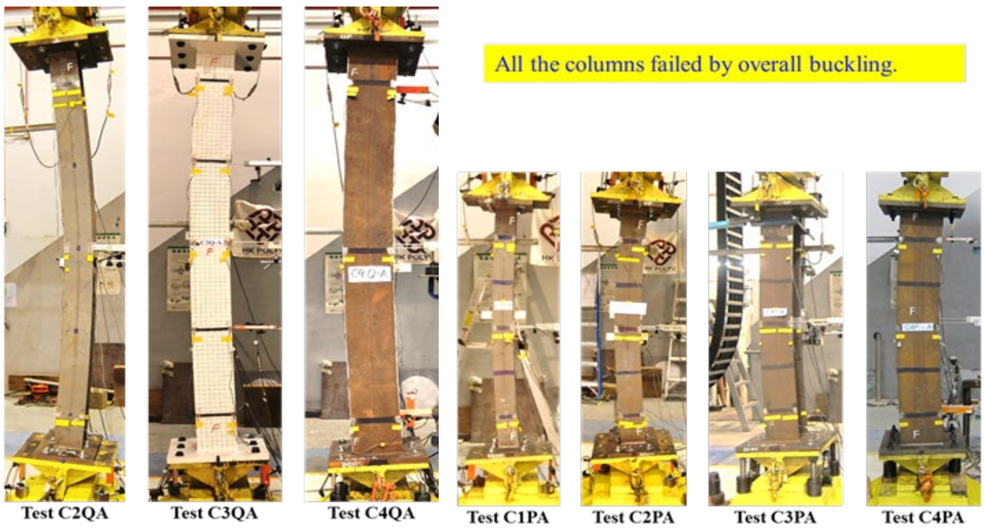


Figure 9.9 Slender columns of S690 welded H-sections under compression

Modern design rules such as EN 1993-1-1 and -12 are found to be readily applicable to these slender columns of welded H-sections under compression, and Figure 9.10 illustrates a good comparison between the measured and the design data.

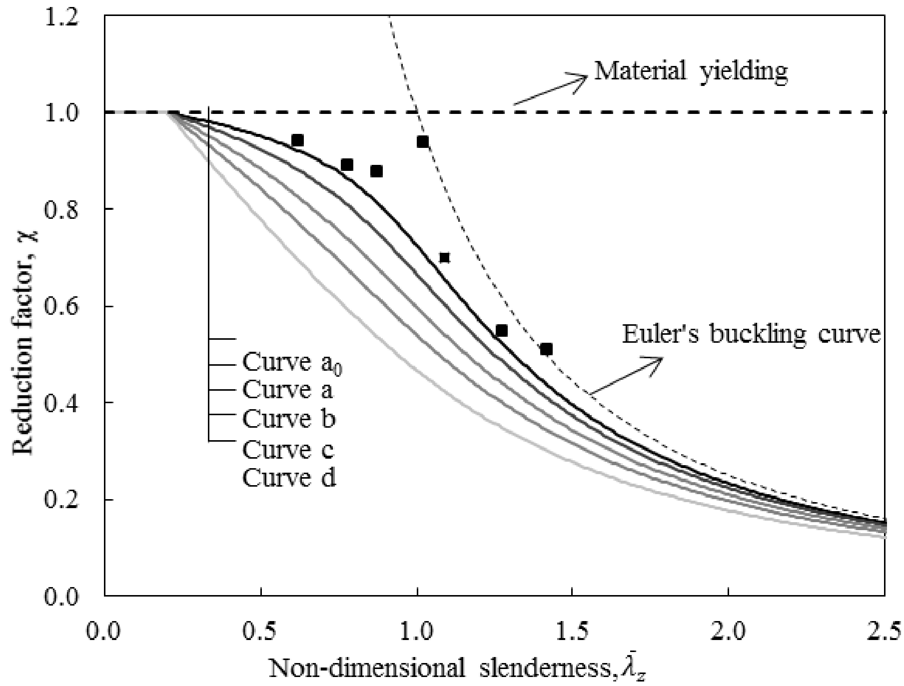


Figure 9.10 Comparison between test data and codified buckling curves to EN1993-1-1

It should be noted that the non-dimensional slenderness ratios of these columns are devised to range from 0.62 to 1.41 of which interaction between material yielding and member buckling is significant.

It is shown in Figure 9.10 that all the test data lie well above the codified buckling curves to EN1993-1-1, in particular, buckling curve c which is recommended for design of welded H-sections undergoing minor axis buckling. It should be noted that the effects of residual stresses is less pronounced in the S690 welded sections when compared with those in S355 welded sections. Moreover, the measured magnitudes of initial geometrical imperfections of all the test specimens, i , are found to be smaller than 0.25 mm, i.e. smaller than L_e / 5000 where L_e is the effective length of the slender column.

In order to adopt practical values of i in all these columns, finite element models have been developed, and the value of i is assigned to be L_e / 1000 and L_e / 1500 (Chung et al., 2022). It should be noted that the values of i are significantly smaller than those commonly adopted because of a full incorporation of the residual stresses predicted with coupled thermomechanical analyses as described in Section 9.3. After extensive parametric studies on the slender columns of S690 welded H-sections with a wide range of effective lengths, Figure 9.11 plots the finite element results onto the same graph of codified buckling curves to EN 1993-1-1.

It is demonstrated that buckling curve b is able to provide conservative and yet efficient design resistances of slender columns of S690 welded H-sections undergoing buckling about minor axes of their cross-sections.

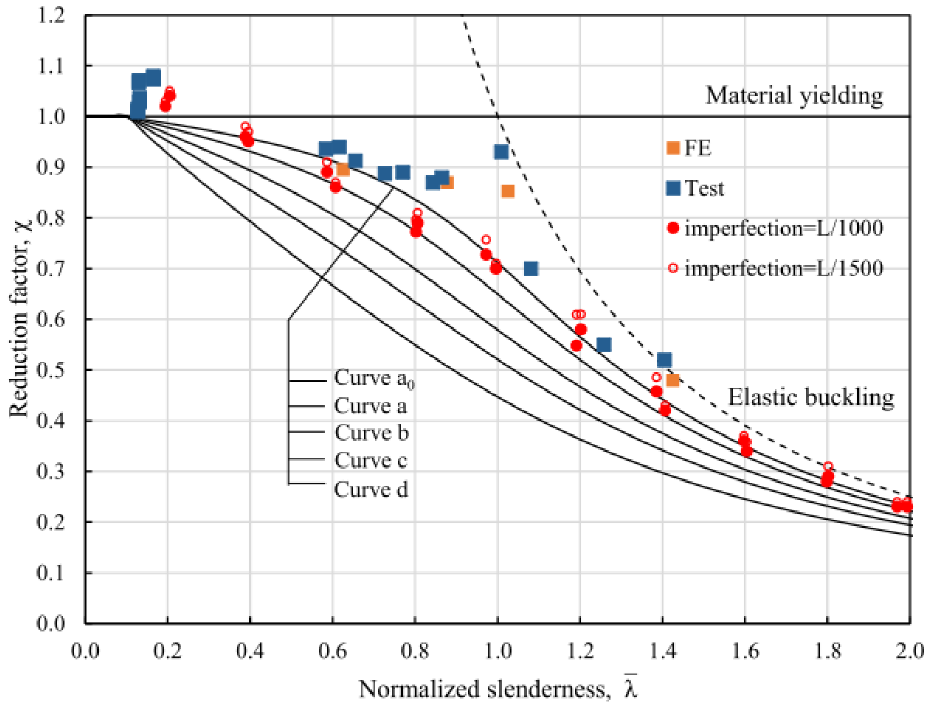
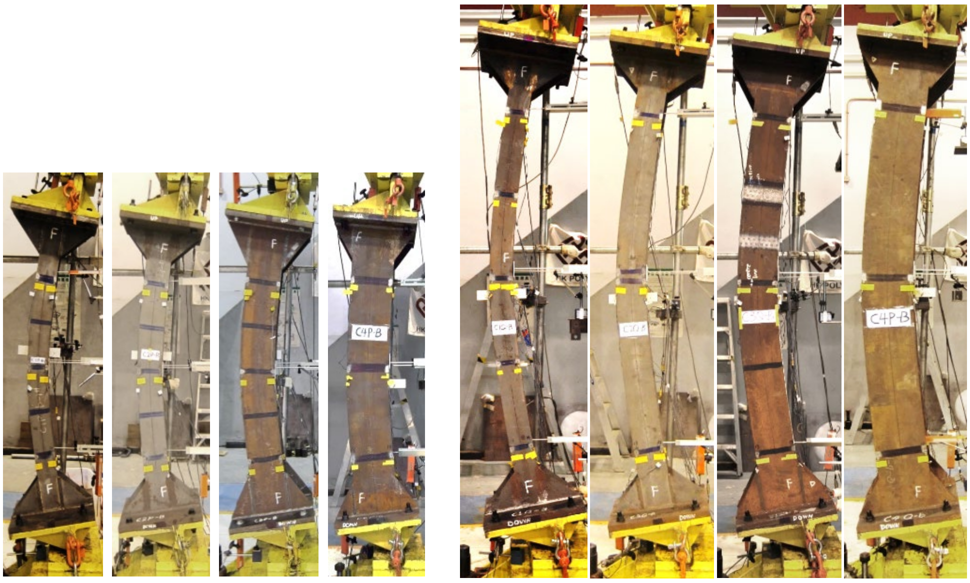


Figure 9.11 Comparison between finite element results and codified buckling curves to EN1993-1-1

9.6 Slender Columns of S690 Welded H-Sections under Combined Compression and Bending

A total of 8 slender columns of S690 welded H-sections with four different cross-sections, namely, Sections C1 to C4 as shown in Figure 9.5, and two different effective lengths were tested under combined compression and bending to investigate their structural behaviour (Ma et al., 2018). Typical set-up of the slender column tests under an eccentric axial load is illustrated in Figure 9.12. It should be noted that all the columns are loaded through eccentric attachments which are pinned at both the top and the bottom ends. As expected, all of them were found to fail in an overall buckling mode under large moments while plastic local plate buckling in flange outstands of the welded H-sections are also observed at large deformations.

Modern design rules such as EN 1993-1-1 and -12 are found to be readily applicable to these slender columns of S690 welded H-sections under combined compression and bending according to the method given in Annex B.



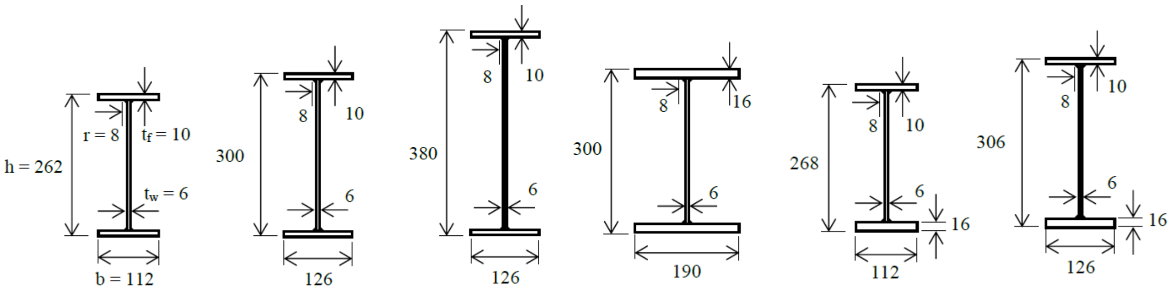
Test C1PB Test C2PB Test C3PB Test C4PB

Test C1QB Test C2QB Test C3QB Test C4QB

Figure 9.12 Slender columns of S690 welded H-sections under combined compression and bending

9.7 Partially Restrained Beams of S690 Welded I-Sections

6 7h.， total of 6 restrained beams and 12 partially restrained beams of S690 welded I-sections under single point loads were conducted to investigate their structural behaviour (Wang et al., 2021). The nominal cross-sectional dimensions of these welded I-sections are illustrated in Figure 9.13.



a) Section B1 Section B2 Section B3 b) Section B4 Section B5 Section B6

Figure 9.13 Nominal cross-sectional dimensions of S690 welded I-sections

Figures 9.14 and 9.15 illustrate typical set-ups for single point load beam tests with full and partial restraints respectively.

In all restrained beam tests, local buckling in compression flange outstands and web plates were observed at failure, as shown in Figure 9.14. Moreover, lateral torsional buckling between lateral restraints of the S690 welded I-sections were observed at failure, as shown in Figure 9.15, in all partially restrained beam tests.

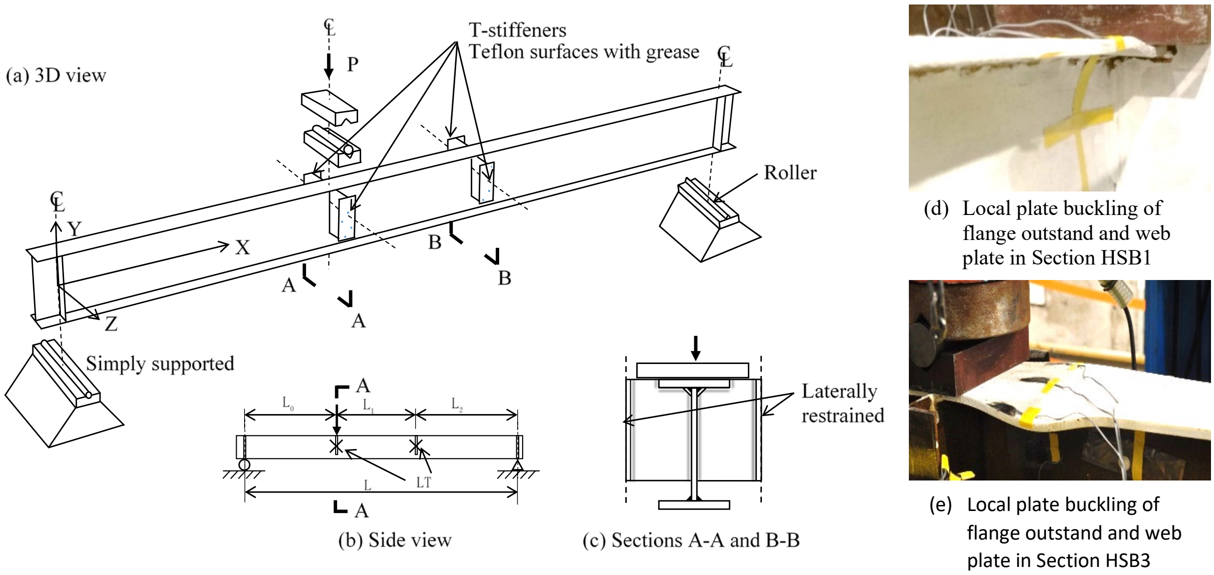


Figure 9.14 Typical set-up and failure modes of restrained beams of S690 welded I-sections

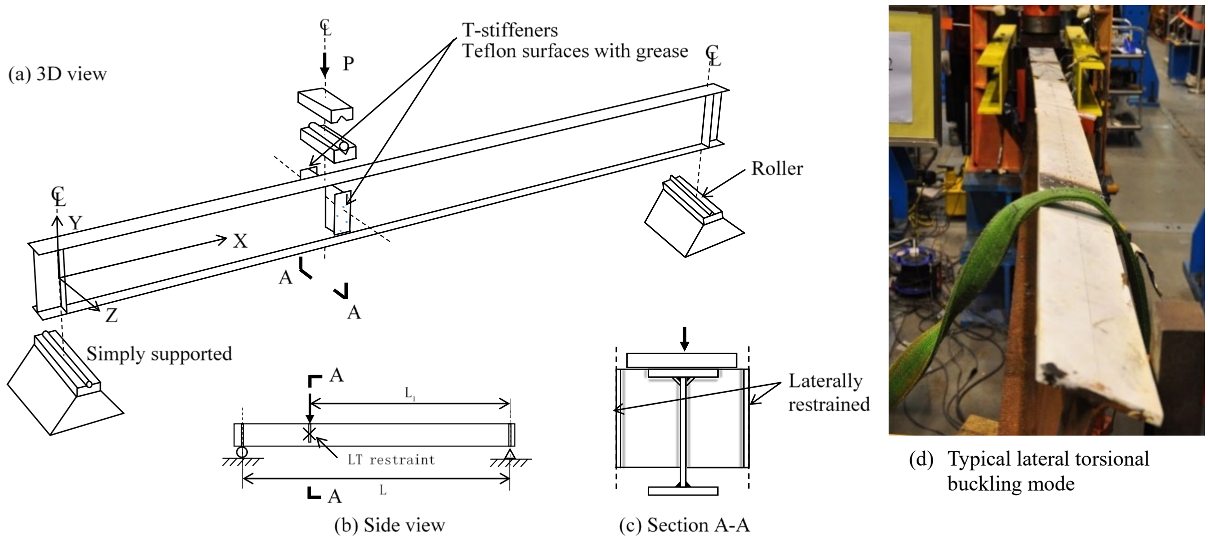


Figure 9.15 Typical set-up and failure modes of partially restrained beams of S690 welded I-sections

Modern design rules such as EN 1993-1-1 and -12 are found to be readily applicable to these beams of S690 welded I-sections under different restraint conditions, and Figure 9.16 illustrates a good comparison between the measured and the design data. It should be noted that the non-dimensional slenderness ratios of these beams are devised to range from 0.20 to 1.24 of which interaction between material yielding and member buckling is significant.

It is shown in Figure 9.16 that all the test data lie well above buckling curve d which is recommended for design of welded I-sections undergoing lateral torsional buckling. It should be noted that the effects of residual stresses is less pronounced in the S690 welded sections when compared with those in S355 welded sections. Moreover, the measured magnitudes of initial geometrical imperfections of all the test specimens, i , are found to be smaller than 0.50 mm, i.e. typically smaller than L_e / 2500 where L_e is an effective length.

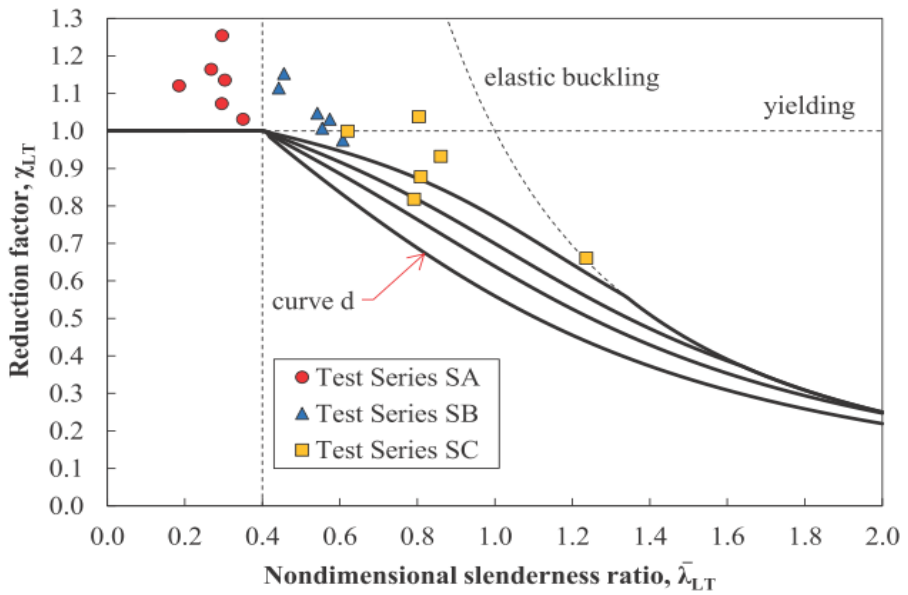


Figure 9.16 Comparison between beam test results and buckling curves in EN 1993

In order to adopt practical values of i in all these beams, finite element models have been developed, and the value of i is assigned to be L_e / 1500. After extensive parametric studies on both restrained and partially restrained beams of S690 welded I-sections with a wide range of effective lengths, Figure 9.17 plots the finite element results onto the same graph of codified buckling curves to EN 1993-1-1. It is demonstrated that buckling curve b is able to provide conservative and yet efficient design resistances of S690 welded I-sections undergoing lateral torsional buckling.

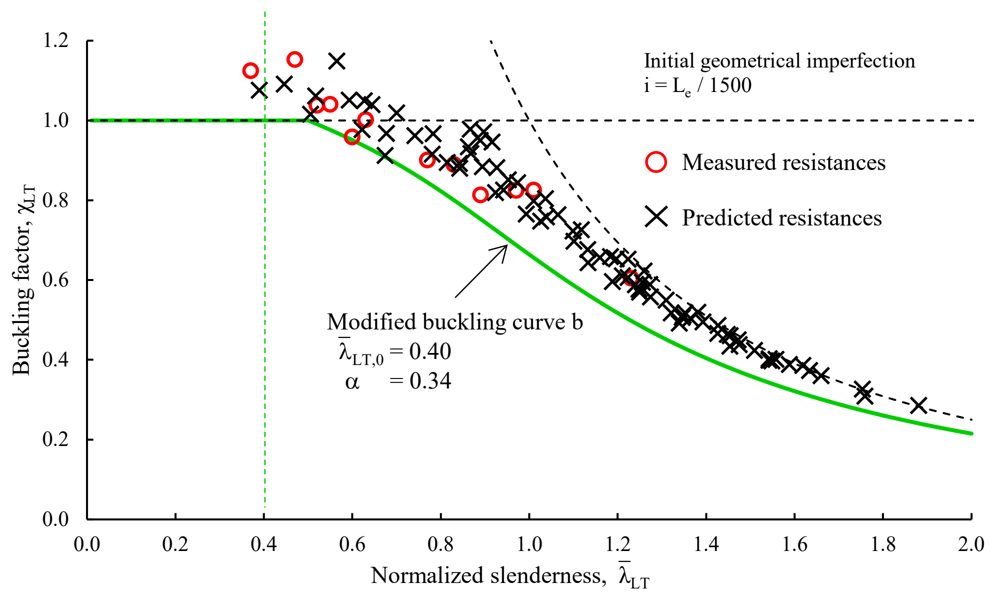
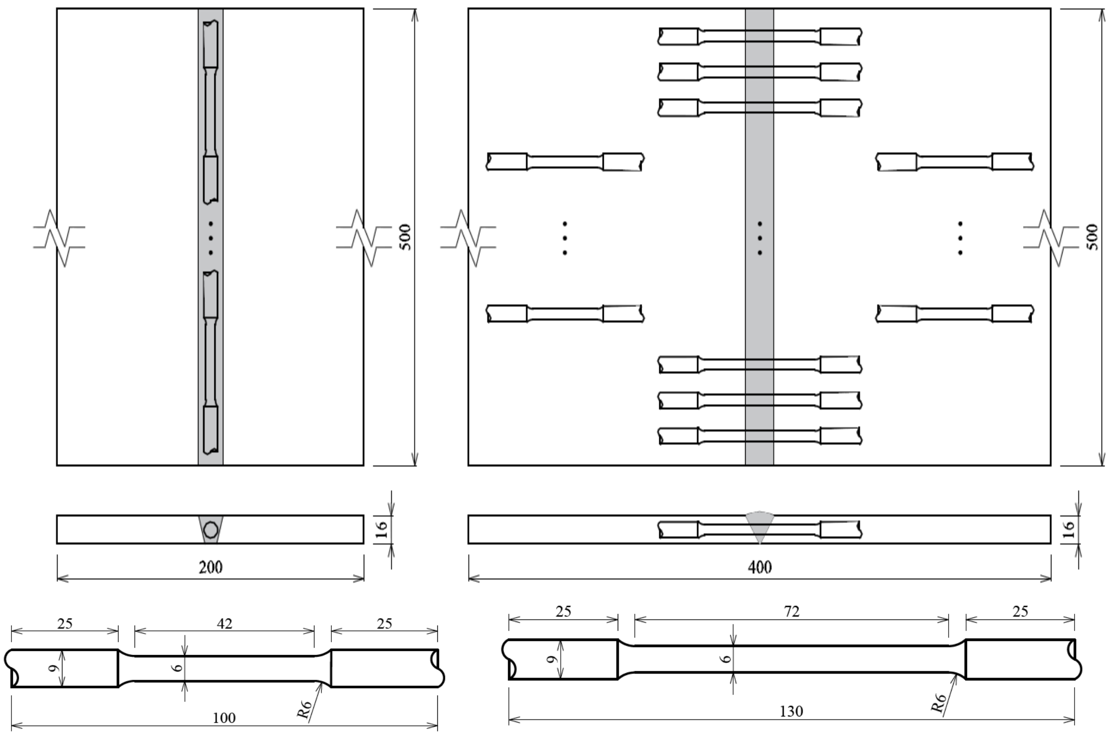


Figure 9.17 Comparison between finite element results and design buckling curves given in EN 1993

9.8 Strength reduction and softening of S690 welded joints after welding

Figure 9.18 illustrates a typical welded joint and various arrangements for machining of coupons of the base plates, the weld metal, and the welded sections.



a) cylindrical coupon of base plates and weld metal

b) cylindrical coupon of welded sections

Figure 9.18 Typical welded joint between S690 steel plates

A total of 15 coupons on S690 welded joints were tested in order to examine their structural behaviour under monotonic actions (Liu et al., 2018). It should be noted that these joints were welded using a robotic welding system with different welding parameters so that the heat input energy per weld run were consistently controlled at 1.0, 1.5, and 2.0 kJ/mm throughout the entire welding process. For a heat input energy per weld run at 5.0 kJ/mm, submerged arc welding was employed. Moreover, a total of 3 coupons of the weld metal were tested to provide basic mechanical data for comparison.

Standard tensile tests on all the 18 coupons were conducted, and all the measured stress strain curves of the base plates, the weld metal and the welded sections are plotted onto the same graph in Figure 9.19 for a direct comparison.

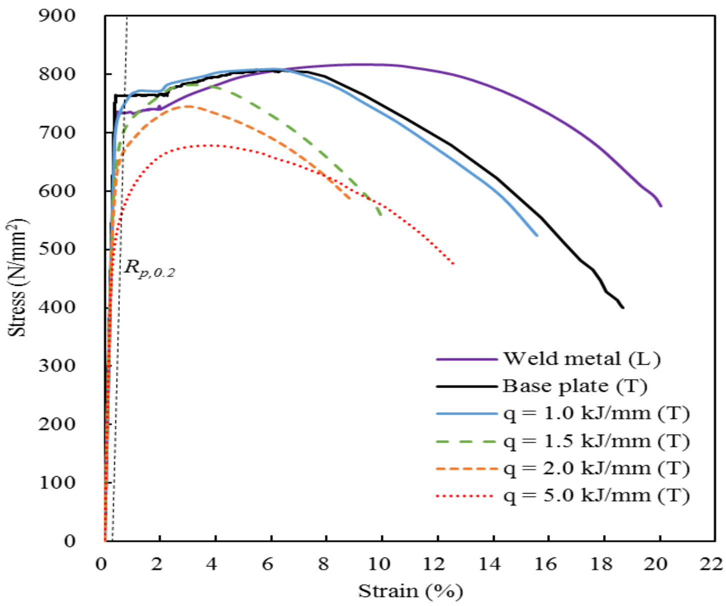


Figure 9.19 Engineering stress-strain curves of coupons of welded joints with different heat input energy

It should be noted that

a) All coupons of welded joints fail at close proximities of the weldment, i.e. within heat affected zones of the steel plates, after exhibiting highly localized elongations.
b) For those coupons machined from a welded joint with a heat input energy at 1.0 kJ/mm, their yield strengths are considered to be the same as those of the base plates as little softening is evident.
c) However, significant softening is apparent in those coupons machined from welded joints with heat input energy at 1.5 and 2.0 kJ/mm, leading to some reduction in their yield strengths. Nevertheless, it is shown that these coupons are able to mobilize a maximum strength equal to the tensile strengths of the base plates, if softening is allowed in the welded joints.
d) For those coupons machined from a welded joint with a heat input energy at 5.0 kJ/mm, significant softening and strength reduction is apparent. It should be noted that their yield and tensile strengths are found to be merely 70 and 85% of those of the base plates.

9.9 Stocky Columns of S690 Welded H-Sections with Splices

In order to investigate structural behaviour of column splices of S690 welded H-sections, a total of 12 stocky columns with 4 different cross-sections, namely, Sections C1 to C4, and 3 heat input energy, namely, 1.0, 1.5 and 2.0 kJ/mm, were tested under compression (Chung et al., 2020). It should be noted that butt-welds were adopted at mid-height of these stocky columns, and welding was performed by a highly experienced welder using various welding parameters developed for robotic welding. Figure 9.20 plots the measured load-shortening curves of these stocky columns with splices under compression, while their typical failure modes are illustrated in Figure 9.21.

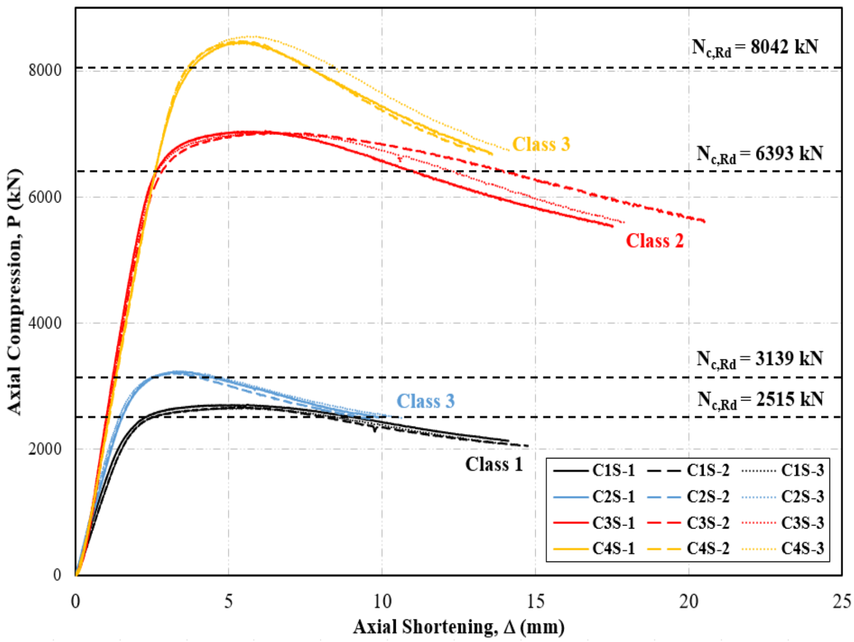


Figure 9.20 Load-shortening curves of stocky columns of S690 welded H-sections with splices

Section C1S Section C2S Section C3S Section C4S

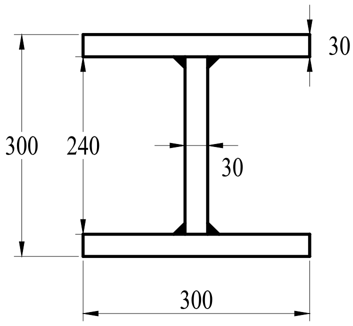
Figure 9.21 Typical failure modes of stocky columns of S690 welded H-sections with splices after testing

It is evident that plastic local plate buckling in both the flange outstands and the webs of these welded H-sections occur at large deformations. As shown in Figure 9.21, all these columns are able to mobilize the design resistances of their cross-sections under compression, $N_{c,Rd}$ (= cross-sectional area x measured yield strength). Moreover, the deformed shapes of these stocky columns with splices shown in Figure 9.21 are found to be very similar to those of stocky columns shown in Figure 9.6.

Consequently, the presence of the splices is shown to have relatively small effects on the cross-section resistances of these stocky columns of S690 welded H-sections. The effects of welding onto the structural behaviour of these stocky columns, in particular, their compression resistances, are shown to be less significant when compared with those yield strengths obtained in standard coupon tests due to various extents of strain hardening.

9.10 Steel Piles of S690 Welded H-Sections with Splices

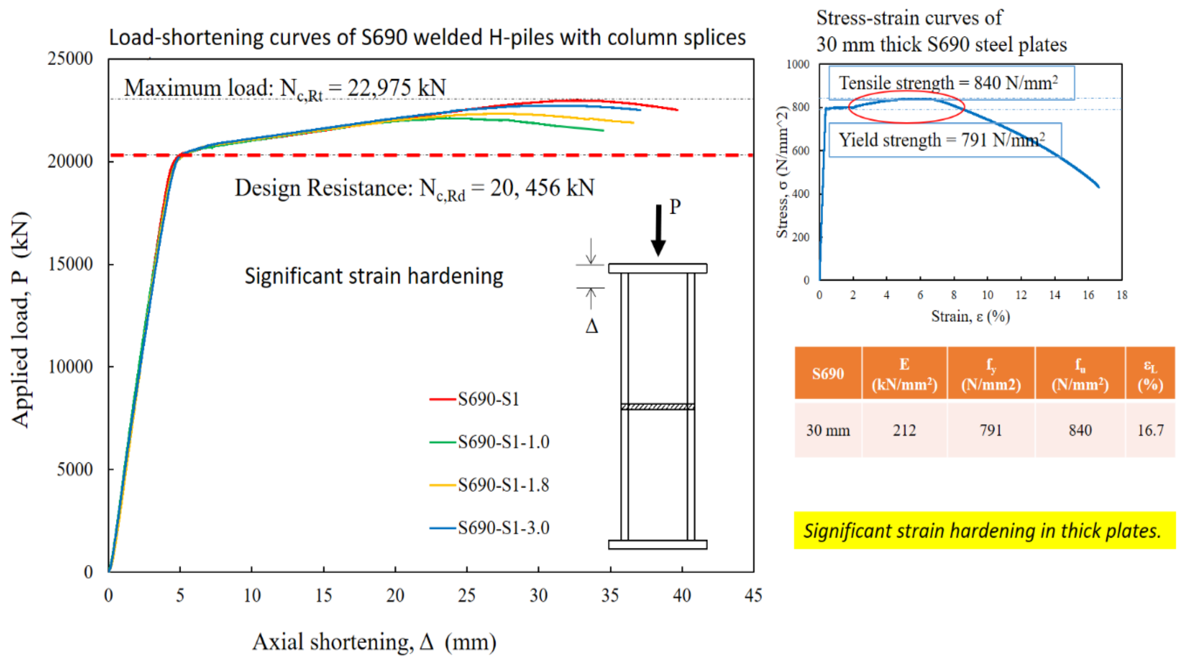
In order to confirm structural adequacy of steel piles of S690 welded H-sections with splices, a total of 4 stocky piles of similar cross-sections, i.e. Section S, as shown in Figure 9.22.



Section S1

Figure 9.22 Nominal cross-sectional dimensions of S690 welded H-sections

It should be noted that 3 of these piles are butt-welded at their mid-height with different heat input energy, namely, 1.0, 1.5 and 2.0 kJ/mm, and welding was performed by a highly experienced welder. All of these steel piles were tested under compression, and Figure 9.23 plots the measured load-shortening curves of these steel piles onto the same graph for direct comparison. It is shown that all of these steel piles with butt-welded splices at their mid-height are able to mobilize the design resistances of their cross-sections under compression, $N_{c,Rd}$ (= cross-sectional area x measured yield strength).



S690	E (kN/mm²)	f_y (N/mm2)	f_u (N/mm²)	ε_L (%)
30 mm	212	791	840	16.7

Figure 9.23 Load-shortening curves of steel piles of S690 welded H-sections with butt-welded splices at mid-height

Moreover, the deformed shapes of these steel piles with splices shown in Figure 9.24 represent typical plastic local plate buckling in both flange outstands and webs of these sections.

Figure 9.24 Stocky columns and steel piles of S690 welded H-sections with butt-welded splices at mid-height

9.11 Conclusions

The Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch) was established to promote effective design and construction of high strength steels, and one of its main objectives is to advance technological capabilities of the Hong Kong Construction Industry in design and construction of super high-rise buildings, long-span bridges and buildings of large enclosure using high-performance materials in Hong Kong and overseas. A series of experimental and numerical investigations into structural behaviour of high strength S690 steels and their welded sections were conducted in the past few years, and a technical report on the current status of these investigations is presented in this Chapter.

In general, the structural behaviour of high strength S690 welded sections are demonstrated to follow closely to that of welded sections of normal strength S355 steels. Modern design rules recommended in EN 1993-1-1 & -12 are shown to be generally applicable to these S690 welded sections when suitably selected design data and parameters are employed.

Both standard tensile tests on standard cylindrical coupons of butt-welded joints and compression tests on stocky columns with butt-welded splices have been conducted. It is shown that while the effects of welding onto yield strengths of these butt-welded joints are significant, their effects on compression resistances of stocky columns are less significant. It is reckoned that with an improved understanding on the effects of welding onto the mechanical properties of the high strength S690 welded joints and sections, design and construction engineers will be able to use high strength steels effectively and efficiently in construction.

Design specifications

European Committee for Standardization, BS EN 1993-1-1. (2005). Eurocode 3—Design of steel structures—Part 1 - 1: General rules and rules for buildings.

European Committee for Standardization, BS EN 1993-1-12. (2007). Eurocode 3—Design of steel structures—Part 1 -12: Additional rules for the extension of EN 1993 up to steel grades S 700.

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Chung, K. F., Ho, H. C., Hu, Y. F., Wang, K., Liu, X., Xiao, M., & Nethercot, D. A. (2020). Experimental evidence on structural adequacy of high strength S690 steel welded joints with different heat input energy. Engineering Structures, 204, 110051.

Hu, Y. F., Chung, K. F., Ban, H., & Nethercot, D. A. (2020). Investigations into residual stresses in S690 cold-formed circular hollow sections due to transverse bending and longitudinal welding. Engineering Structures, 219, 110911.

Liu, X., Chung, K. F., Ho, H. C., Xiao, M., Hou, Z. X., & Nethercot, D. A. (2018). Mechanical behavior of high strength S690-QT steel welded sections with various heat input energy. Engineering Structures, 175, 245-256.

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Ma, T. Y., Hu, Y. F., Liu, X., Li, G. Q., & Chung, K. F. (2017). Experimental investigation into high strength Q690 steel welded H-sections under combined compression and bending. Journal of Constructional Steel Research, 138, 449-462.

Wang, K., Xiao, M., Chung, K. F., & Nethercot, D. A. (2021). Lateral torsional buckling of partially restrained beams of high strength S690 welded I-sections. Journal of Constructional Steel Research, 184, 106777.

Xiao, M., Hu, Y. F., Jin, H., Chung, K. F., & Nethercot, D. A. (2022). Prediction of residual stresses in high-strength S690 cold-formed square hollow sections using integrated numerical simulations. Engineering Structures, 253, 113682.

Book chapter

Chung, K. F., Hu, Y. F., Xiao, M., Liu, X., & Nethercot, D. A. (2022). Structural behavior of welded H-sections made of high-strength S690 steel plates and their applications in construction. Analysis and Design of Plated Structures, pp. 539-591. Woodhead Publishing.

Conference papers

Ho, H. C., Xiao M, Liu X, & Chung, K. F. (2017). Tensile tests on high strength steel materials using high precision measurement. Proceeding of 15th East Asia-pacific conference on structural engineering and construction (EASEC-15), Xi' an, October 2017.

Liu, X., & Chung, K.F. (2016a). Experimental Investigation into Residual Stresses of Welded H-sections made of Q690 Steel Materials. Proceeding of 14th East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-14), January 2016, Ho Chi Minh City, Vietnam, pp559-566.

Liu, X., & Chung, K.F. (2016b). Numerical Investigation into Residual Stresses of Welded H-sections Made of Q690 Steel Materials. The 11th Pacific Structural Steel Conference, Shanghai, China (PSSC-11), 29 - 31 October 2016, Shanghai, China.

Wang, K., Hu, Y. F., Chan, T. M., & Chung, K. F. (2016). Compression tests on stocky welded H-sections made of Q690 steel materials. Proceeding of the 14th East Asia-Pacific Conference, Ho Chi Minh City, January 2016, p552-568.

Wang, K., Liu, X. & Chung, K. F. (2017). Study on structural behaviour of high strength steel S690 welded H-sections under axial compression. Proceeding of the 15th East Asia-Pacific Conference, Xi' an, China, October 2017, p893-901.

Xiao, M., & Chung, K. F. (2019). Structural Behaviour of High Strength S690 Cold-Formed Structural Hollow Sections Under Compression. Proceeding of the 16th East Asia-Pacific Conference, Brisbane, Australia, December 2019, p1559-1568.

Design Tables on Section Dimensions, Properties and Resistances for Rolled and Welded Sections

Rolled sections

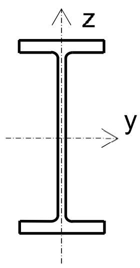
Welded sections

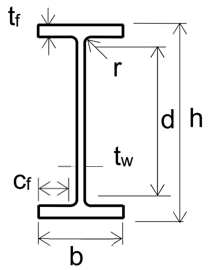
Design Tables 1 to 6 for

section dimensions and properties of rolled sections

- **I-sections**
- **H-sections**
- **CHS**
- **RHS**
- **SHS**

Design Table 01A Section dimensions of rolled I-sections (1)

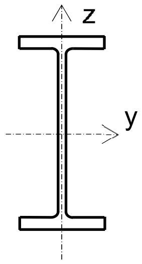


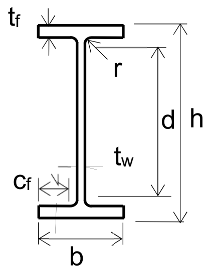


IS	Mass per Meter	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Surface Area	
				Web	Flange					*per Meter*	*per Tonne*
	m	h	b	t_w	t_f	r	d	c_f/t_f	d/t_w		
	kg/m	mm	mm	mm	mm	mm	mm			m^2	m^2
914x419x388#	388.0	921.0	420.5	21.4	36.6	24.1	799.6	4.8	37.4	3.44	8.87
x343#	343.3	911.8	418.5	19.4	32.0	24.1	799.6	5.5	41.2	3.42	10.0
914x305x289#	289.1	926.6	307.7	19.5	32.0	19.1	824.4	3.9	42.3	3.01	10.4
x253#	253.4	918.4	305.5	17.3	27.9	19.1	824.4	4.5	47.7	2.99	11.8
x224#	224.2	910.4	304.1	15.9	23.9	19.1	824.4	5.2	51.8	2.97	13.2
x201#	200.9	903.0	303.3	15.1	20.2	19.1	824.4	6.2	54.6	2.96	14.7
838x292x226#	226.5	850.9	293.8	16.1	26.8	17.8	761.7	4.5	47.3	2.81	12.4
x194#	193.8	840.7	292.4	14.7	21.7	17.8	761.7	5.6	51.8	2.79	14.4
x176#	175.9	834.9	291.7	14.0	18.8	17.8	761.7	6.4	54.4	2.78	15.8
762x267x197	196.8	769.8	268.0	15.6	25.4	16.5	686.0	4.3	44.0	2.55	13.0
x173	173.0	762.2	266.7	14.3	21.6	16.5	686.0	5.1	48.0	2.53	14.6
x147	146.9	754.0	265.2	12.8	17.5	16.5	686.0	6.3	53.6	2.51	17.1
x134	133.9	750.0	264.4	12.0	15.5	16.5	686.0	7.1	57.2	2.51	18.7
686x254x170	170.2	692.9	255.8	14.5	23.7	15.2	615.1	4.5	42.4	2.35	13.8
x152	152.4	687.5	254.5	13.2	21.0	15.2	615.1	5.0	46.6	2.34	15.4
x140	140.1	683.5	253.7	12.4	19.0	15.2	615.1	5.6	49.6	2.33	16.6
x125	125.2	677.9	253.0	11.7	16.2	15.2	615.1	6.5	52.6	2.32	18.5
610x305x238	238.1	635.8	311.4	18.4	31.4	16.5	540.0	4.1	29.3	2.45	10.3
x179	179.0	620.2	307.1	14.1	23.6	16.5	540.0	5.5	38.3	2.41	13.5
x149	149.2	612.4	304.8	11.8	19.7	16.5	540.0	6.6	45.8	2.39	16.0
610x229x140	139.9	617.2	230.2	13.1	22.1	12.7	547.6	4.3	41.8	2.11	15.1
x125	125.1	612.2	229.0	11.9	19.6	12.7	547.6	4.9	46.0	2.09	16.7
x113	113.0	607.6	228.2	11.1	17.3	12.7	547.6	5.5	49.3	2.08	18.4
x101	101.2	602.6	227.6	10.5	14.8	12.7	547.6	6.5	52.2	2.07	20.5
533x210x122	122.0	544.5	211.9	12.7	21.3	12.7	476.5	4.1	37.5	1.89	15.5
x109	109.0	539.5	210.8	11.6	18.8	12.7	476.5	4.6	41.1	1.88	17.2
x101	101.0	536.7	210.0	10.8	17.4	12.7	476.5	5.0	44.1	1.87	18.5
x92	92.1	533.1	209.3	10.1	15.6	12.7	476.5	5.6	47.2	1.86	20.2
x82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	6.6	49.6	1.85	22.5

Limited availability.

Design Table 01B Section properties of rolled I-sections (1)

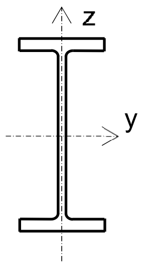


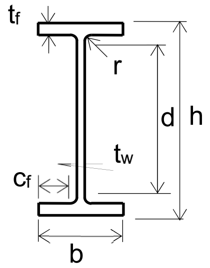


IS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	I_y	I_z	$W_{el,y}$	$W_{el,z}$	$W_{pl,y}$	$W_{pl,z}$	u	x	I_w	I_T	A
	cm^4	cm^4	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
914x419x388#	720000	45400	15600	2160	17700	3340	0.885	26.7	88.9	1730	494
x343#	626000	39200	13700	1870	15500	2890	0.883	30.1	75.8	1190	437
914x305x289#	504000	15600	10900	1010	12600	1600	0.867	31.9	31.2	926	368
x253#	436000	13300	9500	871	10900	1370	0.865	36.2	26.4	626	323
x224#	376000	11200	8270	739	9530	1160	0.860	41.3	22.1	422	286
x201#	325000	9420	7200	621	8350	982	0.853	46.9	18.4	291	256
838x292x226#	340000	11400	7980	773	9160	1210	0.869	35.0	19.3	514	289
x194#	279000	9070	6640	620	7640	974	0.862	41.6	15.2	306	247
x176#	246000	7800	5890	535	6810	842	0.856	46.5	13.0	221	224
762x267x197	240000	8170	6230	610	7170	958	0.869	33.1	11.3	404	251
x173	205000	6850	5390	514	6200	807	0.865	38.0	9.39	267	220
x147	169000	5460	4470	411	5160	647	0.858	45.2	7.40	159	187
x134	151000	4790	4020	362	4640	570	0.853	49.8	6.46	119	171
686x254x170	170000	6630	4920	518	5630	811	0.872	31.8	7.42	308	217
x152	150000	5780	4370	455	5000	710	0.871	35.4	6.42	220	194
x140	136000	5180	3990	409	4560	638	0.870	38.6	5.72	169	178
x125	118000	4380	3480	346	3990	542	0.863	43.8	4.80	116	159
610x305x238	209000	15800	6590	1020	7490	1570	0.886	21.3	14.5	785	303
x179	153000	11400	4930	743	5550	1140	0.885	27.7	10.2	340	228
x149	126000	9310	4110	611	4590	937	0.886	32.7	8.17	200	190
610x229x140	112000	4510	3620	391	4140	611	0.875	30.6	3.99	216	178
x125	98600	3930	3220	343	3680	535	0.875	34.0	3.45	154	159
x113	87300	3430	2870	301	3280	469	0.870	38.0	2.99	111	144
x101	75800	2910	2520	256	2880	400	0.863	43.0	2.52	77	129
533x210x122	76000	3390	2790	320	3200	500	0.878	27.6	2.32	178	155
x109	66800	2940	2480	279	2830	436	0.875	30.9	1.99	126	139
x101	61500	2690	2290	256	2610	399	0.874	33.1	1.81	101	129
x92	55200	2390	2070	228	2360	355	0.873	36.4	1.60	75.7	117
x82	47500	2010	1800	192	2060	300	0.863	41.6	1.33	51.5	105

\# Limited availability.

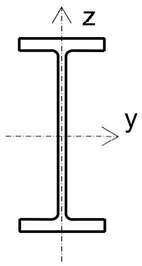
Design Table 02A   Section dimensions of rolled I-sections (2)

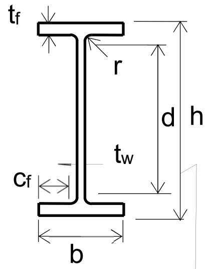




IS	Mass per Meter	Depth of Section	Width of Section	Thickness		Root Radius	Depth between Fillets	Ratios for Local Buckling		Surface Area	
				Web	Flange					*per Meter*	*per Tonne*
	m	h	b	t_w	t_f	r	d	c_f/t_f	d/t_w		
	kg/m	Mm	mm	mm	mm	mm	mm			m^2	m^2
457x191x98	98.3	467.2	192.8	11.4	19.6	10.2	407.6	4.1	35.8	1.67	17.0
x89	89.3	463.4	191.9	10.5	17.7	10.2	407.6	4.6	38.8	1.66	18.6
x82	82.0	460.0	191.3	9.9	16.0	10.2	407.6	5.0	41.2	1.65	20.1
x74	74.3	457.0	190.4	9.0	14.5	10.2	407.6	5.6	45.3	1.64	22.1
x67	67.1	453.4	189.9	8.5	12.7	10.2	407.6	6.3	48.0	1.63	24.3
457x152x82	82.1	465.8	155.3	10.5	18.9	10.2	407.6	3.3	38.8	1.51	18.4
x74	74.2	462.0	154.4	9.6	17.0	10.2	407.6	3.7	42.5	1.50	20.2
x67	67.2	458.0	153.8	9.0	15.0	10.2	407.6	4.2	45.3	1.50	22.3
x60	59.8	454.6	152.9	8.1	13.3	10.2	407.6	4.7	50.3	1.49	24.9
x52	52.3	449.8	152.4	7.6	10.9	10.2	407.6	5.7	53.6	1.48	28.3
406x178x74	74.2	412.8	179.5	9.5	16.0	10.2	360.4	4.7	37.9	1.51	20.4
x67	67.1	409.4	178.8	8.8	14.3	10.2	360.4	5.2	41.0	1.50	22.3
x60	60.1	406.4	177.9	7.9	12.8	10.2	360.4	5.8	45.6	1.49	24.8
x54	54.1	402.6	177.7	7.7	10.9	10.2	360.4	6.7	46.8	1.48	27.3
406x140x46	46.0	403.2	142.2	6.8	11.2	10.2	360.4	5.1	53.0	1.34	29.1
x39	39.0	398.0	141.8	6.4	8.6	10.2	360.4	6.7	56.3	1.33	34.1
356x171x67	67.1	363.4	173.2	9.1	15.7	10.2	311.6	4.6	34.2	1.38	20.6
x57	57.0	358.0	172.2	8.1	13.0	10.2	311.6	5.5	38.5	1.37	24.1
x51	51.0	355.0	171.5	7.4	11.5	10.2	311.6	6.3	42.1	1.36	26.7
x45	45.0	351.4	171.1	7.0	9.7	10.2	311.6	7.4	44.5	1.36	30.2
356x127x39	39.1	353.4	126.0	6.6	10.7	10.2	311.6	4.6	47.2	1.18	30.2
x33	33.1	349.0	125.4	6.0	8.5	10.2	311.6	5.8	51.9	1.17	35.4
305x165x54	54.0	310.4	166.9	7.9	13.7	8.9	265.2	5.2	33.6	1.26	23.3
x46	46.1	306.6	165.7	6.7	11.8	8.9	265.2	6.0	39.6	1.25	27.1
x40	40.3	303.4	165.0	6.0	10.2	8.9	265.2	6.9	44.2	1.24	30.8
305x127x48	48.1	311.0	125.3	9.0	14.0	8.9	265.2	3.5	29.5	1.09	22.7
x42	41.9	307.2	124.3	8.0	12.1	8.9	265.2	4.1	33.2	1.08	25.8
x37	37.0	304.4	123.4	7.1	10.7	8.9	265.2	4.6	37.4	1.07	28.9
305x102x33	32.8	312.7	102.4	6.6	10.8	7.6	275.9	3.7	41.8	1.01	30.8
x28	28.2	308.7	101.8	6.0	8.8	7.6	275.9	4.6	46.0	1.00	35.5
x25	24.8	305.1	101.6	5.8	7.0	7.6	275.9	5.8	47.6	0.992	40.0
254x146x43	43.0	259.6	147.3	7.2	12.7	7.6	219.0	4.9	30.4	1.08	25.1
x37	37.0	256.0	146.4	6.3	10.9	7.6	219.0	5.7	34.8	1.07	28.9
x31	31.1	251.4	146.1	6.0	8.6	7.6	219.0	7.3	36.5	1.06	34.0
254x102x28	28.3	260.4	102.2	6.3	10.0	7.6	225.2	4.0	35.7	0.904	31.9
x25	25.2	257.2	101.9	6.0	8.4	7.6	225.2	4.8	37.5	0.897	35.7
x22	22.0	254.0	101.6	5.7	6.8	7.6	225.2	5.9	39.5	0.890	40.5
203x133x30	30.0	206.8	133.9	6.4	9.6	7.6	172.4	5.9	26.9	0.923	30.8
x25	25.1	203.2	133.2	5.7	7.8	7.6	172.4	7.2	30.2	0.915	36.5
203x102x23	23.1	203.2	101.8	5.4	9.3	7.6	169.4	4.4	31.4	0.790	34.2
178x102x19	19.0	177.8	101.2	4.8	7.9	7.6	146.8	5.1	30.6	0.738	38.7
152x89x16	16.0	152.4	88.7	4.5	7.7	7.6	121.8	4.5	27.1	0.638	40.0
127x76x13	13.0	127.0	76.0	4.0	7.6	7.6	96.6	3.7	24.2	0.537	41.4

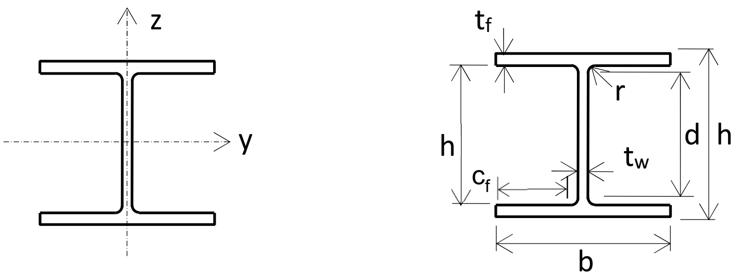
Design Table 02B Section properties of rolled I-sections (2)





IS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	I_y	I_z	$W_{el,y}$	$W_{el,z}$	$W_{pl,y}$	$W_{pl,z}$	u	x	I_w	I_T	A
	cm^4	cm^4	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
457x191x98	45700	2350	1960	243	2230	379	0.881	25.8	1.18	121	125
x89	41000	2090	1770	218	2010	338	0.878	28.3	1.04	90.7	114
x82	37100	1870	1610	196	1830	304	0.879	30.8	0.922	69.2	104
x74	33300	1670	1460	176	1650	272	0.877	33.8	0.818	51.8	94.6
x67	29400	1450	1300	153	1470	237	0.873	37.8	0.705	37.1	85.5
457x152x82	36600	1180	1570	153	1810	240	0.872	27.4	0.591	89.2	105
x74	32700	1050	1410	136	1630	213	0.872	30.1	0.518	65.9	94.5
x67	28900	913	1260	119	1450	187	0.868	33.6	0.448	47.7	85.6
x60	25500	795	1120	104	1290	163	0.868	37.5	0.387	33.8	76.2
x52	21400	645	950	84.6	1100	133	0.859	43.8	0.311	21.4	66.6
406x178x74	27300	1550	1320	172	1500	267	0.882	27.5	0.608	62.8	94.5
x67	24300	1360	1190	153	1350	237	0.880	30.4	0.533	46.1	85.5
x60	21600	1200	1060	135	1200	209	0.880	33.7	0.466	33.3	76.5
x54	18700	1020	930	115	1050	178	0.871	38.3	0.392	23.1	69.0
406x140x46	15700	538	778	75.7	888	118	0.871	39.0	0.207	19.0	58.6
x39	12500	410	629	57.8	724	90.8	0.858	47.4	0.155	10.7	49.7
356x171x67	19500	1360	1070	157	1210	243	0.886	24.4	0.412	55.7	85.5
x57	16000	1110	896	129	1010	199	0.882	28.8	0.330	33.4	72.6
x51	14100	968	796	113	896	174	0.881	32.1	0.286	23.8	64.9
x45	12100	811	687	94.8	775	147	0.874	36.8	0.237	15.8	57.3
356x127x39	10200	358	576	56.8	659	89.0	0.871	35.2	0.105	15.1	49.8
x33	8250	280	473	44.7	543	70.2	0.863	42.1	0.081	8.79	42.1
305x165x54	11700	1060	754	127	846	196	0.889	23.6	0.234	34.8	68.8
x46	9900	896	646	108	720	166	0.890	27.1	0.195	22.2	58.7
x40	8500	764	560	92.6	623	142	0.889	31.0	0.164	14.7	51.3
305x127x48	9570	461	616	73.6	711	116	0.873	23.3	0.102	31.8	61.2
x42	8200	389	534	62.6	614	98.4	0.872	26.5	0.0846	21.1	53.4
x37	7170	336	471	54.5	539	85.4	0.872	29.7	0.0725	14.8	47.2
305x102x33	6500	194	416	37.9	481	60.0	0.867	31.6	0.0442	12.2	41.8
x28	5370	155	348	30.5	403	48.4	0.859	37.3	0.0349	7.40	35.9
x25	4460	123	292	24.2	342	38.8	0.846	43.4	0.0270	4.77	31.6
254x146x43	6540	677	504	92.0	566	141	0.891	21.1	0.103	23.9	54.8
x37	5540	571	433	78.0	483	119	0.890	24.3	0.0857	15.3	47.2
x31	4410	448	351	61.3	393	94.1	0.879	29.6	0.0660	8.55	39.7
254x102x28	4000	179	308	34.9	353	54.8	0.873	27.5	0.0280	9.57	36.1
x25	3410	149	266	29.2	306	46.0	0.866	31.4	0.0230	6.42	32.0
x22	2840	119	224	23.5	259	37.3	0.856	36.3	0.0182	4.15	28.0
203x133x30	2900	385	280	57.5	314	88.2	0.882	21.5	0.0374	10.3	38.2
x25	2340	308	230	46.2	258	70.9	0.876	25.6	0.0294	5.96	32.0
203x102x23	2100	164	207	32.2	234	49.7	0.888	22.4	0.0154	7.02	29.4
178x102x19	1360	137	153	27.0	171	41.6	0.886	22.6	0.00990	4.41	24.3
152x89x16	834	89.8	109	20.2	123	31.2	0.890	19.5	0.00470	3.56	20.3
127x76x13	473	55.7	74.6	14.7	84.2	22.6	0.894	16.3	0.00200	2.85	16.5

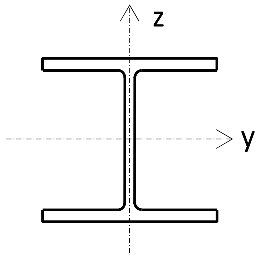
Design Table 03A Section dimensions of rolled H-sections

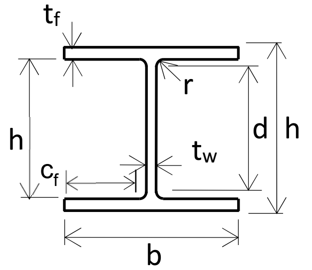


HS	Mass per Meter m kg/m	Depth of Section h mm	Width of Section b mm	Thickness Web t_w mm	Thickness Flange t_f mm	Root Radius r mm	Depth between Fillets d mm	Ratios for Local Buckling c_f/t_f	Ratios for Local Buckling d/t_w	Surface Area per Meter m²	Surface Area per Tonne m²
356x406x634#	633.9	474.6	424.0	47.6	77.0	15.2	290.2	2.3	6.1	2.52	3.98
x551#	551.0	455.6	418.5	42.1	67.5	15.2	290.2	2.6	6.9	2.47	4.48
x467#	467.0	436.6	412.2	35.8	58.0	15.2	290.2	3.0	8.1	2.42	5.18
x393#	393.0	419.0	407.0	30.6	49.2	15.2	290.2	3.5	9.5	2.38	6.06
x340#	339.9	406.4	403.0	26.6	42.9	15.2	290.2	4.0	10.9	2.35	6.91
x287#	287.1	393.6	399.0	22.6	36.5	15.2	290.2	4.7	12.8	2.31	8.05
x235#	235.1	381.0	394.8	18.4	30.2	15.2	290.2	5.7	15.8	2.28	9.70
356x368x202#	201.9	374.6	374.7	16.5	27.0	15.2	290.2	6.1	17.6	2.19	10.8
x177#	177.0	368.2	372.6	14.4	23.8	15.2	290.2	6.9	20.2	2.17	12.3
x153#	152.9	362.0	370.5	12.3	20.7	15.2	290.2	7.9	23.6	2.16	14.1
x129#	129.0	355.6	368.6	10.4	17.5	15.2	290.2	9.4	27.9	2.14	16.6
305x305x283	282.9	365.3	322.2	26.8	44.1	15.2	246.7	3.0	9.2	1.94	6.86
x240	240.0	352.5	318.4	23.0	37.7	15.2	246.7	3.5	10.7	1.91	7.96
x198	198.1	339.9	314.5	19.1	31.4	15.2	246.7	4.2	12.9	1.87	9.44
x158	158.1	327.1	311.2	15.8	25.0	15.2	246.7	5.3	15.6	1.84	11.6
x137	136.9	320.5	309.2	13.8	21.7	15.2	246.7	6.1	17.9	1.82	13.3
x118	117.9	314.5	307.4	12.0	18.7	15.2	246.7	7.1	20.6	1.81	15.4
x97	96.9	307.9	305.3	9.9	15.4	15.2	246.7	8.6	24.9	1.79	18.5
254x254x167	167.1	289.1	265.2	19.2	31.7	12.7	200.3	3.5	10.4	1.58	9.46
x132	132.0	276.3	261.3	15.3	25.3	12.7	200.3	4.4	13.1	1.55	11.7
x107	107.1	266.7	258.8	12.8	20.5	12.7	200.3	5.4	15.6	1.52	14.2
x89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	6.4	19.4	1.50	16.9
x73	73.1	254.1	254.6	8.6	14.2	12.7	200.3	7.8	23.3	1.49	20.4
203x203x86	86.1	222.2	209.1	12.7	20.5	10.2	160.8	4.3	12.7	1.24	14.4
x71	71.0	215.8	206.4	10.0	17.3	10.2	160.8	5.1	16.1	1.22	17.2
x60	60.0	209.6	205.8	9.4	14.2	10.2	160.8	6.2	17.1	1.21	20.2
x52	52.0	206.2	204.3	7.9	12.5	10.2	160.8	7.0	20.4	1.20	23.1
x46	46.1	203.2	203.6	7.2	11.0	10.2	160.8	8.0	22.3	1.19	25.8
152x152x37	37.0	161.8	154.4	8.0	11.5	7.6	123.6	5.7	15.5	0.912	24.7
x30	30.0	157.6	152.9	6.5	9.4	7.6	123.6	7.0	19.0	0.901	30.0
x23	23.0	152.4	152.2	5.8	6.8	7.6	123.6	9.7	21.3	0.889	38.7

\# Limited availability.

Design Table 03B Section properties of rolled H-sections

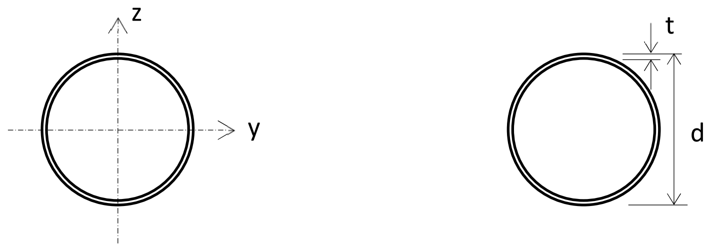




HS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
	I_y cm^4	I_z cm^4	$W_{el,y}$ cm^3	$W_{el,z}$ cm^3	$W_{pl,y}$ cm^3	$W_{pl,z}$ cm^3	u	x	I_w dm^6	I_T cm^4	A cm^2
356x406x634#	275000	98100	11600	4630	14200	7110	0.843	5.46	38.8	13700	808
x551#	227000	82700	9960	3950	12100	6060	0.841	6.05	31.1	9240	702
x467#	183000	67800	8380	3290	10000	5030	0.839	6.86	24.3	5810	595
x393#	147000	55400	7000	2720	8220	4150	0.837	7.86	18.9	3550	501
x340#	123000	46900	6030	2330	7000	3540	0.836	8.85	15.5	2340	433
x287#	99900	38700	5070	1940	5810	2950	0.835	10.2	12.3	1440	366
x235#	79100	31000	4150	1570	4690	2380	0.834	12.1	9.54	812	299
356x368x202#	66300	23700	3540	1260	3970	1920	0.844	13.4	7.16	558	257
x177#	57100	20500	3100	1100	3460	1670	0.844	15.0	6.09	381	226
x153#	48600	17600	2680	948	2960	1430	0.844	17.0	5.11	251	195
x129#	40200	14600	2260	793	2480	1200	0.844	19.9	4.18	153	164
305x305x283	78900	24600	4320	1530	5110	2340	0.855	7.65	6.35	2030	360
x240	64200	20300	3640	1280	4250	1950	0.854	8.74	5.03	1270	306
x198	50900	16300	3000	1040	3440	1580	0.854	10.2	3.88	734	252
x158	38700	12600	2370	808	2680	1230	0.851	12.5	2.87	378	201
x137	32800	10700	2050	692	2300	1050	0.851	14.2	2.39	249	174
x118	27700	9060	1760	589	1960	895	0.850	16.2	1.98	161	150
x97	22200	7310	1450	479	1590	726	0.850	19.3	1.56	91.2	123
254x254x167	30000	9870	2080	744	2420	1140	0.851	8.49	1.63	626	213
x132	22500	7530	1630	576	1870	878	0.850	10.3	1.19	319	168
x107	17500	5930	1310	458	1480	697	0.848	12.4	0.898	172	136
x89	14300	4860	1100	379	1220	575	0.850	14.5	0.717	102	113
x73	11400	3910	898	307	992	465	0.849	17.3	0.562	57.6	93.1
203x203x86	9450	3130	850	299	977	456	0.850	10.2	0.318	137	110
x71	7620	2540	706	246	799	374	0.853	11.9	0.250	80.2	90.4
x60	6130	2070	584	201	656	305	0.846	14.1	0.197	47.2	76.4
x52	5260	1780	510	174	567	264	0.848	15.8	0.167	31.8	66.3
x46	4570	1550	450	152	497	231	0.847	17.7	0.143	22.2	58.7
152x152x37	2210	706	273	91.5	309	140	0.848	13.3	0.0399	19.2	47.1
x30	1750	560	222	73.3	248	112	0.849	16.0	0.0308	10.5	38.3
x23	1250	400	164	52.6	182	80.1	0.840	20.7	0.0212	4.63	29.2

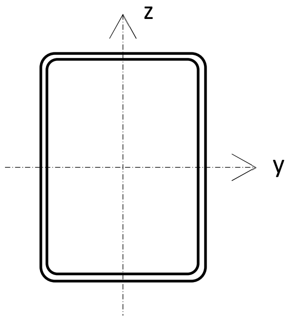
\# Limited availability.

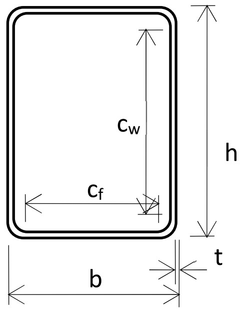
Design Table 04 Section dimensions and properties of hot-finished CHS



CHS	Mass per Meter	Area of Section	Ratio for Local Buckling	Second Moment of Area	Elastic Modulus	Plastic Modulus	Torsional Constants		Surface Area	
dxt	*m*	*A*	*d/t*	*I*	W_{el}	W_{pl}	I_T	W_t	*per Meter*	*per Tonne*
mmxmm	kg/m	cm²		cm⁴	cm³	cm³	cm⁴	cm³	m²	m²
139.7x6.3	20.7	26.4	22.2	589	84.3	112	1180	169	0.439	21.2
x8.0	26.0	33.1	17.5	720	103	139	1440	206	0.439	16.9
168.3x6.3	25.2	32.1	26.7	1050	125	165	2110	250	0.529	21.0
x8.0	31.6	40.3	21.0	1300	154	206	2600	308	0.529	16.7
x10.0	39.0	49.7	16.8	1560	186	251	3130	372	0.529	13.6
x12.5	48.0	61.2	13.5	1870	222	304	3740	444	0.529	11.0
219.1x6.3	33.1	42.1	34.8	2390	218	285	4770	436	0.688	20.8
x8.0	41.6	53.1	27.4	2960	270	357	5920	540	0.688	16.5
x10.0	51.6	65.7	21.9	3600	328	438	7200	657	0.688	13.3
x12.5	63.7	81.1	17.5	4350	397	534	8690	793	0.688	10.8
273.0x6.3	41.4	52.8	43.3	4700	344	448	9390	688	0.858	20.7
x8.0	52.3	66.6	34.1	5850	429	562	11700	857	0.858	16.4
x10.0	64.9	82.6	27.3	7150	524	692	14300	1050	0.858	13.2
x12.5	80.3	102	21.8	8700	637	849	17400	1270	0.858	10.7
323.9x6.3	49.3	62.9	51.4	7930	490	636	15900	979	1.02	20.7
x8.0	62.3	79.4	40.5	9910	612	799	19800	1220	1.02	16.4
x10.0	77.4	98.6	32.4	12200	751	986	24300	1500	1.02	13.2
x12.5	96.0	122	25.9	14800	917	1210	29700	1830	1.02	10.6
x16.0	121	155	20.2	18400	1140	1520	36800	2270	1.02	8.40
355.6x6.3	54.3	69.1	56.4	10500	593	769	21100	1190	1.12	20.6
x8.0	68.6	87.4	44.5	13200	742	967	26400	1480	1.12	16.3
x10.0	85.2	109	35.6	16200	912	1200	32400	1820	1.12	13.1
x12.5	106	135	28.4	19900	1120	1470	39700	2230	1.12	10.5
x16.0	134	171	22.2	24700	1390	1850	49300	2770	1.12	8.31
406.4x8.0	78.6	100	50.8	19900	978	1270	39700	1960	1.28	16.3
x10.0	97.8	125	40.6	24500	1210	1570	49000	2410	1.28	13.1
x12.5	121	155	32.5	30000	1480	1940	60100	2960	1.28	10.5
x16.0	154	196	25.4	37500	1840	2440	74900	3690	1.28	8.31
x20.0	191	243	20.3	45430	2240	2989	90860	4470	1.28	6.70
457.0x8.0	88.6	113	57.1	28500	1250	1610	56900	2490	1.44	16.3
x10.0	110	140	45.7	35100	1540	2000	70200	3070	1.44	13.1
x12.5	137	175	36.6	43100	1890	2470	86300	3780	1.44	10.5
x16.0	174	222	28.6	54000	2360	3110	108000	4730	1.44	8.28
x20.0	216	275	22.9	65680	2870	3822	131400	5750	1.44	6.67
508.0x8.0	98.6	126	63.5	39300	1550	2000	78600	3090	1.60	16.2
x10.0	123	156	50.8	48500	1910	2480	97000	3820	1.60	13.0
x12.5	153	195	40.6	59800	2350	3070	120000	4710	1.60	10.5
x16.0	194	247	31.8	74900	2950	3870	150000	5900	1.60	8.25
x20.0	241	307	25.4	91400	3600	4770	183000	7200	1.60	6.64
610.8x8.0	119	151	76.3	84900	2250	2900	137100	4495	1.90	16.1
x10.0	148	188	61.0	104800	2780	3600	169700	5564	1.90	12.9
x12.5	184	235	48.8	118000	3450	4460	209600	6869	1.90	10.4
x16.0	234	299	38.1	131800	4320	5650	263600	8641	1.90	8.19
x20.0	291	371	30.5	161500	5300	6970	323000	10590	1.90	6.59
711.0x10.0	173	220	71.1	135300	3810	4914	270600	7612	2.23	12.9
x12.5	215	274	56.9	167300	4710	6100	334700	9415	2.23	10.4
x16.0	274	349	44.4	211000	5940	7730	422100	11870	2.23	8.15
x20.0	341	434	35.6	259400	7300	9550	518700	14590	2.23	6.54
813.0x10.0	198	252	81.3	203400	5000	6450	406800	10010	2.55	12.9
x12.5	247	314	65.0	251900	6200	8010	503700	12390	2.55	10.3
x16.0	314	401	50.8	318200	7830	10200	636400	15660	2.55	8.13
x20.0	391	498	40.7	391900	9640	12600	783800	19280	2.55	6.53

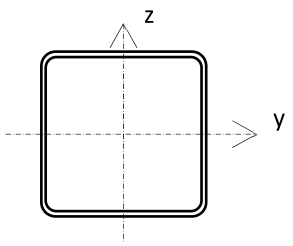
Design Table 05 Section dimensions and properties of hot-finished RHS

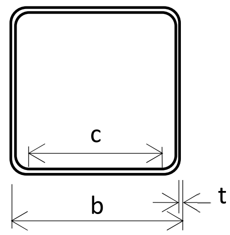




RHS	Mass per Meter	Area of Section	Ratio for Local Buckling		Second Moment of Area		Elastic Modulus		Plastic Modulus		Surface Area	
bxhxt mmxmmxmm	***m*** **kg/m**	***A*** **cm^2**	c_w/t	c_f/t	I_y cm^4	I_z cm^4	$W_{el,y}$ cm^3	$W_{el,z}$ cm^3	$W_{pl,y}$ cm^3	$W_{pl,z}$ cm^3	***per Meter*** **m^2**	***per Tonne*** **m^2**
120x80x6.3	18.2	23.2	16.0	9.70	440	230	73.3	57.6	91.0	68.2	0.384	21.1
x8.0	22.6	28.8	12.0	7.00	525	273	87.5	68.1	111	82.6	0.379	16.8
160x80x6.3	22.2	28.2	22.4	9.70	903	299	113	74.8	142	86.8	0.464	20.9
x8.0	27.6	35.2	17.0	7.00	1090	356	136	89.0	175	106	0.459	16.6
x10.0	33.7	42.9	13.0	5.00	1280	411	161	103	209	125	0.454	13.5
200x100x6.3	28.1	35.8	28.7	12.9	1830	613	183	123	228	140	0.584	20.8
x8.0	35.1	44.8	22.0	9.50	2230	739	223	148	282	172	0.579	16.5
x10.0	43.1	54.9	17.0	7.00	2660	869	266	174	341	206	0.574	13.3
200x150x6.3	33.0	42.1	28.7	20.8	2420	1550	242	207	289	237	0.684	20.7
x8.0	41.4	52.8	22.0	15.8	2970	1890	297	253	359	294	0.679	16.4
x10.0	51.0	64.9	17.0	12.0	3570	2260	357	302	436	356	0.674	13.2
250x150x6.3	38.0	48.4	36.7	20.8	4140	1870	331	249	402	282	0.784	20.6
x8.0	47.7	60.8	28.3	15.8	5110	2300	409	307	501	350	0.779	16.4
x10.0	58.8	74.9	22.0	12.0	6170	2760	494	367	611	426	0.774	13.2
x12.5	72.3	92.1	17.0	9.00	7390	3260	591	435	740	514	0.768	10.6
260x180x6.3	40.9	52.1	35.3	22.6	4950	2820	381	313	458	357	0.860	23.8
x8.0	52.7	67.2	29.5	19.5	6390	3610	492	401	592	459	0.859	16.3
x10.0	65.1	82.9	23.0	15.0	7740	4350	595	483	724	560	0.854	13.1
x12.5	80.1	102	17.8	11.4	9300	5200	715	577	879	679	0.848	10.6
x16.0	100	128	13.3	8.25	11200	6230	865	692	1080	831	0.839	8.39
300x200x6.3	47.9	61.0	44.6	28.7	7830	4190	522	419	624	472	0.984	20.5
x8.0	60.3	76.8	34.5	22.0	9720	5180	648	518	779	589	0.979	16.2
x10.0	74.5	94.9	27.0	17.0	11800	6280	788	628	956	721	0.974	13.1
x12.5	91.9	117	21.0	13.0	14300	7540	952	754	1160	877	0.968	10.5
x16.0	115	147	15.8	9.50	17400	9110	1160	911	1440	1080	0.959	8.34
350x250x6.3	57.8	73.6	52.6	36.7	13200	7880	754	631	892	709	1.180	20.4
x8.0	72.8	92.8	40.8	28.3	16400	9800	940	784	1120	888	1.180	16.2
x10.0	90.2	115	32.0	22.0	20100	11900	1150	955	1380	1090	1.170	13.0
x12.5	112	142	25.0	17.0	24400	14400	1400	1160	1680	1330	1.170	10.4
x16.0	141	179	18.9	12.6	30000	17700	1720	1410	2100	1660	1.160	8.23
400x200x6.3	57.8	73.6	60.5	28.7	15700	5380	785	538	960	594	1.18	20.4
x8.0	72.8	92.8	47.0	22.0	19600	6660	978	666	1200	743	1.18	16.2
x10.0	90.2	115	37.0	17.0	23900	8080	1200	808	1480	911	1.17	13.0
x12.5	112	142	29.0	13.0	29100	9740	1450	974	1810	1110	1.17	10.4
x16.0	141	179	22.0	9.50	35700	11800	1790	1180	2260	1370	1.16	8.23
450x250x8.0	85.4	109	53.3	28.3	30100	12100	1340	971	1620	1080	1.38	16.2
x10.0	106	135	42.0	22.0	36900	14800	1640	1180	2000	1330	1.37	12.9
x12.5	131	167	33.0	17.0	45000	18000	2000	1440	2460	1630	1.37	10.5
x16.0	166	211	25.1	12.6	55700	22000	2480	1760	3070	2030	1.36	8.19
500x300x8.0	97.9	125	59.5	34.5	43700	20000	1750	1330	2100	1480	1.58	16.1
x10.0	122	155	47.0	27.0	53800	24400	2150	1630	2600	1830	1.57	12.9
x12.5	151	192	37.0	21.0	65800	29800	2630	1980	3200	2240	1.57	10.4
x16.0	191	243	28.3	15.8	81800	36800	3270	2450	4000	2800	1.56	8.17
x20.0	225	287	19.0	9.0	90700	41200	3630	2750	4560	3210	1.55	6.59

Design Table 06 Section dimensions and properties of hot-finished SHS





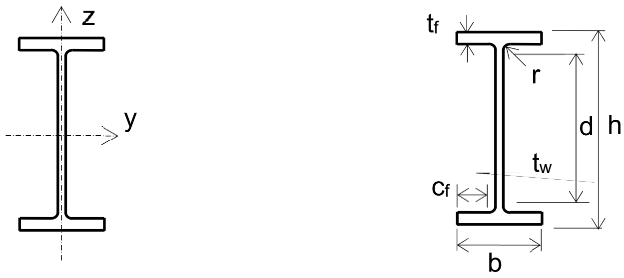
SHS	Mass per Meter	Area of Section	Ratio for Local Buckling	Second Moment of Area	Elastic Modulus	Plastic Modulus	Surface Area	
bxbxt mmxmmxmm	*m* kg/m	*A* cm²	*c/t*	*I* cm⁴	W_{el} cm³	W_{pl} cm³	*per Meter* m²	*per Tonne* m²
100x100x6.3	18.2	23.2	12.9	336	67.1	80.9	0.384	21.1
x8.0	22.6	28.8	9.50	400	79.9	98.2	0.379	16.8
150x150x6.3	28.1	35.8	20.8	1220	163	192	0.584	20.8
x8.0	35.1	44.8	15.8	1490	199	237	0.579	16.5
x10.0	43.1	54.9	12.0	1770	236	286	0.574	13.3
200x200x6.3	38.0	48.4	28.7	3010	301	350	0.784	20.6
x8.0	47.7	60.8	22.0	3710	371	436	0.779	16.3
x10.0	58.8	74.9	17.0	4470	447	531	0.774	13.2
x12.5	72.3	92.1	13.0	5340	534	643	0.768	10.6
220x220x6.3	40.9	52.1	28.9	3890	354	413	0.848	20.7
x8.0	52.7	67.2	24.5	5000	455	532	0.859	16.3
x10.0	65.1	82.9	19.0	6050	550	650	0.854	13.1
x12.5	80.1	102	14.6	7250	659	789	0.848	10.6
x16.0	100	128	10.8	8750	795	969	0.839	8.39
250x250x6.3	47.9	61.0	36.7	6010	481	556	0.984	20.5
x8.0	60.3	76.8	28.3	7460	596	694	0.979	16.2
x10.0	74.5	94.9	22.0	9060	724	851	0.974	13.1
x12.5	91.9	117	17.0	10900	873	1040	0.968	10.5
x16.0	115	147	12.6	13300	1060	1280	0.959	8.34
300x300x6.3	57.8	73.6	44.6	10500	703	809	1.18	20.4
x8.0	72.8	92.8	34.5	13100	875	1010	1.18	16.2
x10.0	90.2	115	27.0	16000	1070	1250	1.17	13.0
x12.5	112	142	21.0	19400	1290	1520	1.17	10.4
x16.0	141	179	15.8	23800	1590	1900	1.16	8.23
350x350x8.0	85.4	109	40.8	21100	1210	1390	1.38	16.2
x10.0	106	135	32.0	25900	1480	1720	1.37	12.9
x12.5	131	167	25.0	31500	1800	2110	1.37	10.5
x16.0	166	211	18.9	38900	2220	2630	1.36	8.19
400x400x8.0	97.9	125	47.0	31900	1590	1830	1.58	16.1
x10.0	122	155	37.0	39100	1960	2260	1.57	12.9
x12.5	151	192	29.0	47800	2390	2780	1.57	10.4
x16.0	191	243	22.0	59300	2970	3480	1.56	8.17
x20.0	235	300	14.0	71540	3580	4250	1.55	6.60

Design Tables 07 to 12 for

Section Resistances of Rolled Sections: S275 Steel

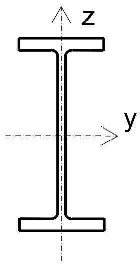
- I-sections
- H-sections
- CHS
- RHS
- SHS

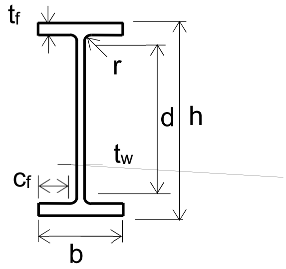
Design Table 07   Section resistances of rolled I-sections: S275 steel (1)



IS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	***EI**$_y$* 10^3*kNm^2	***EI**$_z$* 10^3*kNm^2	**Bending** *y-y*	**Bending** *z-z*	***M**$_{y,Rd}$* **kNm**	***M**$_{z,Rd}$* **kNm**	***V**$_{z,Rd}$* **kN**	***N**$_{a,Rd}$* **kN**
914x419x388#	1512	95.3	1	1	4690	885	3240	13100
x343#	1315	82.3	1	1	4110	766	2920	11200
914x305x289#	1058	32.8	1	1	3340	424	2900	9340
x253#	916	27.9	1	1	2890	363	2570	7930
x224#	790	23.5	1	1	2530	307	2350	6840
x201#	683	19.8	1	1	2210	260	2210	5990
838x292x226#	714	23.9	1	1	2430	321	2220	7140
x194#	586	19.0	1	1	2020	258	2000	5920
x176#	517	16.4	1	1	1800	223	1890	5260
762x267x197	504	17.2	1	1	1900	254	1950	6310
x173	431	14.4	1	1	1640	214	1760	5390
x147	355	11.5	1	1	1370	171	1560	4420
x134	317	10.1	1	1	1280	157	1520	4100
686x254x170	357	13.9	1	1	1490	215	1630	5520
x152	315	12.1	1	1	1330	188	1470	4810
x140	286	10.9	1	1	1210	169	1370	4340
x125	248	9.20	1	1	1060	144	1280	3790
610x305x238	439	33.2	1	1	1980	416	1890	8030
x179	321	23.9	1	1	1470	302	1440	6040
x149	265	19.6	1	1	1220	248	1200	4790
610x229x140	235	9.47	1	1	1100	162	1300	4540
x125	207	8.25	1	1	975	142	1170	3960
x113	183	7.20	1	1	869	124	1090	3520
x101	159	6.11	1	1	792	110	1060	3200
533x210x122	160	7.12	1	1	848	133	1110	4110
x109	140	6.17	1	1	750	116	1020	3560
x101	129	5.65	1	1	692	106	952	3250
x92	116	5.02	1	1	649	97.6	909	3010
x82	100	4.22	1	1	567	82.5	865	2650

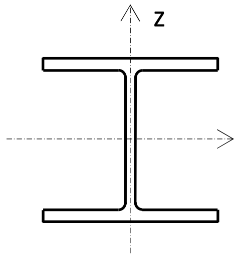
Design Table 08 Section resistances of rolled I-sections: S275 steel (2)

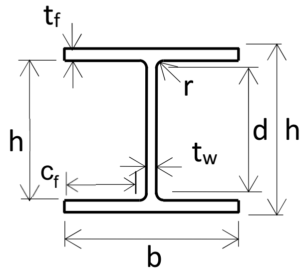




IS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
457x191x98	96.0	4.94	1	1	591	100	852	3310
x89	86.1	4.39	1	1	533	89.6	789	3020
x82	77.9	3.93	1	1	503	83.6	756	2770
x74	69.9	3.51	1	1	454	74.8	693	2460
x67	61.7	3.05	1	1	404	65.2	650	2190
457x152x82	76.9	2.48	1	1	480	63.6	798	2780
x74	68.7	2.21	1	1	432	56.4	721	2400
x60	60.7	1.92	1	1	399	51.4	697	2220
x60	53.6	1.67	1	1	355	44.8	624	1920
x52	44.9	1.35	1	1	303	36.6	578	1630
406x178x74	57.3	3.26	1	1	398	70.8	635	2500
x67	51.0	2.86	1	1	371	65.2	574	2270
x60	45.4	2.52	1	1	330	57.5	532	1970
x54	39.3	2.14	1	1	289	49.0	478	1770
406x140x46	33.0	1.13	1	1	244	32.5	473	1460
x39	26.3	0.861	1	1	199	25.0	438	1200
356x171x67	41.0	2.86	1	1	333	66.8	568	2350
x57	33.6	2.33	1	1	278	54.7	501	2000
x51	29.6	2.03	1	1	246	47.9	455	1720
x45	25.4	1.70	1	1	213	40.4	425	1500
356x127x39	21.4	0.752	1	1	181	24.5	408	1280
x33	17.3	0.588	1	1	149	19.3	366	1050
305x165x54	24.6	2.23	1	1	233	53.9	422	1890
x46	20.8	1.88	1	1	198	45.7	357	1580
x40	17.9	1.60	1	1	171	39.1	319	1360
305x127x48	20.1	0.968	1	1	196	31.9	474	1680
x42	17.2	0.817	1	1	169	27.1	420	1470
x37	15.1	0.706	1	1	148	23.5	372	1300
305x102x33	13.7	0.407	1	1	132	16.5	350	1100
x28	11.3	0.326	1	1	111	13.3	315	921
x25	9.37	0.258	1	1	94.1	10.7	299	797
254x146x43	13.7	1.42	1	1	156	38.8	321	1510
x37	11.6	1.20	1	1	133	32.7	280	1300
x31	9.26	0.941	1	1	108	25.9	260	1090
254x102x28	8.40	0.376	1	1	97.1	15.1	283	993
x25	7.16	0.313	1	1	84.2	12.7	265	880
x22	5.96	0.250	1	1	71.2	10.3	248	749
203x133x30	6.09	0.809	1	1	86.4	24.3	231	1050
x26	4.91	0.647	1	1	71.0	19.5	204	880
203x102x23	4.41	0.344	1	1	64.4	13.7	197	809
178x102x19	2.86	0.288	1	1	47.0	11.4	157	668
152x89x16	1.75	0.189	1	1	33.8	8.58	130	558
127x76x13	0.993	0.117	1	1	23.2	6.22	102	454

Design Table 09   Section resistances of rolled H-sections: S275 steel

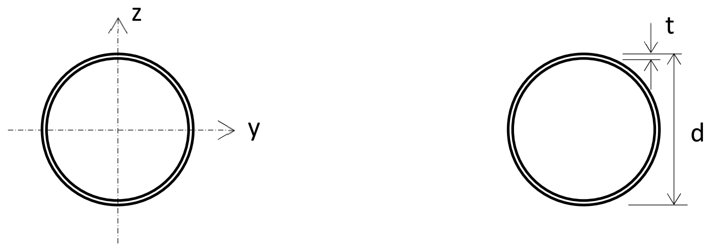




HS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	***EI*$_y$** 10^3*kNm^2	***EI*$_z$** 10^3*kNm^2	**Bending** *y-y*	**Bending** *z-z*	***M*$_{y,Rd}$** kNm	***M*$_{z,Rd}$** kNm	***V*$_{z,Rd}$** kN	***N*$_{a,Rd}$** kN
356x406x634#	578	206	1	1	3480	1740	3040	19800
x551#	477	174	1	1	2960	1480	2630	17200
x467#	384	142	1	1	2550	1280	2290	15200
x393#	309	116	1	1	2100	1060	1920	12800
x340#	258	98.5	1	1	1790	903	1640	11000
x287#	210	81.3	1	1	1540	782	1440	9700
x235#	166	65.1	1	1	1240	631	1150	7920
356x368x202#	139	49.8	1	1	1050	509	1030	6810
x177#	120	43.1	1	1	917	443	907	5990
x153#	102	37.0	1	1	784	379	772	5170
x129#	84.4	30.7	3	3	599	210	645	4350
305x305x283	166	51.7	1	1	1300	597	1490	9180
x240	135	42.6	1	1	1130	517	1320	8110
x198	107	34.2	1	1	912	419	1070	6680
x158	81.3	26.5	1	1	710	326	871	5330
x137	68.9	22.5	1	1	610	278	756	4610
x118	58.2	19.0	1	1	519	237	657	3980
x97	46.6	15.4	2	2	437	200	558	3380
254x254x167	63.0	20.7	1	1	641	302	903	5640
x132	47.3	15.8	1	1	496	233	705	4450
x107	36.8	12.5	1	1	392	185	577	3600
x89	30.0	10.2	1	1	323	152	467	2990
x73	23.9	8.21	1	1	273	128	407	2560
203x203x86	19.8	6.57	1	1	259	121	475	2920
x71	16.0	5.33	1	1	212	99.1	371	2400
x60	12.9	4.35	1	1	180	83.9	352	2100
x52	11.0	3.74	1	1	156	72.6	298	1820
x46	9.60	3.26	1	1	137	63.5	269	1610
152x152x37	4.64	1.48	1	1	85.0	38.5	226	1300
x30	3.68	1.18	1	1	68.2	30.8	184	1050
x23	2.63	0.840	3	3	45.1	14.5	158	803

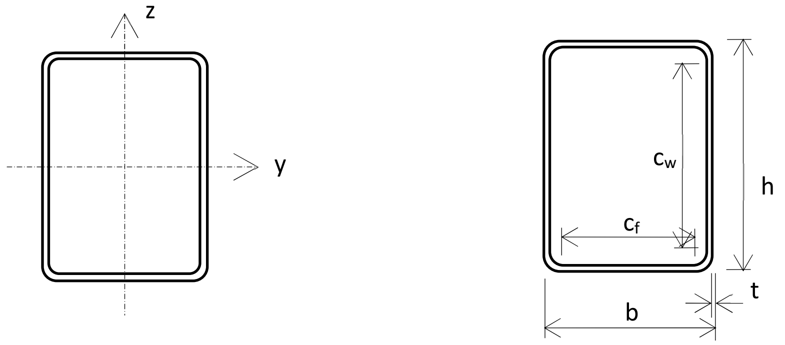
\#   Limited availability.

Design Table 10 Section resistances of hot-finished CHS: S275 steel



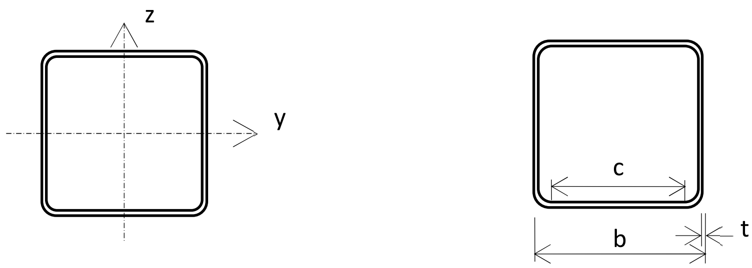
CHS	Flexural rigidity	S275			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
dxt mmxmm	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
139.7x6.3	1.24	1	30.8	267	726
x8.0	1.51	1	38.2	335	910
168.3x6.3	2.21	1	45.4	324	883
x8.0	2.73	1	56.7	407	1110
x10.0	3.28	1	69.0	502	1370
x12.5	3.93	1	83.6	619	1680
219.1x6.3	5.02	1	78.4	426	1160
x8.0	6.22	1	98.2	537	1460
x10.0	7.56	1	120	664	1810
x12.5	9.14	1	147	820	2230
273.0x6.3	9.87	2	123	534	1450
x8.0	12.3	1	155	673	1830
x10.0	15.0	1	190	835	2270
x12.5	18.3	1	233	1030	2810
323.9x6.3	16.7	2	175	636	1730
x8.0	20.8	1	220	803	2180
x10.0	25.6	1	271	997	2710
x12.5	31.1	1	333	1230	3360
x16.0	38.6	1	418	1570	4260
355.6x6.3	22.1	2	211	698	1900
x8.0	27.7	2	266	883	2400
x10.0	34.0	1	330	1100	3000
x12.5	41.8	1	404	1360	3710
x16.0	51.9	1	509	1730	4700
406.4x8.0	41.8	2	349	1010	2750
x10.0	51.5	1	432	1260	3440
x12.5	63.0	1	534	1570	4260
x16.0	78.5	1	671	1980	5390
x20.0	95.4	1	792	2370	6440
457.0x8.0	59.6	2	443	1140	3110
x10.0	73.7	2	550	1420	3850
x12.5	90.5	1	679	1770	4810
x16.0	113	1	855	2240	6110
x20.0	138	1	1010	2680	7290
508.0x8.0	82.5	3	426	1270	3470
x10.0	102	2	682	1580	4290
x12.5	126	1	844	1970	5360
x16.0	157	1	1060	2500	6790
x20.0	192	1	1260	2990	8140
610.8x8.0	178	3	619	1530	4150
x10.0	220	3	765	1900	5170
x12.5	248	2	1230	2380	6460
x16.0	277	1	1550	3020	8220
x20.0	339	1	1850	3610	9830
711.0x10.0	284	3	1050	1750	4760
x12.5	351	2	1680	2170	5910
x16.0	443	2	2130	2770	7540
x20.0	545	1	2530	3320	9040
813.0x10.0	427	4	-	-	-
x12.5	529	3	1710	3170	8640
x16.0	668	2	2810	4050	11000
x20.0	823	1	3340	4850	13200

Design Table 11 Section resistances of hot-finished RHS: S275 steel



RHS	Flexural rigidity		S275					
			Section Classification		Moment resistance		Shear Resistance	Axial Resistance
hxbxt mmxmmxmm	**EI_y 10^3*kNm^2**	**EI_z 10^3*kNm^2**	**Bending** *y-y*	**Bending** *z-z*	**$M_{y,Rd}$ kNm**	**$M_{z,Rd}$ kNm**	**$V_{z,Rd}$ kN**	**$N_{a,Rd}$ kN**
120x80x6.3	0.923	0.483	1	1	25.0	18.8	221	638
x8.0	1.10	0.571	1	1	30.4	22.7	274	792
160x80x6.3	1.90	0.628	1	1	39.2	23.9	298	776
x8.0	2.29	0.748	1	1	48.0	29.0	373	968
x10.0	2.69	0.863	1	1	57.4	34.4	454	1180
200x100x6.3	3.84	1.29	1	1	62.8	38.5	379	985
x8.0	4.68	1.55	1	1	77.5	47.2	474	1230
x10.0	5.59	1.82	1	1	93.7	56.7	581	1510
200x150x6.3	5.08	3.26	1	1	79.6	65.2	382	1160
x8.0	6.24	3.97	1	1	98.6	80.7	479	1450
x10.0	7.50	4.75	1	1	120	97.9	589	1780
250x150x6.3	8.69	3.93	1	3	111	68.6	480	1330
x8.0	10.7	4.83	1	1	138	96.4	603	1670
x10.0	13.0	5.78	1	1	168	117	743	2060
x12.5	15.5	6.85	1	1	203	141	914	2530
260x180x6.3	10.9	6.15	1	3	131	89.5	489	1430
x8.0	13.4	7.58	1	1	163	126	630	1850
x10.0	16.3	9.14	1	1	199	154	778	2280
x12.5	19.5	10.9	1	1	242	187	957	2810
x16.0	23.6	13.1	1	1	297	228	1200	3520
300x200x6.3	16.4	8.80	1	4	172	-	581	1550
x8.0	20.4	10.9	1	2	214	162	732	2110
x10.0	24.8	13.2	1	1	263	198	904	2610
x12.5	30.0	15.8	1	1	322	241	1110	3220
x16.0	36.5	19.1	1	1	396	297	1400	4040
350x250x6.3	27.7	16.5	3	4	207	-	682	1770
x8.0	34.4	20.6	1	4	308	-	859	2440
x10.0	42.2	25.0	1	2	380	300	1070	3160
x12.5	51.2	30.2	1	1	462	366	1320	3910
x16.0	63.0	37.0	1	1	575	454	1660	4920
400x200x6.3	33.0	11.3	1	4	264	-	779	1630
x8.0	41.2	14.0	1	4	330	-	982	2290
x10.0	50.2	17.0	1	3	407	222	1220	3160
x12.5	61.1	20.4	1	1	498	305	1500	3910
x16.0	75.0	24.8	1	1	622	377	1890	4920
450x250x8.0	63.2	25.4	1	4	446	-	1110	2570
x10.0	77.5	31.1	1	4	550	-	1380	3500
x12.5	94.5	37.8	1	2	677	448	1700	4590
x16.0	117	46.2	1	1	844	558	2150	5800
500x300x8.0	91.8	41.8	2	4	578	-	1240	2830
x10.0	113	51.2	1	4	712	-	1540	3860
x12.5	138	62.6	1	3	880	547	1910	5280
x16.0	172	77.3	1	1	1100	770	2410	6680
x20.0	207	92.6	1	1	1290	904	2740	7610

Design Table 12 Section resistances of hot-finished SHS: S275 steel



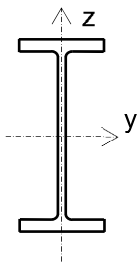
SHS	Flexural rigidity	S275			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
bxbxt **mmxmmxmm**	***EI***$_y$ **10^3*kNm^2**	**Bending** *y-y*	***M***$_{y,Rd}$ **kNm**	***V***$_{z,Rd}$ **kN**	***N***$_{a,Rd}$ **kN**
100x100x6.3	0.704	1	22.2	184	638
x8.0	0.838	1	27.0	229	792
150x150x6.3	2.56	1	52.8	284	985
x8.0	3.13	1	65.1	356	1230
x10.0	3.72	1	78.7	436	1510
200x200x6.3	6.32	1	96.3	384	1330
x8.0	7.79	1	120	483	1670
x10.0	9.39	1	146	595	2060
x12.5	11.2	1	177	731	2530
220x220x6.3	8.51	2	117	414	1430
x8.0	10.5	1	146	533	1850
x10.0	12.7	1	179	658	2280
x12.5	15.2	1	217	810	2810
x16.0	18.4	1	266	1020	3520
250x250x6.3	12.6	3	132	484	1680
x8.0	15.6	1	191	610	2110
x10.0	19.0	1	234	753	2610
x12.5	22.9	1	285	929	3220
x16.0	27.9	1	352	1170	4040
300x300x6.0	22.1	4	-	584	1770
x8.0	27.5	2	279	737	2550
x10.0	33.6	1	344	913	3160
x12.5	40.7	1	418	1130	3910
x16.0	50.0	1	520	1420	4920
350x350x8.0	44.3	4	-	865	2780
x10.0	54.4	2	473	1070	3710
x12.5	66.2	1	580	1330	4590
x16.0	81.7	1	723	1680	5800
400x400x8.0	67.0	4	-	992	2920
x10.0	82.1	3	539	1230	4260
x12.5	100	1	765	1520	5280
x16.0	125	1	957	1930	6680
x20.0	150	1	1130	2290	7950

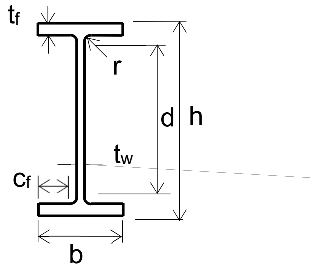
Design Tables 13 to 18 for

Section Resistances of Rolled Sections: S355 Steel

- I-sections
- H-sections
- CHS
- RHS
- SHS

Design Table 13 Section resistances of rolled I-sections: S355 steel (1)



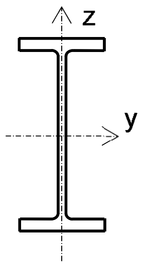


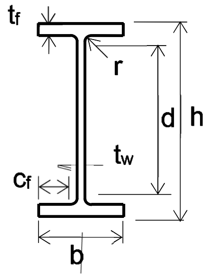
IS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI$_y$ **10^3*kNm^2**	***EI$_z$*** **10^3*kNm^2**	**Bending** *y-y*	**Bending** *z-z*	***M$_{y,Rd}$*** **kNm**	***M$_{z,Rd}$*** **kNm**	***V$_{z,Rd}$*** **kN**	***N$_{a,Rd}$*** **kN**
914x419x388#	1512	95.3	1	1	6110	1150	4220	16500
x343#	1315	82.3	1	1	5350	997	3800	14200
914x305x289#	1058	32.8	1	1	4350	552	3780	11800
x253#	916	27.9	1	1	3760	473	3350	9970
x224#	790	23.5	1	1	3290	400	3060	8590
x201#	683	19.8	1	1	2880	339	2870	7500
838x292x226#	714	23.9	1	1	3160	417	2900	8990
x194#	586	19.0	1	1	2640	336	2610	7430
x176#	517	16.4	1	1	2350	290	2460	6590
762x267x197	504	17.2	1	1	2470	331	2530	7950
x173	431	14.4	1	1	2140	278	2290	6770
x147	355	11.5	1	1	1780	223	2040	5540
x134	317	10.1	1	1	1650	202	1720*	5090
686x254x170	357	13.9	1	1	1940	280	2120	6960
x152	315	12.1	1	1	1730	245	1920	6060
x140	286	10.9	1	1	1570	220	1790	5460
x125	248	9.20	1	1	1380	187	1670	4760
610x305x238	439	33.2	1	1	2580	542	2460	10500
x179	321	23.9	1	1	1910	393	1880	7570
x149	265	19.6	1	1	1580	323	1570	6080
610x229x140	235	9.47	1	1	1430	211	1690	5740
x125	207	8.25	1	1	1270	185	1520	5000
x113	183	7.20	1	1	1130	162	1420	4430
x101	159	6.11	1	1	1020	142	1370	3990
533x210x122	160	7.12	1	1	1100	173	1450	5140
x109	140	6.27	1	1	976	150	1330	4500
x101	129	5.65	1	1	900	138	1240	4110
x92	116	5.02	1	1	838	126	1170	3760
x82	100	4.22	1	1	731	107	1120	3310

\# Limited availability.

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 14   Section resistances of rolled I-sections: S355 steel (2)

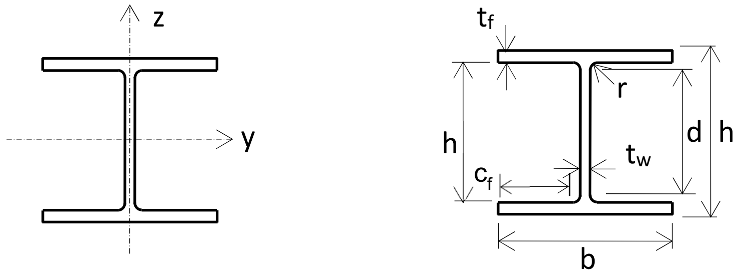




IS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	**EI_z** 10^3*kNm^2	**Bending** *y-y*	**Bending** *z-z*	**$M_{y,Rd}$** kNm	**$M_{z,Rd}$** kNm	**$V_{z,Rd}$** kN	**$N_{a,Rd}$** kN
457x191x98	96.0	4.94	1	1	769	131	1110	4190
x89	86.1	4.39	1	1	693	117	1030	3750
x82	77.9	3.93	1	1	650	108	976	3470
x74	69.9	3.51	1	1	586	96.6	895	3090
x67	61.7	3.05	1	1	522	84.1	839	2740
457x152x82	76.9	2.48	1	1	624	82.8	1040	3440
x74	68.7	2.21	1	1	562	73.5	938	3030
x67	60.7	1.92	1	1	515	66.4	899	2770
x60	53.6	1.67	1	1	458	57.9	806	2390
x52	44.9	1.35	1	1	391	47.2	747	2030
406x178x74	57.3	3.26	1	1	518	92.1	827	3090
x67	51.0	2.86	1	1	479	84.1	741	2840
x60	45.4	2.52	1	1	426	74.2	687	2450
x54	39.3	2.14	1	1	373	63.2	617	2210
406x140x46	33.0	1.13	1	1	315	41.9	611	1820
x39	26.3	0.861	1	1	257	32.2	490*	1500
356x171x67	41.0	2.86	1	1	430	86.3	733	3040
x57	33.6	2.33	1	1	359	70.6	646	2470
x51	29.6	2.03	1	1	318	61.8	587	2170
x45	25.4	1.70	2	2	275	52.2	549	1880
356x127x39	21.4	0.752	1	1	234	31.6	527	1600
x33	17.3	0.588	1	1	193	24.9	472	1310
305x165x54	24.6	2.23	1	1	300	69.6	545	2440
x46	20.8	1.88	1	1	256	58.9	461	2000
x40	17.9	1.60	1	1	221	50.4	411	1710
305x127x48	20.1	0.968	1	1	252	41.2	612	2170
x42	17.2	0.817	1	1	218	34.9	542	1900
x37	15.1	0.706	1	1	191	30.3	481	1610
305x102x33	13.7	0.407	1	1	171	21.3	452	1380
x28	11.3	0.326	1	1	143	17.2	407	1150
x25	9.37	0.258	1	1	121	13.8	386	987
254x146x43	13.7	1.42	1	1	201	50.1	415	1950
x37	11.6	1.20	1	1	171	42.2	361	1650
x31	9.26	0.941	1	1	140	33.4	336	1370
254x102x28	8.40	0.376	1	1	125	19.5	365	1240
x25	7.16	0.313	1	1	109	16.3	341	1090
x22	5.96	0.250	1	1	91.9	13.2	320	934
203x133x30	6.09	0.809	1	1	111	31.3	299	1360
x26	4.91	0.647	1	1	91.6	25.2	263	1140
203x102x23	4.41	0.344	1	1	83.1	17.6	254	1040
178x102x19	2.86	0.288	1	1	60.7	14.8	203	863
152x89x16	1.75	0.189	1	1	43.7	11.1	167	721
127x76x13	0.993	0.117	1	1	29.9	8.02	131	586

*   indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

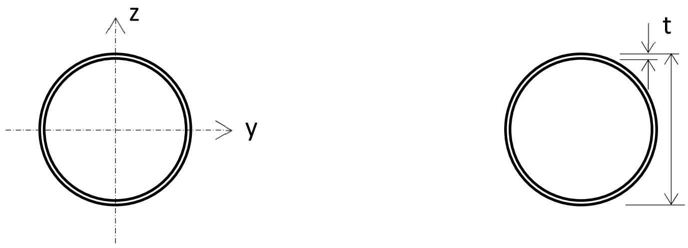
Design Table 15 Section resistances of rolled H-sections: S355 steel



HS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
356x406x634#	578	206	1	1	4620	2310	4040	26300
x551#	477	174	1	1	3930	1970	3490	22800
x467#	384	142	1	1	3350	1690	3000	19900
x393#	309	116	1	1	2750	1390	2520	16800
x340#	258	98.5	1	1	2350	1190	2160	14500
x287#	210	81.3	1	1	2000	1020	1870	12600
x235#	166	65.1	1	1	1620	821	1500	10300
356x368x202#	139	49.8	1	1	1370	662	1340	8870
x177#	120	43.1	1	1	1190	576	1180	7800
x153#	102	37.0	2	2	1020	493	1000	6730
x129#	84.4	30.7	3	3	780	274	839	5660
305x305x283	166	51.7	1	1	1710	784	1950	12100
x240	135	42.6	1	1	1470	673	1710	10600
x198	107	34.2	1	1	1190	545	1400	8690
x158	81.3	26.5	1	1	925	424	1130	6930
x137	68.9	22.5	1	1	794	362	984	6000
x118	58.2	19.0	1	1	676	309	856	5180
x97	46.6	15.4	3	3	515	170	721	4400
254x254x167	63.0	20.7	1	1	835	393	1180	7350
x132	47.3	15.8	1	1	645	303	918	5800
x107	36.8	12.5	1	1	511	240	751	4690
x89	30.0	10.2	1	1	421	198	607	3900
x73	23.9	8.21	2	2	352	165	525	3310
203x203x86	19.8	6.57	1	1	337	157	619	3800
x71	16.0	5.33	1	1	276	129	483	3120
x60	12.9	4.35	1	1	233	108	455	2710
x52	11.0	3.74	1	1	201	93.7	385	2350
x46	9.60	3.26	2	2	176	82.0	347	2080
152x152x37	4.64	1.48	1	1	110	49.7	292	1670
x30	3.68	1.18	1	1	88.0	39.8	238	1360
x23	2.63	0.840	3	3	58.2	18.7	204	1040

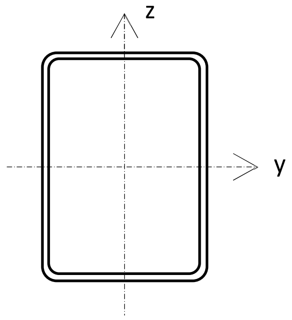
\# Limited availability.

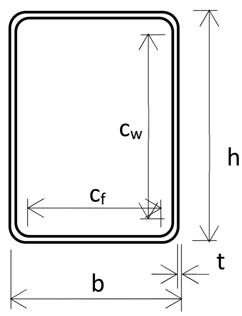
Design Table 16 Section resistances of hot-finished CHS: S355 steel



		S355			
CHS	Flexural rigidity	Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
dxt mmxmm	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
139.7x6.3	1.24	1	39.8	344	937
x8.0	1.51	1	49.3	432	1180
168.3x6.3	2.21	1	58.6	419	1140
x8.0	2.73	1	73.1	526	1430
x10.0	3.28	1	89.1	648	1760
x12.5	3.93	1	108	799	2170
219.1x6.3	5.02	2	101	549	1490
x8.0	6.22	1	127	693	1890
x10.0	7.56	1	155	857	2330
x12.5	9.14	1	190	1058	2880
273.0x6.3	9.87	2	159	689	1870
x8.0	12.3	2	200	869	2360
x10.0	15.0	1	246	1080	2930
x12.5	18.3	1	301	1330	3620
323.9x6.3	16.7	3	174	821	2230
x8.0	20.8	2	284	1040	2820
x10.0	25.6	1	350	1290	3500
x12.5	31.1	1	430	1590	4330
x16.0	38.6	1	540	2020	5500
355.6x6.3	22.1	3	211	902	2450
x8.0	27.7	2	343	1140	3100
x10.0	34.0	2	426	1420	3870
x12.5	41.8	1	522	1760	4790
x16.0	51.9	1	657	2230	6070
406.4x8.0	41.8	3	347	1300	3550
x10.0	51.5	2	557	1630	4440
x12.5	63.0	1	689	2020	5500
x16.0	78.5	1	866	2560	6960
x20.0	95.4	1	1061	3080	8380
457.0x8.0	59.6	3	444	1470	4010
x10.0	73.7	2	710	1830	4970
x12.5	90.5	2	877	2280	6210
x16.0	113	1	1100	2900	7880
x20.0	138	1	1360	3490	9490
508.0x8.0	82.5	4	-	1640	4470
x10.0	102	3	678	2040	5540
x12.5	126	2	1090	2540	6920
x16.0	157	1	1370	3220	8770
x20.0	192	1	1690	3890	10600
610.8x8.0	178	4	-	1970	5360
x10.0	220	4	-	2450	6670
x12.5	248	3	1220	3070	8340
x16.0	277	2	2010	3900	10600
x20.0	339	1	2470	4700	12800
711.0x10.0	284	4	-	2260	6140
x12.5	351	3	1670	2810	7630
x16.0	443	2	2740	3580	9730
x20.0	545	2	3390	4320	11800
813.0x10.0	427	4	-	3290	8950
x12.5	529	4	-	4100	11100
x16.0	668	3	3420	5230	14200
x20.0	823	2	4470	6310	17200

Design Table 17 Section resistances of hot-finished RHS: S355 steel

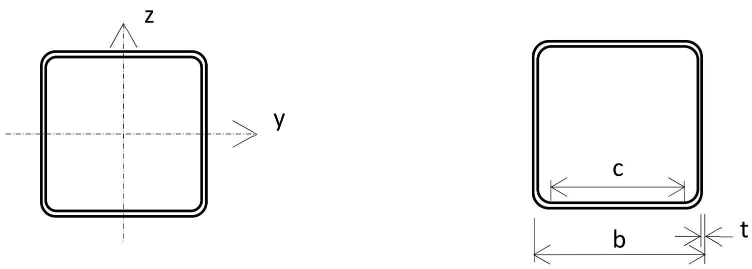




RHS	Flexural rigidity		S355					
			Section Classification		Moment resistance		Shear Resistance	Axial Resistance
hxbxt mmxmmxmm	***EI_y*** **10^3*kNm^2**	***EI_z*** **10^3*kNm^2**	**Bending** *y-y*	**Bending** *z-z*	$M_{y,Rd}$ **kNm**	$M_{z,Rd}$ **kNm**	$V_{z,Rd}$ **kN**	$N_{a,Rd}$ **kN**
120x80x6.3	0.923	0.483	1	1	32.3	24.2	285	824
x8.0	1.10	0.571	1	1	39.3	29.3	354	1020
160x80x6.3	1.90	0.628	1	1	50.6	30.8	385	1000
x8.0	2.29	0.748	1	1	61.9	37.5	481	1250
x10.0	2.69	0.863	1	1	74.2	44.4	586	1520
200x100x6.3	3.84	1.29	1	2	81.0	49.7	489	1270
x8.0	4.68	1.55	1	1	100	61.0	612	1590
x10.0	5.59	1.82	1	1	121	73.2	750	1950
200x150x6.3	5.08	3.26	1	2	103	84.2	493	1490
x8.0	6.24	3.97	1	1	127	104	618	1870
x10.0	7.50	4.75	1	1	155	126	760	2300
250x150x6.3	8.69	3.93	1	4	143	-	620	1630
x8.0	10.7	4.83	1	2	178	124	779	2160
x10.0	13.0	5.78	1	1	217	151	959	2660
x12.5	15.5	6.85	1	1	263	182	1180	3270
260x180x6.3	10.9	6.15	1	4	169	-	631	1730
x8.0	13.4	7.58	1	2	210	163	814	2390
x10.0	16.3	9.14	1	1	257	199	1000	2940
x12.5	19.5	10.9	1	1	312	241	1240	3620
x16.0	23.6	13.1	1	1	384	295	1550	4540
300x200x6.3	16.4	8.80	2	4	221	-	750	1910
x8.0	20.4	10.9	1	4	277	-	944	2640
x10.0	24.8	13.2	1	2	339	256	1170	3370
x12.5	30.0	15.8	1	1	415	311	1440	4150
x16.0	36.5	19.1	1	1	511	383	1810	5220
350x250x6.3	27.7	16.5	4	4	-	-	880	2090
x8.0	34.4	20.6	2	4	398	-	1110	3020
x10.0	42.2	25.0	1	3	490	338	1370	4080
x12.5	51.2	30.2	1	1	596	472	1700	5040
x16.0	63.0	37.0	1	1	742	586	2140	6350
400x200x6.3	33.0	11.3	2	4	341	-	950*	1990
x8.0	41.2	14.0	1	4	426	-	1270	2800
x10.0	50.2	17.0	1	4	525	-	1570	3840
x12.5	61.1	20.4	1	2	643	394	1940	5040
x16.0	75.0	24.8	1	1	802	486	2450	6350
450x250x8.0	63.2	25.4	2	4	575	-	1440	3150
x10.0	77.5	31.1	1	4	710	-	1780	4300
x12.5	94.5	37.8	1	3	873	511	2200	5930
x16.0	117	46.2	1	1	1090	721	2780	7490
500x300x8.0	91.8	41.8	4	4	-	-	1530*	3390
x10.0	113	51.2	2	4	920	-	1990	4740
x12.5	138	62.6	1	4	1140	-	2460	6430
x16.0	172	77.3	1	2	1420	994	3110	8630
x20.0	207	92.6	1	1	1680	1180	3570	9900

* indicates the value of shear buckling resistance $V_{\mathrm{bw,Rd}} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 18   Section resistances of hot-finished SHS: S355 steel



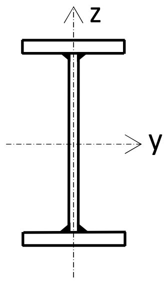
SHS	Flexural rigidity	S355				
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
bxbxt **mmxmmxmm**	***EI***$_y$ **10**^{3}***kNm**^{2}	**Bending** *y-y*	$M_{y,Rd}$ **kNm**	$V_{z,Rd}$ **kN**	$N_{a,Rd}$ **kN**
100x100x6.3	0.704	1	28.7	238	824
x8.0	0.838	1	34.8	295	1020
150x150x6.3	2.56	1	68.1	367	1270
x8.0	3.13	1	84.1	459	1590
x10.0	3.72	1	102	563	1950
200x200x6.3	6.32	2	124	496	1720
x8.0	7.79	1	155	623	2160
x10.0	9.39	1	188	768	2660
x12.5	11.2	1	228	944	3270
220x220x6.3	8.51	3	131	534	1830
x8.0	10.5	1	189	689	2390
x10.0	12.7	1	231	850	2940
x12.5	15.2	1	280	1050	3620
x16.0	18.4	1	344	1310	4540
250x250x6.3	12.6	4	-	625	1980
x8.0	15.6	2	246	787	2730
x10.0	19.0	1	302	973	3370
x12.5	22.9	1	368	1200	4150
x16.0	27.9	1	454	1510	5220
300x300x6.0	22.1	4	-	754	2110
x8.0	27.5	4	-	951	3120
x10.0	33.6	2	444	1180	4080
x12.5	40.7	1	540	1460	5040
x16.0	50.0	1	671	1830	6350
350x350x8.0	44.3	4	-	1120	3320
x10.0	54.4	3	525	1380	4730
x12.5	66.2	1	749	1710	5930
x16.0	81.7	1	934	2160	7490
400x400x8.0	67.0	4	-	1280	3460
x10.0	82.1	4	-	1590	5010
x12.5	100	2	987	1970	6820
x16.0	125	1	1240	2490	8630
x20.0	150	1	1470	2990	10400

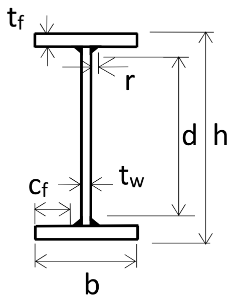
Design Tables 19 to 24 for

Section Dimensions and Properties of Welded Sections

- **I-sections (EWIS)**
- **H-sections (EWHS)**
- **EWCHS**
- **EWRHS**
- **EWSHS**

Design Table 19A Section dimensions of welded I-sections (1)

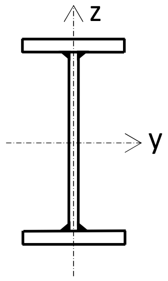


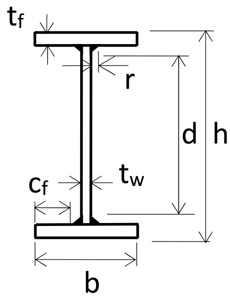


IS	EWIS	Mass per Meter	Depth of Section	Width of Section	Thickness		Fillet Height	Depth between Fillets	Ratios for Local Buckling		Surface Area	
					Web	Flange					*per Meter*	*per Tonne*
		m	h	b	t_w	t_f	r	d	b/t_f	d/t_w		
		kg/m	mm	mm	mm	mm	mm	mm			m^2	m^2
914x419x388#	920x450x420	418.5	920	450	20	40	16	808	5.0	40.4	3.60	8.60
x343#	x353	350.2	915	450	20	30	16	823	6.6	41.2	3.59	10.3
914x305x289#	920x360x312	310.2	930	360	20	30	16	838	5.1	41.9	3.26	10.5
x253#	x282	281.9	920	360	20	25	16	838	6.2	41.9	3.24	11.5
x224#	x249	249.4	910	350	16	25	16	828	6.0	51.8	3.19	12.8
x201#	x218	217.9	900	340	16	20	14	832	7.4	52.0	3.13	14.4
838x292x226#	840x350x246	245.8	850	360	16	25	16	768	6.2	48.0	3.11	12.6
x194#	x214	213.5	840	350	16	20	14	772	7.7	48.3	3.05	14.3
x176#	x184	184.4	840	340	12	20	12	776	7.6	64.7	3.02	16.4
762x267x197	760x320x220	220.1	770	320	16	25	16	688	5.4	43.0	2.79	12.7
x173	x194	194.0	760	320	16	20	14	692	6.9	43.3	2.77	14.3
x147	x167	166.5	750	310	12	20	12	686	6.9	57.2	2.72	16.3
x134	x147	147.4	750	310	12	16	11	696	8.6	58.0	2.72	18.4
686x254x170	690x280x198	198.2	690	290	16	25	16	608	4.8	38.0	2.51	12.7
x152	x173	172.6	690	280	16	20	14	622	5.9	38.9	2.47	14.3
x140	x151	150.5	680	280	12	20	12	616	6.1	51.3	2.46	16.3
x125	x133	133.3	680	280	12	16	11	626	7.7	52.2	2.46	18.4
610x305x238	620x330x258	257.5	640	340	20	30	20	540	4.7	27.0	2.60	10.1
x179	x186	185.5	620	330	12	25	12	546	5.9	45.5	2.54	13.7
x149	x160	159.6	610	330	12	20	12	546	7.4	45.5	2.52	15.8
610x229x140	610x260x158	158.0	620	260	12	25	12	546	4.5	45.5	2.26	14.3
x125	x138	137.6	610	260	12	20	12	546	5.6	45.5	2.24	16.3
x113	x122	121.7	610	260	12	16	11	556	7.1	46.3	2.24	18.4
x101	x112	111.5	600	260	10	16	10	548	7.2	54.8	2.22	19.9
533x210x122	540x250x148	147.5	550	250	12	25	12	476	4.3	39.7	2.08	14.1
x109	x128	127.9	540	250	12	20	12	476	5.4	39.7	2.06	16.1
x101	x113	112.6	540	250	12	16	11	486	6.8	40.5	2.06	18.3
x92	x104	103.5	530	250	10	16	10	478	6.9	47.8	2.04	19.7
x82	x88	87.8	530	250	10	12	8	490	9.3	49.0	2.04	23.2

\# Limited availability.

Design Table 19B Section properties of welded I-sections (1)

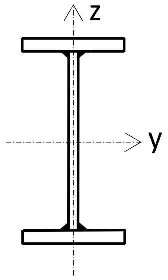


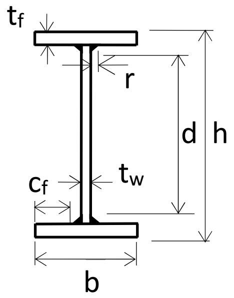


IS	EWIS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
		I_y	I_z	$W_{el,y}$	$W_{el,z}$	$W_{pl,y}$	$W_{pl,z}$	u	x	I_w	I_T	A
		cm^4	cm^4	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
914x419x388#	920x450x420	805000	60800	17300	2700	19600	4140	0.896	24.4	117.7	2140	533
x343#	x353	642000	45600	13800	2030	15800	3130	0.878	32.3	89.3	1040	446
914x305x289#	920x360x312	557000	23400	11800	1300	13700	2040	0.868	33.5	47.4	880	395
x253#	x282	480000	19500	10220	1083	12100	1710	0.859	38.3	39.0	607	359
x224#	x249	437000	17900	9410	1023	10900	1590	0.872	39.9	35.0	482	318
x201#	x218	355000	13130	7730	772	9110	1216	0.855	47.2	25.4	299	278
838x292x226#	840x350x246	383000	19500	8820	1083	10200	1680	0.876	36.9	33.2	484	313
x194#	x214	310000	14310	7240	818	8450	1280	0.860	43.7	24.1	296	272
x176#	x184	284000	13110	6670	771	7610	1190	0.878	46.3	22.0	227	235
762x267x197	760x320x220	278000	13670	7060	854	8220	1330	0.876	33.4	19.0	432	280
x173	x194	230000	10940	5920	684	6950	1080	0.861	39.4	14.98	269	247
x147	x167	205000	9940	5360	641	6140	989	0.879	41.2	13.24	206	212
x134	x147	174000	7950	4560	513	5270	797	0.864	49.8	10.71	126	188
686x254x170	690x280x198	200000	10180	5650	702	6620	1100	0.876	29.8	11.25	389	253
x152	x173	166000	7330	4700	524	5570	831	0.859	35.8	8.23	238	220
x140	x151	151000	7320	4350	523	5020	810	0.880	37.2	7.97	186	192
x125	x133	128000	5860	3710	419	4310	653	0.864	45.1	6.46	114	170
610x305x238	620x330x258	229000	19700	6940	1159	8130	1810	0.881	22.2	18.3	767	328
x179	x186	167000	15000	5320	909	5960	1380	0.899	26.2	13.3	377	236
x149	x160	136000	11980	4360	726	4950	1110	0.887	32.4	10.43	209	203
610x229x140	610x260x158	136000	7330	4320	564	4920	868	0.894	26.9	6.49	304	201
x125	x138	111000	5860	3580	451	4120	699	0.880	33.2	5.10	171	175
x113	x122	94700	4690	3040	361	3540	564	0.866	40.3	4.14	104	155
x101	x112	87800	4690	2870	361	3290	557	0.878	40.8	4.00	89.9	142
533x210x122	540x250x148	100400	6510	3590	521	4100	802	0.896	23.5	4.49	289	188
x109	x128	81900	5210	2970	417	3420	646	0.884	29.0	3.52	162	163
x101	x113	69600	4170	2520	334	2930	521	0.869	35.3	2.86	97.5	143
x92	x104	64400	4170	2380	334	2730	514	0.883	35.6	2.75	84.9	132
x82	x88	51900	3120	1930	250	2230	389	0.864	45.1	2.09	45.7	112

\# Limited availability.

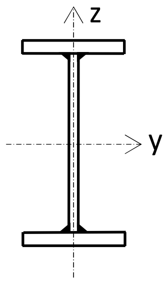
Design Table 20A Section dimensions of welded I-sections (2)

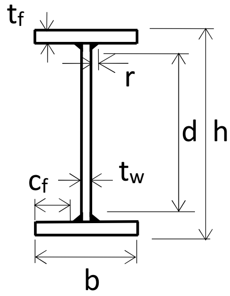




IS	EWIS	Mass per Meter	Depth of Section	Width of Section	Thickness		Fillet Height	Depth between Fillets	Ratios for Local Buckling		Surface Area	
					Web	Flange					*per Meter*	*per Tonne*
		m	h	b	t_w	t_f	r	d	b/t_f	d/t_w		
		kg/m	mm	mm	mm	mm	mm	mm			m^2	m^2
457x191x98	460x220x112	111.8	470	220	12	20	12	406	4.6	31.8	1.80	16.1
x89	x104	103.6	460	220	10	20	10	400	4.8	38.0	1.78	17.2
x82	x90	90.4	460	220	10	16	10	408	5.9	38.8	1.78	19.7
x74	x83	83.1	460	220	8	16	8	412	6.1	49.5	1.78	21.5
x67	x70	69.8	460	220	8	12	8	420	8.2	50.5	1.78	25.5
457x152x82	460x180x91	91.1	460	180	10	20	10	400	3.8	38.0	1.62	17.8
x74	x80	80.4	460	180	10	16	10	408	4.7	38.8	1.62	20.2
x67	x73	73.1	460	180	8	16	8	412	4.9	49.5	1.62	22.2
x60	x62	62.3	460	180	8	12	8	420	6.5	50.5	1.62	26.1
x52	x56	56.3	450	180	8	10	8	414	7.8	49.8	1.60	28.5
406x178x74	410x210x85	84.8	420	210	10	16	10	368	5.6	34.8	1.66	19.6
x67	x78	77.5	410	210	8	16	8	362	5.8	43.3	1.64	21.2
x60	x65	64.8	410	210	8	12	8	370	7.8	44.3	1.64	25.4
x54	x59	58.5	410	210	8	10	8	374	9.3	44.8	1.64	28.1
406x140x46	400x150x49	48.9	400	160	6	12	8	360	5.8	57.3	1.43	29.2
x39	x43	42.5	400	150	6	10	8	364	6.4	58.0	1.39	32.7
x67	355x180x73	72.5	360	180	10	16	10	308	4.7	28.8	1.42	19.6
x57	x62	61.3	360	180	10	12	8	320	6.4	30.4	1.42	23.2
x51	x54	53.8	355	170	8	12	8	315	6.1	37.4	1.37	25.5
x45	x49	48.7	355	170	8	10	8	319	7.3	37.9	1.37	28.2
356x127x39	355x170x44	43.5	355	170	6	10	8	319	7.4	50.5	1.38	31.7
x33	x38	38.1	350	170	6	8	8	318	9.3	50.3	1.37	35.9
305x165x54	310x160x59	58.7	310	160	8	16	8	262	4.3	30.8	1.24	21.2
x46	x45	44.6	310	160	6	12	8	270	5.8	42.3	1.25	28.0
x40	x39	39.3	300	160	6	10	8	264	6.9	41.3	1.23	31.2
305x127x48	310x140x54	53.6	310	140	8	16	8	262	3.6	30.8	1.16	21.7
x42	x45	45.3	310	140	8	12	8	270	4.8	31.8	1.16	25.7
x37	x41	40.6	300	140	8	10	8	264	5.8	31.0	1.14	28.2
305x102x33	310x110x37	36.8	315	110	8	10	8	279	4.3	32.9	1.05	28.6
x28	x33	33.3	310	110	8	8	8	278	5.4	32.8	1.04	31.4
x25	x28	28.4	305	110	6	8	8	273	5.5	42.8	1.04	36.5
254x146x43	260x170x48	47.9	260	170	8	12	8	220	6.1	25.5	1.18	24.7
x37	x43	42.8	260	170	8	10	8	224	7.3	26.0	1.18	27.7
x31	x33	33.4	250	170	6	8	8	218	9.3	33.7	1.17	35.0
254x102x28	260x130x33	32.7	260	130	6	10	8	224	5.4	34.7	1.028	31.4
x25	x29	28.8	260	130	6	8	8	228	6.8	35.3	1.028	35.7
x22	x25	24.7	255	130	6	6	8	227	9.0	35.2	1.018	41.2
203x133x30	210x150x34	33.5	210	150	6	10	8	174	6.4	26.3	1.008	30.1
x26	x29	28.5	200	150	6	8	8	168	8.0	25.3	0.988	34.7
203x102x23	200x110x27	26.8	200	110	6	10	8	164	4.4	24.7	0.828	31.0
178x102x19	180x100x19	18.7	180	100	4	8	8	148	5.0	33.0	0.752	40.2
152x89x16	150x100x18	17.8	150	100	4	8	8	118	5.0	25.5	0.692	38.9
127x76x13	130x80x15	14.6	130	80	4	8	8	98	3.8	20.5	0.572	39.1

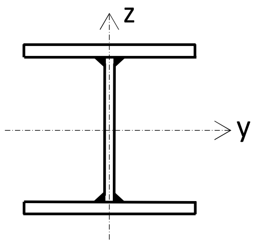
Design Table 20B Section properties of welded I-sections (2)

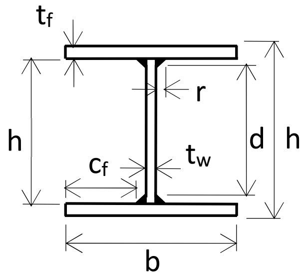




IS	EWIS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
		I_y cm^4	I_z cm^4	$W_{el,y}$ cm^3	$W_{el,z}$ cm^3	$W_{pl,y}$ cm^3	$W_{pl,z}$ cm^3	u	x	I_w dm^6	I_T cm^4	A cm^2
457x191x98	460x220x112	53800	3550	2290	323	2600	502	0.885	25.1	1.80	142	142
x89	x104	49600	3550	2160	323	2420	496	0.896	24.5	1.72	131	132
x82	x90	42100	2840	1830	258	2060	400	0.882	30.7	1.400	74.3	115
x74	x83	40500	2840	1760	258	1960	395	0.897	31.0	1.400	67.4	105.9
x67	x70	32600	2130	1420	194	1590	298	0.878	41.0	1.069	32.8	89.0
457x152x82	460x180x91	41900	1940	1820	216	2070	336	0.890	25.1	0.939	110.0	116
x74	x80	35800	1550	1560	172	1780	272	0.875	31.4	0.764	63.4	102.4
x67	x73	34200	1550	1490	172	1670	267	0.888	31.7	0.764	56.5	93.1
x60	x62	27800	1160	1210	129	1380	202	0.872	41.8	0.582	28.2	79.4
x52	x56	23300	970	1036	108	1190	170	0.860	47.1	0.469	19.3	71.7
406x178x74	410x210x85	33000	2470	1570	235	1770	364	0.883	27.8	1.008	70.3	108.0
x67	x78	30100	2470	1470	235	1630	360	0.896	27.3	0.959	63.8	98.7
x60	x65	24300	1850	1190	176	1330	272	0.882	36.2	0.733	30.8	82.6
x54	x59	21200	1540	1034	147	1170	228	0.870	42.2	0.616	20.7	74.5
406x140x46	400x150x49	17600	810	880	101	981	158	0.891	37.0	0.305	21.1	62.2
x39	x43	14600	560	730	75	826	117	0.876	44.7	0.213	12.7	54.1
x67	355x180x73	20500	1550	1140	172	1290	269	0.883	23.7	0.459	60.1	92.4
x57	x62	16600	1160	922	129	1060	204	0.867	30.3	0.351	31.9	78.1
x51	x54	14800	980	834	115	940	180	0.879	31.4	0.288	25.2	68.6
x45	x49	13000	820	732	96	832	151	0.867	36.6	0.244	17.1	62.1
356x127x39	355x170x44	12300	810	693	95	776	148	0.886	38.5	0.241	13.7	55.4
x33	x38	10200	650	583	76	653	119	0.873	46.2	0.190	8.21	48.5
305x165x54	310x160x59	12700	1090	819	136	925	210	0.897	20.3	0.236	48.4	74.7
x46	x45	10000	810	645	101	713	157	0.898	27.6	0.180	20.5	56.8
x40	x39	8070	680	538	85	599	131	0.889	32.0	0.143	12.7	50.1
305x127x48	310x140x54	11400	730	735	104	831	162	0.894	20.6	0.158	43.0	68.3
x42	x45	9280	550	599	79	682	123	0.877	27.5	0.1221	21.0	57.8
x37	x41	7590	450	506	64	580	103	0.867	30.9	0.0946	14.1	51.7
305x102x33	310x110x37	7100	220	451	40	528	66	0.853	33.0	0.0512	12.4	46.9
x28	x33	5970	170	385	31	457	54	0.839	36.9	0.0388	8.77	42.4
x25	x28	5350	170	351	31	405	52	0.861	41.1	0.0375	5.84	36.2
254x146x43	260x170x48	7320	980	563	115	632	178	0.882	22.2	0.1507	23.6	61.0
x37	x43	6410	810	493	95	555	149	0.873	26.1	0.1266	15.4	54.5
x31	x33	4790	650	383	76	426	118	0.877	32.1	0.0952	7.49	42.5
254x102x28	260x130x33	4930	360	379	55	426	87	0.888	27.8	0.0563	10.39	41.7
x25	x29	4210	290	324	45	367	71	0.874	34.1	0.0460	6.19	36.7
x22	x25	3320	220	260	34	298	54	0.857	40.8	0.0341	3.62	31.5
203x133x30	210x150x34	3450	560	329	75	366	115	0.886	21.5	0.0560	11.4	42.7
x26	x29	2630	450	263	60	293	92	0.874	25.3	0.0415	6.44	36.3
203x102x23	200x110x27	2380	220	238	40	269	63	0.889	21.0	0.0199	8.63	34.1
178x102x19	180x100x19	1410	130	157	26	175	41	0.901	24.1	0.00961	3.76	23.8
152x89x16	150x100x18	940	130	125	26	140	41	0.898	19.5	0.00655	3.70	22.6
127x76x13	130x80x15	560	60	86	15	98	27	0.903	17.0	0.00223	2.97	18.6

Design Table 21A Section dimensions of welded H-sections

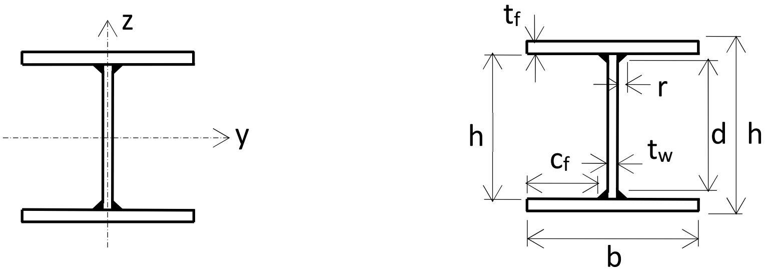




HS	EWHS	Mass per Meter	Depth of Section	Width of Section	Thickness		Fillet Height	Depth between Fillets	Ratios for Local Buckling		Surface Area	
					Web	Flange					per Meter	per Tonne
		m	h	b	t_w	t_f	r	d	c_f/t_f	d/t_w		
		kg/m	mm	mm	mm	mm	mm	mm			m^2	m^2
356x406x634#	420x480x716	716	470	470	50	80	16	278	2.4	5.6	2.72	3.80
x551#	x563	563	460	480	40	60	16	308	3.4	7.7	2.72	4.83
x467#	x532	532	440	480	30	60	16	288	3.5	9.6	2.66	5.00
x393#	x456	456	420	480	30	50	16	288	4.2	9.6	2.58	5.66
x340#	x368	368	400	480	25	40	16	288	5.3	11.5	2.51	6.82
x287#	x300	300	390	490	25	30	16	298	7.2	11.9	2.49	8.31
x235#	x275	275	380	490	16	30	16	288	7.4	18.0	2.47	9.0
356x368x202#	360x440x218	218	380	440	16	25	16	298	7.8	18.6	2.37	10.9
x177#	x217	217	370	440	16	25	16	288	7.8	18.0	2.33	10.7
x153#	x172	172	360	440	12	20	16	288	9.9	24.0	2.30	13.3
x129#	x167	167	360	440	10	20	16	288	10.0	28.8	2.30	13.7
305x305x283	370x330x316	316	370	330	25	50	16	238	2.7	9.5	2.09	6.61
x240	x260	260	350	340	20	40	16	238	3.6	11.9	2.04	7.85
x198	x213	213	340	350	20	30	16	248	5.0	12.4	2.02	9.5
x158	x177	177	330	350	16	25	16	248	6.0	15.5	1.99	11.3
x137	x152	152	320	360	16	20	16	248	7.8	15.5	1.97	12.9
x118	x142	142	320	360	12	20	12	256	8.1	21.3	1.98	13.9
x97	x114	114	310	360	10	16	10	258	10.3	25.8	1.94	17.0
254x254x167	270x310x184	184	290	300	20	30	20	190	4.0	9.5	1.72	9.36
x132	x151	151	280	300	16	25	16	198	5.0	12.4	1.69	11.2
x107	x121	121	270	310	12	20	12	206	6.9	17.2	1.68	13.8
x89	x116	116	260	310	10	20	10	200	7.0	20.0	1.64	14.1
x73	x93	93.2	260	310	8	16	8	212	8.9	26.5	1.64	17.6
203x203x86	210x230x101	101	220	210	12	25	12	146	3.5	12.2	1.28	12.7
x71	x85	84.8	220	220	10	20	10	160	4.8	16.0	1.30	15.3
x60	x73	73.3	210	230	10	16	10	158	6.3	15.8	1.28	17.5
x52	x58	57.9	210	240	8	12	8	170	9.0	21.3	1.30	22.5
x46	x50	49.8	200	240	8	10	8	164	10.8	20.5	1.26	25.3
152x152x37	170x170x42	42.2	170	170	8	12	8	130	6.1	16.3	1.00	23.8
x30	x34	33.8	160	170	6	10	8	124	7.4	20.7	0.97	28.2
x23	x29	28.5	160	170	6	8	8	128	9.3	21.3	0.97	33.2

\# Limited availability.

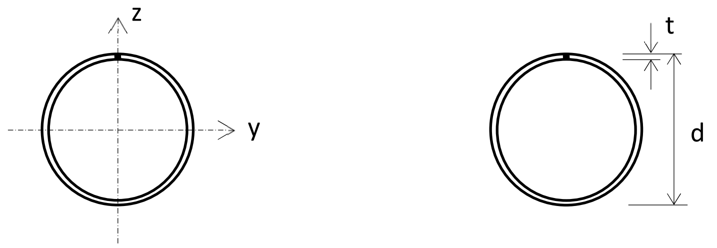
Design Table 21B   Section properties of welded H-sections



HS	EWHS	Second Moment of Area		Elastic Modulus		Plastic Modulus		Buckling Parameter	Torsional Index	Warping Constant	Torsional Constant	Area of Section
		I_y	I_z	$W_{el,y}$	$W_{el,z}$	$W_{pl,y}$	$W_{pl,z}$	u	x	I_w	I_T	A
		cm^4	cm^4	cm^3	cm^3	cm^3	cm^3			dm^6	cm^4	cm^2
356x406x634#	420x480x716	304000	138800	12900	5910	15900	9050	0.812	5.0	52.8	17300	912
x551#	x563	247000	110800	10740	4620	12800	7060	0.814	6.8	44.3	7640	717
x467#	x532	219000	110700	9950	4610	11800	6990	0.803	6.5	40.0	7200	677
x393#	x456	175000	92200	8330	3840	9730	5840	0.789	7.6	31.6	4290	581
x340#	x368	133000	73800	6650	3080	7630	4670	0.776	9.2	23.9	2210	469
x287#	x300	104000	58900	5330	2400	6050	3660	0.762	12.0	19.1	1054	382
x235#	x275	95900	58800	5050	2400	5630	3630	0.756	12.0	18.0	926	350
356x368x202#	360x440x218	75500	35500	3970	1610	4420	2450	0.808	14.7	11.18	503	278
x177#	x217	71200	35500	3850	1610	4280	2450	0.797	14.2	10.56	502	276
x153#	x172	55400	28400	3080	1291	3380	1950	0.795	17.6	8.21	253	220
x129#	x167	54900	28400	3050	1291	3330	1950	0.799	17.6	8.21	245	213
305x305x283	370x330x316	90100	30000	4870	1820	5800	2770	0.858	6.6	7.68	2890	403
x240	x260	69900	26200	3990	1540	4650	2350	0.846	8.0	6.29	1520	331
x198	x213	55200	21500	3250	1229	3720	1870	0.832	10.7	5.17	705	271
x158	x177	44600	17900	2700	1023	3050	1560	0.829	12.7	4.16	403	225
x137	x152	36300	15560	2270	864	2540	1321	0.812	15.3	3.50	230	194
x118	x142	35200	15550	2200	864	2430	1309	0.819	15.5	3.50	208	180
x97	x114	27100	12440	1750	691	1910	1045	0.812	19.0	2.69	108	145
254x254x167	270x310x184	33500	13510	2310	901	2690	1390	0.826	9.0	2.28	601	234
x132	x151	26700	11250	1910	750	2180	1147	0.823	10.6	1.83	344	192
x107	x121	21000	9930	1560	641	1740	972	0.809	12.9	1.55	179	154
x89	x116	19000	9930	1460	641	1630	968	0.796	12.4	1.43	173	148
x73	x93	15700	7940	1208	512	1330	773	0.803	15.7	1.18	88.5	119
203x203x86	210x230x101	10700	3860	973	368	1130	560	0.850	8.1	0.367	229	128
x71	x85	9470	3550	861	323	978	490	0.846	10.4	0.355	123.3	108
x60	x73	7560	3240	720	282	810	429	0.822	12.6	0.305	68.7	93.4
x52	x58	6190	2760	590	230	651	349	0.815	17.0	0.271	30.8	73.8
x46	x50	4820	2300	482	192	532	292	0.797	19.3	0.208	19.1	63.7
152x152x37	170x170x42	2820	983	332	116	374	177	0.843	13.7	0.0614	22.1	53.8
x30	x34	2110	819	264	96	293	146	0.836	15.7	0.0461	12.3	43.7
x23	x29	1780	655	223	77	247	117.6	0.834	19.7	0.0379	6.84	37.1

\# Limited availability.

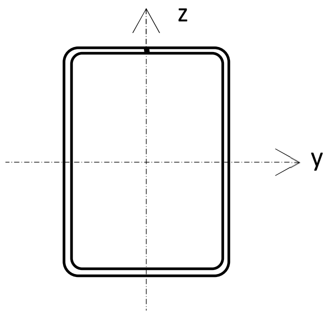
Design Table 22   Section dimensions and properties of cold-formed CHS

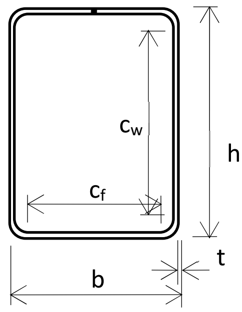


CHS	EWCHS	Mass per Meter	Area of Section	Ratio for Local Buckling	Second Moment of Area	Elastic Modulus	Plastic Modulus	Torsional Constants		Surface Area	
	dxt	*m*	*A*	*d/t*	*I*	W_{el}	W_{pl}	I_T	W_t	*per Meter*	*per Tonne*
	mm	kg/m	cm^2		cm^4	cm^3	cm^3	cm^4	cm^3	m^2	m^2
139.7x6.3	140x6.0	19.8	25.3	23.3	568	81.1	108	1140	162	0.440	22.2
x8.0	x8.0	26.0	33.2	17.5	725	104	139	1450	207	0.440	16.9
x10.0	x10.0	32.1	40.8	14.0	868	124	169	1740	248	0.440	13.7
168.3x6.3	170x6.0	24.3	30.9	28.3	1041	122	161	2080	245	0.535	22.0
x8.0	x8.0	32.0	40.7	21.3	1340	158	210	2680	315	0.535	16.7
x10.0	x10.0	39.5	50.3	17.0	1610	189	256	3220	379	0.535	13.6
x12.5	x12.0	46.8	59.6	14.2	1870	220	300	3740	440	0.535	11.4
219.1x6.3	220x6.0	31.7	40.3	36.7	2310	210	275	4620	420	0.692	21.9
x8.0	x8.0	41.8	53.3	27.5	3000	273	360	6000	545	0.692	16.5
x10.0	x10.0	51.8	66.0	22.0	3640	331	441	7280	662	0.692	13.4
x12.5	x12.0	61.6	78.4	18.3	4250	386	519	8500	773	0.692	11.2
273.0x6.3	270x6.0	39.1	49.8	45.0	4340	321	418	8680	643	0.849	21.7
x8.0	x8.0	51.7	65.8	33.8	5660	419	549	11300	839	0.849	16.4
x10.0	x10.0	64.1	81.7	27.0	6910	512	676	13800	1020	0.849	13.2
x12.5	x12.0	76.3	97.3	22.5	8110	601	799	16200	1200	0.849	11.1
x16.0	x16.0	100	128	16.9	10300	766	1030	20700	1530	0.849	8.47
323.9x6.3	320x6.0	46.5	59.2	53.3	7300	456	592	14600	913	1.01	21.7
x8.0	x8.0	61.6	78.4	40.0	9500	594	779	19000	1190	1.01	16.3
x10.0	x10.0	76.4	97.4	32.0	11700	731	961	23400	1460	1.01	13.2
x12.5	x12.0	91.1	116	26.7	13800	863	1140	27600	1730	1.01	11.0
x16.0	x16.0	120	153	20.0	17700	1110	1480	35400	2220	1.01	8.39
x20.0	x20.0	148	188	16.0	21300	1330	1800	42600	2660	1.01	6.80
355.6x6.3	360x6.0	52.4	66.7	60.0	10460	581	752	20900	1160	1.13	21.6
x8.0	x8.0	69.4	88.5	45.0	13700	761	991	27400	1520	1.13	16.3
x10.0	x10.0	86.3	110	36.0	16900	939	1230	33800	1880	1.13	13.1
x12.5	x12.0	103	131	30.0	19900	1110	1450	39800	2220	1.13	11.0
x16.0	x16.0	136	173	22.5	25600	1420	1890	51200	2840	1.13	8.33
x20.0	x20.0	168	214	18.0	31000	1720	2310	62000	3440	1.13	6.74
406.4x8.0	400x8.0	77.3	98.5	50.0	18900	945	1230	37800	1890	1.26	16.3
x10.0	x10.0	96.2	123	40.0	23300	1170	1520	46600	2340	1.26	13.1
x12.5	x12.0	115	146	33.3	27600	1380	1810	55200	2760	1.26	10.9
x16.0	x16.0	152	193	25.0	35600	1780	2360	71200	3560	1.26	8.30
x20.0	x20.0	187	239	20.0	43200	2160	2890	86400	4320	1.26	6.71
x25.0	x25.0	231	295	16.0	52000	2600	3520	104000	5200	1.26	5.44
457.0x8.0	460x8.0	89.2	114	57.5	29000	1260	1630	58000	2520	1.45	16.2
x10.0	x10.0	111	141	46.0	35800	1560	2030	71600	3120	1.45	13.0
x12.5	x12.0	133	169	38.3	42400	1840	2410	84800	3680	1.45	10.9
x16.0	x16.0	175	223	28.8	55100	2400	3150	110000	4800	1.45	8.25
x20.0	x20.0	217	276	23.0	67000	2910	3870	134000	5820	1.45	6.66
x25.0	x25.0	268	342	18.4	81100	3530	4730	162000	7060	1.45	5.39
508.0x8.0	500x8.0	97.1	124	62.5	37400	1500	1940	74800	3000	1.57	16.2
x10.0	x10.0	121	154	50.0	46200	1850	2400	92400	3700	1.57	13.0
x12.5	x12.0	144	184	41.7	54800	2190	2860	110000	4380	1.57	10.9
x16.0	x16.0	191	243	31.3	71300	2850	3750	143000	5700	1.57	8.23
x20.0	x20.0	237	302	25.0	87000	3480	4610	174000	6960	1.57	6.64
x25.0	x25.0	293	373	20.0	105500	4220	5640	211000	8440	1.57	5.36
x30.0	x30.0	348	443	16.7	122800	4910	6630	246000	9820	1.57	4.52
610.0x8.0	610x8.0	119	151	76.3	68500	2250	2900	137000	4500	1.92	16.1
x10.0	x10.0	148	188	61.0	84800	2780	3600	170000	5560	1.92	13.0
x12.5	x12.0	177	225	50.8	100800	3300	4290	202000	6600	1.92	10.8
x16.0	x16.0	234	299	38.1	131800	4320	5650	264000	8640	1.92	8.18
x20.0	x20.0	291	371	30.5	161000	5280	6960	322000	10560	1.92	6.59
x25.0	x25.0	361	459	24.4	196900	6460	8560	394000	12920	1.92	5.32
x30.0	x30.0	429	547	20.3	230000	7540	10090	460000	15080	1.92	4.47

CHS	EWCHS	Mass per Meter	Area of Section	Ratio for Local Buckling	Second Moment of Area	Elastic Modulus	Plastic Modulus	Torsional Constants		Surface Area	
	dxt **mm**	***m*** **kg/m**	***A*** **cm^2**	***d/t***	***I*** **cm^4**	**W_{el}** **cm^3**	**W_{pl}** **cm^3**	**I_T** **cm^4**	**W_t** **cm^3**	***per Meter*** **m^2**	***per Tonne*** **m^2**
711.0x10.0	710x10.0	173	220	71.0	135000	3800	4900	270000	7600	2.23	12.9
x12.5	x12.0	207	263	59.2	160000	4510	5850	320000	9020	2.23	10.8
x16.0	x16.0	274	349	44.4	210000	5920	7710	420000	11800	2.23	8.15
x20.0	x20.0	340	434	35.5	258000	7270	9520	516000	14500	2.23	6.56
x25.0	x25.0	422	538	28.4	316000	8900	11730	632000	17800	2.23	5.28
x30.0	x30.0	503	641	23.7	371000	10450	13870	742000	20900	2.23	4.43
x36.0	x36.0	598	762	19.7	434000	12230	16350	868000	24500	2.23	3.73
813.0x10.0	810x10.0	197	251	81.0	201000	4960	6400	402000	9920	2.55	12.9
x12.5	x12.0	236	301	67.5	240000	5930	7640	480000	11900	2.55	10.8
x16.0	x16.0	313	399	50.6	315000	7780	10100	630000	15600	2.55	8.12
x20.0	x20.0	390	496	40.5	387000	9560	12500	774000	19100	2.55	6.53
x25.0	x25.0	484	617	32.4	475000	11730	15410	950000	23500	2.55	5.26
x30.0	x30.0	577	735	27.0	560000	13830	18250	1120000	27700	2.55	4.41
x36.0	x36.0	687	875	22.5	657000	16220	21570	1314000	32400	2.55	3.70

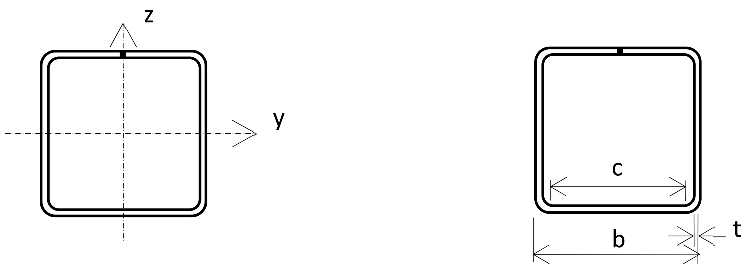
Design Table 23 Section dimensions and properties of cold-formed RHS





RHS	EWRHS		Mass per Meter	Area of Section	Ratio for Local Buckling		Second Moment of Area		Elastic Modulus		Plastic Modulus		Surface Area	
	hxb	*t*	*m*	*A*	c_w/t	c_f/t	I_y	I_z	$W_{el,y}$	$W_{el,z}$	$W_{pl,y}$	$W_{pl,z}$	*per Meter*	*per Tonne*
	mm	mm	kg/m	cm^2			cm^4	cm^4	cm^3	cm^3	cm^3	cm^3	m^2	m^2
120x80x6.3	120x80	x6.0	16.5	21.0	14.0	7.3	382	204	63.6	51.0	80.3	60.9	0.369	22.4
x8.0		x8.0	21.0	26.7	9.0	4.0	451	240	75.2	60.1	98.2	74.2	0.359	17.1
160x80x6.3	160x80	x6.0	20.3	25.8	20.7	7.3	793	270	99.2	67.4	127	78.6	0.449	22.2
x8.0		x8.0	26.0	33.1	14.0	4.0	959	324	120	80.9	158	97.3	0.439	16.9
x10.0		x10.0	31.2	39.7	10.0	2.0	1080	362	135	90.4	183	112	0.428	13.7
200x100x6.3	200x100	x6.0	25.9	33.0	27.3	10.7	1640	560	164	112	207	128	0.569	22.0
x8.0		x8.0	33.5	42.7	19.0	6.5	2030	688	203	138	261	161	0.559	16.7
x10.0		x10.0	40.6	51.7	14.0	4.0	2340	790	234	158	309	190	0.548	13.5
200x150x6.3	200x150	x6.0	30.6	39.0	27.3	19.0	2200	1420	220	189	265	218	0.669	21.8
x8.0		x8.0	39.8	50.7	19.0	12.8	2760	1780	276	237	338	278	0.659	16.6
x10.0		x10.0	48.4	61.7	14.0	9.0	3250	2080	325	277	404	332	0.648	13.4
250x150x6.3	250x150	x6.0	35.3	45.0	35.7	19.0	3780	1730	302	231	370	261	0.769	21.8
x8.0		x8.0	46.1	58.7	25.3	12.8	4790	2180	383	291	475	335	0.759	16.5
x10.0		x10.0	56.3	71.7	19.0	9.0	5670	2570	454	343	570	402	0.748	13.3
x12.5		x12.0	64.0	81.6	12.8	4.5	5980	2740	478	365	623	440	0.718	11.2
260x180x6.3	260x180	x6.0	39.1	49.8	37.3	24.0	4750	2710	365	301	439	342	0.849	21.7
x8.0		x8.0	51.1	65.1	26.5	16.5	6040	3440	465	382	565	440	0.839	16.4
x10.0		x10.0	62.6	79.7	20.0	12.0	7200	4090	554	454	682	531	0.828	13.2
x12.5		x12.0	71.6	91.2	13.7	7.0	7730	4420	595	491	753	588	0.798	11.1
300x200x6.3	300x200	x6.0	44.8	57.0	44.0	27.3	7220	3900	481	390	578	440	0.969	21.7
x8.0		x8.0	58.6	74.7	31.5	19.0	9250	4980	617	498	748	568	0.959	16.4
x10.0		x10.0	72.0	91.7	24.0	14.0	11090	5960	739	596	907	689	0.948	13.2
x12.5		x12.0	82.9	106	17.0	8.7	12100	6540	807	654	1010	772	0.918	11.1
x16.0		x16.0	105	134	10.8	4.5	14400	7750	960	775	1240	946	0.890	8.44
350x250x6.3	350x250	x6.0	54.2	69.0	52.3	35.7	12300	7360	703	589	830	663	1.17	21.6
x8.0		x8.0	71.2	90.7	37.8	25.3	15800	9480	903	758	1080	862	1.16	16.3
x10.0		x10.0	87.7	112	29.0	19.0	19100	11400	1091	912	1320	1050	1.15	13.1
x12.5		x12.0	102	130	21.2	12.8	21300	12800	1217	1020	1500	1190	1.12	11.0
x16.0		x16.0	131	166	13.9	7.6	25900	15500	1480	1240	1870	1490	1.09	8.35
400x200x6.3	400x200	x6.0	54.2	69.0	60.7	27.3	14500	5030	725	503	893	556	1.17	21.6
x8.0		x8.0	71.2	90.7	44.0	19.0	18700	6450	935	645	1160	722	1.16	16.3
x10.0		x10.0	87.7	112	34.0	14.0	22600	7760	1130	776	1420	879	1.15	13.1
x12.5		x12.0	102	130	25.3	8.7	25100	8660	1260	866	1600	1000	1.12	11.0
x16.0		x16.0	131	166	17.0	4.5	30400	10500	1520	1050	1990	1240	1.09	8.35
450x250x8.0	450x250	x8.0	83.8	107	50.3	25.3	29000	11800	1290	940	1580	1060	1.36	16.2
x10.0		x10.0	103	132	39.0	19.0	35300	14300	1570	1140	1930	1290	1.35	13.0
x12.5		x12.0	121	154	29.5	12.8	39700	16200	1760	1300	2200	1480	1.32	10.9
x16.0		x16.0	156	198	20.1	7.6	49000	19900	2180	1590	2780	1860	1.29	8.28
500x300x8.0	500x300	x8.0	96.3	123	56.5	31.5	42400	19500	1700	1300	2050	1450	1.56	16.2
x10.0		x10.0	119	152	44.0	24.0	51700	23700	2070	1580	2510	1780	1.55	13.0
x12.5		x12.0	139	178	33.7	17.0	58900	27100	2360	1810	2900	2050	1.52	10.9
x16.0		x16.0	181	230	23.3	10.8	73600	33800	2940	2250	3680	2610	1.49	8.24
x20.0		x20.0	220	280	17.0	7.0	85800	39300	3430	2620	4380	3100	1.46	6.66

Design Table 24 Section dimensions and properties of cold-formed SHS



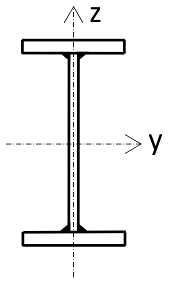
SHS	EWSHS		Mass per Meter	Area of section	Ratio for Local Buckling	Second Moment of area	Elastic Modulus	Plastic Modulus	Surface Area	
	b mm	*t* mm	*m* kg/m	*A* cm^2	*c/t*	I_y cm^4	W_{el} cm^3	W_{pl} cm^3	*per Meter* m^2	*per Tonne* m^2
100x100x6.3	100	x6.0	16.5	21.0	10.7	294	58.9	71.8	0.369	22.4
x8.0		x8.0	21.0	26.7	6.5	349	69.7	87.8	0.359	17.1
150x150x6.3	150	x6.0	25.9	33.0	19.0	1110	148	175	0.569	22.0
x8.0		x8.0	33.5	42.7	12.8	1370	183	221	0.559	16.7
x10.0		x10.0	40.6	51.7	9.0	1590	212	262	0.548	13.5
200x200x6.3	200	x6.0	35.3	45.0	27.3	2770	277	323	0.769	21.8
x8.0		x8.0	46.1	58.7	19.0	3500	350	415	0.759	16.5
x10.0		x10.0	56.3	71.7	14.0	4150	415	499	0.748	13.3
x12.5		x12.0	64.0	81.6	8.7	4410	441	546	0.718	11.2
220x220x6.3	220	x6.0	39.1	49.8	30.7	3730	339	395	0.849	21.7
x8.0		x8.0	51.1	65.1	21.5	4750	432	509	0.839	16.4
x10.0		x10.0	62.6	79.7	16.0	5660	515	614	0.828	13.2
x12.5		x12.0	71.6	91.2	10.3	6110	555	680	0.798	11.1
x16.0		x16.0	90.4	115.2	5.8	7110	646	822	0.770	8.52
250x250x6.3	250	x6.0	44.8	57.0	35.7	5570	446	516	0.969	21.7
x8.0		x8.0	58.6	74.7	25.3	7130	570	668	0.959	16.4
x10.0		x10.0	72.0	91.7	19.0	8550	684	810	0.948	13.2
x12.5		x12.0	82.9	105.6	12.8	9380	750	908	0.918	11.1
x16.0		x16.0	105	134.4	7.6	11160	893	1114	0.890	8.44
300x300x6.3	300	x6.0	54.2	69.0	44.0	9820	655	755	1.17	21.6
x8.0		x8.0	71.2	90.7	31.5	12700	847	982	1.16	16.3
x10.0		x10.0	87.7	112	24.0	15300	1020	1200	1.15	13.1
x12.5		x12.0	102	130	17.0	17100	1140	1360	1.12	11.0
x16.0		x16.0	131	166	10.8	20800	1390	1700	1.09	8.35
350x350x8.0	350	x8.0	83.8	107	37.8	20500	1170	1360	1.36	16.2
x10.0		x10.0	103	132	29.0	24900	1420	1660	1.35	13.0
x12.5		x12.0	121	154	21.2	28200	1610	1900	1.32	10.9
x16.0		x16.0	156	198	13.9	34900	1990	2400	1.29	8.28
400x400x8.0	400	x8.0	96.3	123	44.0	31000	1550	1790	1.56	16.2
x10.0		x10.0	119	152	34.0	37800	1890	2200	1.55	13.0
x12.5		x12.0	139	178	25.3	43200	2160	2530	1.52	10.9
x16.0		x16.0	181	230	17.0	54000	2700	3220	1.49	8.24
x20.0		x20.0	220	280	12.0	63200	3160	3840	1.46	6.66

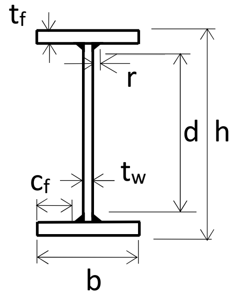
Design Tables 25 to 27 for

Section Resistances of Welded Sections: Q235 Steel

- **EWI-sections (EWIS)**
- **EWH-sections (EWHS)**

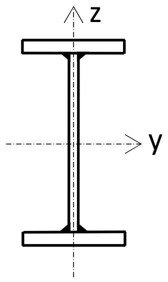
Design Table 25 Section resistances of welded I-sections: Q235 steel (1)

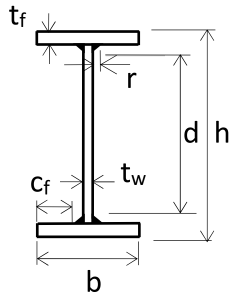




EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
920x450x420	1690	128	1	1	4010	847	1980	10900
x353	1350	95.8	1	1	3230	640	2020	9130
920x360x312	1170	49.1	1	1	2800	417	2050	8080
x282	1008	41.0	1	1	2480	350	2050	7350
x249	918	37.6	1	1	2230	325	1700	6050
x218	746	27.6	1	1	1860	249	1700	5210
840x350x246	804	41.0	1	1	2080	344	1580	6090
x214	651	30.1	1	1	1730	262	1580	5240
x184	596	27.5	1	1	1560	243	1180	4250
760x320x220	584	28.7	1	1	1680	273	1420	5590
x194	483	23.0	1	1	1420	220	1420	4900
x167	431	20.9	1	1	1260	202	1050	3960
x147	365	16.7	1	1	1130	170	1060	3600
690x280x198	420	21.4	1	1	1350	225	1260	5170
x173	349	15.4	1	1	1140	170	1280	4500
x151	317	15.4	1	1	1030	166	947	3670
x133	269	12.3	1	1	921	139	959	3350
620x330x258	481	41.4	1	1	1660	370	1370	6710
x186	351	31.5	1	1	1220	282	844	4710
x160	286	25.2	1	1	1010	228	844	4030
610x260x158	286	15.4	1	1	1010	178	844	3990
x138	233	12.3	1	1	843	143	844	3460
x122	199	9.85	1	1	756	120	856	3160
x112	184	9.85	1	1	703	119	701	2800
540x250x148	211	13.7	1	1	839	164	740	3840
x128	172	10.9	1	1	700	132	740	3330
x113	146	8.76	1	1	626	111	752	3060
x104	135	8.76	1	1	583	110	614	2690
x88	109	6.55	2	2	476	83.0	624	2250

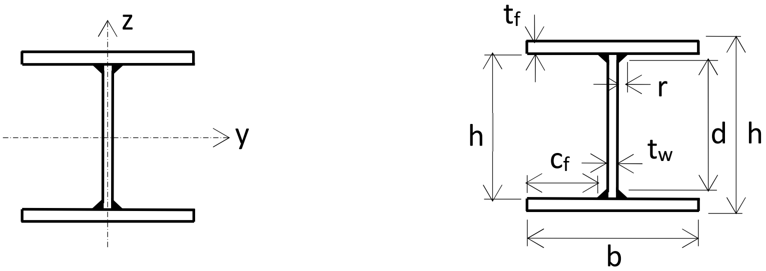
Design Table 26 Section resistances of welded I-sections: Q235 steel (2)





EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	EI_z	Bending	Bending	$M_{y,Rd}$	$M_{z,Rd}$	$V_{z,Rd}$	$N_{a,Rd}$
	10^3*kNm^2	10^3*kNm^2	*y-y*	*z-z*	kNm	kNm	kN	kN
460x220x112	113	7.46	1	1	532	103	636	2910
x104	104	7.46	1	1	495	101	518	2700
x90	88.4	5.96	1	1	440	85.4	528	2460
x83	85.1	5.96	1	1	419	84.4	422	2170
x70	68.5	4.47	1	1	340	63.7	430	1790
460x180x91	88.0	4.07	1	1	423	68.8	518	2370
x80	75.2	3.26	1	1	380	58.0	528	2190
x73	71.8	3.26	1	1	357	57.0	422	1890
x62	58.4	2.44	1	1	295	43.2	430	1590
x56	48.9	2.04	1	1	254	36.3	424	1430
410x210x85	69.3	5.19	1	1	378	77.8	479	2310
x78	63.2	5.19	1	1	348	76.8	373	2070
x65	51.0	3.89	1	1	284	58.0	381	1720
x59	44.5	3.23	2	2	250	48.6	385	1540
400x150x49	37.0	1.70	1	1	210	33.7	278	1230
x43	30.7	1.18	1	1	176	24.9	281	1050
355x180x73	43.1	3.26	1	1	276	57.5	405	1970
x62	34.9	2.44	1	1	226	43.5	414	1670
x54	31.1	2.06	1	1	201	38.4	327	1460
x49	27.3	1.72	1	1	178	32.2	331	1330
355x170x44	25.8	1.70	1	1	166	31.7	248	1120
x38	21.4	1.37	2	2	140	25.5	247	978
310x160x59	26.7	2.29	1	1	198	44.9	274	1600
x45	21.0	1.70	1	1	152	33.5	212	1200
x39	16.9	1.43	1	1	128	28.0	207	1070
310x140x54	23.9	1.53	1	1	177	34.6	274	1460
x45	19.5	1.16	1	1	146	26.3	282	1230
x41	15.9	0.945	1	1	124	22.1	276	1100
310x110x37	14.9	0.462	1	1	113	14.1	291	1000
x33	12.5	0.357	1	1	97.7	11.5	290	906
x28	11.2	0.357	1	1	86.5	11.1	214	754
260x170x48	15.4	2.06	1	1	135	38.0	233	1300
x43	13.5	1.70	1	1	119	31.9	237	1160
x33	10.06	1.37	2	2	91.0	25.3	173	908
260x130x33	10.4	0.756	1	1	91.1	18.7	178	890
x29	8.84	0.609	1	1	78.3	15.1	181	784
x25	6.97	0.462	1	1	63.7	11.5	180	672
210x150x34	7.25	1.18	1	1	78.2	24.6	141	912
x29	5.52	0.945	1	1	62.5	19.7	136	776
200x110x27	5.00	0.462	1	1	57.4	13.4	133	728
180x100x19	2.96	0.273	1	1	37.3	8.81	80.9	509
150x100x18	1.97	0.273	1	1	29.9	8.79	66.1	484
130x80x15	1.18	0.126	1	1	20.9	5.69	56.2	398

Design Table 27 Section resistances of welded H-sections: Q235 steel



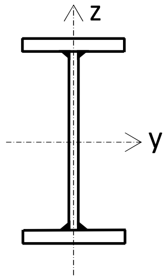
EWHS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	EI_z	Bending	Bending	$M_{y,Rd}$	$M_{z,Rd}$	$V_{z,Rd}$	$N_{a,Rd}$
	10^3*kNm^2	10^3*kNm^2	*y-y*	*z-z*	kNm	kNm	kN	kN
420x480x716	638	291	1	1	3110	1770	1750	17800
x563	519	233	1	1	2500	1380	1610	14000
x532	460	232	1	1	2310	1370	1130	13200
x456	368	194	1	1	1900	1140	1130	11400
x368	279	155	1	1	1560	955	945	9600
x300	218	124	1	1	1240	749	974	7810
x275	201	123	1	1	1150	743	632	7170
360x440x218	159	74.6	1	1	904	501	651	5680
x217	150	74.6	1	1	875	501	632	5650
x172	116	59.6	2	2	691	399	474	4490
x167	115	59.6	2	2	681	399	395	4360
370x330x316	189	63.0	1	1	1130	541	797	7870
x260	147	55.0	1	1	951	481	638	6770
x213	116	45.2	1	1	761	383	661	5550
x177	93.7	37.6	1	1	624	319	553	4600
x152	76.2	32.7	1	1	520	270	553	3970
x142	73.9	32.7	1	1	497	268	414	3690
x114	56.9	26.1	3	3	408	223	343	3100
270x310x184	70.4	28.4	1	1	550	284	543	4790
x151	56.1	23.6	1	1	446	235	454	3930
x121	44.1	20.9	1	1	356	199	340	3160
x116	39.9	20.9	1	1	333	198	271	3030
x93	33.0	16.7	1	1	284	165	225	2540
210x230x101	22.5	8.11	1	1	231	115	252	2620
x85	19.9	7.46	1	1	200	100	222	2210
x73	15.9	6.80	1	1	173	91.7	220	2000
x58	13.0	5.80	1	1	139	74.7	184	1580
x50	10.12	4.83	3	3	114	62.3	178	1360
170x170x42	5.92	2.06	1	1	79.9	37.7	144	1150
x34	4.43	1.72	1	1	62.6	31.3	104	933
x29	3.74	1.38	2	2	52.7	25.1	107	793

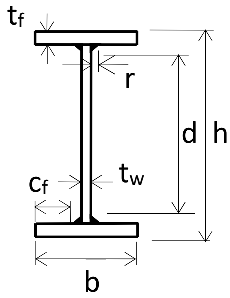
Design Tables 28 to 33 for

Section Resistances of Welded Sections: Q275 steel

- **EWI-sections (EWIS)**
- **EWH-sections (EWHS)**
- **EWCHS**
- **EWRHS**
- **EWSHS**

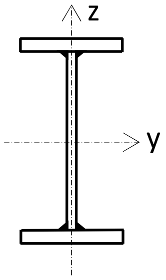
Design Table 28 Section resistances of welded I-sections: Q275 steel (1)

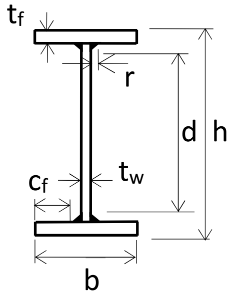




EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
920x450x420	1690	128	1	1	4720	1000	2340	12600
x353	1350	95.8	1	1	3810	754	2380	10400
920x360x312	1170	49.1	1	1	3300	491	2420	9150
x282	1008	41.0	1	1	2920	412	2420	8280
x249	918	37.6	1	1	2630	383	1990	6980
x218	746	27.6	1	1	2190	293	1990	6000
840x350x246	804	41.0	1	1	2450	405	1850	7040
x214	651	30.1	1	1	2040	309	1850	6040
x184	596	27.5	1	1	1830	286	1390	4910
760x320x220	584	28.7	1	1	1980	321	1660	6470
x194	483	23.0	1	1	1670	259	1660	5660
x167	431	20.9	1	1	1480	238	1230	4580
x147	365	16.7	2	2	1320	199	1240	4120
690x280x198	420	21.4	1	1	1590	265	1480	6080
x173	349	15.4	1	1	1340	200	1500	5180
x151	317	15.4	1	1	1210	195	1110	4250
x133	269	12.3	1	1	1080	163	1120	3840
620x330x258	481	41.4	1	1	1960	436	1610	7900
x186	351	31.5	1	1	1440	332	990	5470
x160	286	25.2	1	1	1190	268	990	4680
610x260x158	286	15.4	1	1	1190	209	990	4630
x138	233	12.3	1	1	990	168	990	4000
x122	199	9.85	1	1	885	141	1000	3630
x112	184	9.85	1	1	823	139	820	3220
540x250x148	211	13.7	1	1	990	193	866	4440
x128	172	10.9	1	1	824	156	866	3840
x113	146	8.76	1	1	733	130	880	3480
x104	135	8.76	1	1	683	129	719	3100
x88	109	6.55	3	3	558	97.2	730	2580

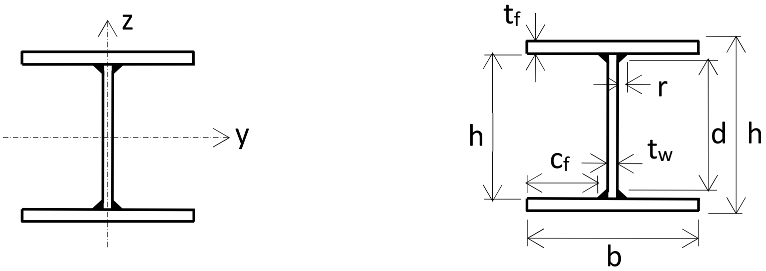
Design Table 29 Section resistances of welded I-sections: Q275 steel (2)





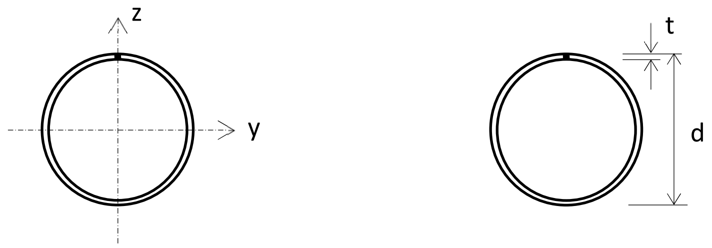
EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
460x220x112	113	7.46	1	1	626	121	814	3430
x104	104	7.46	1	1	583	120	664	3180
x90	88.4	5.96	1	1	515	100	664	2880
x83	85.1	5.96	1	1	490	99	531	2500
x70	68.5	4.47	1	1	398	74.6	531	2060
460x180x91	88.0	4.07	1	1	499	81.0	664	2790
x80	75.2	3.26	1	1	445	67.9	664	2560
x73	71.8	3.26	1	1	418	66.7	531	2180
x62	58.4	2.44	1	1	345	50.6	531	1820
x56	48.9	2.04	1	1	298	42.4	520	1640
410x210x85	69.3	5.19	1	1	443	91.0	606	2700
x78	63.2	5.19	1	1	408	89.9	473	2390
x65	51.0	3.89	1	1	333	67.9	473	1980
x59	44.5	3.23	3	3	293	56.9	473	1770
400x150x49	37.0	1.70	1	1	245	39.4	346	1420
x43	30.7	1.18	1	1	206	29.2	346	1210
355x180x73	43.1	3.26	1	1	323	67.3	520	2310
x62	34.9	2.44	1	1	265	51.0	520	1950
x54	31.1	2.06	1	1	235	44.9	410	1710
x49	27.3	1.72	1	1	208	37.7	410	1550
355x170x44	25.8	1.70	1	1	194	37.1	307	1290
x38	21.4	1.37	3	3	163	29.8	303	1120
310x160x59	26.7	2.29	1	1	231	52.5	358	1870
x45	21.0	1.70	1	1	178	39.2	268	1380
x39	16.9	1.43	1	1	150	32.8	260	1220
310x140x54	23.9	1.53	1	1	208	40.5	358	1710
x45	19.5	1.16	1	1	171	30.8	358	1440
x41	15.9	0.945	1	1	145	25.8	346	1290
310x110x37	14.9	0.462	1	1	132	16.5	364	1170
x33	12.5	0.357	1	1	114	13.5	358	1060
x28	11.2	0.357	1	1	101	12.9	264	865
260x170x48	15.4	2.06	1	1	158	44.5	300	1520
x43	13.5	1.70	1	1	139	37.3	300	1360
x33	10.1	1.37	3	3	106	29.6	217	1060
260x130x33	10.4	0.756	1	1	107	21.9	225	1040
x29	8.84	0.609	1	1	91.7	17.6	225	918
x25	6.97	0.462	2	2	74.5	13.4	221	787
210x150x34	7.25	1.18	1	1	91.5	28.7	182	1070
x29	5.52	0.945	1	1	73.2	23.1	173	908
200x110x27	5.00	0.462	1	1	67.2	15.7	173	852
180x100x19	2.96	0.273	1	1	43.7	10.3	104	596
150x100x18	1.97	0.273	1	1	34.9	10.3	86.6	566
130x80x15	1.18	0.126	1	1	24.5	6.66	75.1	466

Design Table 30 Section resistances of welded H-sections: Q275 steel



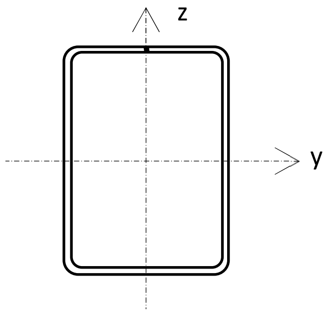
EWHS	Flexural Rigidity EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Section Classification Bending *y-y*	Bending *z-z*	Moment Resistance $M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	Shear Resistance $V_{z,Rd}$ kN	Axial Resistance $N_{a,Rd}$ kN
420x480x716	638	291	1	1	3540	2020	2070	20300
x563	519	233	1	1	2970	1640	1890	16600
x532	460	232	1	1	2740	1620	1340	15700
x456	368	194	1	1	2260	1350	1340	13500
x368	279	155	1	1	1840	1130	1110	11300
x300	218	124	1	1	1460	882	1150	9190
x275	201	123	1	1	1360	875	739	8440
360x440x218	159	74.6	1	1	1060	590	762	6700
x217	150	74.6	1	1	1030	590	739	6660
x172	116	59.6	3	3	814	470	554	5290
x167	115	59.6	3	3	802	470	462	5130
370x330x316	189	63.0	1	1	1340	642	939	9330
x260	147	55.0	1	1	1120	566	751	7980
x213	116	45.2	1	1	896	451	779	6530
x177	93.7	37.6	1	1	735	376	647	5420
x152	76.2	32.7	1	1	612	318	647	4670
x142	73.9	32.7	1	1	585	315	485	4350
x114	56.9	26.1	3	3	478	261	401	3630
270x310x184	70.4	28.4	1	1	648	335	640	5640
x151	56.1	23.6	1	1	525	276	531	4620
x121	44.1	20.9	1	1	419	234	398	3720
x116	39.9	20.9	1	1	393	233	318	3570
x93	33.0	16.7	2	2	333	193	263	2970
210x230x101	22.5	8.11	1	1	272	135	294	3090
x85	19.9	7.46	1	1	236	118	260	2600
x73	15.9	6.80	1	1	203	107	257	2340
x58	13.0	5.80	2	2	163	87.4	215	1840
x50	10.1	4.83	3	3	133	72.9	208	1590
170x170x42	5.92	2.06	1	1	93.5	44.2	169	1340
x34	4.43	1.72	1	1	73.3	36.6	121	1090
x29	3.74	1.38	3	3	61.7	29.4	125	928

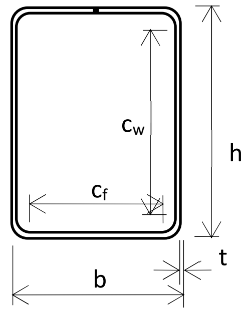
Design Table 31 Section resistances of cold-formed CHS: Q275 steel



EWCHS	Flexural rigidity	Q275			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
140x6.0	1.19	1	26.9	232	631
x8.0	1.52	1	34.8	305	829
x10.0	1.82	1	42.3	375	1020
170x6.0	2.19	1	40.3	284	773
x8.0	2.81	1	52.5	374	1020
x10.0	3.38	1	64.0	462	1260
x12.0	3.93	1	74.9	547	1490
220x6.0	4.85	1	68.7	371	1010
x8.0	6.30	1	89.9	490	1330
x10.0	7.64	1	110	606	1650
x12.0	8.93	1	130	721	1960
270x6.0	9.11	2	105	457	1240
x8.0	11.9	1	137	605	1650
x10.0	14.5	1	169	751	2040
x12.0	17.0	1	200	894	2430
x16.0	21.7	1	258	1170	3190
320x6.0	15.3	2	148	544	1480
x8.0	20.0	1	195	721	1960
x10.0	24.6	1	240	895	2430
x12.0	29.0	1	285	1070	2900
x16.0	37.2	1	370	1400	3820
360x6.0	22.0	3	145	613	1670
x8.0	28.8	2	248	813	2210
x10.0	35.5	1	308	1010	2750
x12.0	41.8	1	363	1210	3280
x16.0	53.8	1	473	1590	4320
400x8.0	39.7	2	308	905	2460
x10.0	48.9	1	380	1130	3060
x12.0	58.0	1	453	1340	3660
x16.0	74.8	1	590	1770	4830
x20.0	90.7	1	696	2110	5750
460x8.0	60.9	2	408	1040	2840
x10.0	75.2	2	508	1300	3530
x12.0	89.0	1	603	1550	4220
x16.0	116	1	788	2050	5580
x20.0	141	1	932	2450	6660
500x8.0	78.5	3	375	1140	3090
x10.0	97.0	2	600	1410	3850
x12.0	115	1	715	1690	4600
x16.0	150	1	938	2240	6080
x20.0	183	1	1110	2670	7270
610x8.0	144	3	563	1390	3780
x10.0	178	3	695	1730	4710
x12.0	212	2	1070	2070	5640
x16.0	277	1	1410	2740	7460
x20.0	338	1	1680	3280	8930
710x10.0	284	3	950	2020	5500
x12.0	336	2	1460	2420	6580
x16.0	441	2	1930	3210	8720
x20.0	542	1	2290	3840	10400
810x10.0	422	4	-	2310	6280
x12.0	504	3	1480	2760	7520
x16.0	662	2	2520	3670	9980
x20.0	813	1	3010	4400	12000

Design Table 32 Section resistances of cold-formed RHS: Q275 steel

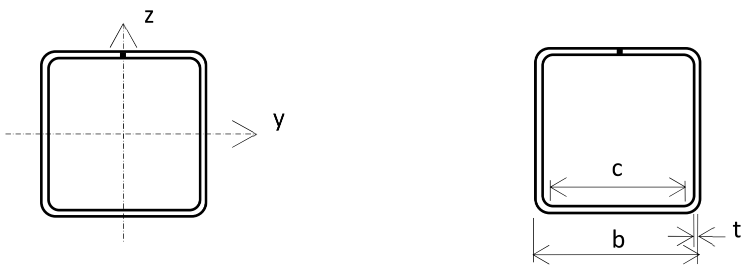




EWRHS	Flexural rigidity		Q275					
			Section Classification		Moment resistance		Shear Resistance	Axial Resistance
	EIEI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
120x80x6.0	0.801	0.428	1	1	20.1	15.2	187	525
x8.0	0.948	0.505	1	1	24.5	18.6	240	667
160x80x6.0	1.67	0.566	1	1	31.8	19.7	256	645
x8.0	2.01	0.679	1	1	39.5	24.3	333	827
x10.0	2.27	0.760	1	1	45.8	28.1	404	993
200x100x6.0	3.44	1.18	1	1	51.7	32.0	326	825
x8.0	4.26	1.44	1	1	65.3	40.4	425	1070
x10.0	4.91	1.66	1	1	77.1	47.5	520	1290
200x150x6.0	4.62	2.98	1	1	66.3	54.6	326	975
x8.0	5.80	3.74	1	1	84.5	69.5	425	1270
x10.0	6.83	4.37	1	1	101	82.9	520	1540
250x150x6.0	7.94	3.63	1	3	92.5	57.7	412	1130
x8.0	10.1	4.58	1	1	119	83.7	540	1470
x10.0	11.9	5.40	1	1	143	100	664	1790
x12.0	12.6	5.75	1	1	156	110	783	2040
260x180x6.0	9.98	5.69	1	3	110	75.3	430	1250
x8.0	12.7	7.22	1	1	141	110	563	1630
x10.0	15.1	8.59	1	1	170	133	693	1990
x12.0	16.2	9.28	1	1	188	147	818	2280
300x200x6.0	15.2	8.19	1	4	145	-	499	1330
x8.0	19.4	10.5	1	2	187	142	656	1870
x10.0	23.3	12.5	1	1	227	172	808	2290
x12.0	25.4	13.7	1	1	253	193	956	2640
x16.0	30.2	16.3	1	1	310	236	1240	3360
350x250x6.0	25.8	15.5	3	4	176	-	585	1520
x8.0	33.2	19.9	1	3	270	190	771	2270
x10.0	40.1	23.9	1	1	330	263	953	2790
x12.0	44.7	26.9	1	1	375	298	1130	3240
x16.0	54.4	32.6	1	1	468	373	1470	4160
400x200x6.0	30.5	10.6	1	4	223	-	672*	1400
x8.0	39.3	13.5	1	4	290	-	887	2100
x10.0	47.5	16.3	1	2	355	220	1100	2790
x12.0	52.7	18.2	1	1	400	250	1300	3240
x16.0	63.8	22.1	1	1	498	310	1700	4160
450x250x8.0	60.9	24.8	1	4	395	-	1000	2350
x10.0	74.1	30.0	1	4	483	-	1240	3190
x12.0	83.4	34.0	1	1	550	370	1480	3840
x16.0	103	41.8	1	1	695	465	1930	4960
500x300x8.0	89.0	41.0	2	4	513	-	1120	2600
x10.0	109	49.8	1	4	628	-	1390	3530
x12.0	124	56.9	1	2	725	513	1650	4440
x16.0	155	71.0	1	1	920	653	2160	5760
x20.0	180	82.5	1	1	1060	747	2560	6740

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 33 Section resistances of cold-formed SHS: Q275 steel



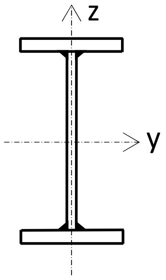
EWSHS	Flexural rigidity	Q275			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
100x100x6.0	0.618	1	17.9	152	525
x8.0	0.732	1	22.0	194	667
150x150x6.0	2.33	1	43.8	239	825
x8.0	2.88	1	55.3	309	1070
x10.0	3.34	1	65.4	375	1290
200x200x6.0	5.82	1	80.8	326	1130
x8.0	7.35	1	104	425	1470
x10.0	8.72	1	125	520	1790
x12.0	9.26	1	137	610	2040
220x220x6.0	7.83	2	99	360	1250
x8.0	9.98	1	127	471	1630
x10.0	11.9	1	154	577	1990
x12.0	12.8	1	170	679	2280
x16.0	14.9	1	205	868	2880
250x250x6.0	11.7	3	111	412	1420
x8.0	15.0	1	167	540	1870
x10.0	18.0	1	203	664	2290
x12.0	19.7	1	227	783	2640
x16.0	23.4	1	279	1010	3360
300x300x6.0	20.6	4	-	499	1540
x8.0	26.7	2	245	656	2270
x10.0	32.1	1	300	808	2790
x12.0	35.9	1	340	956	3240
x16.0	43.7	1	425	1240	4160
350x350x8.0	43.1	3	293	771	2580
x10.0	52.3	1	415	953	3290
x12.0	59.2	1	475	1130	3840
x16.0	73.3	1	600	1470	4960
400x400x8.0	65.1	4	-	887	2730
x10.0	79.4	2	550	1100	3790
x12.0	90.7	1	633	1300	4440
x16.0	113	1	805	1700	5760
x20.0	133	1	925	2000	6740

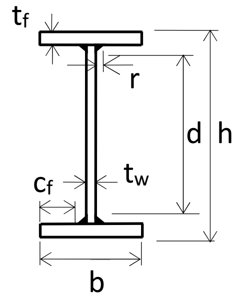
Design Tables 34 to 39 for

Section Resistances of Welded Sections: Q355 steel

- **EWI-sections (EWIS)**
- **EWH-sections (EWHS)**
- **EWCHS**
- **EWRHS**
- **EWSHS**

Design Table 34 Section resistances of welded I-sections: Q355 steel (1)

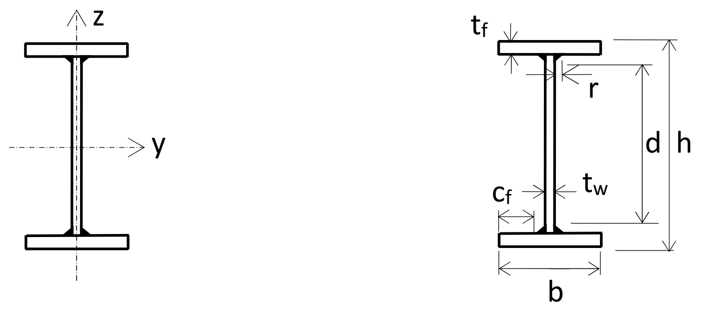




EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	EI_z	Bending	Bending	$M_{y,Rd}$	$M_{z,Rd}$	$V_{z,Rd}$	$N_{a,Rd}$
	10^3*kNm^2	10^3*kNm^2	*y-y*	*z-z*	kNm	kNm	kN	kN
920x450x420	1690	128	1	1	6150	1300	3040	16000
x353	1350	95.8	1	1	4670	925	3100	12400
920x360x312	1170	49.1	1	1	4050	603	3150	10900
x282	1008	41.0	1	1	3580	505	3150	9790
x249	918	37.6	1	1	3230	470	2560	8280
x218	746	27.6	1	1	2690	359	2560	7080
840x350x246	804	41.0	1	1	3010	496	2380	8380
x214	651	30.1	2	2	2500	379	2380	7140
x184	596	27.5	2	2	2250	351	1640*	5850
760x320x220	584	28.7	1	1	2430	394	2150	7700
x194	483	23.0	1	1	2050	318	2150	6700
x167	431	20.9	1	1	1810	292	1590	5450
x147	365	16.7	3	3	1370	152	1610	4700
690x280x198	420	21.4	1	1	1960	325	1910	7150
x173	349	15.4	1	1	1650	245	1940	6140
x151	317	15.4	1	1	1480	239	1430	5060
x133	269	12.3	2	2	1270	193	1450	4380
620x330x258	481	41.4	1	1	2400	535	2100	9690
x186	351	31.5	1	1	1760	408	1270	6570
x160	286	25.2	1	1	1460	329	1270	5600
610x260x158	286	15.4	1	1	1450	257	1270	5540
x138	233	12.3	1	1	1220	207	1270	4770
x122	199	9.85	1	1	1050	167	1290	4140
x112	184	9.85	1	1	972	164	1060	3690
540x250x148	211	13.7	1	1	1210	237	1120	5330
x128	172	10.9	1	1	1010	191	1120	4590
x113	146	8.76	1	1	866	154	1140	3990
x104	135	8.76	1	1	807	152	928	3560
x88	109	6.55	3	3	579	73.7	943	2940

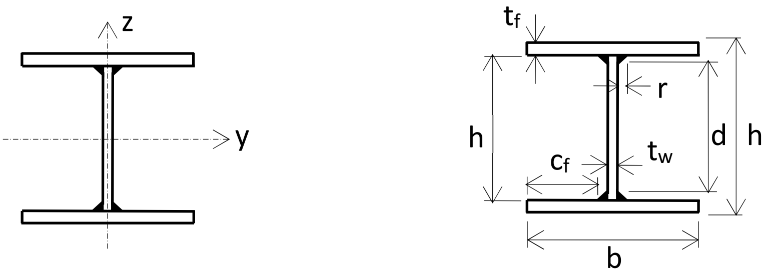
* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 35 Section resistances of welded I-sections: Q355 steel (2)



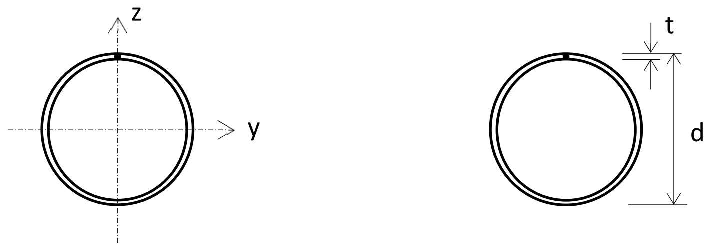
EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
460x220x112	113	7.46	1	1	815	158	961	4470
x104	104	7.46	1	1	759	156	783	4010
x90	88.4	5.96	1	1	665	129	797	3570
x83	85.1	5.96	1	1	633	127	638	3150
x70	68.5	4.47	2	2	458	62.5	650	2590
460x180x91	88.0	4.07	1	1	649	105.4	783	3510
x80	75.2	3.26	1	1	574	87.6	797	3150
x73	71.8	3.26	1	1	539	86.1	638	2740
x62	58.4	2.44	1	1	445	65.3	650	2280
x56	48.9	2.04	2	2	384	54.8	641	2050
410x210x85	69.3	5.19	1	1	571	118	723	3420
x78	63.2	5.19	1	1	526	116	563	3020
x65	51.0	3.89	2	2	429	87.7	575	2490
x59	44.5	3.23	3	3	334	47.3	581	2220
400x150x49	37.0	1.70	1	1	317	50.9	420	1780
x43	30.7	1.18	1	1	266	37.6	425	1510
355x180x73	43.1	3.26	1	1	416	86.8	611	2980
x62	34.9	2.44	1	1	342	65.8	626	2520
x54	31.1	2.06	1	1	303	57.9	493	2140
x49	27.3	1.72	1	1	269	48.6	499	1920
355x170x44	25.8	1.70	1	1	250	47.8	375	1630
x38	21.4	1.37	3	3	188	24.7	373	1410
310x160x59	26.7	2.29	1	1	298	67.8	414	2410
x45	21.0	1.70	1	1	230	50.6	320	1750
x39	16.9	1.43	1	1	193	42.4	313	1540
310x140x54	23.9	1.53	1	1	268	52.3	414	2200
x45	19.5	1.16	1	1	220	39.7	426	1860
x41	15.9	0.945	1	1	187	33.3	417	1670
310x110x37	14.9	0.462	1	1	170	21.3	440	1510
x33	12.5	0.357	1	1	148	17.4	438	1370
x28	11.2	0.357	1	1	131	16.7	323	1080
260x170x48	15.4	2.06	1	1	204	57.5	352	1970
x43	13.5	1.70	1	1	179	48.1	358	1760
x33	10.06	1.37	3	3	124	24.7	262	1370
260x130x33	10.4	0.756	1	1	138	28.2	268	1320
x29	8.84	0.609	1	1	118	22.8	273	1160
x25	6.97	0.462	3	3	84.0	10.9	272	988
210x150x34	7.25	1.18	1	1	118	37.1	212	1380
x29	5.52	0.945	2	2	94.4	29.8	206	1170
200x110x27	5.00	0.462	1	1	86.7	20.3	201	1100
180x100x19	2.96	0.273	1	1	56.4	13.3	122	769
150x100x18	1.97	0.273	1	1	45.1	13.27	99.9	731
130x80x15	1.18	0.126	1	1	31.6	8.60	85.0	602

Design Table 36 Section resistances of welded H-sections: Q355 steel



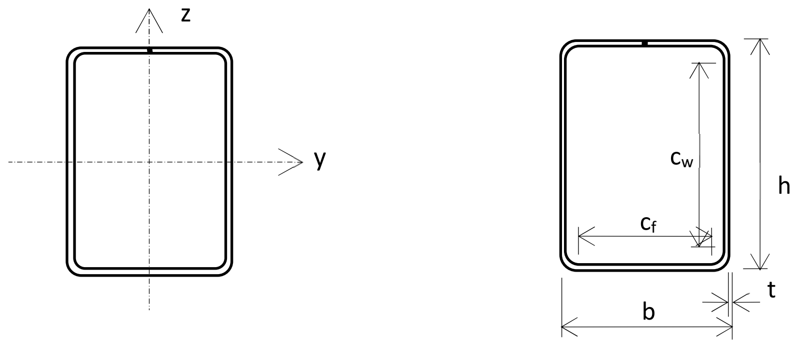
EWHS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
420x480x716	638	291	1	1	4700	2670	2730	26900
x563	519	233	1	1	3900	2150	2460	21800
x532	460	232	1	1	3590	2130	1740	20600
x456	368	194	1	1	2960	1780	1740	17700
x368	279	155	1	1	2390	1460	1450	14700
x300	218	124	1	1	1900	1150	1490	12000
x275	201	123	1	1	1770	1140	954	11000
360x440x218	159	74.6	2	2	1390	768	984	8720
x217	150	74.6	2	2	1340	768	954	8670
x172	116	59.6	3	3	966	405	715	6880
x167	115	59.6	3	3	957	405	596	6680
370x330x316	189	63.0	1	1	1770	844	1220	12300
x260	147	55.0	1	1	1460	737	978	10400
x213	116	45.2	1	1	1170	587	1010	8500
x177	93.7	37.6	1	1	957	489	835	7050
x152	76.2	32.7	2	2	797	414	835	6080
x142	73.9	32.7	2	2	762	411	626	5660
x114	56.9	26.1	3	3	565	223	518	4680
270x310x184	70.4	28.4	1	1	844	436	833	7340
x151	56.1	23.6	1	1	684	360	686	6020
x121	44.1	20.9	1	1	546	305	514	4850
x116	39.9	20.9	1	1	511	304	410	4640
x93	33.0	16.7	3	3	390	165	340	3830
210x230x101	22.5	8.11	1	1	354	176	380	4020
x85	19.9	7.46	1	1	307	154	335	3390
x73	15.9	6.80	1	1	261	139	332	3010
x58	13.0	5.80	3	3	190	74.2	277	2380
x50	10.12	4.83	3	3	156	61.9	268	2060
170x170x42	5.92	2.06	1	1	121	57.0	218	1730
x34	4.43	1.72	1	1	94.6	47.3	157	1410
x29	3.74	1.38	3	3	71.8	24.9	161	1200

Design Table 37 Section resistances of cold-formed CHS: Q355 steel



EWCHS	Flexural rigidity	Q355			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
140x6.0	1.19	1	34.8	300	815
x8.0	1.52	1	45	394	1070
x10.0	1.82	1	54.5	484	1320
170x6.0	2.19	1	52.1	367	1000
x8.0	2.81	1	67.8	483	1310
x10.0	3.38	1	82.6	596	1620
x12.0	3.93	1	96.7	707	1920
220x6.0	4.85	2	88.7	478	1300
x8.0	6.30	1	116	632	1720
x10.0	7.64	1	142	783	2130
x12.0	8.93	1	168	930	2530
270x6.0	9.11	2	135	590	1610
x8.0	11.9	1	177	781	2130
x10.0	14.5	1	218	969	2640
x12.0	17.0	1	258	1150	3140
x16.0	21.7	1	333	1510	4120
320x6.0	15.3	3	147	702	1910
x8.0	20.0	2	251	930	2530
x10.0	24.6	1	310	1160	3140
x12.0	29.0	1	367	1380	3750
x16.0	37.2	1	477	1810	4930
360x6.0	22.0	3	-	792	2150
x8.0	28.8	2	320	1049	2850
x10.0	35.5	2	397	1300	3550
x12.0	41.8	1	468	1560	4230
x16.0	53.8	1	610	2050	5580
400x8.0	39.7	3	305	1170	3180
x10.0	48.9	2	491	1450	3950
x12.0	58.0	1	584	1740	4720
x16.0	74.8	1	762	2290	6230
x20.0	90.7	1	906	2750	7490
460x8.0	60.9	3	407	1350	3670
x10.0	75.2	2	655	1680	4560
x12.0	89.0	2	778	2000	5450
x16.0	116	1	1017	2650	7200
x20.0	141	1	1210	3190	8670
500x8.0	78.5	4	-	1470	3990
x10.0	97.0	3	597	1830	4970
x12.0	115	2	923	2180	5940
x16.0	150	1	1210	2890	7850
x20.0	183	1	1450	3480	9460
610x8.0	144	4	-	1790	4880
x10.0	178	3	0	2240	6080
x12.0	212	3	1070	2670	7280
x16.0	277	2	1820	3540	9640
x20.0	338	1	2180	4270	11600
710x10.0	284	4	-	2610	7100
x12.0	336	3	1460	3120	8490
x16.0	441	2	2490	4140	11300
x20.0	542	2	2990	5000	13600
810x10.0	422	4	-	2980	8110
x12.0	504	4	-	3570	9710
x16.0	662	3	2510	4730	12900
x20.0	813	2	3910	5720	15600

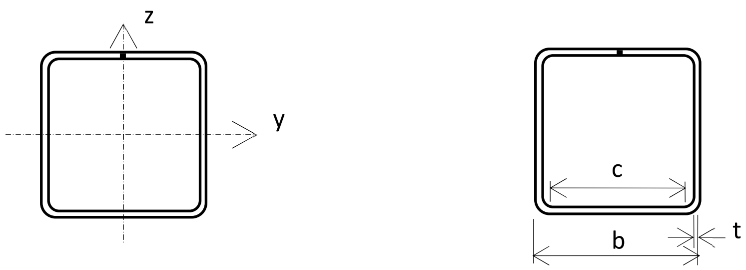
Design Table 38 Section resistances of cold-formed RHS: Q355 steel



EWRHS	Flexural rigidity		Q355					
			Section Classification		Moment resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
120x80x6.0	0.801	0.428	1	1	25.9	19.6	241	678
x8.0	0.948	0.505	1	1	31.7	24	310	861
160x80x6.0	1.67	0.566	1	1	41	25.4	331	833
x8.0	2.01	0.679	1	1	51	31.4	429	1070
x10.0	2.27	0.760	1	1	59.1	36.2	522	1280
200x100x6.0	3.44	1.18	1	2	66.8	41.4	420	1070
x8.0	4.26	1.44	1	1	84.3	52.1	549	1380
x10.0	4.91	1.66	1	1	99.6	61.3	671	1670
200x150x6.0	4.62	2.98	1	2	85.5	70.4	420	1260
x8.0	5.80	3.74	1	1	109	89.8	549	1640
x10.0	6.83	4.37	1	1	130	107	671	1990
250x150x6.0	7.94	3.63	1	4	119	-	532	1390
x8.0	10.1	4.58	1	1	153	108	698	1890
x10.0	11.9	5.40	1	1	184	130	857	2310
x12.0	12.6	5.75	1	1	201	142	1011	2630
260x180x6.0	9.98	5.69	1	4	142	-	555	1520
x8.0	12.7	7.22	1	1	182	142	727	2100
x10.0	15.1	8.59	1	1	220	171	894	2570
x12.0	16.2	9.28	1	1	243	190	1055	2940
300x200x6.0	15.2	8.19	2	4	187	-	644	1640
x8.0	19.4	10.5	1	3	242	161	847	2410
x10.0	23.3	12.5	1	1	293	222	1043	2960
x12.0	25.4	13.7	1	1	326	249	1230	3410
x16.0	30.2	16.3	1	1	400	305	1600	4340
350x250x6.0	25.8	15.5	4	4	-	-	756	1820
x8.0	33.2	19.9	1	4	349	-	996	2760
x10.0	40.1	23.9	1	2	426	339	1230	3610
x12.0	44.7	26.9	1	1	484	384	1460	4180
x16.0	54.4	32.6	1	1	604	481	1900	5370
400x200x6.0	30.5	10.6	2	4	288	-	821*	1710
x8.0	39.3	13.5	1	4	374	-	1140	2580
x10.0	47.5	16.3	1	3	458	250	1420	3610
x12.0	52.7	18.2	1	1	516	323	1680	4180
x16.0	63.8	22.1	1	1	642	400	2190	5370
450x250x8.0	60.9	24.8	1	4	510	-	1290	2890
x10.0	74.1	30.0	1	4	623	-	1600	3940
x12.0	83.4	34.0	1	2	710	478	1910	4960
x16.0	103	41.8	1	1	897	600	2490	6400
500x300x8.0	89.0	41.0	3	4	549	-	1440	3190
x10.0	109	49.8	1	4	810	-	1790	4350
x12.0	124	56.9	1	3	936	584	2130	5730
x16.0	155	71.0	1	1	1190	842	2790	7430
x20.0	180	82.5	1	1	1370	972	3330	8780

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 39 Section resistances of cold-formed SHS: Q355 steel

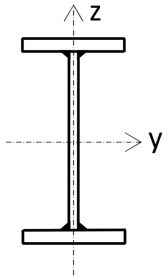


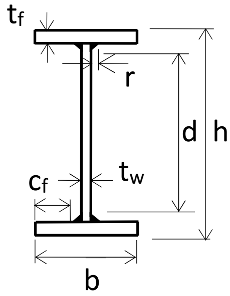
EWSHS	Flexural rigidity	Q355			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
100x100x6.0	0.618	1	23.2	197	678
x8.0	0.732	1	28.3	250	861
150x150x6.0	2.33	1	56.5	309	1070
x8.0	2.88	1	71.4	399	1380
x10.0	3.34	1	84.5	484	1670
200x200x6.0	5.82	2	104	420	1450
x8.0	7.35	1	134	549	1890
x10.0	8.72	1	161	671	2310
x12.0	9.26	1	176	787	2630
220x220x6.0	7.83	2	128	465	1610
x8.0	9.98	1	164	608	2100
x10.0	11.9	1	198	745	2570
x12.0	12.8	1	220	876	2940
x16.0	14.9	1	265	1120	3720
250x250x6.0	11.7	4	-	532	1720
x8.0	15.0	1	216	698	2410
x10.0	18.0	1	262	857	2960
x12.0	19.7	1	293	1010	3410
x16.0	23.4	1	360	1300	4340
300x300x6.0	20.6	4	-	644	1830
x8.0	26.7	3	273	847	2930
x10.0	32.1	1	387	1043	3610
x12.0	35.9	1	439	1230	4180
x16.0	43.7	1	549	1600	5370
350x350x8.0	43.1	4	-	996	3110
x10.0	52.3	2	536	1230	4250
x12.0	59.2	1	613	1460	4960
x16.0	73.3	1	775	1900	6400
400x400x8.0	65.1	4	-	1140	3260
x10.0	79.4	3	610	1420	4900
x12.0	90.7	1	817	1680	5730
x16.0	113	1	1040	2190	7430
x20.0	133	1	1200	2610	8780

Design Tables 40 to 45 for section resistances of welded sections: Q460 steel

- EWI-sections (EWIS)
- EWH-sections (EWHS)
- EWCHS
- EWRHS
- EWSHS

Design Table 40 Section resistances of welded I-sections: Q460 steel (1)

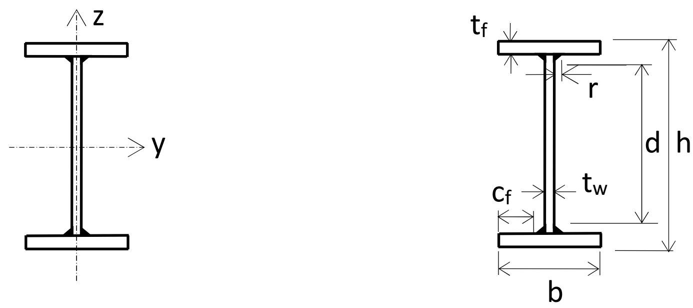




EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
920x450x420	1690	128	1	1	7840	1660	3880	19900
x353	1350	95.8	2	2	6320	1250	3950	16300
920x360x312	1170	49.1	1	1	5480	816	4020	14200
x282	1008	41.0	1	1	4840	684	4020	12800
x249	918	37.6	2	1	4370	636	3320*	10800
x218	746	27.6	3	3	3160	309	3320*	9220
840x350x246	804	41.0	1	1	4080	672	3090	11000
x214	651	30.1	3	3	2950	327	3090	9320
x184	596	27.5	3	3	2700	308	1870*	7690
760x320x220	584	28.7	1	1	3290	533	2780	10100
x194	483	23.0	2	2	2780	430	2780	8750
x167	431	20.9	2	2	2460	396	1870*	7160
x147	365	16.7	3	3	1940	214	1870*	6430
690x280x198	420	21.4	1	1	2650	440	2470	9390
x173	349	15.4	1	1	2230	332	2510	8020
x151	317	15.4	1	1	2010	324	1850	6640
x133	269	12.3	3	3	1570	175	1870*	5980
620x330x258	481	41.4	1	1	3250	724	2680	13100
x186	351	31.5	1	1	2380	552	1650	8710
x160	286	25.2	3	3	1780	290	1650	7390
610x260x158	286	15.4	1	1	1970	347	1650	7310
x138	233	12.3	1	1	1650	280	1650	6270
x122	199	9.85	2	2	1480	236	1670	5660
x112	184	9.85	3	3	1230	151	1300*	5070
540x250x148	211	13.7	1	1	1640	321	1450	7040
x128	172	10.9	1	1	1370	258	1450	6040
x113	146	8.76	2	2	1230	218	1470	5460
x104	135	8.76	2	2	1140	215	1200	4890
x88	109	6.55	3	3	820	104	1220	4020

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

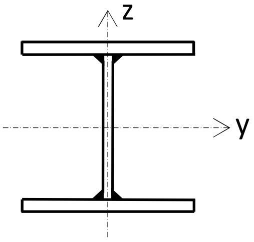
Design Table 41   Section resistances of welded I-sections: Q460 steel (2)

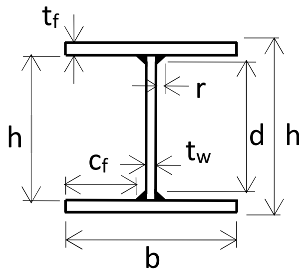


EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	EI_z	Bending	Bending	$M_{y,Rd}$	$M_{z,Rd}$	$V_{z,Rd}$	$N_{a,Rd}$
	10^3*kNm^2	10^3*kNm^2	*y-y*	*z-z*	kNm	kNm	kN	kN
460x220x112	113	7.46	1	1	1040	201	1250	5550
x104	104	7.46	1	1	968	198	1014	5000
x90	88.4	5.96	1	1	861	167	1030	4500
x83	85.1	5.96	1	1	820	165	827	4000
x70	68.5	4.47	3	3	594	81.0	842	3260
460x180x91	88.0	4.07	1	1	828	134	1014	4360
x80	75.2	3.26	1	1	744	114	1030	3970
x73	71.8	3.26	1	1	698	112	827	3460
x62	58.4	2.44	2	2	577	84.6	842	2860
x56	48.9	2.04	3	3	433	45.1	831	2560
410x210x85	69.3	5.19	1	1	740	152	937	4320
x78	63.2	5.19	1	1	682	150	730	3830
x65	51.0	3.89	3	3	498	73.7	746	3140
x59	44.5	3.23	3	3	432	61.3	753	2790
400x150x49	37.0	1.70	2	1	410	66.0	468*	2260
x43	30.7	1.18	2	1	345	48.8	468*	1910
355x180x73	43.1	3.26	1	1	539	113	792	3860
x62	34.9	2.44	2	2	443	85.2	811	3190
x54	31.1	2.06	1	1	393	75.1	639	2690
x49	27.3	1.72	3	3	306	40.3	647	2410
355x170x44	25.8	1.70	3	3	290	39.9	485	2060
x38	21.4	1.37	3	3	244	32.0	484	1770
310x160x59	26.7	2.29	1	1	387	87.9	537	3070
x45	21.0	1.70	1	1	298	65.6	414	2220
x39	16.9	1.43	2	2	251	54.9	406	1950
310x140x54	23.9	1.53	1	1	347	67.8	537	2800
x45	19.5	1.16	1	1	285	51.4	552	2350
x41	15.9	0.945	1	1	243	43.2	541	2100
310x110x37	14.9	0.462	1	1	221	27.6	570	1870
x33	12.5	0.357	1	1	191	22.6	568	1690
x28	11.2	0.357	1	1	169	21.6	419	1350
260x170x48	15.4	2.06	1	1	264	74.4	456	2550
x43	13.5	1.70	3	3	206	39.9	464	2280
x33	10.06	1.37	3	3	160	32.0	339	1720
260x130x33	10.4	0.756	1	1	178	36.5	348	1670
x29	8.84	0.609	2	2	153	29.5	353	1460
x25	6.97	0.462	3	3	109	14.2	352	1240
210x150x34	7.25	1.18	1	1	153	48.1	275	1780
x29	5.52	0.945	3	3	110	25.1	267	1520
200x110x27	5.00	0.462	1	1	112	26.3	261	1430
180x100x19	2.96	0.273	1	1	73.0	17.3	158	974
150x100x18	1.97	0.273	1	1	58.5	17.2	129	947
130x80x15	1.18	0.126	1	1	41.0	11.1	110	779

*   indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

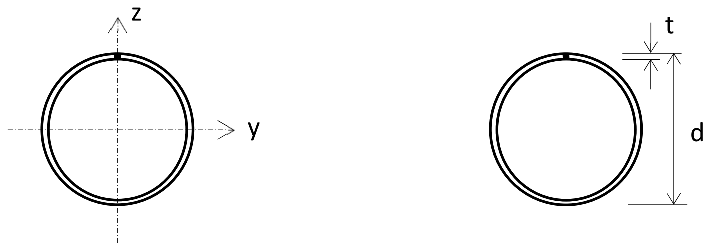
Design Table 42 Section resistances of welded H-sections: Q460 steel





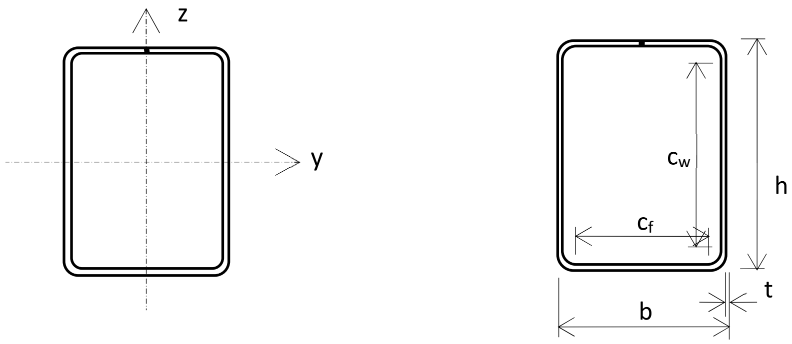
EWHS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	EI_z	Bending	Bending	$M_{y,Rd}$	$M_{z,Rd}$	$V_{z,Rd}$	$N_{a,Rd}$
	10^3*kNm^2	10^3*kNm^2	*y-y*	*z-z*	kNm	kNm	kN	kN
420x480x716	638	291	1	1	5780	3290	3420	33200
x563	519	233	1	1	4890	2700	3140	27400
x532	460	232	1	1	4510	2670	2220	25900
x456	368	194	1	1	3720	2230	2220	22200
x368	279	155	1	1	3050	1870	1850	18800
x300	218	124	3	3	2130	960	1910	15300
x275	201	123	3	3	2020	960	1240	14000
360x440x218	159	74.6	3	3	1590	644	1270	11100
x217	150	74.6	3	3	1540	644	1240	11100
x172	116	59.6	3	3	1230	516	927	8780
x167	115	59.6	3	3	1220	516	773	8520
370x330x316	189	63.0	1	1	2210	1060	1560	15400
x260	147	55.0	1	1	1860	940	1250	13200
x213	116	45.2	1	1	1490	748	1290	10800
x177	93.7	37.6	1	1	1220	624	1080	9000
x152	76.2	32.7	3	3	908	346	1080	7760
x142	73.9	32.7	3	3	880	346	811	7220
x114	56.9	26.1	4	4	-	-	671	5960
270x310x184	70.4	28.4	1	1	1080	556	1060	9360
x151	56.1	23.6	1	1	872	459	888	7680
x121	44.1	20.9	2	2	696	389	666	6180
x116	39.9	20.9	2	2	652	387	531	5920
x93	33.0	16.7	3	3	505	214	440	4960
210x230x101	22.5	8.11	1	1	452	224	493	5130
x85	19.9	7.46	1	1	391	196	435	4320
x73	15.9	6.80	1	1	339	180	430	3910
x58	13.0	5.80	3	3	247	96.2	359	3080
x50	10.12	4.83	4	4	-	-	348	2570
170x170x42	5.92	2.06	1	1	156	73.8	282	2250
x34	4.43	1.72	3	3	110	40.3	203	1830
x29	3.74	1.38	3	3	93.0	32.2	209	1550

Design Table 43 Section resistances of cold-formed CHS: Q460 steel



EWCHS	Flexural rigidity	Q460			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
140x6.0	1.19	1	45.1	388	1060
x8.0	1.52	1	58.3	510	1390
x10.0	1.82	1	70.7	628	1710
170x6.0	2.19	2	67.5	475	1290
x8.0	2.81	1	87.8	626	1700
x10.0	3.38	1	107	773	2100
x12.0	3.93	1	125	916	2490
220x6.0	4.85	3	87.8	620	1690
x8.0	6.30	2	150	819	2230
x10.0	7.64	1	184	1010	2760
x12.0	8.93	1	217	1210	3280
270x6.0	9.11	3	134	765	2080
x8.0	11.9	2	230	1010	2750
x10.0	14.5	2	283	1260	3420
x12.0	17.0	1	334	1490	4070
x16.0	21.7	1	432	1960	5340
320x6.0	15.3	4	-	910	2480
x8.0	20.0	3	248	1210	3280
x10.0	24.6	2	402	1500	4070
x12.0	29.0	2	476	1780	4860
x16.0	37.2	1	618	2350	6390
360x6.0	22.0	4	-	1030	2790
x8.0	28.8	3	318	1360	3700
x10.0	35.5	3	393	1690	4600
x12.0	41.8	2	606	2020	5490
x16.0	53.8	1	790	2660	7230
400x8.0	39.7	4	-	1510	4120
x10.0	48.9	3	489	1880	5120
x12.0	58.0	2	757	2250	6120
x16.0	74.8	1	987	2970	8070
x20.0	90.7	1	1160	3510	9550
460x8.0	60.9	4	-	1750	4750
x10.0	75.2	4	-	2170	5910
x12.0	89.0	3	769	2600	7060
x16.0	116	2	1320	3430	9330
x20.0	141	1	1550	4060	11100
500x8.0	78.5	4	-	1900	5170
x10.0	97.0	4	-	2370	6440
x12.0	115	3	916	2830	7690
x16.0	150	2	1570	3740	10170
x20.0	183	1	1840	4430	12100
610x8.0	144	4	-	2330	6330
x10.0	178	4	-	2900	7880
x12.0	212	4	-	3470	9430
x16.0	277	3	1810	4590	12500
x20.0	338	2	2780	5450	14800
710x10.0	284	4	-	3380	9200
x12.0	336	4	-	4040	11000
x16.0	441	3	2480	5360	14600
x20.0	542	2	3810	6370	17300
810x10.0	422	4	-	3860	10500
x12.0	504	4	-	4620	12600
x16.0	662	4	-	6130	16700
x20.0	813	3	3820	7300	19900

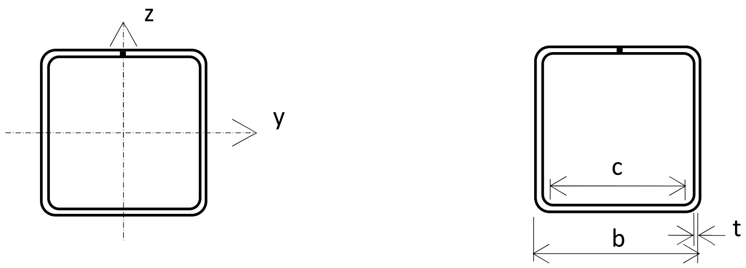
Design Table 44 Section resistances of cold-formed RHS: Q460 steel



EWRHS	Flexural rigidity		Q460					
			Section Classification		Moment resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
120x80x6.0	0.801	0.428	1	1	33.6	25.4	313	879
x8.0	0.948	0.505	1	1	41.1	31.0	402	1120
160x80x6.0	1.67	0.566	1	1	53.2	32.9	429	1080
x8.0	2.01	0.679	1	1	66.1	40.7	556	1380
x10.0	2.27	0.760	1	1	76.6	47.0	676	1660
200x100x6.0	3.44	1.18	1	1	86.5	53.6	545	1380
x8.0	4.26	1.44	1	1	109	67.5	711	1790
x10.0	4.91	1.66	1	1	129	79.4	869	2160
200x150x6.0	4.62	2.98	1	1	111	91.3	545	1630
x8.0	5.80	3.74	1	1	141	116	711	2120
x10.0	6.83	4.37	1	1	169	139	869	2580
250x150x6.0	7.94	3.63	1	1	155	109	690	1720
x8.0	10.1	4.58	1	1	199	140	904	2450
x10.0	11.9	5.40	1	1	238	168	1110	3000
x12.0	12.6	5.75	1	1	260	184	1310	3410
260x180x6.0	9.98	5.69	2	2	183	143	719	1890
x8.0	12.7	7.22	1	1	236	184	943	2720
x10.0	15.1	8.59	1	1	285	222	1160	3330
x12.0	16.2	9.28	1	1	315	246	1370	3810
300x200x6.0	15.2	8.19	3	3	201	163	834	2030
x8.0	19.4	10.5	1	1	313	238	1100	2990
x10.0	23.3	12.5	1	1	379	288	1350	3840
x12.0	25.4	13.7	1	1	422	323	1600	4420
x16.0	30.2	16.3	1	1	519	395	2070	5620
350x250x6.0	25.8	15.5	4	4	-	-	935*	2170
x8.0	33.2	19.9	2	2	452	360	1290	3430
x10.0	40.1	23.9	1	1	552	439	1590	4670
x12.0	44.7	26.9	1	1	627	498	1890	5420
x16.0	54.4	32.6	1	1	782	623	2460	6960
400x200x6.0	30.5	10.6	3	3	303	210	935*	2100
x8.0	39.3	13.5	1	1	485	302	1480	3170
x10.0	47.5	16.3	1	1	594	367	1830	4330
x12.0	52.7	18.2	1	1	669	418	2180	5420
x16.0	63.8	22.1	1	1	832	519	2840	6960
450x250x8.0	60.9	24.8	2	2	661	443	1680	3560
x10.0	74.1	30.0	1	1	807	539	2080	4860
x12.0	83.4	34.0	1	1	920	619	2470	6420
x16.0	103	41.8	1	1	1160	778	3230	8300
500x300x8.0	89.0	41.0	4	4	-	-	1660*	3810
x10.0	109	49.8	2	2	1050	744	2320	5370
x12.0	124	56.9	1	1	1210	857	2760	6960
x16.0	155	71.0	1	1	1540	1090	3620	9630
x20.0	180	82.5	1	1	1750	1240	4250	11200

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 45 Section resistances of cold-formed SHS: Q460 steel

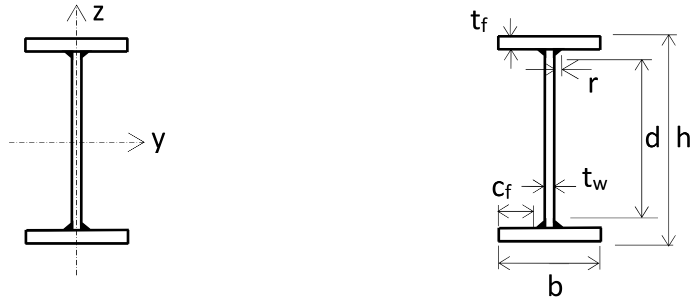


EWSHS	Flexural rigidity	Q460			
		Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
100x100x6.0	0.618	1	30.0	255	879
x8.0	0.732	1	36.7	324	1120
150x150x6.0	2.33	1	73.2	400	1380
x8.0	2.88	1	92.6	518	1790
x10.0	3.34	1	109	628	2160
200x200x6.0	5.82	3	116	545	1880
x8.0	7.35	1	173	711	2450
x10.0	8.72	1	208	869	3000
x12.0	9.26	1	229	1020	3410
220x220x6.0	7.83	4	-	603	1970
x8.0	9.98	1	213	788	2720
x10.0	11.9	1	257	966	3330
x12.0	12.8	1	285	1140	3810
x16.0	14.9	1	344	1450	4820
250x250x6.0	11.7	4	-	690	2070
x8.0	15.0	2	280	904	3120
x10.0	18.0	1	339	1110	3840
x12.0	19.7	1	380	1310	4420
x16.0	23.4	1	466	1680	5620
300x300x6.0	20.6	4	-	834	2180
x8.0	26.7	4	-	1100	3530
x10.0	32.1	2	502	1350	4670
x12.0	35.9	1	569	1600	5420
x16.0	43.7	1	711	2070	6960
350x350x8.0	43.1	4	-	1290	3740
x10.0	52.3	3	594	1590	5510
x12.0	59.2	1	795	1890	6420
x16.0	73.3	1	1000	2460	8300
400x400x8.0	65.1	4	-	1480	3880
x10.0	79.4	4	-	1830	5660
x12.0	90.7	2	1060	2180	7430
x16.0	113	1	1350	2840	9630
x20.0	133	1	1540	3330	11200

Design Tables 46 to 51 for section resistances of welded sections: Q690 steel

- **EWI-sections (EWIS)**
- **EWH-sections (EWHS)**
- **EWCHS**
- **EWRHS**
- **EWSHS**

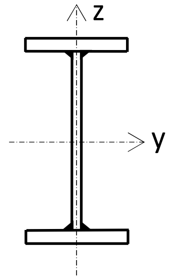
Design Table 46 Section resistances of welded I-sections: Q690 steel (1)

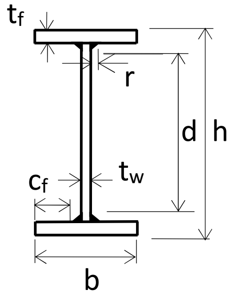


EWIS	Flexural Rigidity EI_y 10^3*kNm^2	Flexural Rigidity EI_z 10^3*kNm^2	Section Classification Bending *y-y*	Section Classification Bending *z-z*	Moment Resistance $M_{y,Rd}$ kNm	Moment Resistance $M_{z,Rd}$ kNm	Shear Resistance $V_{z,Rd}$ kN	Axial Resistance $N_{a,Rd}$ kN
920x450x420	1690	128	1	1	13300	2820	6600	32700
x353	1350	95.8	3	3	9520	1380	6710	26600
920x360x312	1170	49.1	1	1	9320	1390	6830	22900
x282	1008	41.0	3	3	7090	737	6830	20500
x249	918	37.6	3	3	6530	696	4480*	17600
x218	746	27.6	3	3	5370	525	4480*	14800
840x350x246	804	41.0	3	3	6130	737	4480*	17800
x214	651	30.1	3	3	5020	556	4480*	15000
x184	596	27.5	3	3	4600	524	2520*	12500
760x320x220	584	28.7	2	2	5590	906	4480*	16400
x194	483	23.0	3	3	4110	465	4480*	14100
x167	431	20.9	3	3	3720	436	2520*	11700
x147	365	16.7	4	4	-	-	2520*	9800
690x280x198	420	21.4	1	1	4500	747	4080	15200
x173	349	15.4	3	3	3270	356	4140	12900
x151	317	15.4	3	3	3020	356	2520*	10790
x133	269	12.3	3	3	2590	289	2520*	9360
620x330x258	481	41.4	1	1	5530	1230	4550	21500
x186	351	31.5	3	3	3670	618	2520*	14300
x160	286	25.2	3	3	3030	494	2520*	12100
610x260x158	286	15.4	2	1	3350	591	2520*	12000
x138	233	12.3	2	2	2800	476	2520*	10190
x122	199	9.85	3	3	2140	249	2520*	8870
x112	184	9.85	3	3	2020	249	1750*	8000
540x250x148	211	13.7	1	1	2790	545	2390	11500
x128	172	10.9	2	2	2330	439	2390	9840
x113	146	8.76	3	3	1780	230	2430	8570
x104	135	8.76	3	3	1680	230	1750*	7730
x88	109	6.55	4	4	-	-	1750*	5950

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 47 Section resistances of welded I-sections: Q690 steel (2)

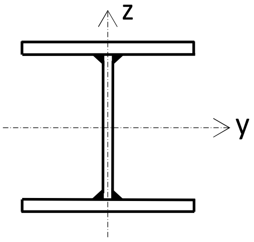


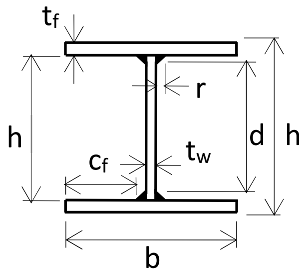


EWIS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2	EI_z 10^3*kNm^2	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
460x220x112	113	7.46	1	1	1770	342	2060	9070
x104	104	7.46	1	1	1650	337	1670	8210
x90	88.4	5.96	3	3	1260	178	1710	7130
x83	85.1	5.96	3	3	1210	178	1070*	6370
x70	68.5	4.47	3	3	980	134	1070*	5160
460x180x91	88.0	4.07	1	1	1410	229	1670	7130
x80	75.2	3.26	1	1	1230	187	1710	6250
x73	71.8	3.26	3	1	1028	184	1070*	5490
x62	58.4	2.44	3	3	835	88.9	1070*	4500
x56	48.9	2.04	3	3	715	74.4	1070*	4000
410x210x85	69.3	5.19	2	2	1220	251	1550	6860
x78	63.2	5.19	2	2	1120	248	1070*	6120
x65	51.0	3.89	3	3	821	122	1070*	4970
x59	44.5	3.23	4	4	-	-	1070*	4160
400x150x49	37.0	1.70	3	2	607	109	601*	3590
x43	30.7	1.18	3	3	504	51.5	601*	3010
355x180x73	43.1	3.26	1	1	890	186	1310	6100
x62	34.9	2.44	3	3	636	88.9	1340	5030
x54	31.1	2.06	3	3	575	79.6	1055	4260
x49	27.3	1.72	3	3	505	66.6	1068	3790
355x170x44	25.8	1.70	3	3	478	65.8	601*	3270
x38	21.4	1.37	4	4	-	-	601*	2660
310x160x59	26.7	2.29	1	1	638	145	886	4910
x45	21.0	1.70	2	2	492	108	601*	3550
x39	16.9	1.43	3	3	371	58.7	669	3110
310x140x54	23.9	1.53	1	1	573	112	886	4470
x45	19.5	1.16	1	1	471	84.9	911	3710
x41	15.9	0.945	2	2	400	71.3	892	3320
310x110x37	14.9	0.462	1	1	364	45.6	940	2920
x33	12.5	0.357	2	2	315	37.2	937	2620
x28	11.2	0.357	2	2	279	35.7	601*	2120
260x170x48	15.4	2.06	3	3	389	79.6	752	4130
x43	13.5	1.70	3	3	340	65.8	765	3660
x33	10.06	1.37	4	4	-	-	559	2600
260x130x33	10.4	0.756	2	2	294	60.3	574	2660
x29	8.84	0.609	3	3	223	30.8	583	2310
x25	6.97	0.462	4	4	-	-	581	1890
210x150x34	7.25	1.18	3	3	227	51.5	454	2890
x29	5.52	0.945	3	3	181	41.4	440	2460
200x110x27	5.00	0.462	1	1	185	43.4	430	2320
180x100x19	2.96	0.273	1	1	121	28.5	261	1570
150x100x18	1.97	0.273	1	1	96.5	28.4	214	1540
130x80x15	1.18	0.126	1	1	67.6	18.4	182	1290

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

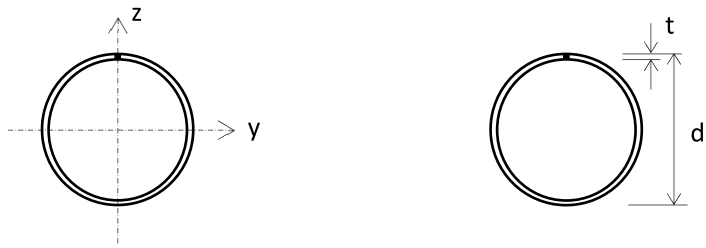
Design Table 48 Section resistances of welded H-sections: Q690 steel





EWHS	Flexural Rigidity		Section Classification		Moment Resistance		Shear Resistance	Axial Resistance
	EI_y	**EI_z**	**Bending**	**Bending**	**$M_{y,Rd}$**	**$M_{z,Rd}$**	**$V_{z,Rd}$**	**$N_{a,Rd}$**
	10^3*kNm^2	**10^3*kNm^2**	*y-y*	*z-z*	**kNm**	**kNm**	**kN**	**kN**
420x480x716	638	291	1	1	10300	5880	6000	59300
x563	519	233	1	1	8580	4730	5340	48000
x532	460	232	1	1	7910	4680	3770	45400
x456	368	194	1	1	6520	3910	3770	38900
x368	279	155	2	2	5190	3180	3140	31900
x300	218	124	3	3	3620	1630	3240	26000
x275	201	123	3	3	3430	1630	2040	23800
360x440x218	159	74.6	3	3	2700	1095	2100	18900
x217	150	74.6	3	3	2620	1095	2040	18800
x172	116	59.6	4	4	-	-	1530	13500
x167	115	59.6	4	4	-	-	1270	12800
370x330x316	189	63.0	1	1	3890	1860	2650	27000
x260	147	55.0	1	1	3160	1600	2120	22500
x213	116	45.2	1	1	2530	1270	2200	18400
x177	93.7	37.6	3	3	1840	696	1780	15300
x152	76.2	32.7	3	3	1540	588	1780	13200
x142	73.9	32.7	3	3	1500	587	1340	12300
x114	56.9	26.1	4	4	-	-	1110	8700
270x310x184	70.4	28.4	1	1	1830	945	1810	15900
x151	56.1	23.6	1	1	1480	780	1470	13100
x121	44.1	20.9	3	3	1061	436	1100	10500
x116	39.9	20.9	3	3	993	436	876	10100
x93	33.0	16.7	4	4	-	-	727	7700
210x230x101	22.5	8.11	1	1	768	381	813	8720
x85	19.9	7.46	1	1	665	333	717	7340
x73	15.9	6.80	3	3	497	194	709	6440
x58	13.0	5.80	4	4	-	-	593	4850
x50	10.12	4.83	4	4	-	-	574	3850
170x170x42	5.92	2.06	3	3	229	79.8	465	3710
x34	4.43	1.72	3	3	182	66.5	335	3010
x29	3.74	1.38	4	4	-	-	344	2420

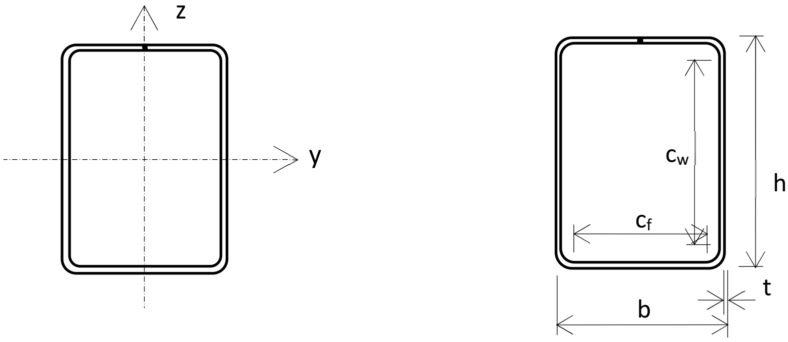
Design Table 49 Section resistances of cold-formed CHS: Q690 steel



EWCHS	Flexural rigidity EI_y 10^3*kNm^2	Section Classification	Moment Resistance $M_{y,Rd}$ kNm	Shear Resistance $V_{z,Rd}$ kN	Axial Resistance $N_{a,Rd}$ kN
140x6.0	1.19	2	74.3	641	1740
x8.0	1.52	2	96.2	841	2290
x10.0	1.82	1	117	1040	2820
170x6.0	2.19	3	84.5	784	2130
x8.0	2.81	2	145	1030	2810
x10.0	3.38	1	177	1270	3470
x12.0	3.93	1	207	1510	4110
220x6.0	4.85	4	-	1020	2780
x8.0	6.30	3	188	1350	3680
x10.0	7.64	2	304	1670	4550
x12.0	8.93	2	358	1990	5410
270x6.0	9.11	4	-	1260	3430
x8.0	11.9	4	-	1670	4540
x10.0	14.5	3	353	2070	5640
x12.0	17.0	2	551	2470	6710
x16.0	21.7	1	712	3240	8810
320x6.0	15.3	4	-	1500	4080
x8.0	20.0	4	-	1990	5410
x10.0	24.6	4	-	2470	6720
x12.0	29.0	3	595	2940	8010
x16.0	37.2	2	1020	3880	10500
x20.0	44.7	1	1220	4710	12800
360x6.0	22.0	4	-	1690	4600
x8.0	28.8	4	-	2240	6100
x10.0	35.5	4	-	2790	7590
x12.0	41.8	3	766	3330	9050
x16.0	53.8	2	1300	4390	11900
X20.0	53.8	2	1570	5340	14500
400x8.0	39.7	4	-	2500	6800
x10.0	48.9	4	-	3110	8450
x12.0	58.0	4	-	3710	10100
x16.0	74.8	3	1230	4900	13300
x20.0	90.7	2	1970	5970	16200
x25.0	109	1	2390	7360	20000
460x8.0	60.9	4	-	2880	7840
x10.0	75.2	4	-	3590	9750
x12.0	89.0	4	-	4280	11650
x16.0	116	3	1660	5660	15400
x20.0	141	2	2630	6910	18800
x25.0	170	2	3220	8540	23200
500x8.0	78.5	4	-	3140	8530
x10.0	97.0	4	-	3900	10600
x12.0	115	4	-	4670	12700
x16.0	150	4	-	6170	16800
x20.0	183	3	2370	7540	20510
x25.0	222	2	3840	9320	25370
x30.0	258	1	4510	11100	30120
610x8.0	144	4	-	3840	10440
x10.0	178	4	-	4780	13010
x12.0	212	4	-	5720	15550
x16.0	277	4	-	7570	20600
x20.0	338	3	3600	9270	25200
x25.0	413	3	4390	11500	31200
x30.0	484	2	6860	13700	37200

EWCHS	Flexural rigidity	Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
710x10.0	284	4	-	5580	15200
x12.0	336	4	-	6670	18200
x16.0	441	4	-	8850	24100
x20.0	542	4	-	10800	29500
x25.0	664	3	6050	13400	36600
x30.0	779	2	9430	16000	43600
x36.0	911	2	11100	19100	51800
810x10.0	422	4	-	6370	17300
x12.0	504	4	-	7630	20800
x16.0	662	4	-	10100	27500
x20.0	813	4	-	12400	33800
x25.0	998	4	-	15400	41900
x30.0	1176	3	9400	18400	50000
x36.0	1380	2	14700	21900	59500

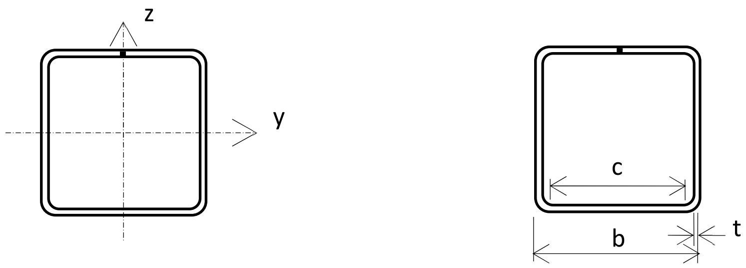
Design Table 50   Section resistances of cold-formed RHS: Q690 steel



EWRHS	Flexural rigidity		Section Classification		Moment resistance		Shear Resistance	Axial Resistance
	EI_y 10³*kNm²	EI_z 10³*kNm²	Bending *y-y*	Bending *z-z*	$M_{y,Rd}$ kNm	$M_{z,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
120x80x6.0	0.801	0.428	1	1	55.4	42.0	516	1450
x8.0	0.948	0.505	1	1	67.7	51.2	663	1840
160x80x6.0	1.67	0.566	1	2	87.7	54.2	708	1780
x8.0	2.01	0.679	1	1	109	67.1	918	2280
x10.0	2.27	0.760	1	1	126	77.5	1120	2740
200x100x6.0	3.44	1.18	1	4	143	-	899	2130
x8.0	4.26	1.44	1	1	180	111	1170	2950
x10.0	4.91	1.66	1	1	213	131	1430	3570
200x150x6.0	4.62	2.98	1	4	183	-	899	2540
x8.0	5.80	3.74	1	1	233	192	1170	3500
x10.0	6.83	4.37	1	1	278	229	1430	4260
250x150x6.0	7.94	3.63	1	4	255	-	1140	2640
x8.0	10.1	4.58	1	4	328	-	1490	3900
x10.0	11.9	5.40	1	1	393	277	1830	4950
x12.0	12.6	5.75	1	1	430	304	2160	5630
260x180x6.0	9.98	5.69	3	4	252	-	1190	2910
x8.0	12.7	7.22	1	4	390	-	1560	4270
x10.0	15.1	8.59	1	2	470	366	1910	5500
x12.0	16.2	9.28	1	1	520	406	2260	6290
300x200x6.0	15.2	8.19	4	4	-	-	1380*	2970
x8.0	19.4	10.5	1	4	516	-	1810	4620
x10.0	23.3	12.5	1	3	626	411	2230	6330
x12.0	25.4	13.7	1	1	697	533	2640	7290
x16.0	30.2	16.3	1	1	856	652	3420	9270
350x250x6.0	25.8	15.5	4	4	-	-	1200*	3120
x8.0	33.2	19.9	4	4	-	-	2130	5140
x10.0	40.1	23.9	1	4	911	-	2630	7130
x12.0	44.7	26.9	1	2	1040	821	3120	8940
x16.0	54.4	32.6	1	1	1290	1030	4050	11500
400x200x6.0	30.5	10.6	4	4	-	-	1200*	3050
x8.0	39.3	13.5	2	4	800	-	2450*	4810
x10.0	47.5	16.3	1	4	980	-	3030	6610
x12.0	52.7	18.2	1	4	1100	-	3590	8600
x16.0	63.8	22.1	1	1	1370	856	4690	11500
450x250x8.0	60.9	24.8	4	4	-	-	2770*	5280
x10.0	74.1	30.0	1	4	1330	-	3430	7420
x12.0	83.4	34.0	1	4	1520	-	4070	9690
x16.0	103	41.8	1	2	1920	1280	5330	13700
500x300x8.0	89.0	41.0	4	4	-	-	2140*	5490
x10.0	109	49.8	3	4	1430	-	3820*	8210
x12.0	124	56.9	1	4	2000	-	4550	10700
x16.0	155	71.0	1	3	2540	1550	5970	15900
x20.0	180	82.5	1	1	2980	2110	7220	19000

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

Design Table 51 Section resistances of cold-formed SHS: Q690 steel



EWSHS	Flexural rigidity	Section Classification	Moment Resistance	Shear Resistance	Axial Resistance
	EI_y 10^3*kNm^2		$M_{y,Rd}$ kNm	$V_{z,Rd}$ kN	$N_{a,Rd}$ kN
100x100x6.0	0.618	1	49.5	421	1450
x8.0	0.732	1	60.6	535	1840
150x150x6.0	2.33	1	121	660	2280
x8.0	2.88	1	153	854	2950
x10.0	3.34	1	181	1036	3570
200x200x6.0	5.82	4	191	899	2810
x8.0	7.35	1	286	1170	4050
x10.0	8.72	1	344	1430	4950
x12.0	9.26	1	377	1680	5630
220x220x6.0	7.83	4	-	994	2900
x8.0	9.98	2	351	1300	4490
x10.0	11.9	1	424	1590	5500
x12.0	12.8	1	469	1870	6290
x16.0	14.9	1	567	2400	7950
250x250x6.0	11.7	4	-	1140	3010
x8.0	15.0	4	461	1490	4860
x10.0	18.0	1	559	1830	6330
x12.0	19.7	1	627	2160	7290
x16.0	23.4	1	769	2780	9270
300x300x6.0	20.6	4	-	1200*	3140
x8.0	26.7	4	-	1810	5190
x10.0	32.1	3	828	2230	7450
x12.0	35.9	1	938	2640	8940
x16.0	43.7	1	1173	3420	11500
350x350x8.0	43.1	4	-	2130	5420
x10.0	52.3	4	980	2630	7930
x12.0	59.2	2	1310	3120	10600
x16.0	73.3	1	1660	4050	13700
400x400x8.0	65.1	4	-	2140*	5580
x10.0	79.4	4	-	3030	8270
x12.0	90.7	4	1750	3590	11600
x16.0	113	1	2220	4690	15900
x20.0	133	1	2610	5650	19000

* indicates the value of shear buckling resistance $V_{bw,Rd} = \frac{\chi_m f_{yw} h_w t}{\sqrt{3}\gamma_{m1}}$, where χ_m is the shear buckling factor.

References

European Steel Material Specifications:

EN 10020, Definition and classification of grades of steel. 2000.

EN 10021, General technical delivery requirements for steel and iron products. 2006.

EN 10025, Hot rolled products of structural steels —
Part 1: General technical delivery conditions. 2004.
Part 2: Technical delivery conditions for non-alloy structural steels. 2019.
Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels. 2019.
Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels. 2019.
Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance. 2019.
Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition. 2019.

BS EN 10029, Hot-rolled steel plates 3 mm thick or above: Tolerances on dimensions and shape. 2010.

EN 10034, Structural steel I and H sections: Tolerances on shape and dimensions. 1993.

EN 10149, Specification for hot-rolled flat products made of high yield strength steels for cold forming —
Part 1: General delivery conditions. 2013.
Part 2: Delivery conditions for thermomechanically rolled steels. 2013.
Part 3. Delivery conditions for normalized or normalized rolled steels. 2013.

EN 10210, Hot finished structural hollow sections of non-alloy and fine grain steels —
Part 1: Technical delivery conditions. 2006.
Part 2: Tolerances, dimensions and sectional properties. 2019.
Part 3: Technical delivery conditions for high strength and weather resistant steels. 2020.

EN 10219, Cold formed welded structural hollow sections of non-alloy and fine grain steels —
Part 1: Technical delivery conditions. 2006.
Part 2: Tolerances, dimensions and sectional properties. 2019.
Part 3: Technical delivery conditions for high strength and weather resistant steels. 2020.

BS 4-1, Structural steel sections —
Part 1: Specification for hot-rolled sections. 2005.

Chinese Steel Material Specifications:

GB/T 700, Carbon structural steels. 2006.

GB/T 709, Dimension, shape, weight and tolerance for hot-rolled steel strip, plate and sheet. 2019.

GB/T 1591, High strength low alloy structural steels. 2018.

GB/T 3274, Hot-rolled plates, sheets and strips of carbon structural steels and high strength low alloy structural steels. 2017.

GB/T 4171, Atmospheric corrosion resisting structural steel. 2008.

GB/T 5313, Steel plates with through-thickness characteristics. 2010.

GB/T 6725, General requirements of cold forming steel sections. 2017.

GB/T 6728, Cold forming hollow sectional steel for general structure. 2017.

GB/T 8162, Seamless steel tubes for structural purposes. 2018.

GB/T 17395, Dimensions, shapes, masses, and tolerances of seamless steel tubes. 2008.

GB/T 19879, Steel plates for building structures. 2015.

Code of Practices for Design of Steel Structures:

BS5950, Structural use of steelwork in building, Part 1: Code of practice for design of hot rolled and welded sections, 2012.

Code of practice for the structural use of steel, 2011. Building Department, Government of Hong Kong SAR.

EN 1993, Eurocode 3: Design of steel structures —

Part 1-1: General rules and rules for buildings. 2005+A1:2014.

Part 1-2: General rules - Structural fire design. 2005.

Part 1-3: General rules - Supplementary rules for cold-formed members and sheeting. 2006.

Part 1-5: Plated structural elements. 2006+A2:2019.

Part 1-8: Design of joints. 2005.

Part 1-12: Additional rules for the extension of EN1993 up to steel grades S700. 2007.

GB 50017, Standard for design of steel structures. 2017. Ministry of Housing and Urban-Rural Development of PRC; General Administration of Quality Supervision Inspection and Quarantine of PRC.

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Chung, K.F., Harmonized member buckling design in Structural Eurocodes. Innovation in Construction, Research Journal 2014, Construction Industry Council, Hong Kong, 2014.

Design of Steel Structures with Worked Examples to EN 1993-1-1 and EN 1993-1-8. F. Wald, K. H. Tan and S. P. Chiew. Research Publishing. 2012.

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Steel Building Design: Design Data. The Steel Construction Institute, Tata Steel and British Constructional Steelwork Association, SCI Publication No. P363, 2013.

Steel Designers' Manual. 7th Edition. Buick Davidson & Graham W. Owens. Steel Construction Institute, Wiley-Blackwell, 2012.

Structural Steelwork, Design to Limit State Theory. 4th Edition. D. Lam, T. C. Ang, and S. P. Chiew. CRC Press, Taylor & Francis Group, 2013.

The Behaviour and Design of Steel Structures to EC3. 4th Edition. N S Trahair, M A Bradford, D A Nethercot, and L Gardner. Taylor & Francis Group, 2008.

Appendices

Appendix A Design procedure for a pinned-pinned column to EN 1993-1

A1 Design of a steel column against axial buckling

1. Determine the buckling length of the steel column for both axes.

2. Calculate N_{cr} and Af_y.

3. Calculate the non-dimensional slenderness, $\bar{\lambda}$ of the steel column.

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i} \times \frac{1}{\lambda_1} \qquad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \frac{L_{cr}}{i} \times \frac{1}{\lambda_1} \sqrt{\frac{A_{eff}}{A}} \qquad \text{for Class 4 cross-sections}$$

where A is the cross-sectional area,

A_{eff} is the effective cross-sectional area of Class 4 sections,

f_y is the yield strength,

$N_{cr} = \dfrac{\pi^2 EI}{L_{cr}^{\;2}}$ which is the critical flexural buckling load/elastic critical force

and

L_{cr} is the buckling length in the buckling plane considered,

$\lambda_1 = \pi \sqrt{\dfrac{E}{f_y}} = 93.9\varepsilon$, where $\varepsilon = \sqrt{\dfrac{235}{f_y}}$

4. Choose a suitable flexural buckling curve for rolled and equivalent welded sections in Table A1, and hence, the imperfection factor, α, is obtained from Table A2.

5. Determine the parameter ϕ.

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

6. Calculate the buckling reduction factor, χ.

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \le 1.0$$

7. Calculate the design buckling resistance, $N_{b,Rd}$.

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

where γ_{M1} is the partial factor for resistance of the steel column to instability.

Table A1: Selection of flexural buckling curves for rolled and equivalent welded cross-section

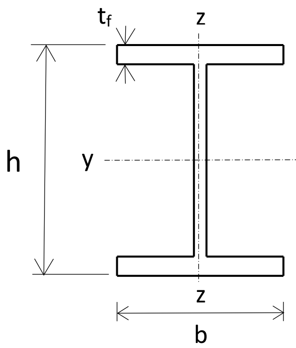
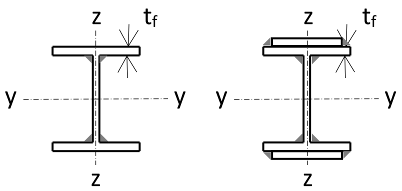
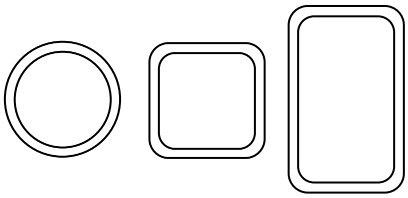
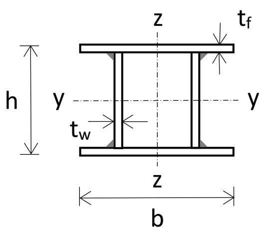
Cross section		Limits		Buckling about axis	Buckling curve		
					S 235 S 275 S 355 S 420	S460	S690
Rolled sections		h/b > 1.2	$t_f \le 40$ mm	y-y z-z	a b	a_0 a_0	- -
			$40 \le t_f \le 100$ mm	y-y z-z	b c	a a	- -
		h/b ≤ 1.2	$t_f \le 100$ mm	y-y z-z	b c	a a	- -
			$t_f > 100$ mm	y-y z-z	d d	c c	- -
Welded I-sections		$t_f \le 40$ mm		y-y z-z	b c	b c	a* a*
		$t_f > 40$ mm		y-y z-z	c d	c d	c d
Hollow sections		hot-finished		any	a	a_0	-
		cold-formed		any	c	c	b*
Welded box sections		generally applicable except as below		any	b	b	b
		thick welds: $a > 0.5\,t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c	c

Table A2: Recommended values for imperfection factor, α, for various flexural buckling curves

Buckling curve	a_0	a, a*	b, b*	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

Table A3: Reduction factor, χ for flexural buckling

$\bar{\lambda}$	Reduction factor, χ				
	Buckling curve				
	a_0	a, a*	b, b*	c	d
0.00	1.000	1.000	1.000	1.000	1.000
0.02	1.000	1.000	1.000	1.000	1.000
0.04	1.000	1.000	1.000	1.000	1.000
0.06	1.000	1.000	1.000	1.000	1.000
0.08	1.000	1.000	1.000	1.000	1.000
0.10	1.000	1.000	1.000	1.000	1.000
0.12	1.000	1.000	1.000	1.000	1.000
0.14	1.000	1.000	1.000	1.000	1.000
0.16	1.000	1.000	1.000	1.000	1.000
0.18	1.000	1.000	1.000	1.000	1.000
0.20	1.000	1.000	1.000	1.000	1.000
0.22	0.997	0.996	0.993	0.990	0.984
0.24	0.995	0.991	0.986	0.980	0.969
0.26	0.992	0.987	0.979	0.969	0.954
0.28	0.989	0.982	0.971	0.959	0.938
0.30	0.986	0.977	0.964	0.949	0.923
0.32	0.983	0.973	0.957	0.939	0.909
0.34	0.980	0.968	0.949	0.929	0.894
0.36	0.977	0.963	0.942	0.918	0.879
0.38	0.973	0.958	0.934	0.908	0.865
0.40	0.970	0.953	0.926	0.897	0.850
0.42	0.967	0.947	0.918	0.887	0.836
0.44	0.963	0.942	0.910	0.876	0.822
0.46	0.959	0.936	0.902	0.865	0.808
0.48	0.955	0.930	0.893	0.854	0.793
0.50	0.951	0.924	0.884	0.843	0.779
0.52	0.947	0.918	0.875	0.832	0.765
0.54	0.943	0.911	0.866	0.820	0.751
0.56	0.938	0.905	0.857	0.809	0.738
0.58	0.933	0.897	0.847	0.797	0.724
0.60	0.928	0.890	0.837	0.785	0.710
0.62	0.922	0.882	0.827	0.773	0.696
0.64	0.916	0.874	0.816	0.761	0.683
0.66	0.910	0.866	0.806	0.749	0.670
0.68	0.903	0.857	0.795	0.737	0.656
0.70	0.896	0.848	0.784	0.725	0.643
0.72	0.889	0.838	0.772	0.712	0.630
0.74	0.881	0.828	0.761	0.700	0.617
0.76	0.872	0.818	0.749	0.687	0.605
0.78	0.863	0.807	0.737	0.675	0.592
0.80	0.853	0.796	0.724	0.662	0.580
0.82	0.843	0.784	0.712	0.650	0.568
0.84	0.832	0.772	0.699	0.637	0.556
0.86	0.821	0.760	0.687	0.625	0.544
0.88	0.809	0.747	0.674	0.612	0.532
0.90	0.796	0.734	0.661	0.600	0.521
0.92	0.783	0.721	0.648	0.588	0.510
0.94	0.769	0.707	0.635	0.575	0.499
0.96	0.755	0.693	0.623	0.563	0.488
0.98	0.740	0.680	0.610	0.552	0.477
1.00	0.725	0.666	0.597	0.540	0.467

$\bar{\lambda}$	Reduction factor, χ				
	Buckling curve				
	a_0	a, a*	b, b*	c	d
1.00	0.725	0.666	0.597	0.540	0.467
1.02	0.710	0.652	0.584	0.528	0.457
1.04	0.695	0.638	0.572	0.517	0.447
1.06	0.679	0.624	0.559	0.506	0.438
1.08	0.664	0.610	0.547	0.495	0.428
1.10	0.648	0.596	0.535	0.484	0.419
1.12	0.633	0.582	0.523	0.474	0.410
1.14	0.618	0.569	0.512	0.463	0.401
1.16	0.603	0.556	0.500	0.453	0.393
1.18	0.588	0.543	0.489	0.443	0.384
1.20	0.573	0.530	0.478	0.434	0.376
1.22	0.559	0.518	0.467	0.424	0.368
1.24	0.545	0.505	0.457	0.415	0.361
1.26	0.531	0.493	0.447	0.406	0.353
1.28	0.518	0.482	0.437	0.397	0.346
1.30	0.505	0.470	0.427	0.389	0.339
1.32	0.493	0.459	0.417	0.380	0.332
1.34	0.481	0.448	0.408	0.372	0.325
1.36	0.469	0.438	0.399	0.364	0.318
1.38	0.457	0.428	0.390	0.357	0.312
1.40	0.446	0.418	0.382	0.349	0.306
1.42	0.435	0.408	0.373	0.342	0.299
1.44	0.425	0.399	0.365	0.335	0.293
1.46	0.415	0.390	0.357	0.328	0.288
1.48	0.405	0.381	0.350	0.321	0.282
1.50	0.395	0.372	0.342	0.315	0.277
1.52	0.386	0.364	0.335	0.308	0.271
1.54	0.377	0.356	0.328	0.302	0.266
1.56	0.369	0.348	0.321	0.296	0.261
1.58	0.360	0.341	0.314	0.290	0.256
1.60	0.352	0.333	0.308	0.284	0.251
1.62	0.344	0.326	0.302	0.279	0.247
1.64	0.337	0.319	0.295	0.273	0.242
1.66	0.329	0.312	0.289	0.268	0.237
1.68	0.322	0.306	0.284	0.263	0.233
1.70	0.315	0.299	0.278	0.258	0.229
1.72	0.308	0.293	0.273	0.253	0.225
1.74	0.302	0.287	0.267	0.248	0.221
1.76	0.295	0.281	0.262	0.243	0.217
1.78	0.289	0.276	0.257	0.239	0.213
1.80	0.283	0.270	0.252	0.235	0.209
1.82	0.277	0.265	0.247	0.230	0.206
1.84	0.272	0.260	0.243	0.226	0.202
1.86	0.266	0.255	0.238	0.222	0.199
1.88	0.261	0.250	0.234	0.218	0.195
1.90	0.256	0.245	0.229	0.214	0.192
1.92	0.251	0.240	0.225	0.210	0.189
1.94	0.246	0.236	0.221	0.207	0.186
1.96	0.241	0.231	0.217	0.203	0.183
1.98	0.237	0.227	0.213	0.200	0.180
2.00	0.232	0.223	0.209	0.196	0.177

Appendix B Design procedures for an unrestrained beam to EN 1993-1

B1 Design of a steel beam against lateral torsional buckling using general design method to Clause 6.3.2.2 of EN 1993-1-1

1. Determine the buckling length of the steel beam.

2. Calculate M_{cr} and $W_{pl,y}f_y$.

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr}^2} \left\{ \left[\frac{I_w}{I_z} + \frac{L_{cr}^2 GI_t}{\pi^2 EI_z} + \left(C_2 z_g - C_3 z_j\right)^2 \right]^{0.5} - \left(C_2 z_g - C_3 z_j\right) \right\}$$

where I_z, I_t, I_w are the section properties,

E is the Young's modulus,

G is the shear modulus, $G = \frac{E}{2(1+\nu)}$,

L_{cr} is the buckling length of the steel beam, $L_{cr} = kL$, and k is the effective length coefficient,

C_1, C_2, C_3 are the factors depending on the shape of the bending moment diagram, end restraint conditions and loading conditions as listed in Table B1.1,

z_g is the vertical distance of the loading position above the shear centre,

z_j is the relative distance to the shear centre. It is simply taken as 0 for uniform doubly symmetric cross-sections.

3. Calculate the non-dimensional slenderness, $\overline{\lambda}_{LT}$ of the steel beam.

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \frac{\lambda_{LT}}{\lambda_1}\sqrt{\beta_w}$$

where $\beta_w = \frac{W_y}{W_{pl,Rd}}$, and

$W_y = W_{pl,y}$ for Class 1 and 2 cross-sections,

$= W_{el,y}$ for Class 3 cross-sections,

$= W_{eff,y}$ for Class 4 cross-sections,

$W_{pl,y}$ is the plastic section modulus for Class 1 and 2 sections,

$W_{el,y}$ is the elastic section modulus for Class 3 sections,

$W_{eff,y}$ is the effective elastic section modulus for Class 4 sections,

f_y is the yield strength.

Table B1.1a Values of factors C_1, C_2 and C_3 corresponding to k factor under different end moment loading

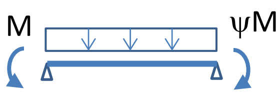
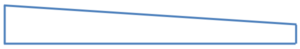
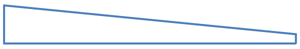
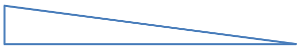
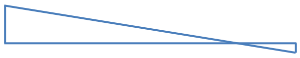
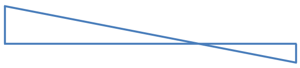
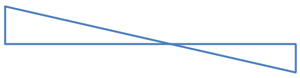
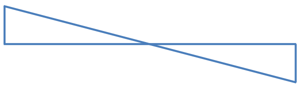
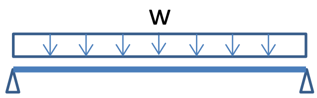
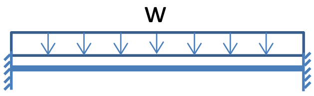
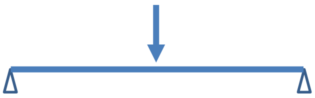
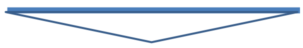
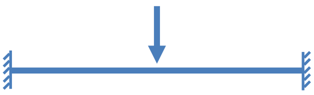
Loading and support conditions	Bending moment Diagram	Value of k	Values of factors C_1	C_2	C_3
	$\psi = +1$ 	1.0	1.000	-	1.000
		0.7	1.000		1.113
		0.5	1.000		1.144
	$\psi = +3/4$ 	1.0	1.141	-	0.998
		0.7	1.270		1.565
		0.5	1.305		2.283
	$\psi = +1/2$ 	1.0	1.323	-	0.992
		0.7	1.473		1.556
		0.5	1.514		2.271
	$\psi = +1/4$ 	1.0	1.563	-	0.977
		0.7	1.739		1.531
		0.5	1.788		2.235
	$\psi = 0$ 	1.0	1.879	-	0.939
		0.7	2.092		1.473
		0.5	2.150		2.150
	$\psi = -1/4$ 	1.0	2.281	-	0.855
		0.7	2.538		1.340
		0.5	2.609		1.957
	$\psi = -1/2$ 	1.0	2.704	-	0.676
		0.7	3.009		1.059
		0.5	3.093		1.546
	$\psi = -3/4$ 	1.0	2.927	-	0.366
		0.7	3.009		0.575
		0.5	3.093		0.837
	$\psi = -1$ 	1.0	2.752	-	0.000
		0.7	3.063		0.000
		0.5	3.149		0.000

Table B1.1b Values of factors C1, C2 and C3 corresponding to k factor under transverse loading cases

Loading and support conditions	Bending moment Diagram	Value of k	Values of factors		
			C_1	C_2	C_3
		1.0 0.5	1.132 0.972	0.459 0.304	0.525 0.980
		1.0 0.5	1.285 0.712	1.562 0.652	0.753 1.070
		1.0 0.5	1.365 1.070	0.553 0.432	1.730 3.050
		1.0 0.5	1.565 0.938	1.267 0.715	2.640 4.800
		1.0 0.5	1.046 1.010	0.430 0.410	1.120 1.890

4. Choose a suitable lateral buckling curve for rolled sections or equivalent welded sections from Table B1.2, and hence, the imperfection factor, α_{LT}, can be obtained from Table B1.3.

Table B1.2. Selection of buckling curves for rolled sections and equivalent welded sections

Cross-section	Limits	Buckling curve	
Rolled I-sections	$h / b \leq 2$ $h / b > 2$	a b	
Welded sections		S235 ~ S460	S690
	$h / b \leq 2$ $h / b > 2$	c d	b b

Table B1.3. Recommended imperfection factor values for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor, α_{LT}	0.21	0.34	0.49	0.76

5. Determine the parameter Φ_{LT}.

$$\Phi_{LT} = 0.5 \left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - 0.2\right) + \overline{\lambda}_{LT}{}^{2}\right]$$

6. Calculate the reduction factor, χ_{LT}.

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{2} - \overline{\lambda}_{LT}^{2}}} \quad \text{but} \quad \chi_{LT} \leq 1.0$$

7. Alternatively, reduction factor, χ_{LT} can be obtained from Table B1.4.

8. Calculate the buckling moment resistance, $M_{b,Rd}$.

$$M_{b,Rd} = \frac{\chi_{LT} W_{y} f_{y}}{\gamma_{M1}}$$

where γ_{M1} is the partial factor for resistance of the beam to instability.

Table B1.4. Reduction factor, χ_{LT} for lateral torsional buckling

$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT} Buckling curve a	b	c	d
0.00	1.000	1.000	1.000	1.000
0.02	1.000	1.000	1.000	1.000
0.04	1.000	1.000	1.000	1.000
0.06	1.000	1.000	1.000	1.000
0.08	1.000	1.000	1.000	1.000
0.10	1.000	1.000	1.000	1.000
0.12	1.000	1.000	1.000	1.000
0.14	1.000	1.000	1.000	1.000
0.16	1.000	1.000	1.000	1.000
0.18	1.000	1.000	1.000	1.000
0.20	1.000	1.000	1.000	1.000
0.22	0.996	0.993	0.990	0.984
0.24	0.991	0.986	0.980	0.969
0.26	0.987	0.979	0.969	0.954
0.28	0.982	0.971	0.959	0.938
0.30	0.977	0.964	0.949	0.923
0.32	0.973	0.957	0.939	0.909
0.34	0.968	0.949	0.929	0.894
0.36	0.963	0.942	0.918	0.879
0.38	0.958	0.934	0.908	0.865
0.40	0.953	0.926	0.897	0.850
0.42	0.947	0.918	0.887	0.836
0.44	0.942	0.910	0.876	0.822
0.46	0.936	0.902	0.865	0.808
0.48	0.930	0.893	0.854	0.793
0.50	0.924	0.884	0.843	0.779
0.52	0.918	0.875	0.832	0.765
0.54	0.911	0.866	0.820	0.751
0.56	0.905	0.857	0.809	0.738
0.58	0.897	0.847	0.797	0.724
0.60	0.890	0.837	0.785	0.710
0.62	0.882	0.827	0.773	0.696
0.64	0.874	0.816	0.761	0.683
0.66	0.866	0.806	0.749	0.670
0.68	0.857	0.795	0.737	0.656
0.70	0.848	0.784	0.725	0.643
0.72	0.838	0.772	0.712	0.630
0.74	0.828	0.761	0.700	0.617
0.76	0.818	0.749	0.687	0.605
0.78	0.807	0.737	0.675	0.592
0.80	0.796	0.724	0.662	0.580
0.82	0.784	0.712	0.650	0.568
0.84	0.772	0.699	0.637	0.556
0.86	0.760	0.687	0.625	0.544
0.88	0.747	0.674	0.612	0.532
0.90	0.734	0.661	0.600	0.521
0.92	0.721	0.648	0.588	0.510
0.94	0.707	0.635	0.575	0.499
0.96	0.693	0.623	0.563	0.488
0.98	0.680	0.610	0.552	0.477
1.00	0.666	0.597	0.540	0.467

$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT} Buckling curve a	b	c	d
1.00	0.666	0.597	0.540	0.467
1.02	0.652	0.584	0.528	0.457
1.04	0.638	0.572	0.517	0.447
1.06	0.624	0.559	0.506	0.438
1.08	0.610	0.547	0.495	0.428
1.10	0.596	0.535	0.484	0.419
1.12	0.582	0.523	0.474	0.410
1.14	0.569	0.512	0.463	0.401
1.16	0.556	0.500	0.453	0.393
1.18	0.543	0.489	0.443	0.384
1.20	0.530	0.478	0.434	0.376
1.22	0.518	0.467	0.424	0.368
1.24	0.505	0.457	0.415	0.361
1.26	0.493	0.447	0.406	0.353
1.28	0.482	0.437	0.397	0.346
1.30	0.470	0.427	0.389	0.339
1.32	0.459	0.417	0.380	0.332
1.34	0.448	0.408	0.372	0.325
1.36	0.438	0.399	0.364	0.318
1.38	0.428	0.390	0.357	0.312
1.40	0.418	0.382	0.349	0.306
1.42	0.408	0.373	0.342	0.299
1.44	0.399	0.365	0.335	0.293
1.46	0.390	0.357	0.328	0.288
1.48	0.381	0.350	0.321	0.282
1.50	0.372	0.342	0.315	0.277
1.52	0.364	0.335	0.308	0.271
1.54	0.356	0.328	0.302	0.266
1.56	0.348	0.321	0.296	0.261
1.58	0.341	0.314	0.290	0.256
1.60	0.333	0.308	0.284	0.251
1.62	0.326	0.302	0.279	0.247
1.64	0.319	0.295	0.273	0.242
1.66	0.312	0.289	0.268	0.237
1.68	0.306	0.284	0.263	0.233
1.70	0.299	0.278	0.258	0.229
1.72	0.293	0.273	0.253	0.225
1.74	0.287	0.267	0.248	0.221
1.76	0.281	0.262	0.243	0.217
1.78	0.276	0.257	0.239	0.213
1.80	0.270	0.252	0.235	0.209
1.82	0.265	0.247	0.230	0.206
1.84	0.260	0.243	0.226	0.202
1.86	0.255	0.238	0.222	0.199
1.88	0.250	0.234	0.218	0.195
1.90	0.245	0.229	0.214	0.192
1.92	0.240	0.225	0.210	0.189
1.94	0.236	0.221	0.207	0.186
1.96	0.231	0.217	0.203	0.183
1.98	0.227	0.213	0.200	0.180
2.00	0.223	0.209	0.196	0.177

Appendix B Design procedures for an unrestrained beam to EN 1993-1

B2 Design of a steel beam against lateral torsional buckling using alternative design method to Clause 6.3.2.3 of EN 1993-1-1

1. Determine the buckling length of the steel beam.

2. Calculate M_{cr} and $W_{pl,y}f_y$.

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr}^2}\left\{\left[\frac{I_w}{I_z} + \frac{L_{cr}^2 GI_t}{\pi^2 EI_z} + \left(C_2 z_g - C_3 z_j\right)2\right]^{0.5} - \left(C_2 z_g - C_3 z_j\right)\right\}$$

where I_z, I_t, I_w are the section properties,

E is the Young's modulus,

G is the shear modulus, $G = \dfrac{E}{2(1+\nu)}$,

L_{cr} is the buckling length of the steel beam, $L_{cr} = kL$, and k is the effective length coefficient,

C_1, C_2, C_3 are the factors depending on the shape of the bending moment diagram, end restraint conditions and loading conditions as listed in Table B1.1,

z_g is the vertical distance of the loading position above the shear centre,

z_j is the relative distance to the shear centre. It is simply taken as 0 for uniform doubly symmetric cross-sections.

3. Calculate the non-dimensional slenderness, $\bar{\lambda}_{LT}$ of the steel beam.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \frac{\lambda_{LT}}{\lambda_1}\sqrt{\beta_w}$$

where $\beta_w = \dfrac{W_y}{W_{pl,Rd}}$,

$W_y = W_{pl,y}$ for Class 1 and 2 cross-sections,

$= W_{el,y}$ for Class 3 cross-sections,

$= W_{eff,y}$ for Class 4 cross-sections,

$W_{pl,y}$ is the plastic section modulus for Class 1 and 2 sections,

$W_{el,y}$ is the elastic section modulus for Class 3 sections,

$W_{eff,y}$ is the effective elastic section modulus for Class 4 sections,

f_y is the yield strength.

4. Choose a suitable lateral buckling curve for rolled sections or equivalent welded sections from Table B2.1, and hence, the imperfection factor, α_{LT} can be obtained from Table B2.2.

Table B2.1. Selection of buckling curves for rolled sections and equivalent welded sections

Cross-section	Limits	Buckling curve	
Rolled I-sections	h / b ≤ 2 h / b > 2	b c	
Welded I-sections		S235 ~ S460	S690
	h / b ≤ 2 h / b > 2	c d	b b

Table B2.2. Recommended imperfection factor values for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor, α_{LT}	0.21	0.34	0.49	0.76

5. Determine the parameter Φ_{LT}.

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - \lambda_{LT,0}\right) + \beta\lambda_{LT}^2\right]$$

For rolled sections or equivalent welded sections:

$\bar{\lambda}_{LT,0}$ = 0.4 (maximum value)

β = 0.75 (minimum value)

6. Calculate the reduction factor, χ_{LT}.

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\ 2} - \beta\bar{\lambda}_{LT}^{\ 2}}} \quad \text{but} \quad \chi_{LT} \le 1.0 \text{ and } \chi_{LT} \le \frac{1}{\bar{\lambda}_{LT}^{\ 2}}$$

Reduction factor, $\bar{\lambda}_{LT}$ can also be obtained from Tables B2.4.

7. Calculate the modified reduction factor, $\chi_{LT,mod}$

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \quad \text{but} \quad \chi_{LT,mod} \le 1.0 \text{ and } \chi_{LT,mod} \le \frac{1}{\bar{\lambda}_{LT}^{\,2}}$$

where f is the correction factor for the moment distribution

f $= 1 - 0.5(1 - k_c)\left[1 - 2.0(\bar{\lambda}_{LT} - 0.8)^2\right] \le 1.0$

k_c is a correction factor according to Table B2.3.

Table B2.3. Correction factors k_c

Moment distribution	k_c
 $\psi = 1$  $-1 \le \psi \le 1$	1.0 $\dfrac{1}{1.33 - 0.33\psi}$
  	0.94 0.90 0.91
  	0.86 0.77 0.82

8. Calculate the buckling moment resistance, $M_{b,Rd}$

$$M_{b,Rd} = \chi_{LT,mod}\, W_y \frac{f_y}{\gamma_{M1}}$$

where γ_{M1} is the partial factor for resistance of the steel beam to instability.

Table B2.4. Reduction factor, χ_{LT} for lateral torsional buckling

$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT}		
	Buckling curve		
	b	c	d
0.00	x	1.000	1.000
0.02	1.000	1.000	1.000
0.04	1.000	1.000	1.000
0.06	1.000	1.000	1.000
0.08	1.000	1.000	1.000
0.10	1.000	1.000	1.000
0.12	1.000	1.000	1.000
0.14	1.000	1.000	1.000
0.16	1.000	1.000	1.000
0.18	1.000	1.000	1.000
0.20	1.000	1.000	1.000
0.22	1.000	1.000	1.000
0.24	1.000	1.000	1.000
0.26	1.000	1.000	1.000
0.28	1.000	1.000	1.000
0.30	1.000	1.000	1.000
0.32	1.000	1.000	1.000
0.34	1.000	1.000	1.000
0.36	1.000	1.000	1.000
0.38	1.000	1.000	1.000
0.40	1.000	1.000	1.000
0.42	0.992	0.989	0.983
0.44	0.984	0.978	0.966
0.46	0.976	0.966	0.949
0.48	0.968	0.955	0.932
0.50	0.960	0.944	0.916
0.52	0.952	0.932	0.900
0.54	0.943	0.921	0.883
0.56	0.935	0.909	0.867
0.58	0.926	0.898	0.852
0.60	0.917	0.886	0.836
0.62	0.908	0.874	0.820
0.64	0.899	0.862	0.805
0.66	0.889	0.850	0.790
0.68	0.880	0.838	0.775
0.70	0.870	0.826	0.760
0.72	0.860	0.813	0.745
0.74	0.849	0.801	0.730
0.76	0.839	0.789	0.716
0.78	0.828	0.776	0.702
0.80	0.817	0.764	0.688
0.82	0.806	0.751	0.674
0.84	0.795	0.739	0.660
0.86	0.783	0.726	0.647
0.88	0.772	0.713	0.634
0.90	0.760	0.701	0.621
0.92	0.748	0.688	0.608
0.94	0.736	0.676	0.596
0.96	0.724	0.664	0.584
0.98	0.712	0.651	0.572
1.00	0.700	0.639	0.560

$\bar{\lambda}_{LT}$	Reduction factor, χ_{LT}		
	Buckling curve		
	b	c	d
1.00	0.700	0.639	0.560
1.02	0.687	0.627	0.548
1.04	0.675	0.615	0.537
1.06	0.663	0.603	0.526
1.08	0.651	0.592	0.515
1.10	0.639	0.580	0.505
1.12	0.626	0.569	0.494
1.14	0.614	0.557	0.484
1.16	0.603	0.546	0.474
1.18	0.591	0.536	0.465
1.20	0.579	0.525	0.455
1.22	0.568	0.514	0.446
1.24	0.556	0.504	0.437
1.26	0.545	0.494	0.428
1.28	0.534	0.484	0.420
1.30	0.524	0.475	0.412
1.32	0.513	0.465	0.403
1.34	0.503	0.456	0.395
1.36	0.493	0.447	0.388
1.38	0.483	0.438	0.380
1.40	0.473	0.429	0.373
1.42	0.463	0.421	0.366
1.44	0.454	0.413	0.359
1.46	0.445	0.405	0.352
1.48	0.436	0.397	0.345
1.50	0.427	0.389	0.339
1.52	0.419	0.382	0.332
1.54	0.410	0.374	0.326
1.56	0.402	0.367	0.320
1.58	0.394	0.360	0.314
1.60	0.387	0.353	0.309
1.62	0.379	0.347	0.303
1.64	0.372	0.340	0.298
1.66	0.363	0.334	0.292
1.68	0.354	0.328	0.287
1.70	0.346	0.322	0.282
1.72	0.338	0.316	0.277
1.74	0.330	0.310	0.272
1.76	0.323	0.305	0.268
1.78	0.316	0.299	0.263
1.80	0.309	0.294	0.259
1.82	0.302	0.289	0.254
1.84	0.295	0.284	0.250
1.86	0.289	0.279	0.246
1.88	0.283	0.274	0.242
1.90	0.277	0.269	0.238
1.92	0.271	0.265	0.234
1.94	0.266	0.260	0.230
1.96	0.260	0.256	0.227
1.98	0.255	0.252	0.223
2.00	0.250	0.247	0.219

Appendix B Design procedures for an unrestrained beam to EN 1993-1

B3 Design of a steel beam against lateral torsional buckling for rolled or equivalent welded I, H or channel sections using the design method given in the Steel Designer's Manual

1. Determine the buckling length of the steel beam.

2. Calculate the non-dimensional slenderness $\bar{\lambda}_{LT}$ of the steel beam.

$\bar{\lambda}_{LT} = \dfrac{1}{\sqrt{C_1}} UV\bar{\lambda}_z\sqrt{\beta_w}$ for rolled I-, H- and channel sections

where C_1 is a factor that depends on the shape of bending moment diagram as listed in Table B3.1,

U is a section property (given in section property tables, which may conservatively be taken as 0.9),

V is a parameter related to slenderness, and for symmetric rolled sections where the loads are not destabilising, may be conservatively taken as 1.0,

or as $V = \dfrac{1}{\sqrt[4]{1+\dfrac{1}{20}\left(\dfrac{\lambda_z}{h/t_f}\right)^2}}$,

where $\lambda_z = \dfrac{L_{cr}}{i_z}$ in which L_{cr} is the buckling length in the buckling plane considered.

$\bar{\lambda}_z$ $= \dfrac{\lambda_z}{\lambda_1}$,

L is the distance between points of lateral restraints,

$\lambda_1 = \pi\sqrt{\dfrac{E}{f_y}} = 93.9\varepsilon$, where $\varepsilon = \sqrt{\dfrac{235}{f_y}}$,

$\beta_w = \dfrac{W_y}{W_{pl,Rd}}$, and

$W_y = W_{pl,y}$ for Class 1 and 2 cross-sections,

$= W_{el,y}$ for Class 3 cross-sections,

$= W_{eff,y}$ for Class 4 cross-sections,

$W_{pl,y}$ is the plastic section modulus of Class 1 and 2 sections,

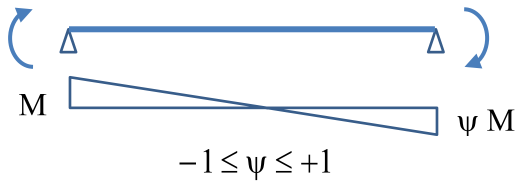
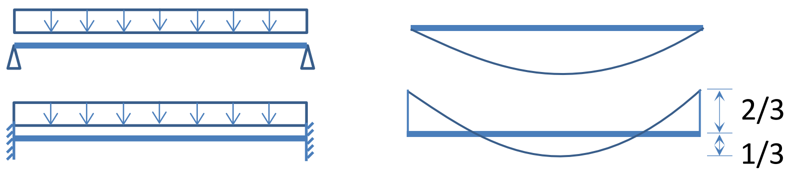
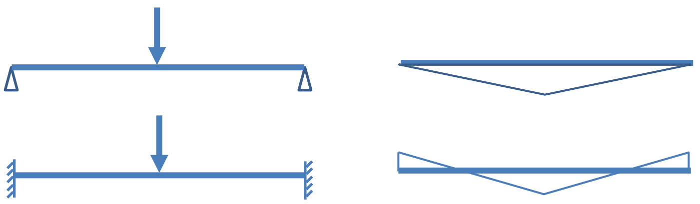
$W_{el,y}$ is the elastic section modulus of Class 3 sections,

$W_{eff,y}$ is the effective elastic section modulus of Class 4 sections,

f_y is the yield strength.

It is conservative to assume that the product $UV = 0.9$ and that $\beta_w = 1.0$

Table B3.1. Values of C_1 for various moment conditions (load is not destabilising)

End Moment Loading	ψ	$\frac{1}{\sqrt{C_1}}$	C_1
	+1.00	1.00	1.00
	+0.75	0.92	1.17
	+0.50	0.86	1.36
	+0.25	0.80	1.56
	0.00	0.75	1.77
	-0.25	0.71	2.00
	-0.50	0.67	2.24
	-0.75	0.63	2.49
	-1.00	0.60	2.76
Intermediate transverse loading 		0.94	1.13
		0.62	2.60
		0.86	1.35
		0.77	1.69

3. Choose a suitable lateral buckling curve for rolled sections or equivalent welded sections from Table B3.2, and hence, the imperfection factor, α_{LT}, is obtained from Table B3.3.

Table B3.2. Selection of buckling curves for rolled sections and equivalent welded sections

Cross-section	Limits	Buckling curve	
Rolled I-sections	$h/b \le 2$	b	
	$2 < h/b \le 3.1$	c	
	$h/b > 3.1$	d	
Welded sections		S235 ~ S460	S690
	$h/b \le 2$	c	b
	$h/b > 2$	d	b

Table B3.3. Recommended values for imperfection factor for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor, α_{LT}	0.21	0.34	0.49	0.76

4. Determine the parameter Φ_{LT}.

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}\right) + \beta\overline{\lambda}_{LT}^{2}\right]$$

For rolled sections,

$\overline{\lambda}_{LT,0}$ = 0.4 (maximum value)

β = 0.75 (minimum value)

For welded sections,

$\overline{\lambda}_{LT,0}$ = 0.2 (maximum value)

β = 1.0 (minimum value)

5. Calculate the reduction factor, χ_{LT}.

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{2} - \beta\overline{\lambda}_{LT}^{2}}} \quad \text{but} \quad \chi_{LT} \leq 1.0 \text{ and } \chi_{LT} \leq \frac{1}{\overline{\lambda}_{LT}^{2}}$$

6. Calculate the buckling moment resistance, $M_{b,Rd}$.

$$M_{b,Rd} = \frac{\chi_{LT}\ W_y f_y}{\gamma_{M1}}$$

where γ_{M1} is the partial factor for resistance of the beam to instability.

Appendix C Design procedures for a column member under combined axial compression and bending to EN 1993-1

C1 Interaction of combined axial compression and bending to Clause 6.3.3 using the design method given in the U.K. National Annex

1. Members subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \text{ , and}$$

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\dfrac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

where N_{Ed} is the design value of the compression force,
$M_{y,Ed}$ is the design value of the maximum moment about the y-y axis,
$M_{z,Ed}$ is the design value of the maximum moment about the z-z axis,
$\Delta M_{y,Ed}$ is the moment due to the shift of the centroidal axis about the major axis for Class 4 sections,
$\Delta M_{z,Ed}$ is the moment due to the shift of the centroidal axis about the minor axis for Class 4 sections,
N_{Rk} is the design resistance of the cross-section for uniform compression.
$M_{y,Rk}$ is the design moment resistance of the cross-section about the y-y axis.
$M_{z,Rk}$ is the design moment resistance of the cross-section about the z-z axis.
$k_{yy}, k_{yz}, k_{zy}, k_{zz}$ are the interaction factors to be calculated by Method A and B as illustrated in Annexes A and B of EN 1993-1-1: 2005.

2. Method B is recommended by SCI-P362 as a simplified approach for manual calculations. Use of either method is permitted by the U. K. National Annex.

3. Calculate interaction factors, k_{ij} by Method A

Table C.1 Interaction factors for combined axial compression and bending

Interaction factors	Design assumptions: Elastic cross-sectional properties Class 3, class 4	Design assumptions: Plastic cross-sectional properties Class 1, class 2
k_{yy}	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{yz}}0.6\sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my}C_{mLT}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my}C_{mLT}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\dfrac{1}{C_{zy}}0.6\sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{zz}}$
Auxiliary terms:		
$\mu_y=\dfrac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-\chi_y\dfrac{N_{Ed}}{N_{cr,y}}}$ $\mu_z=\dfrac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-\chi_z\dfrac{N_{Ed}}{N_{cr,z}}}$ $w_y=\dfrac{W_{pl,y}}{W_{el,y}}\le 1.5$ $w_z=\dfrac{W_{pl,z}}{W_{el,z}}\le 1.5$ $n_{pl}=\dfrac{N_{Ed}}{N_{Rk}/\gamma_{M1}}$ C_{my} see Table C.2 $a_{LT}=1-\dfrac{I_T}{I_y}\ge 0$	$C_{yy}=1+(w_y-1)\left[\left(2-\dfrac{1.6}{w_y}C_{my}^2\bar{\lambda}_{max}-\dfrac{1.6}{w_y}C_{my}^2\bar{\lambda}_{max}^2\right)n_{pl}-b_{LT}\right]\ge\dfrac{W_{el,y}}{W_{pl,y}}$ with $b_{LT}=0.5a_{LT}\bar{\lambda}_0^2\dfrac{M_{y,Ed}}{\chi_{LT}M_{pl,y,Rd}}\dfrac{M_{z,Ed}}{M_{pl,z,Rd}}$ $C_{yz}=1+(w_z-1)\left[\left(2-14\dfrac{C_{mz}^2\bar{\lambda}_{max}^2}{w_z^5}\right)n_{pl}-c_{LT}\right]\ge 0.6\sqrt{\dfrac{w_z}{w_y}}\dfrac{W_{el,z}}{W_{pl,z}}$ with $c_{LT}=10\ a_{LT}\dfrac{\bar{\lambda}_0^2}{5+\bar{\lambda}_z^4}\dfrac{M_{y,Ed}}{C_{my}\ \chi_{LT}\ M_{pl,y,Rd}}$ $C_{zy}=1+(w_y-1)\left[\left(2-14\dfrac{C_{my}^2\bar{\lambda}_{max}^2}{w_y^5}\right)n_{pl}-d_{LT}\right]\ge 0.6\sqrt{\dfrac{w_y}{w_z}}\dfrac{W_{el,y}}{W_{pl,y}}$ with $d_{LT}=2a_{LT}\dfrac{\bar{\lambda}_0}{0.1+\bar{\lambda}_z^4}\dfrac{M_{y,Ed}}{C_{my}\chi_{LT}M_{pl,y,Rd}}\dfrac{M_{z,Ed}}{C_{mz}M_{pl,z,Rd}}$ $C_{zz}=1+(w_z-1)\left(2-\dfrac{1.6}{w_z}C_{mz}^2\bar{\lambda}_{max}-\dfrac{1.6}{w_z}C_{mz}^2\bar{\lambda}_{max}^2-e_{LT}\right)n_{pl}\ge\dfrac{W_{el,z}}{W_{pl,z}}$ with $e_{LT}=1.7a_{LT}\dfrac{\bar{\lambda}_0}{0.1+\bar{\lambda}_z^4}\dfrac{M_{y,Ed}}{C_{my}\chi_{LT}M_{pl,y,Rd}}$	

Table C.1 (continued)

$$\bar{\lambda}_{max} = \max\begin{cases}\bar{\lambda}_y \\ \bar{\lambda}_z\end{cases}$$

$\bar{\lambda}_0$ = non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, i.e. $\psi_y = 1.0$ in Table C.2

$\bar{\lambda}_{LT}$ = non-dimensional slenderness for lateral-torsional buckling

If $\bar{\lambda}_0 \leq 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0}$$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = 1.0$$

If $\bar{\lambda}_0 > 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0} + \left(1 - C_{my,0}\right)\frac{\sqrt{\varepsilon_y}\,a_{LT}}{1+\sqrt{\varepsilon_y}\,a_{LT}}$$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = C_{my}^2 \frac{a_{LT}}{\sqrt{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}} \geq 1$$

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A}{W_{el,y}}$ for class 1,2 and 3 cross-sections

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A_{eff}}{W_{eff,y}}$ for class 4 cross-sections

$N_{cr,y}$ = elastic flexural buckling force about the y-y axis

$N_{cr,z}$ = elastic flexural buckling force about the z-z axis

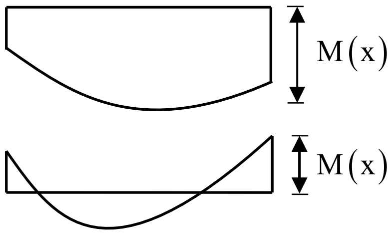
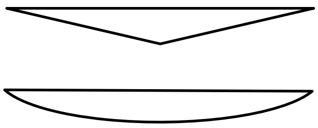
$N_{cr,T}$ = elastic torsional buckling force

I_T = St. Venant torsional constant

I_y = second moment of area about y-y axis

Table C2 Equivalent uniform moment factors, $C_{mi,0}$

<table>
<tr><td>Moment diagram</td><td>$C_{mi,0}$</td></tr>
<tr><td>

$-1 \le \psi \le 1$</td><td>$$C_{mi,0} = 0.79 + 0.21\psi_i + 0.36(\psi_i - 0.33)\frac{N_{Ed}}{N_{cr,i}}$$</td></tr>
<tr><td>
</td><td>$$C_{mi,0} = 1 + \left(\frac{\pi^2 EI_i|\delta_x|}{L^2|M_{i,Ed}(x)|} - 1\right)\frac{N_{Ed}}{N_{cr,i}}$$
$M_{i,Ed}(x)$ is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$
$|\delta_x|$ is the maximum member displacement along the member</td></tr>
<tr><td></td><td>$$C_{mi,0} = 1 - 0.18\frac{N_{Ed}}{N_{cr,i}}$$
$$C_{mi,0} = 1 + 0.03\frac{N_{Ed}}{N_{cr,i}}$$</td></tr>
</table>

4. Calculate interaction factors, k_{ij} by Method B

Table C.3 Interaction factors for combined axial compression and bending

Interaction factors	Type of sections	Design assumptions: Elastic cross-sectional properties Class 3, class 4	Design assumptions: Plastic cross-sectional properties Class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0.6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0.6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0.2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0.6\,k_{zz}$
k_{zy}	I-sections RHS-sections	$0.8\,k_{yy}$	$0.6\,k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0.6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0.6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0.6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+1.4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0.2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0.8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$, the coefficient k_{zy} may be $k_{zy}=0$.			

Table C.4 Interaction factors k_{ij} for members susceptible to torsional deformations

Interaction factors	Design assumptions	
	Elastic cross-sectional properties Class 3, class 4	Plastic cross-sectional properties Class 1, class 2
k_{yy}	k_{yy} from Table C.3	k_{yy} from Table C.3
k_{yz}	k_{yz} from Table C.3	k_{yz} from Table C.3
k_{zy}	$\left[1-\frac{0.05\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\leq\left[1-\frac{0.05}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$\left[1-\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq\left[1-\frac{0.1}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ for $\bar{\lambda}_z<0.4$: $k_{zy}=0.6+\bar{\lambda}_z\leq 1-\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$
k_{zz}	k_{zz} from Table C.3	k_{zz} from Table C.3

Table C.5 Equivalent uniform moment factors, C_m in Tables C.3 and C.4

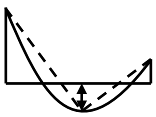
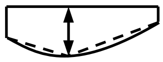
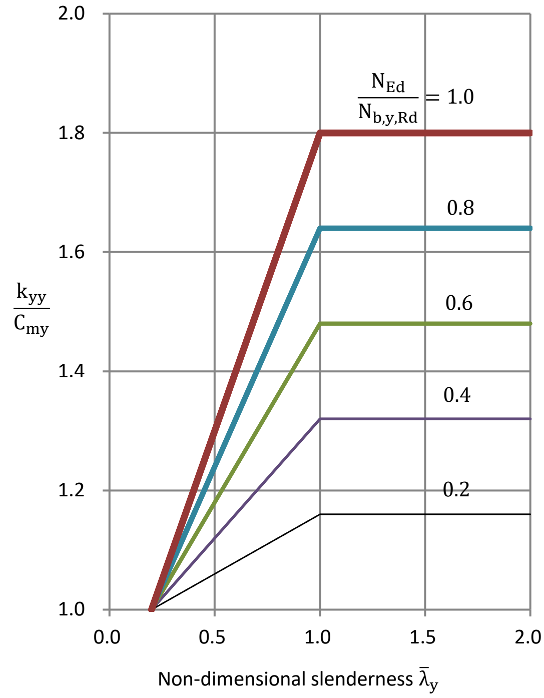
<table>
<tr><th rowspan="2">Moment diagram</th><th rowspan="2" colspan="2">Range</th><th colspan="2">C_{my} and C_{mz} and C_{mLT}</th></tr>
<tr><th>Uniform loading</th><th>Concentrated load</th></tr>
<tr><td>M_1  ψM_1</td><td colspan="2">$-1 \le \psi \le 1$</td><td colspan="2">$0.6 + 0.4\psi \ge 0.4$</td></tr>
<tr><td rowspan="3">M_h  ψM_h
$\alpha_h = M_s / M_h$</td><td>$0 \le \alpha_s \le 1$</td><td>$-1 \le \psi \le 1$</td><td>$0.2 + 0.8\alpha_s \ge 0.4$</td><td>$0.2 + 0.8\alpha_s \ge 0.4$</td></tr>
<tr><td rowspan="2">$-1 \le \alpha_s \le 0$</td><td>$0 \le \psi \le 1$</td><td>$0.1 - 0.8\alpha_s \ge 0.4$</td><td>$-0.8\alpha_s \ge 0.4$</td></tr>
<tr><td>$-1 \le \psi \le 0$</td><td>$0.1(1-\psi) - 0.8\alpha_s \ge 0.4$</td><td>$0.2(-\psi) - 0.8\alpha_s \ge 0.4$</td></tr>
<tr><td rowspan="3">M_h  ψM_h
M_s
$\alpha_h = M_h / M_s$</td><td>$0 \le \alpha_h \le 1$</td><td>$-1 \le \psi \le 1$</td><td>$0.95 + 0.05\alpha_h$</td><td>$0.90 + 0.10\alpha_h$</td></tr>
<tr><td rowspan="2">$-1 \le \alpha_h \le 0$</td><td>$0 \le \psi \le 1$</td><td>$0.95 + 0.05\alpha_h$</td><td>$0.90 + 0.10\alpha_h$</td></tr>
<tr><td>$-1 \le \psi \le 0$</td><td>$0.95 + 0.05\alpha_h(1+2\psi)$</td><td>$0.90 + 0.10\alpha_h(1+2\psi)$</td></tr>
<tr><td colspan="5">For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0.9$ or $C_{mz} = 0.9$ respectively.</td></tr>
<tr><td colspan="5">C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:
moment factor / Bending axis / Points braced in direction
C_{my} / y-y / z-z
C_{mz} / z-z / y-y
C_{mLT} / y-y / y-y</td></tr>
</table>

Table C.6 Interaction factors for combined axial compression and bending

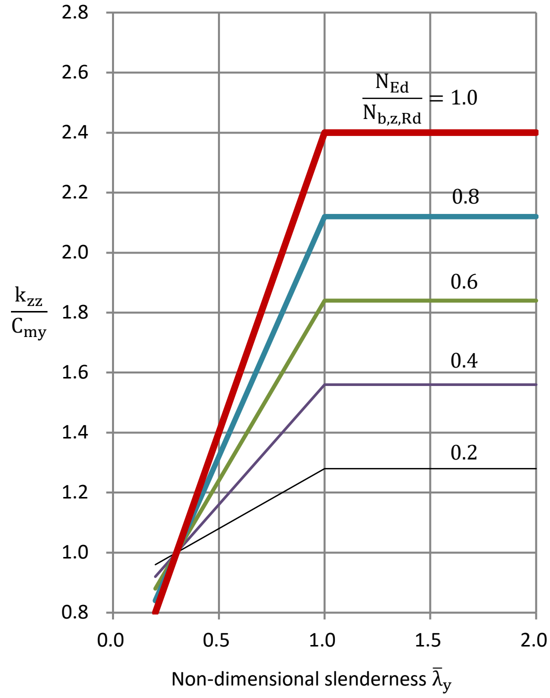
Interaction Factors	Criteria	Section	Design Assumptions		C Factor
			Class 1 and 2 cross-sections	Class 3 cross-sections	
k_{yy}	-	All	Figure C.1	Figure C.2	C_{my}
k_{yz}	-	All	$0.6\ k_{zz}$	k_{zz}	-
k_{zz}	Member not susceptible to torsional deformation	RHS sections	Figure C.1	Figure C.2	C_{mz}
	Member susceptible to torsional deformation	I sections	Figure C.1	Figure C.2	C_{mz}
k_{zy}	Member not susceptible to torsional deformation	All	0.6	0.8	-
	Member susceptible to torsional deformation	All	Figure C.1	Figure C.2	C_{mLT}

(1) C -Factors may be obtained from Table C.5.

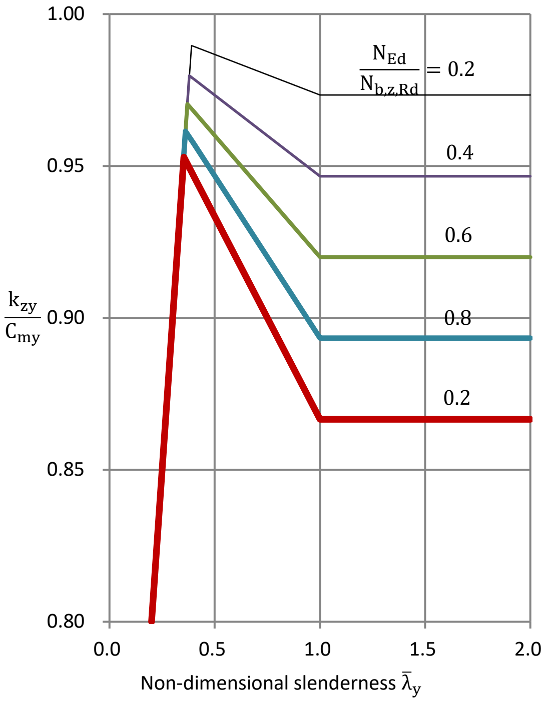
(2) In Figure C.1 and Figure C.2, k_{zy} is based on the conservative assumption that $C_{mLT} = 1.0$.



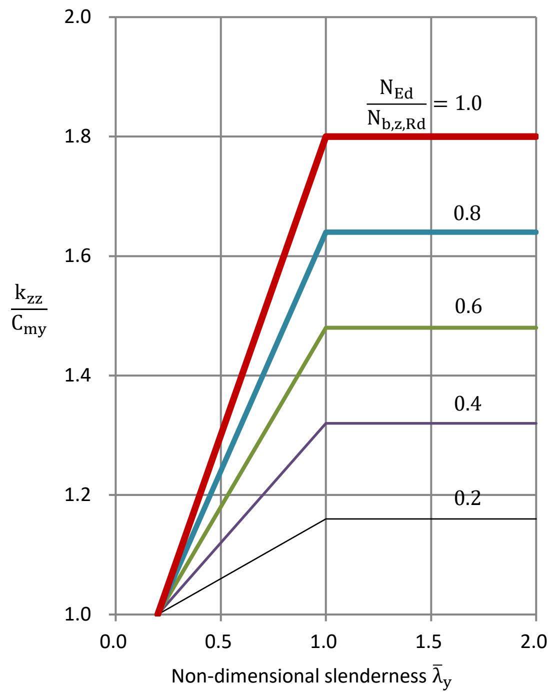
a) Interaction factor k_{yy}



b) Interaction factor k_{zz}

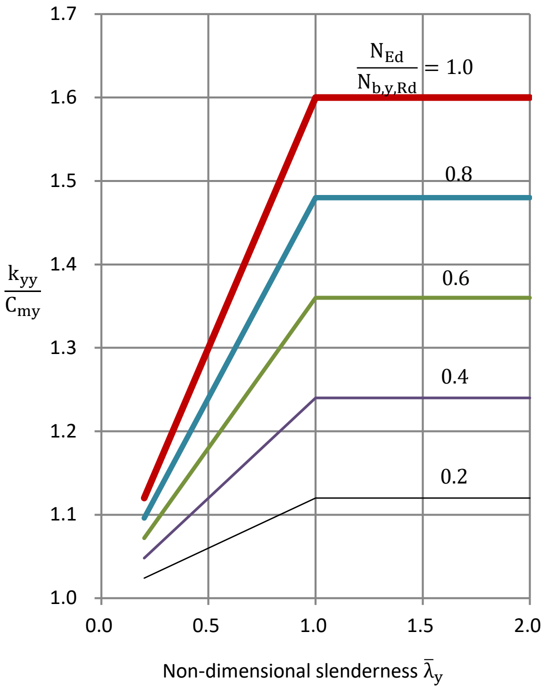


c) Interaction factor k_{zy}

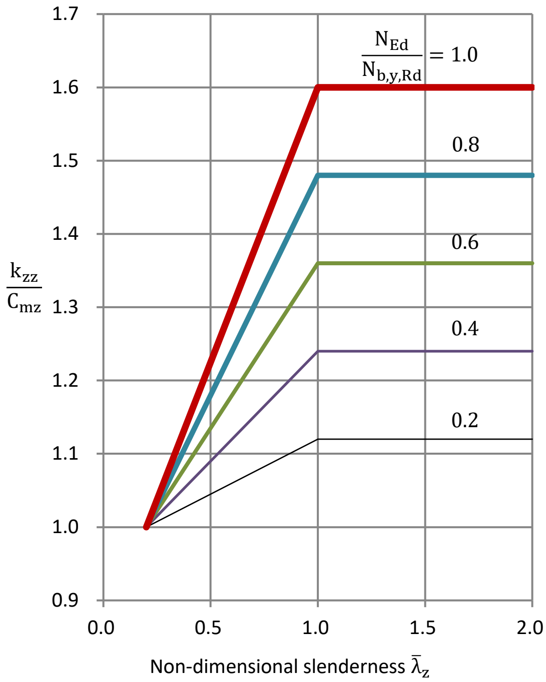


d) Interaction factor k_{zz} for RH Sections

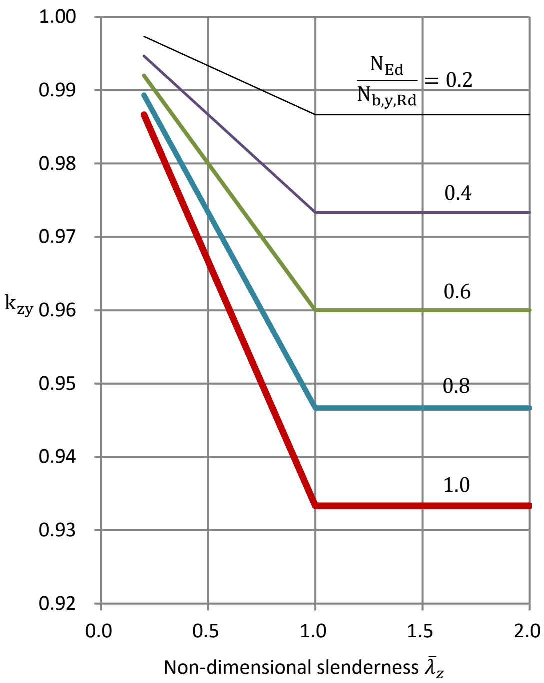
Figure C.1. Interaction factor k_{ij} for Class 1 and 2 sections



a) Interaction factor k_{yy}



b) Interaction factor k_{zz}



c) Interaction factor k_{zy} for I sections

Figure C.2. Interaction factor k_{ij} for Class 3 sections

Appendix D

Worked Examples to EN 1993-1

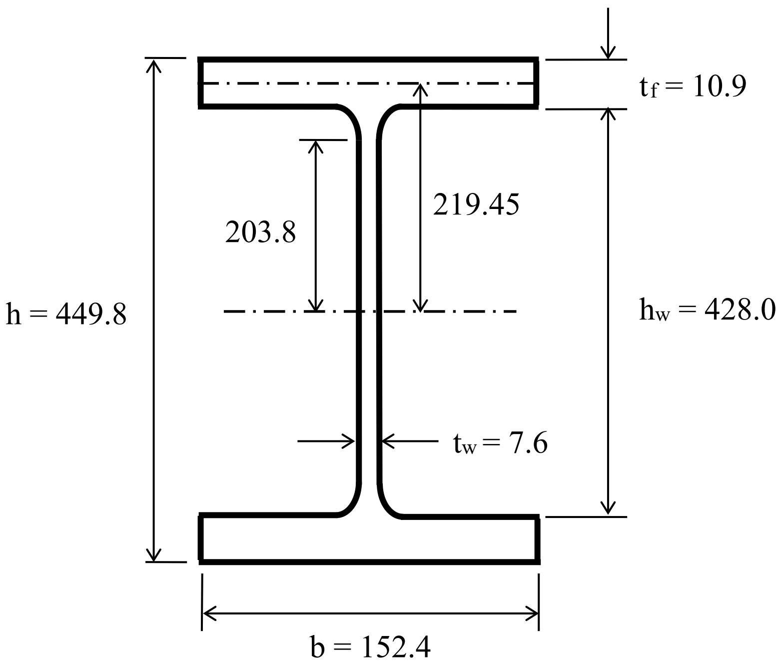
Part I Section analysis and section resistance

Worked Example I-1 Determination of section resistances

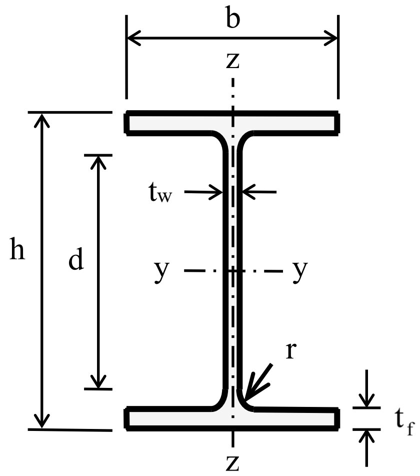
Question

Determine the section resistance of a steel beam as shown:

457 × 152 × 52 kg/m I-section S355



Solution



Section properties of 457 × 152 × 52 kg/m I-section:

h = 449.8 mm		b = 152.4 mm	
d = 407.6 mm			
t_w = 7.6 mm		t_f = 10.9 mm	
r = 10.2 mm			

Calculate cross-sectional area, A

$$A = A_w + 2A_f$$
$$= (h - 2t_f)t_w + 2\,b\,t_f$$
$$= (449.8 - 2\times 10.9)\times 7.6 + 2\times 152.4\times 10.9$$
$$= 3253\ \text{mm}^2 + 3322\ \text{mm}^2$$
$$= 6575\ \text{mm}^2 \qquad \text{(c.f. } 66.6\ \text{cm}^2 \text{ or } 6660\ \text{mm}^2 \text{ from tabulated data)}$$

$$A_{fillet} = 4\times 10.2^2 \times \left(1 - \frac{\pi}{4}\right) = 89.3\ \text{mm}^2$$

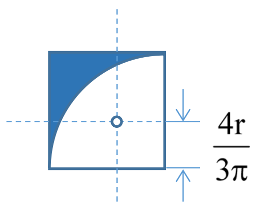
$$A_g = A + A_{fillet}$$
$$= 6575 + 89.3 = 6664.3\ \text{mm}^2$$

In general, fillets are neglected in most design.

Calculate second moment of area, I

$$I_y = 2\times\left(\frac{152.4\times 10.9^3}{12} + 152.4\times 10.9\times 219.45^2\right) + \frac{7.6\times 428.0^3}{12}$$
$$= 2\times\left(16.45\times 10^3 + 80.0\times 10^6\right) + 49.66\times 10^6$$
$$= 160.03\times 10^6 + 49.66\times 10^6$$
$$= 209.69\times 10^6\ \text{mm}^4 \quad \text{or} \quad 20969\ \text{cm}^4$$

$$I_{fillet} = 4\times\left(\frac{10.2^4}{12} + 10.2^2\times\left(214 - \frac{10.2}{2}\right)^2 - \left(\frac{\pi}{16} - \frac{4}{9\pi}\right)\times 10.2^4 - \frac{\pi\times 10.2^2}{4}\right.$$
$$\left.\times\left(214 - 10.2 + \frac{4\times 10.2}{3\pi}\right)^2\right)$$
$$= 4\left[902 + 4540\times 10^3 - 594 - 3540\times 10^3\right]$$
$$= 4.00\times 10^6\ \text{mm}^4$$



$$I_g = I_y + I_{fillet} = 213.69\times 10^6\ \text{mm}^4$$

<u>Calculate the elastic section modulus, W_{el}</u>

$$W_{el,y} = \frac{I_y}{h/2} = \frac{213.69\times10^6}{449.8/2}$$
$$= 950.2\times10^3\ \text{mm}^3 = \quad 950.2\ \text{cm}^3$$

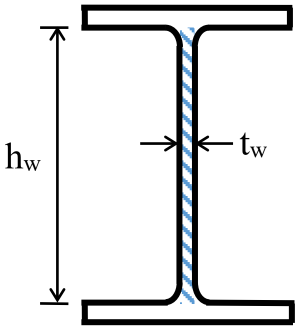
$$W_{el,y,w} = \frac{49.66\times10^6}{449.8/2}$$
$$= 220.8\times10^3\ \text{mm}^3 = \quad 220.8\ \text{cm}^3$$

$$W_{el,y,f} = \frac{160.03\times10^6}{449.8/2}$$
$$= 711.6\times10^3\ \text{mm}^3 = \quad 711.6\ \text{cm}^3$$

$$\frac{W_{el,y,w}}{W_{el,y}} = \frac{220.8}{950.2} = \quad 0.232$$

$$\frac{W_{el,y,f}}{W_{el,y}} = \frac{711.6}{950.2} = \quad 0.749$$

Calculate the plastic section modulus, W_{pl}



$$W_{pl,y0} = b\ t_f\left(h_w + t_f\right) + h_w t_w \frac{h_w}{4}$$

$$= 152.4 \times 10.9 \times \left(428.0 + 10.9\right) + \frac{428.0^2 \times 7.6}{4}$$

$$= 729.1 \times 10^3 + 348.0 \times 10^3$$

$$= 1077.1 \times 10^3\, mm^3 \text{ or } 1077\ cm^3$$

$$W_{pl,y,fillet} = 4\left[r^2\left(\frac{h_w}{2} - \frac{r}{2}\right) - \frac{\pi r^2}{4}\left(\frac{h_w}{2} - r + \frac{4r}{3\pi}\right)^2\right]$$

$$= 4 \times \left[21.7 \times 10^3 - 17.0 \times 10^3\right]$$

$$= 18.8 \times 10^3\, mm^3$$

$$W_{pl,y} = W_{pl,y0} + W_{pl,fillet}$$

$$= 1077.1 \times 10^3 + 18.8 \times 10^3\ mm^3$$

$$= 1095.9 \times 10^3\ mm^3 \text{ or } 1096\ cm^3$$

(c.f. 1100 cm^2 from tabulated data)

$$\frac{W_{pl,y,w}}{W_{pl,y}} = \frac{348.0}{1096} = 0.318$$

$$\frac{W_{pl,y,f}}{W_{pl,y}} = \frac{729.1}{1096} = 0.665$$

The shape factor of I-section $= \frac{W_{pl,y}}{W_{el,y}} = \frac{1096}{950.2} = 1.15$ (included fillets)

$= \frac{W_{pl,y}}{W_{el,y}} = \frac{1077}{932} = 1.16$ (neglected fillets)

$$W_{pl,z} = 2 \times \frac{b^2 t_f}{4} + \frac{h_w t_f^{\ 2}}{4}$$

$$= 2 \times 152.4^2 \times \frac{10.9}{4} + 428 \times \frac{7.6^2}{4}$$

$$= 126.6 \times 10^3 + 6.18 \times 10^3$$

$$= 132.8\, cm^3$$

(c.f. 133 cm^2 from tabulated data)

Typical section properties in an I-section

Elements	**Area** A		**Second moment of area** I	
	(cm^2)	*ratio*	*(cm^4)*	*ratio*
Flanges Web Fillet	33.22 32.53 0.89	0.498 0.488 0.014	16003 4966 400	0.750 0.232 0.018
Total	66.64	1	21369	1

Elements	**Elastic modulus** W_{el}		**Plastic modulus** W_{pl}	
	(cm^3)	*ratio*	*(cm^3)*	*ratio*
Flanges Web Fillet	711.6 220.8 17.8	0.749 0.232 0.019	729.1 348.0 18.8	0.665 0.318 0.017
Total	950.2	1	1095.9	1

Perform section classification

Since t_f = 10.9 mm and t_w = 7.6 mm, i.e. the nominal material thickness is less than 16 mm, the nominal value of yield strength f_y for grade S355 steel is 355 N/mm^2.

$$f_y = 355 \ \ N/mm^2$$

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$$ [Cl. 5.5]

Web – subject to bending: [Table 5.2]

$$c_w = h - 2t_f - 2r = 407.6 \text{ mm}$$

$$\Rightarrow \quad c_w / t_w = 407.6 / 7.6 = 53.6$$

Limit for Class 1 web $= 72\varepsilon = 58.32 \quad \geq \quad 53.6$

$\Rightarrow$ The web is Class 1.

Flange under compression: [Table 5.2]

$$c_f = (b - t_w - 2r)/2 = 62.2 \text{ mm}$$

$$\Rightarrow \quad c_f / t_f = 62.2 / 10.9 = 5.71$$

Limit for Class 1 flange $= 9\varepsilon = 7.3 \geq 5.71$

$\Rightarrow$ The flanges are Class 1.

The overall cross-section classification is Class 1 subject to bending.

Summary

Hence, the design resistance of the cross-section for uniform compression, $N_{c,Rd}$ is

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{6660 \times 355 \times 10^{-3}}{1.0}$$ [Cl. 6.2.4 (2)]

$$= 2364 \text{ kN}$$

The design resistance for bending about y-y axis, $M_{c,y,Rd}$ is

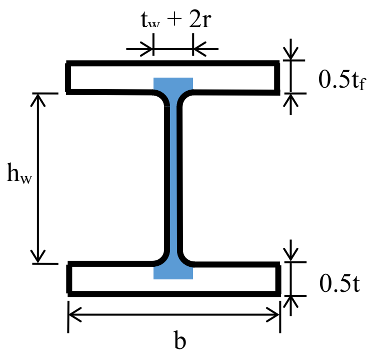
$$M_{c,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{1100 \times 10^3 \times 355 \times 10^{-6}}{1.0}$$ [Cl. 6.2.5 (2)]

$$= 390.5 \text{ kNm}$$

The design resistance for bending about z-z axis, $M_{c,z,Rd}$ is

$$M_{c,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{133 \times 10^3 \times 355 \times 10^{-6}}{1.0}$$

$$= 47.2 \text{ kNm}$$



The design shear resistance, $V_{c,Rd}$ is [Cl. 6.2.6 (2)]

$$V_{pl,Rd} = \frac{A_v \left(f_y / \sqrt{3}\right)}{\gamma_{M0}}, \text{ where } \gamma_{M0} = 1.0$$

where

$$A_v = A - 2\,b\,t_f + (t_w + 2r)\,t_f \text{ but not less than } \eta h_w t_w$$

$$A_v = 6660 - 2 \times 152.4 \times 10.9 + (7.6 + 2 \times 10.2) \times 10.9 = 3642.9 \text{ mm}^2$$

and $A_v > \eta h_w t_w = 1.0 \times (449.8 - 2 \times 10.9) \times 7.6 = 3252.8 \text{ mm}^2$

$$V_{pl,Rd} = \frac{3642.9 \times \left(355 / \sqrt{3}\right)}{1.00} \times 10^{-3} = 746.6 \text{ kN}$$

Consider the axial resistance,
Web – subject to compression: [Table 5.2]

$$c_w = h - 2t_f - 2r = 407.6 \text{ mm}$$
$$\Rightarrow \quad c_w / t_w = 407.6 / 7.6 = 53.6$$

Limit for Class 3 web $= 42\varepsilon = 38.83 < 53.6$

$\Rightarrow$ The web is Class 4.

Axial resistance contributed by the flange and fillets:

$$\mathrm{N}_{f,\mathrm{Rd}} = (6{,}664 - 407.6 \times 7.6) \times 355 \times 10^{-3} = 1{,}266 \text{ kN}$$

Effective width of web:

$$\bar{\lambda}_{\mathrm{p,w}} = \frac{c_w / t_w}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{53.6}{28.4 \times 0.81 \times \sqrt{4}} = 1.165$$

$$\bar{\lambda}_0 > 0.5 + \sqrt{0.085 - 0.055\psi} = 0.673 < \bar{\lambda}_{\mathrm{p,w}}$$

$$\mathrm{d}_{\mathrm{w,eff}} = \frac{\bar{\lambda}_{\mathrm{p,w}} - 0.055(3+\psi)}{\bar{\lambda}_{\mathrm{p,w}}{}^2} = \frac{1.165 - 0.22}{1.165^2} = 0.70$$

Effective area of the web:

$$\mathrm{A}_{w,\mathrm{eff}} = 0.7 \times 407.6 \times 7.6 = 2{,}168 \text{ mm}^2$$

Effective area of the web:

$$\mathrm{A}_{\mathrm{eff}} = 6{,}664 - 407.6 \times 7.6 + 2{,}168 = 5{,}735 \text{ mm}^2$$

Hence, the design resistance of the cross-section for uniform compression, $N_{c,Rd}$ is

$$N_{c,Rd} = \frac{A_{eff} \times f_y}{\gamma_{M0}} = \frac{5735 \times 355 \times 10^{-3}}{1.0} = 2{,}036 \text{ kN}$$ [Cl. 6.2.4 (2)]

Part I Section analysis and section resistance

Worked Example I-2

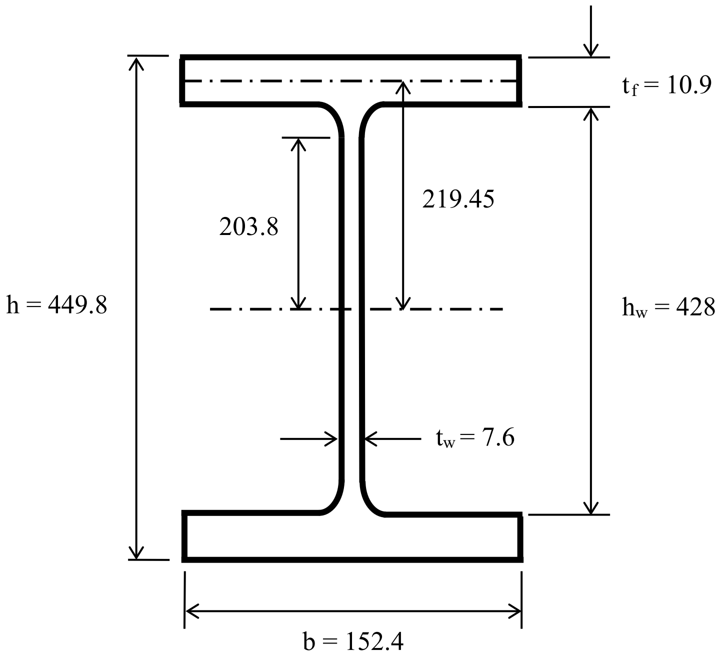
Cross section resistance under combined bending and shear force

Question

Determine the design moment resistance of a steel beam under high shear with the following details:

457 × 152 × 52 kg/m I-section S355

Shear force ratio, $V_{Ed} / V_{pl,Rd} = 0.8$



Solution

Resistance against bending and shear force

For 457 x 152 x 52 kg/m I-section S355

$$M_{y,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w}\right] \times f_y}{\gamma_{M0}} \quad \text{but } M_{y,V,Rd} \le M_{y,c,Rd} \qquad \text{[Cl. 6.2.8]}$$

where

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

$$W_{pl,w} = \frac{A_w^2}{4t_w} = \frac{h_w^2 t_w}{4} = \frac{428^2 \times 7.6}{4} = 348.0 \times 10^3\,\text{mm}^3$$

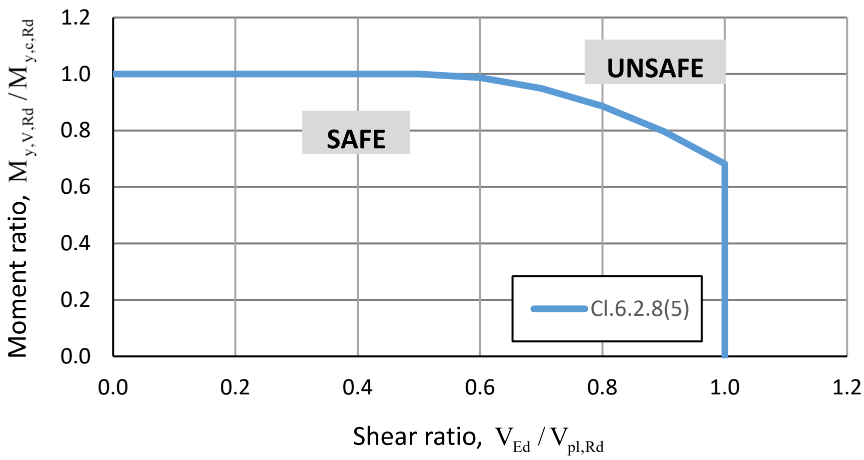
$$W_{pl,y} = 1100 \times 10^3\,\text{mm}^3 \qquad W_{pl,w} / W_{pl,y} = 0.316$$

$$M_{y,c,Rd} = 355 \times 1100 \times 10^3 \times 10^{-6} = 390.5\ \text{kNm}$$

Moment resistance contributed by the top and the bottom flanges:

$$M_{pl,f} = 390.5 \times (1 - 0.316) = 267.1\ \text{kNm}$$

$V_{Ed} / V_{pl,Rd}$	ρ	$M_{y,V,Rd}$ (kNm)	$M_{y,V,Rd} / M_{y,c,Rd}$
0.5	0.00	390.5	1.000
0.6	0.04	385.6	0.987
0.7	0.16	370.7	0.949
0.8	0.36	346.0	0.886
0.9	0.64	311.4	0.798
1.0	1.00	267.0	0.684



For a high shear load at $V_{Ed} / V_{pl,Rd} = 0.8$

$$M_{y,V,Rd} = 346.0\ \text{kNm}$$

Part I Section analysis and section resistance
Worked Example I-3
Cross section resistance under combined bending and axial force

Question

Determine the design moment resistance of a steel beam under combined bending and axial force with the following details:

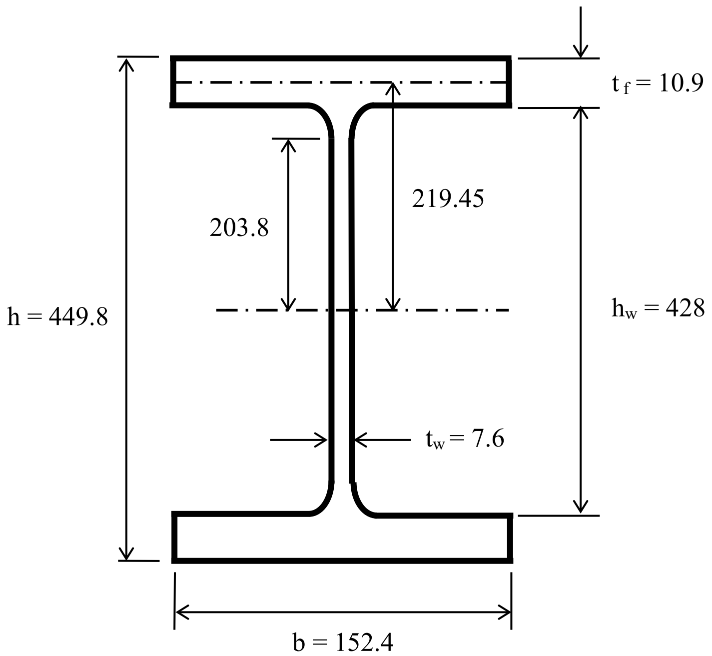
457 × 152 × 52 kg/m I-section S355

Axial compression force ratio, $N_{Ed} / N_{pl,Rd} = 0.8$

a) Determine the design plastic resistance for bending about the y-y axis reduced due to the axial force N_{Ed}.

b) Determine the design plastic resistance for bending about the z-z axis reduced due to the axial force N_{Ed}.

c) Plot the failure criterion of the cross section under an interaction of bi-axial bending and axial force.



Solution

Resistance under combined bending and axial force

a) For 457 × 152 × 52 kg/m I-section S355 subjected to combined major axis bending and axial force

$$N_{c,Rd} = 2{,}334 \text{ kN}$$

If $N_{Ed} \leq 0.25 N_{c,Rd} = 583.5 \text{ kN}$, and [Cl. 6.2.9]

$$N_{Ed} \leq \frac{0.5 h_w t_w f_y}{\gamma_{M0}} = \frac{0.5 \times 428 \times 7.6 \times 355}{1} \times 10^{-3} = 577.4 \text{ kN},$$

Then allowance needs not be made for the effect of axial force on the plastic resistance moment.

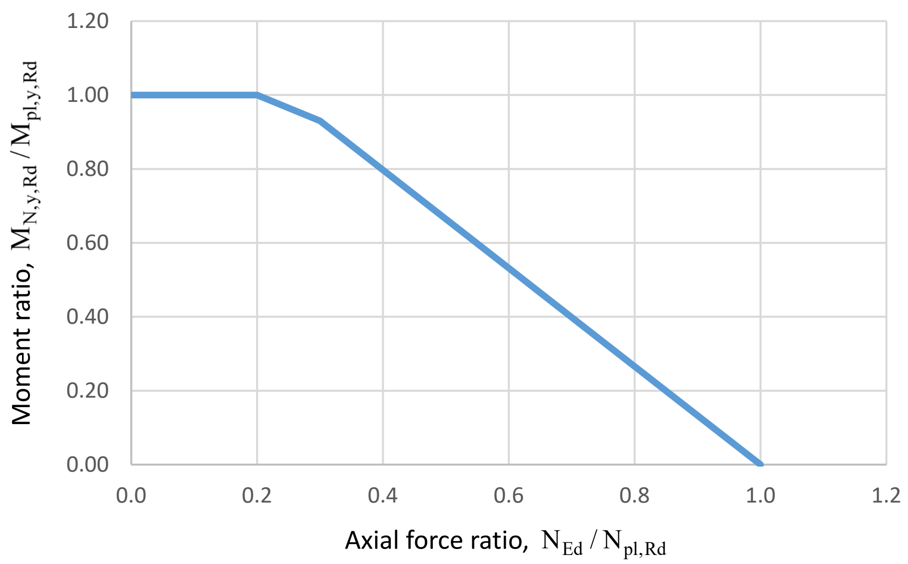
Otherwise, the design plastic resistance for bending about y-y axis reduce due to the axial force is:

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5a}, \text{ but } M_{N,y,Rd} \leq M_{pl,y,Rd}$$

where

$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

$$a = \frac{(A - 2bt_f)}{A} \qquad \text{but } a \leq 0.5$$



N_{Ed}	$N_{pl,Rd}$	n	a	$M_{N,y,Rd} / M_{pl,y,Rd}$
0.0	2,334	0.0	0.49	1.00
233.4	2,334	0.1	0.49	1.00
466.8	2,334	0.2	0.49	1.00
700.2	2,334	0.3	0.49	0.93
933.6	2,334	0.4	0.49	0.80
1167.0	2,334	0.5	0.49	0.66
1400.4	2,334	0.6	0.49	0.53
1633.8	2,334	0.7	0.49	0.40
1867.2	2,334	0.8	0.49	0.27
2100.6	2,334	0.9	0.49	0.13
2,334.0	2,334	1.0	0.49	0.00

For a high axial load at $N_{Ed} / N_{pl,Rd} = 0.8$

$$M_{N,y,Rd} = 0.27 \times 390.5 = 105.4 \text{ kNm}$$

b) For 457 x 152 x 52 kg/m I-section S355 subjected to combined minor axis bending and axial force

$$N_{c,Rd} = 2{,}334 \text{ kNm}$$

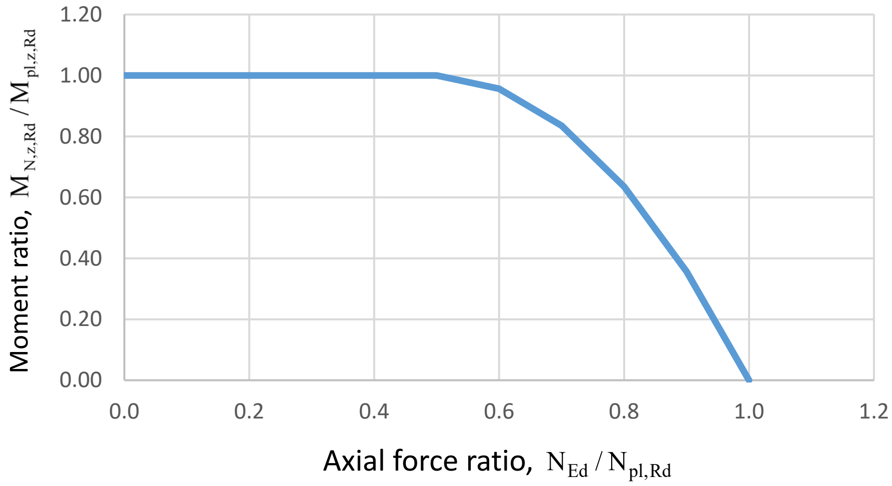
If $N_{Ed} \le \dfrac{h_w t_w f_y}{\gamma_{M0}} = \dfrac{428 \times 7.6 \times 355}{1.00} \times 10^{-3} = 1{,}154.7 \text{ kN}$,

then allowance needs not be made for the effect of axial force on the plastic resistance moment.

Otherwise, the design resistance for bending about the z-z axis reduced due to the axial force is:

For $n \le a$: $M_{N,z,Rd} = M_{pl,z,Rd}$ [Cl. 6.2.9]

For $n > a$: $M_{N,z,Rd} = M_{pl,z,Rd}\left[1 - \left(\dfrac{n-a}{1-a}\right)^2\right]$



N_{Ed}	$N_{pl,Rd}$	n	a	$M_{N,z,Rd} / M_{pl,z,Rd}$
0.0	2,334	0.0	0.49	1.00
233.4	2,334	0.1	0.49	1.00
466.8	2,334	0.2	0.49	1.00
700.2	2,334	0.3	0.49	1.00
933.6	2,334	0.4	0.49	1.00
1167.0	2,334	0.5	0.49	1.00
1400.4	2,334	0.6	0.49	0.96
1633.8	2,334	0.7	0.49	0.83
1867.2	2,334	0.8	0.49	0.63
2100.6	2,334	0.9	0.49	0.36
2,334.0	2,334	1.0	0.49	0.00

For high axial load at $N_{Ed} / N_{pl,Rd} = 0.8$

$$M_{N,z,Rd} = 0.63 \times 47.2 = 29.7 \text{ kNm}$$

c) For biaxial bending, Clause 6.2.9.1 gives

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^2 + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{5n} \leq 1 \qquad \text{[Cl. 6.2.9 (6)]}$$

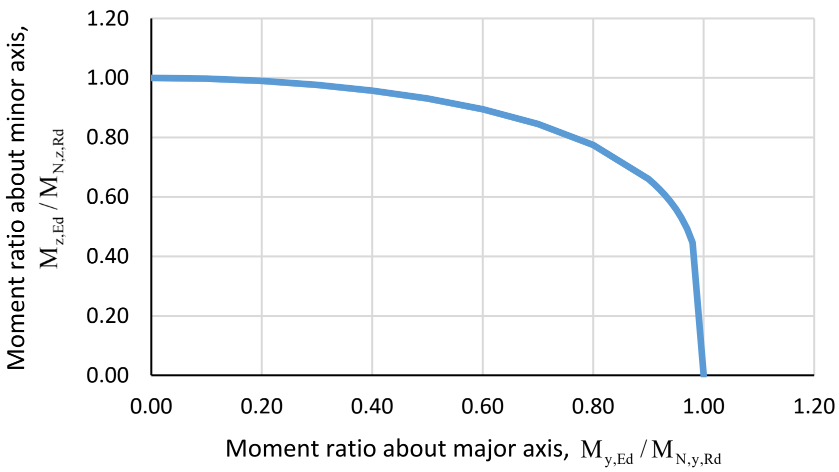
For 457 x 152 x 52 kg/m I-section of S355 subjected to bi-axial bending and axial force

$$N_{c,Rd} = 2{,}334 \text{ kN}$$

For a high axial compression, $N_{Ed} = 0.8N_{c,Rd} = 1867 \text{ kN}$, the following criterion should be used:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^2 + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{4} \leq 1 \qquad \text{as } n = 0.8$$

A graphical presentation of the interaction curve is shown as follows:



$M_{y,Ed} / M_{N,y,Rd}$	$M_{z,Ed} / M_{N,z,Rd}$
0.00	1.00
0.10	1.00
0.20	0.99
0.30	0.98
0.40	0.96
0.50	0.93
0.60	0.89
0.70	0.85
0.80	0.77
0.90	0.66
1.00	0.00

Part II Member design
Worked Example II-1 Design of a fully restrained steel beam

<u>Question</u>

Design a steel beam under the following condition:

Span = 10 m (assuming simply supported)
Beam spacing = 3 m
Loadings

Permanent actions

Dead load, $G_{k,1}$ = $3.0\ kN/m^2$

Superimposed dead load, $G_{k,2}$ = $1.0\ kN/m^2$

Variable actions

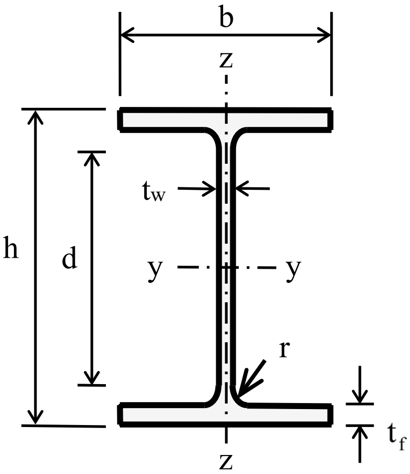
Imposed load, $Q_{k,1}$ = $3.0\ kN/m^2$

Try 457 × 152 × 52 kg/m I-section S355.
Check against bending, shear and deflection.

Note: Deflection limit under variable action $= \dfrac{L}{360}$

Solution

Section properties of 457 × 152 × 52 kg/m I-section:



h	= 449.8 mm	b	= 152.4 mm
t_w	= 7.6 mm	t_f	= 10.9 mm
r	= 10.2 mm		

$W_{pl,y}$ $= 1{,}100 \times 10^3\ mm^3$

I_y $= 21{,}400 \times 10^4\ mm^4$ $\quad I_z = 645 \times 10^4\ mm^4$

I_w $= 311 \times 10^9\ mm^6$ $\quad I_t = 21.4 \times 10^4\ mm^4$

A $= 6660\ mm^2$

Material property:
Since $t_f = 10.9$ mm and $t_w = 7.6$ mm, i.e. the nominal material thickness is less than 16 mm, the nominal value of the yield strength for grade S355 steel is:

f_y = 355 N/mm^2
E = 210,000 N/mm^2
ν = 0.3
G = 81000 N/mm^2

Span = 10 m
Contributive area $= 10 \times 3 = 30\ m^2$
This beam is assumed to be simply supported.

a) Loading.

Dead load, $G_{k,1}$ = 3.0 kN/m^2
Superimposed dead load, $G_{k,2}$ = 1.0 kN/m^2
Live load, $Q_{k,1}$ = 3.0 kN/m^2
Factored load $= 1.40 \times (3+1) + 1.60 \times 3$
= 10.4 kN/m^2
or = 31.2 kN/m for a width of 3 m
Design moment, M_{Ed} $= (31.2 \times 10) \times 10/8$
= 390.0 kNm
Design shear force, V_{Ed} $= (31.2 \times 10) \times 0.5$
= 156.0 kN

b) Try 457 × 152 × 52 kg/m I-section S355.

c) Perform section classification.

As demonstrated in Worked Example I-1, the cross-section classification is Class 1.

d) Check for moment.

As demonstrated in Worked Example I-1, the design resistance for bending about y-y axis, $M_{c,Rd}$ is:

$M_{c,Rd} = 390.5 \text{ kNm}$ [Cl. 6.2.5 (2)]

$\geq M_{Ed} = 390.0 \text{ kNm}$ $\therefore$ OK

e) Check for shear force.

As demonstrated in Worked Example I-1, the design shear resistance $V_{pl,Rd}$ is:

$V_{pl,Rd} = \dfrac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$, where $\gamma_{M0} = 1.0$ [Cl. 6.2.6 (2)]

$V_{pl,Rd} = 746.6 \text{ kN}$

$> V_{Ed} = 156.0 \text{ kN}$ $\therefore$ OK

f) Check for deflection.

Serviceability load $= 3 \text{ kN/m}^2$

$= 9 \text{ kN/m}$ for a width of 3 m

$$\Delta = \frac{5}{384}\frac{WL^4}{EI} = \frac{5}{384} \times \frac{9 \times 10{,}000^4}{210{,}000 \times 21{,}400 \times 10^4} = 26.1 \text{ mm}$$

$< L/360 = 10{,}000/360 = 27.8 \text{ mm}$ $\therefore$ OK

Therefore, 457 × 152 × 52 kg/m I-section S355 satisfies the design.

Part II Member design
Worked Example II-2
Design of an unrestrained steel beam against lateral torsional buckling

Question

As the structural details stated in Worked Example II-1,

Loading during construction = 1.5 kN/m^2

Case I) Span = 10 m with no intermediate restraint
Case II) Span = 10 m with one restraint at mid-span

Other design data are given in Worked Example II-1.

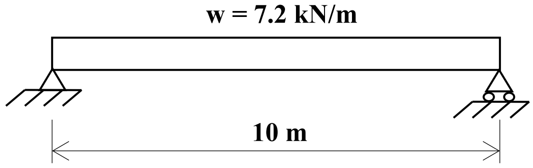
Solution to Procedure B2 –
Use design method given in Appendix B2

Case I) Span = 10 m with no intermediate restraint

a) Evaluate the design load and the design moment.

Factored construction load, w = 1.5 kN/m^2 x 1.6
= 2.4 kN/m^2
= 7.2 kN/m over a width of 3 m

Factored design moment, M_{Ed} = 7.2 x 10^2 / 8 = 90.0 kNm



b) Buckling length, $L_{cr,z} = 10\,\text{m}$.

c) Try 457 x 152 x 52 kg/m I-section S355.

d) Perform cross-section classification – as demonstrated in Worked Example I-1.

e) Calculate the elastic critical moment and the plastic moment resistance.

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr,z}{}^2}\left(\frac{I_w}{I_z} + \frac{L_{cr,z}{}^2 GI_T}{\pi^2 EI_z}\right)^{0.5}$$

For a simply supported beam under uniformly distributed loading, $C_1 = 1.132$.

$$M_{cr} = 1.132 \frac{\pi^2 \times 210{,}000 \times 645 \times 10^4}{10{,}000^2} \times \left(\frac{311 \times 10^9}{645 \times 10^4} + \frac{10{,}000^2 \times 81{,}000 \times 21.4 \times 10^4}{\pi^2 \times 210{,}000 \times 645 \times 10^4}\right)^{0.5} \times 10^{-6}$$

$$= 1.132 \times 133{,}684 \times (48{,}217 + 129{,}664)^{0.5} \times 10^{-6}$$

$$= 63.8\,\text{kNm}$$

For Class 1 section,

$$M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = 1100 \times 10^3 \times 355 \times 10^{-6} = 390.5 \text{ kNm} \quad \text{where } \gamma_{M0} = 1.0 \qquad \text{[Cl. 6.2.5 (2)]}$$

f) Calculate the non-dimensional slenderness, $\bar{\lambda}_{LT}$.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{390.5}{63.8}} = 2.47 \qquad \text{[Cl. 6.3.2.2]}$$

g) Determine the imperfection factor, α_{LT}.

Buckling curve c is used for sections with $h/b > 2$, $\alpha_{LT} = 0.49$ [Cl. 6.3.2.3]

h) Calculate the reduction factor for lateral torsional buckling, χ_{LT}.

For rolled sections, $\bar{\lambda}_{LT,0} = 0.4$ and $\beta = 0.75$ [Cl. 6.3.2.3]

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\,\bar{\lambda}_{LT}^{2}\right]$$

$$= 0.5\left[1 + 0.49\left(2.47 - 0.4\right) + 0.75 \times 2.47^{2}\right] = 3.29$$

$$\chi_{LT} = \frac{1}{3.29 + \sqrt{3.29^{2} - 0.75 \times 2.47^{2}}} = 0.17$$

but $\chi_{LT} \leq 1.0$

and $\chi_{LT} \leq \frac{1}{\bar{\lambda}_{LT}^{2}} = \frac{1}{2.47^{2}} = 0.16$

$\therefore \chi_{LT} = 0.16$

i) Calculate the modified reduction factor, $\chi_{LT,mod}$.

$$f = 1 - 0.5\left(1 - k_c\right)\left[1 - 2.0\left(\bar{\lambda}_{LT} - 0.8\right)^{2}\right]$$ [Cl. 6.3.2.3(2)]

$$= 1 - 0.5\left(1 - 0.94\right)\left[1 - 2.0\left(2.47 - 0.8\right)^{2}\right]$$

$$= 1.14 > 1$$

$\therefore f = 1$

$\therefore \chi_{LT,mod} = \chi_{LT} = 0.16$

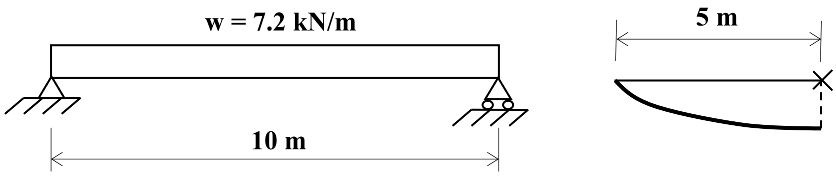
j) Calculate the design buckling resistance moment and check for structural adequacy.

$$M_{b,Rd} = \chi_{LT,mod}\frac{W_{pl,y}f_y}{\gamma_{M1}} = 0.16 \times 390.5 = 62.5\ \text{kNm}$$ where $\gamma_{M1} = 1.0$ [Cl. 6.3.2.1]

$< M_{Ed} = 90.0$ kNm $\therefore$ Not OK.

Case II) Span = 10 m with one restraint at mid-span

a) Evaluate the design load and the design moment.
The factored design moment is the same as that in case I),
i.e. $M_{Ed} = 90.0 \text{ kNm}$



b) Buckling length, $L_{cr,z} = 5 \text{ m}$.

c) Try 457x152x52 I-section S355.

d) Perform cross-section classification – as demonstrated in Worked Example I-1.

e) Calculate the elastic critical moment and the plastic moment resistance.

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr,z}{}^2} \left(\frac{I_w}{I_z} + \frac{L_{cr,z}{}^2 GI_T}{\pi^2 EI_z} \right)^{0.5}$$

Conservatively, take $C_1 = 1.0$.

$$M_{cr} = 1.0 \times \frac{\pi^2 \times 210{,}000 \times 645 \times 10^4}{5{,}000^2} \times \left(\frac{311 \times 10^9}{645 \times 10^4} + \frac{5{,}000^2 \times 81{,}000 \times 21.4 \times 10^4}{\pi^2 \times 210{,}000 \times 645 \times 10^4} \right)^{0.5} \times 10^{-6}$$

$$= 1.0 \times 534{,}735 \times (48{,}217 + 32{,}416)^{0.5} \times 10^{-6}$$

$$= 151.8 \text{ kNm}$$

and $M_{pl,Rd} = \dfrac{W_{pl,y} f_y}{\gamma_{M0}} = 390.5 \text{ kNm}$ where $\gamma_{M0} = 1.0$ [Cl. 6.2.5 (2)]

f) Calculate the non-dimensional slenderness, $\bar{\lambda}_{LT}$.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{390.5}{151.8}} = 1.60$$

g) Determine the imperfection factor, α_{LT}.

Buckling curve c is used for sections with $h/b > 2$, $\alpha_{LT} = 0.49$ [Cl. 6.3.2.3]

h) Calculate the reduction factor of lateral torsional buckling, χ_{LT}.

For rolled sections, $\bar{\lambda}_{LT,0} = 0.4$ and $\beta = 0.75$ [Cl. 6.3.2.3]

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}\left(\bar{\lambda}_{LT}-\bar{\lambda}_{LT,0}\right)+\beta\,\bar{\lambda}_{LT}^{\,2}\right]$$

$$= 0.5\left[1+0.49\,(1.60-0.4)+0.75\times 1.60^2\right] = 1.75$$

$$\chi_{LT} = \frac{1}{1.75+\sqrt{1.75^2-0.75\times 1.60^2}} = 0.35$$

but $\chi_{LT} \le 1.0$

and $\chi_{LT} \le \dfrac{1}{\bar{\lambda}_{LT}^{\,2}} = \dfrac{1}{1.60^2} = 0.39$

$\therefore \chi_{LT} = 0.35$

i) Calculate the modified reduction factor, $\chi_{LT,\mathrm{mod}}$.

$f = 1-0.5\,(1-k_c)\left[1-2.0\,(\bar{\lambda}_{LT}-0.8)^2\right]$ where k_c = 1.0 conservatively. [Cl. 6.3.2.3(2)]

$$= 1-0.5\,(1-1)\left[1-2.0\,(1.31-0.8)^2\right]$$

$$= 1$$

$\therefore \chi_{LT,\mathrm{mod}} = \chi_{LT} = 0.35$

j) Calculate the design buckling resistance and check for structural adequacy.

$M_{b,RD} = \chi_{LT}\dfrac{M_{pl,y} f_y}{\gamma_{M1}} = 0.35 \times 390.5 = 136.7$ kNm where $\gamma_{M1} = 1.0$ [Cl. 6.3.2.1]

$\ge M_{Ed} = 90.0$ kNm $\therefore$ OK.

Therefore, 457 × 152 × 52 kg/m I-section S355 with an intermediate restraint at mid-span satisfies the design check.

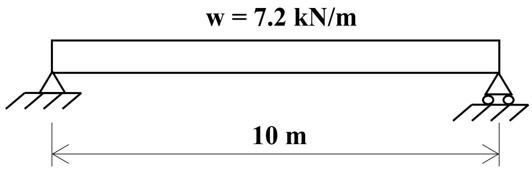
Solution to Procedure B3 –
Use design method given in Appendix B3

Case I) Span = 10 m with no intermediate restraint

a) Evaluate the design load and the design moment.

Factored construction load, w $= 1.5\ kN/m^2 \times 1.6$
$= 2.4\ kN/m^2$
$= 7.2\ kN/m$ over a width of 3 m

Factored design moment, M_{Ed} $= 7.2 \times 10^2 / 8 = 90.0\ kNm$



b) Try 457 x 152 x 52 kg/m I-section S355.

c) Perform cross-section classification – as demonstrated in Worked Example I-1.

d) Determine the buckling length, $L_{cr,z} = 10$ m.

e) Calculate the non-dimensional slenderness, $\bar{\lambda}_{LT}$.

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV\bar{\lambda}_z\sqrt{\beta_w}$$

where

$C_1 = 1.13$;

$U = 0.859$;

$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = \pi\sqrt{\frac{210{,}000}{355}} = 76.4$;

$i = 31.1\ mm$;

$\lambda_z = 10{,}000 / 31.1 = 321.5\ mm$;

$$V = \frac{1}{\sqrt[4]{1 + \frac{1}{20}\left(\frac{\lambda_z}{h/t_f}\right)^2}} = \frac{1}{\sqrt[4]{1 + \frac{1}{20}\left(\frac{321.5}{449.8/10.9}\right)^2}} = 0.706;$$

$\bar{\lambda}_z = \lambda_z / \lambda_1 = 321.5 / 76.4 = 4.21$

$\beta_w = 1$ for Class 1 sections

$$\therefore\ \bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV\bar{\lambda}_z\sqrt{\beta_w}$$

$$= 0.94 \times 0.859 \times 0.706 \times 4.21 \times 1$$
$$= 2.40$$

g) Determine the imperfection factor, α_{LT}, and the paramenter ϕ_{LT}.

Buckling curve b is used for sections with $2 < h/b \leq 3.1$, $\alpha_{LT} = 0.49$.

h) Calculate the reduction factor for lateral torsional buckling, χ_{LT}.

$$\phi_{LT} = 0.5\left[1 + 0.49\left(2.40 - 0.4\right) + 0.75 \times 2.40^2\right] = 3.15$$

$$\chi_{LT} = \frac{1}{3.15 + \sqrt{3.15^2 - 0.75 \times 2.4^2}} = 0.18 \leq 1.0$$

i) Calculate the design buckling resistance moment and check for structural adequacy.

$$M_{b,Rd} = \chi_{LT} \frac{W_{pl,y} f_y}{\gamma_{M1}} = 0.18 \times 390.5 = 70.3 \text{ kNm}$$

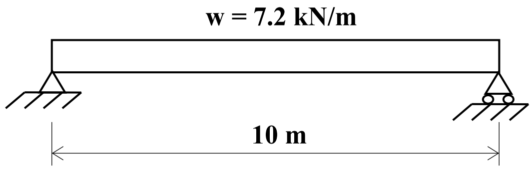
where $\gamma_{M0} = 1.0$

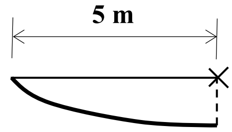
$M_{b,Rd} \leq M_{Ed} = 90.0 \text{ kNm}$ $\therefore$ Not OK.

Case II) Span = 10 m with one restraint at mid-span

a) Evaluate the design load and the design moment.
Factored design moment is same as that in case I),

i.e. $M_{Ed} = 90.0 \text{ kNm}$





b) Determine the buckling length, $L_{cr,z} = 5 \text{ m}$.

c) Try 457 x 152 x 52 kg/m I-section S355.

d) Perform cross-section classification – as demonstrated in Worked Example I-1.

e) Calculate the non-dimensional slenderness, $\bar{\lambda}_{LT}$.

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV\bar{\lambda}_z\sqrt{\beta_w}$$

where $C_1 = 1.00$ for a conservative apprach;

$U = 0.859$;

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = \pi\sqrt{\frac{210{,}000}{355}} = 76.4$$

$i = 31.1 \text{ mm}$

$\lambda_z = L_{cr,z}/i = 5{,}000/31.1 = 160.8$

$$V = \frac{1}{\sqrt[4]{1+\frac{1}{20}\left(\frac{\lambda_z}{h/t_f}\right)^2}} = \frac{1}{\sqrt[4]{1+\frac{1}{20}\left(\frac{160.8}{449.8/10.9}\right)^2}} = 0.87$$

$\bar{\lambda}_z = \lambda_z/\lambda_1 = 160.8/76.4 = 2.10$

$\beta_w = 1$ for Class 1 sections

$$\therefore \bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV\bar{\lambda}_z\sqrt{\beta_w}$$

$$= 1.00 \times 0.859 \times 0.87 \times 2.10 \times 1$$

$$= 1.57$$

g) Determine the imperfection factor, α_{LT}, and the paramenter ϕ_{LT}.
Buckling curve b is used for sections with $2 < h/b \le 3.1$, $\alpha_{LT} = 0.49$.

h) Calculate the reduction factor of lateral torsional buckling, χ_{LT} .

$$\phi_{LT} = 0.5\left[1 + 0.49\left(1.57 - 0.4\right) + 0.75 \times 1.57^2\right] = 1.71$$

$$\chi_{LT} = \frac{1}{1.71 + \sqrt{1.71^2 - 0.75 \times 1.57^2}} = 0.36 \leq 1.0$$

i) Calculate the design buckling resistance moment and check for structural adequacy.

$$M_{b,Rd} = \chi_{LT} \frac{W_{pl,y} f_y}{\gamma_{M1}} = 0.36 \times 390.5 = 140.6 \text{ kNm}$$

where $\gamma_{M1} = 1.0$

$\geq M_{Ed} = 90.0$ kNm $\quad\therefore$ OK.

Therefore, 457 × 152 × 52 kg/m I-section S355 with an intermediate restraint at mid-span satisfies the design check.

Summary of the reduction factors for lateral torsional buckling, χ_{LT}

Procedure B2:
Design method given in Cl.6.3.2.3

Case I:
$\bar{\lambda}_{LT} = 2.48$
$\phi_{LT} = 3.32$
$\chi_{LT} = 0.17$
$f = 0.16$
$\chi_{LT,mod} = 0.16$

Case II:
$\bar{\lambda}_{LT} = 1.60$
$\phi_{LT} = 1.75$
$\chi_{LT} = 0.35$
$f = 1.00$
$\chi_{LT,mod} = 0.35$

Procedure B3:
Design method given in Steel Designers' Manual

Case I:
$\bar{\lambda}_{LT} = 2.40$
$\phi_{LT} = 3.15$
$\chi_{LT} = 0.18$

Case II:
$\bar{\lambda}_{LT} = 1.57$
$\phi_{LT} = 1.71$
$\chi_{LT} = 0.36$

$\therefore$ Design method according to Procedure B3 gives a more safe design to lateral torsional buckling.

Part II   Member design

Worked Example II-3   Design of a steel column under axial compression

<u>Question</u>

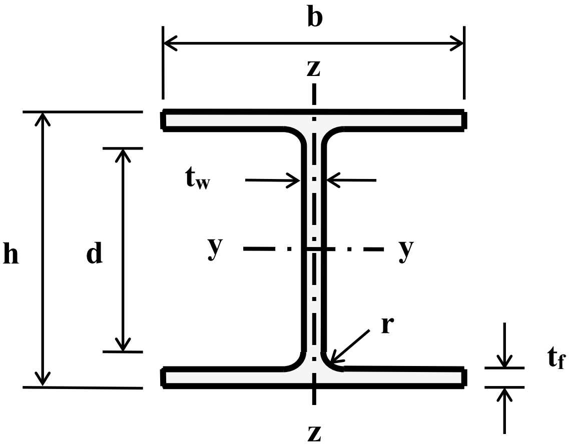
Design a steel column under the following condition:

Factored axial load, N_{Ed}	= 1000 kN
Effective length, $L_{cr,y}$	= 9.0 m
$L_{cr,z}$	= 6.3 m

Try 254 x 254 x 73 kg/m H-section S355.

Solution

Section properties of 254 x 254 x 73 kg/m H-section S355:



$h = 254.1$ mm
$b = 254.6$ mm
$t_w = 8.6$ mm
$t_f = 14.2$ mm
$r = 12.7$ mm
$A = 93.1 \times 10^2\ mm^2$
$I_y = 11{,}400 \times 10^4\ mm^4$
$I_z = 3{,}910 \times 10^4\ mm^4$
$I_w = 562 \times 10^9\ mm^6$
$I_t = 576 \times 10^3\ mm^4$
$W_{el,y} = 898 \times 10^3\ mm^3$
$W_{pl,y} = 992 \times 10^3\ mm^3$

Material properties:

Since $t_f = 14.2$ mm and $t_w = 8.6$ mm, i.e. the nominal material thickness is smaller than 16 mm, the nominal value of the yield strength for grade S355 steel is:

$f_y = 355\ N/mm^2$
$E = 210{,}000\ N/mm^2$
$\nu = 0.3$
$G = 81{,}000\ N/mm^2$

a) Evaluate the design load.

$$N_{Ed} = 1000\ kN$$

b) Try 254 x 254 x 73 kg/m H-section S355.

c) Perform section classification.

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$$

Web – internal compression part: [Table 5.2]

$c_w = h - 2t_f - 2r = 200.3$ mm

$\Rightarrow c_w / t_w = 200.3 / 8.6 = 23.3$

Limit for Class 1 web $= 33\varepsilon = 26.7 \geq 23.3 \Rightarrow$ The web is Class 1.

Outstand flanges: [Table 5.2]

$c_f = (b - t_w - 2r)/2 = 110.3$ mm

$\Rightarrow c_f / t_f = 110.3 / 14.2 = 7.8$

Limit for Class 1 flange $= 10\varepsilon = 8.1 \geq 7.8 \Rightarrow$ The flanges are Class 2.

The overall cross-section classification is Class 2 under pure compression.

d) Determine the effective length for both axes.

Effective length, $L_{cr,y}$ = 9.0 m

$L_{cr,z}$ = 6.3 m

e) Calculate N_{cr} and Af_y.

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr,y}^{\ 2}} = \frac{\pi^2 \times 210{,}000 \times 114{,}000{,}000}{9{,}000^2} \times 10^{-3} = 2{,}917 \ \text{kN}$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr,z}^{\ 2}} = \frac{\pi^2 \times 210{,}000 \times 39{,}100{,}000}{6{,}300^2} \times 10^{-3} = 2{,}042 \ \text{kN}$$

$$N_{c,Rd} = Af_y = 9{,}310 \times 355 \times 10^{-3} = 3{,}305 \ \text{kN}$$

f) Calculate the non-dimensional slenderness, $\bar{\lambda}$.

$$\bar{\lambda}_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \sqrt{\frac{3{,}305}{2{,}917}} = 1.06$$ [Cl.6.3.1.2]

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{cr,z}}} = \sqrt{\frac{3{,}305}{2{,}042}} = 1.27$$ [Cl.6.3.1.2]

g) Determine the imperfection factor, α.

For a section with $h/b \le 1.2$,

- use buckling curve b with $\alpha = 0.34$ for buckling about y-y axis.
- use buckling curve c with $\alpha = 0.49$ for buckling about z-z axis.

h) Calculate the parameter, ϕ and the buckling reduction factor, χ_z.

Buckling about y-y axis:

$$\phi_y = 0.5\left[1 + 0.34\left(1.06 - 0.2\right) + 1.06^2\right] = 1.21$$ [Cl.6.3.1.2]

$$\chi_y = \frac{1}{1.21 + \sqrt{1.21^2 - 1.06^2}} = 0.56$$

Buckling about z-z axis:

$$\phi_z = 0.5\left[1 + 0.49\left(1.27 - 0.2\right) + 1.27^2\right] = 1.57$$ [Cl.6.3.1.2]

$$\chi_y = \frac{1}{1.57 + \sqrt{1.57^2 - 1.27^2}} = 0.40$$

$\therefore \ \chi = \chi_z = 0.40$ critical

i&j) Calculate the design buckling resistance, $N_{b,Rd}$ and check for structural adequacy:

$$N_{b,Rd} = \chi \frac{N_{c,Rd}}{\gamma_{M1}} = \frac{0.40 \times 3{,}305}{1.00} = 1{,}322 \ \text{kN} \ge 1000 \ \text{kN}$$ $\therefore$ OK.

Therefore, 254 x 254 x 73 kg/m H-section S355 steel satisfies the design.

Part II Member design

Worked Example II-4

Design of a beam-column under combined compression and bending

Question

Design a steel column under the following condition:

Design axial load, N_{Ed} = 1000 kN

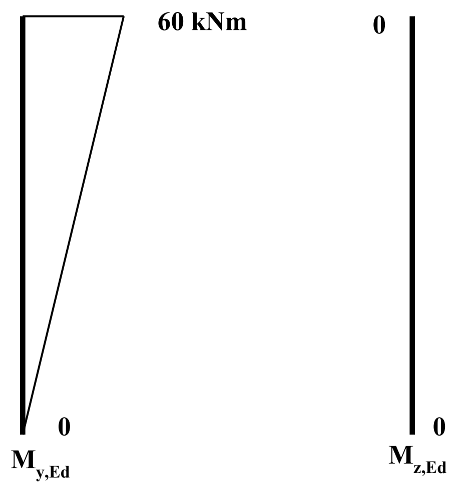
Design moment, $M_{y,Ed}$ = 60 kNm

$M_{z,Ed}$ = 0 kNm

Effective length, $L_{cr,y}$ = 9.0 m

$L_{cr,z}$ = 6.3 m

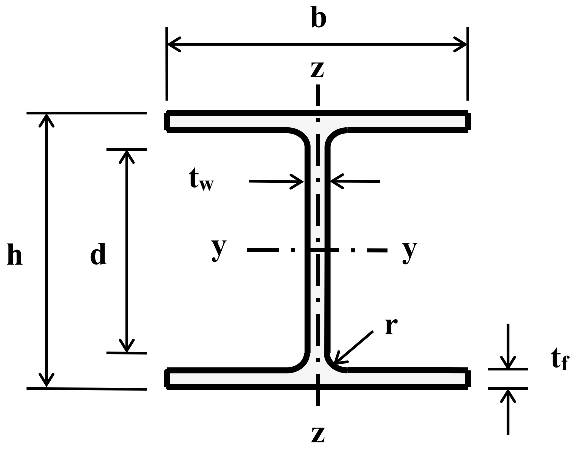
Try 254 x 254 x 73 kg/m H-section S355.



Solution

Section properties of 254 × 254 × 73 kg/m H-section S355:

$h = 254.1\,\text{mm}$
$b = 254.6\,\text{mm}$
$t_w = 8.6\,\text{mm}$
$t_f = 14.2\,\text{mm}$
$r = 12.7\,\text{mm}$
$A = 93.1\times10^2\,\text{mm}^2$
$I_y = 11{,}400\times10^4\,\text{mm}^4$
$I_z = 3{,}910\times10^4\,\text{mm}^4$
$I_w = 562\times10^9\,\text{mm}^6$
$I_t = 576\times10^3\,\text{mm}^4$
$W_{el,y} = 898\times10^3\,\text{mm}^3$
$W_{pl,y} = 992\times10^3\,\text{mm}^3$



Material properties:

Since $t_f = 14.2$ mm and $t_w = 8.6$ mm, i.e. the nominal material thickness is smaller than 16 mm, the nominal value of the yield strength for grade S355 steel is:

$f_y = 355\ \text{N/mm}^2$
$E = 210{,}000\ \text{N/mm}^2$
$\nu = 0.3$
$G = 81{,}000\ \text{N/mm}^2$

a) Evaluate the design load.

$N_{Ed} = 1000\ \text{kN}$
$M_{y,Ed} = 60\ \text{kNm}$
$M_{z,Ed} = 0\ \text{kNm}$

b) Try 254 x 254 x 73 kg/m H-section S355 steel.

c) Perform section classification.

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$$

Web – internal compression part: [Table 5.2]

$c_w = h - 2t_f - 2r = 200.3\ \text{mm}$

$\Rightarrow c_w / t_w = 200.3 / 8.6 = 23.3$

Limit for Class 1 web $= 33\varepsilon = 26.7 \geq 23.3$ $\Rightarrow$ The web is Class 1.

Outstand flanges:

$c_f \;=\; (b - t_w - 2r)/2 \;=\; 110.3 \text{ mm}$

$\Rightarrow\; c_f / t_f \;=\; 110.3 / 14.2 \;=\; 7.8$

Limit for Class 2 flange = $10\varepsilon \;=\; 8.1 \;\geq\; 7.8$ $\Rightarrow$ The flanges are Class 2.

The overall cross-section classification is Class 2. (Under pure compression)

d) Determine the effective length for both axes.

Effective length, $L_{cr,y}$ = 9.0 m

$L_{cr,z}$ = 6.3 m

e) Check the resistance of the cross-section for combined bending and axial force.

Compression:

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \qquad \text{[Cl. 6.2.4 (2)]}$$

The design compression resistance of the cross-section is therefore:

$$N_{c,Rd} = \frac{9{,}310 \times 355}{1.00} \times 10^{-3} = 3{,}305 \text{ kN} > 1{,}000\text{kN} \qquad \therefore \text{OK.}$$

Bending about y-y axis:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} \qquad \text{[Cl. 6.2.5 (2)]}$$

The design resistance of the cross-section for bending is therefore:

$$M_{c,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{992 \times 10^3 \times 355}{1.00} \times 10^{-6}$$

$$= 352.2 \text{ kNm} \;>\; 60 \text{ kNm} \qquad \therefore \text{OK.}$$

Cross-section capacity check for combined bending and axial force:

f) Check the member buckling resistance in combined bending and axial compression.

Buckling resistance in compression:

Calculate the elastic critical force and Af_y.

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr,y}{}^2} = \frac{\pi^2 \times 210{,}000 \times 114{,}000{,}000}{9{,}000^2} \times 10^{-3} = 2{,}917 \text{ kN}$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr,z}{}^2} = \frac{\pi^2 \times 210{,}000 \times 39{,}100{,}000}{6{,}300^2} \times 10^{-3} = 2{,}042 \text{ kN}$$

$$N_{c,Rd} = Af_y = 9{,}310 \times 355 \times 10^{-3} = 3{,}305 \text{ kN}$$

Calculate the non-dimensional slenderness.

$$\bar{\lambda}_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \sqrt{\frac{3{,}305}{2{,}917}} = 1.06$$ [Cl.6.3.1.2]

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{cr,z}}} = \sqrt{\frac{3{,}305}{2{,}042}} = 1.27$$ [Cl.6.3.1.2]

g) Determine the imperfection factor, α.

Choose a suitable buckling curve [Table 6.1]

- use buckling curve b with $\alpha = 0.34$ for buckling about the y-y axis.
- use buckling curve c with $\alpha = 0.49$ for buckling about the z-z axis.

Calculate the buckling reduction factor, χ.
Buckling curve about y-y axis:

$$\phi_y = 0.5\left[1+0.34\left(1.06-0.2\right)+1.06^2\right] = 1.21$$ [Cl.6.3.1.2]

$$\chi_y = \frac{1}{1.21+\sqrt{1.21^2-1.06^2}} = 0.56$$

Buckling curve about z-z axis:

$$\phi_y = 0.5\left[1+0.49\left(1.27-0.2\right)+1.27^2\right] = 1.57$$ [Cl.6.3.1.2]

$$\chi_y = \frac{1}{1.57+\sqrt{1.57^2-1.27^2}} = 0.40$$

Buckling resistance in bending:

h) Calculate the elastic critical moment, M_{cr} and the plastic resistance moment $M_{pl,Rd}$.

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr,z}{}^2}\left(\frac{I_w}{I_z}+\frac{L_{cr,z}{}^2 GI_T}{\pi^2\, EI_z}\right)^{0.5}$$

with a zero moment at one end, i.e. $\psi = 0$, $C_1 = 1.879$.

$$M_{cr} = 1.879 \times \frac{\pi^2 \times 210{,}000 \times 39.1\times10^6}{6{,}300^2} \times \left(\frac{562\times10^9}{39.1\times10^6}+\frac{6{,}300^2\times81{,}000\times576\times10^3}{\pi^2\times210{,}000\times39.1\times10^6}\right)^{0.5} \times 10^{-6}$$

$$= 1.879\times2{,}041{,}807\times\left(14{,}373+22{,}850\right)^{0.5}\times10^{-6} = 740.2\ \text{kNm}$$

$$M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{992\times10^3\times355\times10^{-6}}{1.0} = 352.2\ \text{kNm}$$ where $\gamma_{M0} = 1.0$

i) Calculate the non-dimensional slenderness.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{352.2}{740.2}} = 0.69$$ [Cl.6.3.2.2]

j) Determine the imperfection factor for lateral torsional buckling, α_{LT}.

Buckling curve a is used for sections with $h/b \leq 2.0$, $\alpha_{LT} = 0.21$ [Table 6.3 & 6.4]

k) Calculate the buckling reduction factor. [Cl. 6.3.2.2]

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}\left(\bar{\lambda}_{LT}-0.2\right)+\bar{\lambda}_{LT}^{\,2}\right]$$

$$\phi_{LT} = 0.5\left[1+0.21\left(0.69-0.2\right)+0.69^2\right]=0.79$$

$$\chi_{LT} = \frac{1}{0.79+\sqrt{0.79^2-0.69^2}} = 0.85$$

$$\Rightarrow \quad \chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}} = 0.85\times\frac{352.2}{1.0} = 299.4 \text{ kNm}$$

l) Resistance in combined bending and axial compression:

A member subjected to combined bending and axial compression must satisfy both equations:

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT}M_{y,Rk}/\gamma_{M1}} + \cancel{k_{yz}\frac{M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}}} \leq 1$$ [Cl. 6.3.3]

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT}M_{y,Rk}/\gamma_{M1}} + \cancel{k_{zz}\frac{M_{z,Ed}}{M_{z,Rk}/\gamma_{M1}}} \leq 1$$

$$\chi_y\, N_{Rk}/\gamma_{M1} = 0.56\times 9{,}310\times 355\times 10^{-3} = 1{,}850.8 \text{ kN}$$

$$\chi_y\, N_{Rk}/\gamma_{M1} = 0.40\times 9{,}310\times 355\times 10^{-3} = 1{,}322.0 \text{ kN}$$

m) Determination of interaction factors k_{ij} using Annex B

Since $M_{z,Ed} = 0$ kNm, only k_{yy} and k_{zy} are required.

Since the member is susceptible to lateral torsional buckling, interaction factors k_{yy} and k_{zy} are determined according to Table B.2.

$$\psi = 0,\quad C_{my} = C_{mLT} = 0.6+0.4\psi \geq 0.4$$
$$= 0.6$$

$$k_{yy} = C_{my}\left(1+\left(\bar{\lambda}_y-0.2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) \leq C_{my}\left(1+0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$$

$$=0.60\left(1+\left(1.06-0.2\right)\times\frac{1{,}000}{1{,}850.8}\right)=0.95$$

$$\leq 0.60\left(1+0.8\times\frac{1{,}000}{1{,}850.8}\right)=0.86$$

$$\therefore \quad k_{yy} = 0.86$$

$\because \overline{\lambda_z} = 1.27 \geq 0.4,$

$$k_{zy} = \left(1 - \frac{0.1\ \overline{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}}\right) \geq \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}}\right)$$

$$= \left(1 - \frac{0.1 \times 1.27}{(0.6 - 0.25)} \frac{1{,}000}{1{,}322}\right) = 0.73$$

$$\geq \left(1 - \frac{0.1}{(0.6 - 0.25)} \frac{1{,}000}{1{,}322}\right) = 0.78$$

$$\therefore \quad k_{yy} = 0.78$$

n) Check for structural adequacy.

$$\frac{N_{Ed}}{\chi_y N_{Rk} / \gamma_{M1}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}}$$

$$= \frac{1{,}000}{1{,}850.8} + 0.86 \times \frac{60}{299.4}$$

$$= 0.54 + 0.17 = 0.71 \ \leq \ 1.00 \qquad \therefore \text{OK.}$$

$$\frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} + k_{zy} \frac{M_{y,Ed}}{\chi_z M_{y,Rk} / \gamma_{M1}}$$

$$= \frac{1{,}000}{1{,}322.0} + 0.78 \times \frac{60}{299.4}$$

$$= 0.76 + 0.16 = 0.92 \ \leq \ 1.00 \qquad \therefore \text{OK.}$$

Therefore, 254 x 254 x 73 kg/m H-section S355 steel satisfies the design.

Key parameters in Worked Example II-4.

$C_{my} = 0.60$; $C_{mLT} = 0.60$

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} = 0.47 \le 1$$

$$\frac{N_{Ed}}{\chi_y N_{Rk} / \gamma_{M1}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} = 0.71 \le 1$$

$$\frac{N_{Ed}}{\chi_z N_{Rk} / \gamma_{M1}} + k_{zy} \frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} = 0.92 \le 1$$

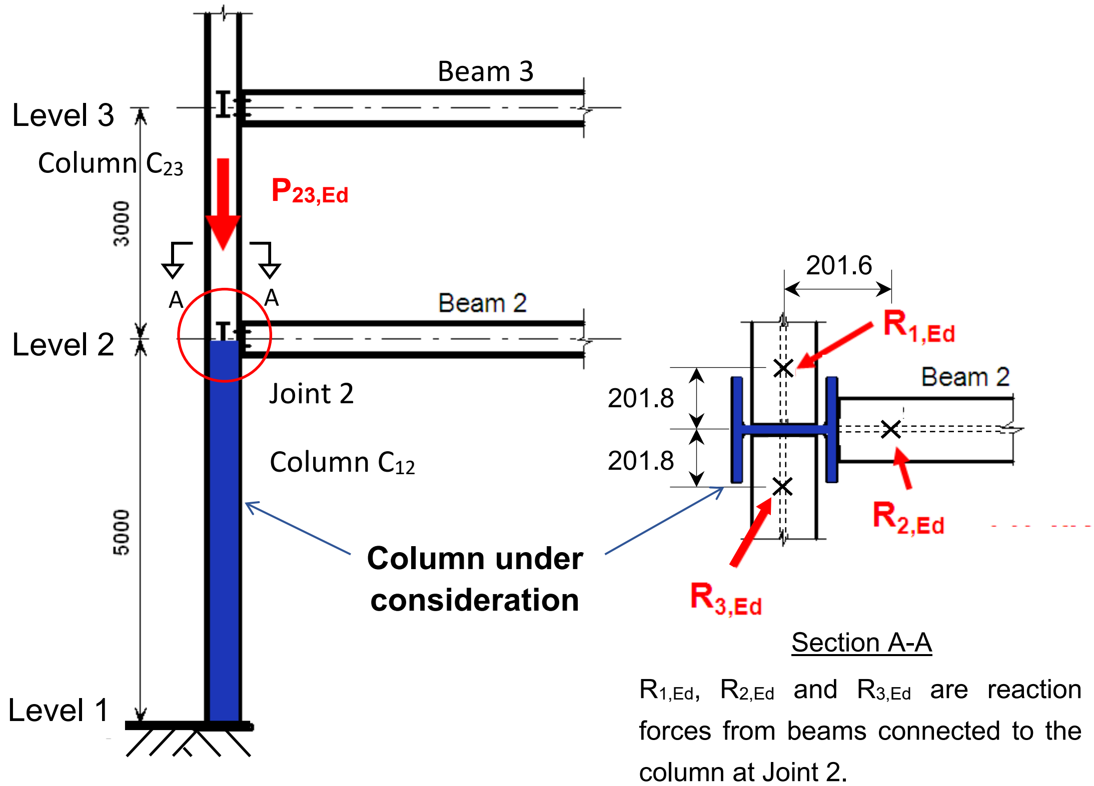
Part II Member design

Worked Example II-5a Column in simple construction

Question

Design the column between Levels 1 and 2, i.e. Column C12, as shown in the figure below, with a S275 H-section. The following assumptions are made:

- The column forms part of a braced structure of simple construction.
- The column is effectively pinned at the base, and continuous at Level 2.
- Beam 2 is connected to the column flange of the column at Joint 2 with flexible end plates.



Design data:

$P_{23,Ed}$ = 377 kN

$P_{1,Ed}$ = 37 kN

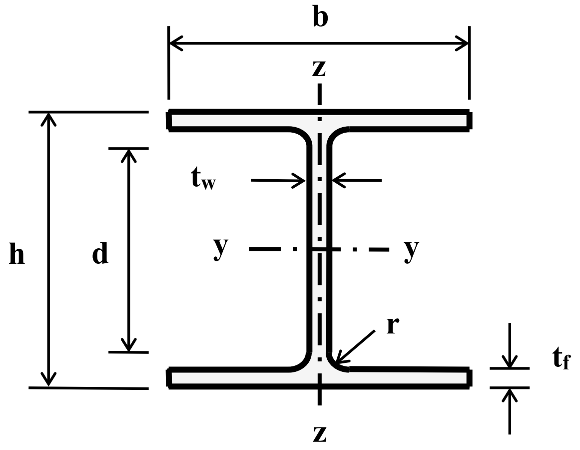
$P_{2,Ed}$ = 147 kN

$P_{3,Ed}$ = 28 kN

Try 203 x 203 x 46 kg/m H-section S275.

Solution

Section properties of 203 x 203 x 46 kg/m H-section S275:



$$
\begin{aligned}
h &= 203.2 \text{ mm} \\
b &= 203.6 \text{ mm} \\
t_w &= 7.2 \text{ mm} \\
t_f &= 11.0 \text{ mm} \\
r &= 10.2 \text{ mm}
\end{aligned}
$$

$$
\begin{aligned}
A &= 5{,}870 \quad \text{mm}^2 \\
I_y &= 4{,}570 \times 10^4 \quad \text{mm}^4 \\
I_z &= 1{,}550 \times 10^4 \quad \text{mm}^4 \\
I_w &= 143 \times 10^9 \quad \text{mm}^6 \\
I_t &= 222 \times 10^3 \quad \text{mm}^4 \\
W_{el,y} &= 450 \times 10^3 \quad \text{mm}^3 \\
W_{pl,y} &= 497 \times 10^3 \quad \text{mm}^3 \\
W_{el,z} &= 152 \times 10^3 \quad \text{mm}^3 \\
W_{pl,z} &= 231 \times 10^3 \quad \text{mm}^3
\end{aligned}
$$

U = 0.847 (buckling parameter)

a) Nominal moments due to connected beams

In simple construction, reaction forces from connected beams are assumed to act at 100 mm from the faces of the web or of the flanges of the column (NCCI SN005a).

Nominal moments at Joint 2

$$M_{2,y,Ed} = \left(\frac{h}{2} + 100\right) \times R_{2,Ed} \times 10^{-3} \text{ kNm} \qquad = 29.6 \text{ kNm}$$

$$M_{2,z,Ed} = \left(\frac{t_w}{2} + 100\right) \times \left(R_{1,Ed} - R_{3,Ed}\right) \times 10^{-3} \text{ kNm} \quad = 0.9 \quad \text{kNm}$$

These nominal moments are distributed between the column members above and below Level 2, i.e. Columns C_{12} and C_{23}, in proportion to their bending stiffnesses, K_{12} and K_{23} respectively.

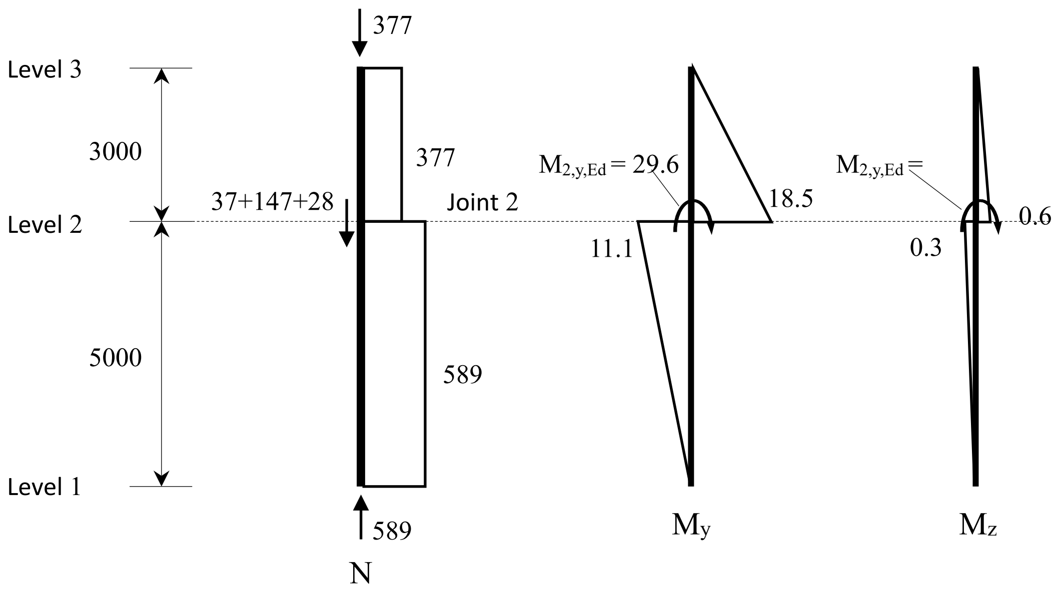
$$\frac{K_{23}}{K_{12} + K_{23}} = \frac{EI/L_{23}}{EI/L_{12} + EI/L_{23}} = \frac{EI/5000}{EI/3000 + EI/5000} = \frac{3}{8}$$

The nominal moments acting onto Column C_{12} at Joint 2 after moment distribution are:

$$M_{y,Ed} = M_{2,y,Ed} \times \frac{3}{8} \; = 11.1 \text{ kNm}$$

$$M_{z,Ed} = M_{2,z,Ed} \times \frac{3}{8} \; = 0.3 \quad \text{kNm}$$

The axial force and the bending moment diagrams are shown below.



b) Buckling lengths

About the y-y axis $\qquad L_{cr,y} = L = 5{,}000\text{ mm}$

About the z-z axis $\qquad L_{cr,z} = L = 5{,}000\text{ mm}$

c) Resistance to flexural buckling

Flexural buckling about the z-z axis is considered to be critical.

Both the elastic critical force and the non-dimensional slenderness for flexural buckling of column C_{12} are evaluated as follows:

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr,z}{}^2} = \frac{\pi^2 \times 210 \times 10^3 \times 1{,}550 \times 10^4}{5{,}000^2} \times 10^{-3} = 1{,}285\text{ kN}$$

$$N_{c,Rd} = Af_y = 5{,}870 \times 275 \times 10^{-3} = 1{,}614\text{ kN}$$

$$\therefore\ \bar{\lambda}_z = \sqrt{\frac{N_{c,Rd}}{N_{cr,z}}} = \sqrt{\frac{1{,}614}{1{,}285}} = 1.12$$

From Table 6.2 of EN 1993-1-1:

For a H-section (with $h/b \le 1.2$) and $t_f \le 100$ mm, use buckling curve 'c', and hence,

$\alpha = 0.49$. [Table 6.1]

According to Table 6.1 of EN 1993-1-1

$$\Phi_z = 0.5\left[1 + \alpha\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z{}^2\right] = 0.5\left[1 + 0.49 \times \left(1.12 - 0.2\right) + 1.12^2\right] = 1.35$$ [Cl. 6.3.1.2]

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z{}^2 - \bar{\lambda}_z{}^2}} = \frac{1}{1.35 + \sqrt{1.35^2 - 1.12^2}} = 0.48$$

$$\therefore\ N_{b,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \sqrt{\frac{0.48 \times 1{,}614}{1.0}} = 774\,\text{kN} \geq N_{Ed} = 589\,\text{kN}$$

$\therefore$ The resistance of Column C_{12} to flexural buckling is adequate.

d) Design buckling resistance moment

$$V = \frac{1}{\sqrt[4]{1 + \frac{1}{20}\left(\frac{\lambda_z}{h/t_f}\right)^2}} = \frac{1}{\sqrt[4]{1 + \frac{1}{20}\left(\frac{5{,}000/51.4}{203.2/11.0}\right)^2}} = 0.80$$

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV\bar{\lambda}_z \sqrt{\beta_w}$$ (Refer to Appendix B3)

$$= \frac{1}{\sqrt{1.77}} \times 0.847 \times 0.80 \times 1.12 \times \sqrt{1.0} = 0.57$$

Alternatively, the non-dimensional slenderness for lateral torsional buckling for the H-section may be approximated (NCCI SN002a) as follows:

$$\bar{\lambda}_{LT} = 0.9\bar{\lambda}_z = 1.01$$

This assumes a uniform bending moment and a section symmetric about its major axis.

From Table B3.2 in Appendix B of this Technical Guide, for a rolled H-section (with $h/b \leq 2$), use buckling curve 'b', and hence, $\alpha_{LT} = 0.34$.

$$\begin{aligned}\Phi_{LT} &= 0.5\left[\,1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\bar{\lambda}_{LT}^{\,2}\,\right] \\ &= 0.5\left[\,1 + 0.34 \times (0.57 - 0.4) + 0.75 \times 0.57^2\,\right] = 0.65\end{aligned}$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\,2} - \beta\bar{\lambda}_{LT}^{\,2}}} = \frac{1}{0.65 + \sqrt{0.65^2 - 0.75 \times 0.57^2}} = 0.93$$

$$\therefore\ M_{b,Rd} = \frac{0.93 \times 497 \times 10^3 \times 275}{1.0} \times 10^{-6} = 127.1\,\text{kNm} \geq M_{y,Ed} = 11.1\,\text{kNm}$$

$\therefore$ The design buckling resistance moment of Column C_{12} is adequate.

e) Resistance for bending about minor axis

There is no reduction for buckling to the minor axis bending resistance $M_{c,z,Rd}$.

$$\therefore\ M_{c,z,Rd} = \frac{W_{pl,z} f_y}{\gamma_{M0}} = \frac{231 \times 10^3 \times 275}{1.0} \times 10^{-6} = 63.5\,\text{kNm}$$

$\therefore$ The resistance of Column C_{12} for bending about minor axis is adequate.

f) Combined compression and bending

Using the simplified buckling check for combined bending and axial compression:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy}\frac{M_{y,Ed}}{M_{b,Rd}} + k_{zz}\frac{M_{z,Ed}}{M_{c,z,Rd}} \leq 1$$ [Clause 6.3.3, Eq. 6.62]

$$= \frac{589}{774} + 1.0 \times \frac{11.1}{127.1} + 1.5 \times \frac{0.3}{63.5} \leq 1$$ (Refer to NCCI SN048b)

Use $k_{zy} = 1.0$ and $k_{zz} = 1.5$ for columns in simple construction

$= 0.76 + 0.09 + 0.01$

$= 0.86 \leq 1.0$

$\therefore$ The member resistance of Column C12 under combined bending and axial compression is adequate.

Therefore, 203 x 203 x 46 kg/m H-section S275 steel satisfies the design.

Part II Member design

Worked Example II-5b Column in simple construction

In Worked Example II-5a, the factors k_{zy} and k_{zz} can be alternatively calculated according to Annex A in EN 1993-1-1 as follow:

$$N_{Rk} = Af_y = 5{,}870 \times 275 \times 10^{-3} = 1{,}614\ \text{kN}$$

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr,y}^{\ 2}} = \frac{\pi^2 \times 210 \times 10^3 \times 4{,}568 \times 10^4}{5{,}000^2} \times 10^{-3} = 3{,}787\ \text{kN}$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr,z}^{\ 2}} = \frac{\pi^2 \times 210 \times 10^3 \times 1{,}548 \times 10^4}{5{,}000^2} \times 10^{-3} = 1{,}283\ \text{kN}$$

$$\bar{\lambda}_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \sqrt{\frac{1{,}614}{3{,}787}} = 0.65$$

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{cr,z}}} = \sqrt{\frac{1{,}614}{1{,}283}} = 1.12$$

For a H-section (with $h/b \le 1.2$) and $t_f \le 100\ \text{mm}$, use curve ‘b’ for buckling about y-y axis, and hence, $\alpha = 0.34$.

$$\Phi_y = 0.5\left[1 + \alpha\left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^{\ 2}\right] = 0.5\left[1 + 0.34 \times (0.65 - 0.2) + 0.65^2\right] = 0.79$$

For a H-section (with $h/b \le 1.2$) and $t_f \le 100\ \text{mm}$, use curve ‘c’ for buckling about z-z axis, and hence, $\alpha = 0.49$.

$$\Phi_z = 0.5\left[1 + \alpha\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^{\ 2}\right] = 0.5\left[1 + 0.49 \times (1.12 - 0.2) + 1.12^2\right] = 1.35$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^{\ 2} - \bar{\lambda}_y^{\ 2}}} = \frac{1}{0.79 + \sqrt{0.79^2 - 0.65^2}} = 0.81$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^{\ 2} - \bar{\lambda}_z^{\ 2}}} = \frac{1}{1.35 + \sqrt{1.35^2 - 1.12^2}} = 0.48$$

For double symmetric H-section with $y_o = z_o = 0$

$$i_o{}^2 = i_y{}^2 + i_z{}^2 + y_o{}^2 + z_o{}^2$$ (Refer to EN 1993-1-3, Eq. 6.33b)

$$= \frac{I_y}{A} + \frac{I_z}{A} = \frac{4{,}570 \times 10^4}{5{,}870} + \frac{1{,}550 \times 10^4}{5{,}870} = 10{,}426\ \text{mm}^2$$

$$N_{cr,T} = \frac{1}{i_o{}^2}\left(GI_T + \frac{\pi^2 EI_w}{L_{cr,T}{}^2}\right)$$ (Refer to EN 1993-1-3, Eq. 6.33a)

$$= \frac{1}{10{,}426}\left(81 \times 10^3 \times 22.2 \times 10^4 + \frac{\pi^2 \times 210 \times 10^3 \times 0.143 \times 10^{12}}{2{,}500^2}\right) \times 10^{-3}\ \text{kN}$$
$$= 6{,}273\ \text{kN}$$

$$\beta = 1 - \left(\frac{y_o}{i_o}\right)^2 = 1$$ (Refer to EN 1993-1-3, Eq. 6.35)

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_o}{i_o}\right)^2\left(\frac{N_{cr,T}}{N_{cr,y}}\right)}\right]$$ (Refer to EN 1993-1-3, Eq. 6.35)

$$= \frac{1}{2 \times 1}\left[N_{cr,y} + N_{cr,T} - \left(N_{cr,y} - N_{cr,T}\right)\right]$$
$$= N_{cr,T}$$
$$= 6{,}273\ \text{kN}$$

$$C_{my,0} = 0.79 + 0.21\Psi_y + 0.36(\Psi_y - 0.33)\frac{N_{Ed}}{N_{cr,y}}$$

$$= 0.79 + 0.21 \times 0 + 0.36 \times (0 - 0.33)\frac{589}{3{,}787} = 0.77$$

$$C_{mz,0} = 0.79 + 0.21\Psi_z + 0.36(\Psi_z - 0.33)\frac{N_{Ed}}{N_{cr,z}}$$

$$= 0.79 + 0.21 \times 0 + 0.36 \times (0 - 0.33)\frac{589}{1{,}283} = 0.74$$

$$\mu_y = \frac{1 - \dfrac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \dfrac{N_{Ed}}{N_{cr,y}}} = \frac{1 - \dfrac{589}{3{,}787}}{1 - 0.81 \times \dfrac{589}{3{,}787}} = 0.97$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_z \frac{N_{Ed}}{N_{cr,z}}} = \frac{1 - \frac{589}{1{,}283}}{1 - 0.48 \times \frac{589}{1{,}283}} = 0.69$$

$$\because w_y = \frac{W_{pl,y}}{W_{el,y}} = \frac{497}{450} = 1.10 < 1.5$$

$$\therefore w_y = 1.10$$

$$\because w_z = \frac{W_{pl,z}}{W_{el,z}} = \frac{231}{152} = 1.52 > 1.50$$

$$\therefore w_z = 1.50$$

$$n_{pl} = \frac{N_{Ed}}{N_{Rk} / \gamma_{M1}} = \frac{589}{1{,}614 / 1.00} = 0.36$$

$$a_{LT} = 1 - \frac{I_T}{I_y} = 1 - \frac{22.2}{4{,}570} = 1.00$$

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}} \frac{A}{W_{el,y}} = \frac{11.1 \times 10^6}{589 \times 10^3} \times \frac{5{,}870}{450 \times 10^3} = 0.25$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{{L_{cr,z}}^2} \left\{ \left[\frac{I_w}{I_z} + \frac{{L_{cr,z}}^2 GI_T}{\pi^2 EI_z} + (C_2 z_g)^2 \right]^{0.5} - (C_2 z_g) \right\}$$ (Refer to NCCI SN003a)

Since $\overline{\lambda_0}$ is the non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, $C_1 = 1.00$ and $C_2 z_g = 0$

$$\begin{aligned} M_{cr} &= 1.00 \times \frac{\pi^2 \times 210 \times 10^3 \times 1{,}550 \times 10^4}{5{,}000^2} \times \left[\frac{0.143 \times 10^{12}}{1{,}550 \times 10^4} + \frac{5{,}000^2 \times 81 \times 10^3 \times 22.2 \times 10^4}{\pi^2 \times 210 \times 10^3 \times 1{,}550 \times 10^4} \right]^{0.5} \times 10^{-6} \text{ kNm} \\ &= 1.00 \times 1{,}285{,}022 \times (9{,}226 + 13{,}994)^{0.5} \times 10^{-6} \text{ kNm} \\ &= 195.8 \text{ kNm} \end{aligned}$$

$$\overline{\lambda_0} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{497 \times 275 \times 10^{-3}}{195.8}} = 0.84$$

$$\because \overline{\lambda}_0 = 0.84$$

$$> 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)} = 0.2\sqrt{1.77}\sqrt[4]{\left(1-\frac{589}{1,283}\right)\left(1-\frac{589}{6,273}\right)} = 0.22$$

$$\therefore C_{my} = C_{my,0} + \left(1 - C_{my,0}\right)\frac{\sqrt{\varepsilon_y}a_{LT}}{1+\sqrt{\varepsilon_y}a_{LT}} = 0.77 + \left(1-0.77\right)\frac{\sqrt{0.25}\times 1}{1+\sqrt{0.25}\times 1} = 0.85$$

$$C_{mz} = C_{mz,0} = 0.74$$

$$\because C_{mLT} = C_{my}{}^{2}\frac{a_{LT}}{\sqrt{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}} = 0.85^2 \times \frac{1.00}{\sqrt{\left(1-\frac{589}{1,283}\right)\left(1-\frac{589}{6,273}\right)}} = 1.03 > 1$$

$$\therefore C_{mLT} = 1.03$$

$\chi_{LT} = 0.93$ (Worked Example II-5a)

$$\lambda_{max} = \max\begin{cases}\overline{\lambda}_y \\ \overline{\lambda}_z\end{cases} = \max\begin{cases}0.65 \\ 1.12\end{cases} = 1.12$$

$$d_{LT} = 2a_{LT}\frac{\overline{\lambda}_0}{0.1+\overline{\lambda}_z{}^4}\frac{M_{y,Ed}}{C_{my}\chi_{LT}M_{pl,y,Rd}}\frac{M_{z,Ed}}{C_{mz}M_{pl,z,Rd}}$$
$$= 2\times 1\times\frac{0.84}{0.1+1.12^4}\frac{11.1}{0.85\times 0.93\times 497\times 275\times 10^{-3}}\frac{0.3}{0.74\times 231\times 275\times 10^{-3}}$$
$$= 0.00066$$

$$\because C_{zy} = 1+\left(w_y - 1\right)\left[\left(2-14\frac{C_{my}{}^{2}\overline{\lambda}_{max}{}^{2}}{w_y{}^{5}}\right)n_{pl} - d_{LT}\right]$$
$$= 1+\left(1.10-1\right)\left[\left(2-14\times\frac{0.85^2\times 1.12^2}{1.10^5}\right)\times 0.36 - 0.00066\right] = 0.79$$
$$> 0.6\sqrt{\frac{w_y}{w_z}\frac{W_{el,y}}{W_{pl,y}}} = 0.6\sqrt{\frac{1.10}{1.50}\frac{450}{497}} = 0.47$$

$$\therefore C_{zy} = 0.79$$

$$e_{LT} = 1.7a_{LT}\frac{\overline{\lambda}_0}{0.1+\overline{\lambda}_z{}^4}\frac{M_{y,Ed}}{C_{my}\chi_{LT}M_{pl,y,Rd}}$$
$$= 1.7\times 1\times\frac{0.84}{0.1+1.12^4}\frac{11.1}{0.85\times 0.93\times 497\times 275\times 10^{-3}} = 0.09$$

$$\because C_{zz} = 1 + (w_z - 1)\left(2 - \frac{1.6}{1.5} C_{mz}^2 \bar{\lambda}_{max} - \frac{1.6}{1.5} C_{mz}^2 \bar{\lambda}_{max}^2 - e_{LT}\right) n_{pl}$$

$$= 1 + (1.5 - 1)\left[\left(2 - \tfrac{1.6}{1.5} \times 0.74^2 \ \times 1.12 - \tfrac{1.6}{1.5} \times 0.74^2 \ \times 1.12^2 - 0.09\right) \times 0.36\right]$$

$$= 1.09$$

$$> \frac{W_{el,z}}{W_{pl,z}} = \frac{152}{231} = 0.66$$

$$\therefore C_{zz} = 1.09$$

$$k_{zy} = C_{my} C_{mLT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,y}}} \frac{1}{C_{zy}} 0.6 \sqrt{\frac{w_y}{w_z}} = 0.85 \times 1.03 \times \frac{0.69}{1 - \frac{589}{3{,}787}} \times \frac{1}{0.79} \times 0.6 \times \sqrt{\frac{1.10}{1.50}} = 0.47$$

$$k_{zz} = C_{mz} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,z}}} \frac{1}{C_{zz}} = 0.74 \frac{0.69}{1 - \frac{589}{1{,}283}} \frac{1}{1.07} = 0.88$$

Part II Member design

Worked Example II-5c Column in simple construction

In Worked Example II-5a, the factors k_{zy} and k_{zz} can be alternatively calculated according to Annex B in BS EN 1993-1-1 as follow:

Since H-section is not susceptible to torsional deformation, use Table B.1.

$$\because \Psi_y = \Psi_z = \Psi_{LT} = 0$$

$$\therefore C_{my} = C_{mz} = C_{mLT} = 0.6$$

As determined from Worked Example II-5a,

$$N_{Rk} = 1{,}614\,kN \qquad N_{cr,y} = 3{,}787\,kN \qquad N_{cr,z} = 1{,}283\,kN$$

$$\overline{\lambda}_y = 0.65 \qquad \overline{\lambda}_z = 1.12 \qquad \chi_y = 0.81 \qquad \chi_z = 0.48$$

$$\because k_{yy} = C_{my}\left(1+\left(\overline{\lambda}_y - 0.2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0.6\left(1+\left(0.65-0.2\right)\frac{589}{0.81\times 1614/1.00}\right) = 0.72$$

$$< C_{my}\left(1+0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) = 0.6\left(1+0.8\frac{589}{0.81\times 1614/1.00}\right) = 0.82$$

$$\therefore k_{yy} = 0.72$$

$$k_{zy} = 0.6k_{yy} = 0.6\times 0.72 = 0.43$$

$$\because k_{zz} = C_{mz}\left(1+\left(2\overline{\lambda}_z - 0.6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) = 0.6\left(1+\left(2\times 1.12-0.6\right)\frac{589}{0.48\times 1614/1.00}\right) = 1.35$$

$$> C_{mz}\left(1+1.4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) = 0.6\left(1+1.4\times\frac{589}{0.48\times 1614/1.00}\right) = 1.24$$

$$\therefore k_{zz} = 1.24$$

The factors k_{zy} and k_{zz} according to different method are summarized in the table below:

	BS EN 1993-1-1 Annex A	BS EN 1993-1-1 Annex B	NCCI SN048b
k_{zy}	0.47	0.43	1.0
k_{zz}	0.88	1.24	1.5

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