STAINLESS STEEL BRIDGE DESIGN MANUAL

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Prepared by:
Dr Nirmalya Bandyopadhyay,
Former Director, STUP Consultants Pvt Ltd

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FOREWORD

Prem Krishna
Former Professor & Head of Civil Engineering
University of Roorkee (now IIT Roorkee)

Structural steel, or carbon steel as it is often called, followed the use of cast iron and wrought iron in structural applications, sometime in the later part of the nineteenth century. This has since been the mainstay of applications such as in the industry and the railways and almost entirely for the longer-span applications of the modern era. Amongst the many merits of steel as a structural material are its high strength-to-weight ratio for larger structures, recyclability, better-defined mechanical properties, and so on.

The major non-metallic materials, namely, plain, reinforced and prestressed concrete entered the fray in the first half of the twentieth century, and, there was a certain period during which steel was pushed onto the back foot due to the advent of prestressed concrete particularly, which overcame some limitations of reinforced concrete too. As a teacher of 'Design of Steel Structures' to students of the Bachelor of Civil Engineering class in the 1960s at the University of Roorkee (now IIT Roorkee), I recall a somewhat disappointing scenario wherein the application of steel structures did not seem to be growing. That, however, tended to be a temporary phase, since steel began to experience a renaissance soon in the following years.

Driven by the enormous need to add civil infrastructure the world over, the last 40–50 years have experienced extraordinary developments in concretes, steels and composites. This trend continues to accentuate to meet the demands of sustainability and resilience to environmental challenges. These developments have also brought to the fore, during the last couple of decades, the idea of deploying stainless steel in structural applications. There is a wide range of stainless steels available, and, chosen appropriately for structural applications stainless steel has mechanical properties reasonably well matched with those of carbon steel. It has the added merits of being almost maintenance-free, greatly corrosion resistant, a high degree of recyclability, and much superior aesthetic appeal.

Internationally, there are a few standards/guidelines for the use of stainless steel in structural applications. These are AISC (American Institute of Steel Construction) Design Guide 27; Eurocode 3 – Design of Steel Structures – Part 1-4: General Rules – Supplementary Rules for Stainless Steel; AS/NZS 4673 – Cold–formed Stainless Steel Structures: Australian and New Zealand Standard; Design Manual for Structural Stainless Steel 4th Edition: SCI Publication P413 – Published by Steel Construction Institute, UK.

There is, however, no Indian standard or guideline yet on the subject to guide designers to utilise stainless steel which it merits. Indian Stainless Steel Development Association (ISSDA) has, thus, taken a commendable initiative to bring out the Stainless Steel Bridge Design Manual. The guidelines by and large cover stainless steel, like what the Indian Standard 800 does for conventional steels, and, go beyond in addressing such issues as life cycle costing and include an illustrative example. The document is well-edited and composed in a designer-friendly manner.

I fervently hope that the publication of this document will encourage the appropriate use of stainless steel in structural applications and thus add the material to designers' repertoire.

PREFACE

Rajamani Krishnamurti
President
Indian Stainless Steel Development Association
(ISSDA)

Welcome to the unveiling of our latest endeavour - the design manual for stainless steel structures, proudly presented by the Indian Stainless Steel Development Association. Within these pages lies a culmination of decades of innovation, dedication, and expertise, aimed at showcasing the unparalleled qualities of stainless steel and empowering professionals to unlock its full potential.

Stainless steel stands as a beacon of excellence in both architectural and engineering realms, embodying traits of design, resilience, and sustainability that are un-rivalled. From towering skyscrapers to intricate sculptures, its influence on the built environment is undeniable, transcending mere functionality to become synonymous with artistry and progress.

This manual serves as a compendium of knowledge, meticulously curated to cover everything from the fundamental properties of stainless steel to the most advanced design principles. Drawing upon the insights of industry leaders and researchers, we have ensured that each page adheres to the highest standards of accuracy and relevance.

As you navigate through these chapters, you will not only gain a deeper understanding of the technical intricacies of stainless steel design, but also its creative and sustainable dimensions. Stainless steel structures, we believe, are not just utilitarian but canvases for innovation and symbols of a greener, more responsible future.

Our heartfelt appreciation goes out to all those whose contributions have made this manual possible - from the professionals who shared their expertise to the researchers whose studies underpin its contents. Together, they have enriched this resource, ensuring its value for architects, engineers, designers, and students alike.

We are confident that this manual will serve as an indispensable companion for anyone venturing into the realm of stainless steel design, empowering them to push boundaries and create structures that inspire awe and admiration. Stainless steel, after all, is more than just a material - it is a medium for innovation, a testament to longevity, and a harbinger of sustainable design.

On behalf of the Indian Stainless Steel Development Association (ISSDA), we extend our warmest wishes to all those embarking on this journey through the realm of stainless steel structures. May this manual ignite your imagination, guide your endeavours, and pave the way for designs that leave an indelible mark on the world.

MESSAGE

V K Pandey
Executive Director
Salem Steel Plant
Steel Authority of India Ltd

We cannot imagine a world today without Steel, Steel is one of the most versatile engineering materials, being extensively used in almost all area of our day to day existence. Within the realm of world of Steel, Stainless Steel adds properties of Anticorrosion, Hygienic, and higher weight to strength ratio, Good Energy absorbing properties, better high temperature resistance and aesthetics as it is available in variety of surface finishes.

It has been seen in past that as countries graduate from Low-income economies to Middle / higher income economies, the use of Stainless Steel increases exponentially. With higher disposable income, investments are made with a view of higher life of structure and better aesthetics. Stainless Steel perfectly matches with these requirements.

My heartfelt congratulations to ISSDA for bringing out Design Handbook for Structural Stainless Steel at most appropriate time. It was one of the most awaited and timely compilation which will prove to be extremely useful for all Structural designers, Architects, Academicians and Students of Engineering. It will greatly help in bringing awareness about advantages of designing structures on the basis of Life Cycle Cost using Stainless Steel. With our long coastal area, use of Stainless Steel will immensely benefit in mitigating losses on account of corrosion, increasing safety of structures and general public.

On behalf of Steel Authority of India Ltd, we extend our warmest wishes to all and hope that the Design handbook for Stainless Steel structures helps in designing New Age Cost effective, innovative, Hygienic, aesthetic and Safe Structures in years to come.

MESSAGE

Anshul Gupta
Former General Manager, Northeast Frontier Railway And Advisor, Innovations and International Business,
RailTel Corporation of India Limited,
Ministry of Railways

It is a matter of great pleasure and pride that Indian Stainless Steel Development Association (ISSDA), a non-profit organization, which since its inception has been at forefront to promote the usage of Stainless Steel in India, is coming out with a 'Manual on the usage of Stainless Steel in Structures'. Though use of Stainless Steel is very common in various applications for more than 100 years, but it is only now that its application in Bridges, Tunnels and other heavy structures is gaining recognition.

The use of stainless steel as standard material for making train coaches on Indian Railways, Underground Metros, light monorail train services has been well accepted due to its high impact strength to avoid crumbling, corrosion resistance, fire safety, ease of cleaning to maintain hygiene and visual aesthetics. On Railways, use of Stainless Steel for the construction of Bridge and other heavy structures is yet to take of in a big way. One reason has been non-availability of a National Standard for use of stainless steel in structural applications in India. This effort of ISSDA, as the nodal organization for taking up the task of filling the gap by drafting the design guideline till such time a National Design Code is published, is highly praiseworthy and shall definitely help Indian Railways and other organizations to adopt Stainless Steel in bridges and other structures. I am sure this effort shall lead to wider acceptance of Stainless use as a preferred material in Bridge construction leading to impressive strength to weight ratio, higher Life expectation of 120 years and thus optimal lifetime cost.

MESSAGE

Tarun Kumar Khulbe
Chief Executive Officer
Jindal Stainless Limited

Stainless steel is revolutionising infrastructure development in India, offering unparalleled durability and resilience in challenging environments. It is increasingly becoming a prime choice for sustainable and long-lasting infrastructure projects across the country. However, there's a dearth of published guidelines in India for the appropriate application of stainless steel in the construction of critical infrastructure like bridges.

I would like to commend ISSDA for identifying this need and contributing towards developing this comprehensive design guideline on the usage of stainless steel in constructing bridges. This guide will help readers understand stainless steel's numerous advantages including its superior strength; its innate corrosion resistance, which reduces the downtime and frequency of maintenance; its enhanced ductility for seismic conditions; and reduced embodied/operational carbon emissions. The use of stainless steel inches us closer to carbon neutrality while constructing bridges.

Although internationally, several standards and guidelines have been published to help engineers leverage the benefits of stainless steel in structural applications, including AISC Design Guide 27, Euro Codes 1993-1 to 4, and SCI Publication P413; I am certain that the information made available in this guidebook will aid engineers and academics transform the infrastructure landscape in India, advancing our country's engineering and technological capabilities.

The mechanical behaviour of stainless steel is similar to carbon steel, though there are some dissimilarities. There are several grades of stainless steel and not all of them are suitable for bridge construction and can withstand prevalent environmental conditions. Moreover, the stress-strain curve of stainless steel isn't strictly bilinear like carbon steel. Its coefficient of linear expansion and thermal conductivity characteristics are unique. Hence, there's an innate need to understand the variables and suitable specifications while using stainless steel in constructing bridges. This guidebook aims to 'bridge' that knowledge gap.

As India continues to innovate and set global benchmarks in infrastructure development, I believe that the publishing of a stainless steel handbook such as this will act as a catalyst for research and innovation in bridge engineering in our country. I am confident that through collaborative efforts involving academia, industry experts, and governmental bodies, we can achieve breakthroughs in stainless steel technology, pushing the boundaries of bridge design and construction in India.

EDITORIAL BOARD

We acknowledge the contribution of the following individuals in the preparation of this document.

- Mr Amitabha Ghoshal, Former Vice President STUP Consultants Pvt Ltd – Mentor
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GUIDELINES FOR DESIGN OF STAINLESS STEEL BRIDGES

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1. INTRODUCTION

Indian Stainless Steel Development Association (ISSDA) is a non-profit organization, founded in 1989 by leading Stainless Steel manufacturers to promote the usage of Stainless Steel in India.

The primary aim of the Indian Stainless Steel Development Association (ISSDA) is to promote and advance the stainless steel industry in India by

- Advocacy & Policy Engagement: We advocate for interests of stainless steel industry by engaging with policymakers, regulatory bodies and government authorities to provide input on policies, standards and regulations related to the stainless steel industry.

- Promotion: The association actively promotes the use of stainless steel across different sectors and industries. We conduct awareness campaigns, seminars, workshops and educational initiatives to inform businesses, engineers, architects and public about the benefit of stainless steel.

- R&D Initiatives: We support research and development initiatives related to stainless steel. It collaborates with research institutions, universities and industry experts to drive innovation in stainless steel technology, applications and processes.

- Quality Standards: ISSDA plays a significant role in establishing and maintaining quality standards foe stainless steel products. We assist in developing testing and certification procedures to ensure that stainless steel products meet industry standards and specifications.

- Technical Guidance & Information Dissemination: By technical guidance and expertise to Engineers, Designers and Fabricators working with stainless steel. This includes information on material selection, fabrication techniques and best practices by offering resources, publications and data to industry professionals and public.

- Market Development: The association explores and develops new markets and applications for stainless steel, identifying emerging trends and industries where stainless steel can be utilized effectively.

- Sustainability: ISSDA promotes the sustainability aspects of stainless steel, emphasizing its recyclability, long life cycle and environmental benefits. We encourage responsible manufacturing and usage of stainless steel.

Although Stainless Steel has been used for more than a hundred years, it is only recently its application in Bridges and other heavy structures is gaining recognition. Stainless Steel, an iron alloy with minimum 10.5% Chromium, presents:

- High Strength

- Corrosion Resistance
- Enhanced Ductility
- Ease of Fabrication
- Infinitely Recyclable
- Negligible to Minimal Maintenance

Although mechanical behaviour of Stainless Steel is similar to Carbon Steel, there are some dissimilarities. There are multitude of grades of Stainless Steel and not all of them are suitable for Bridge construction. The stress-strain curve of Stainless Steel is strictly not bilinear like carbon steel. The coefficient of linear expansion and thermal conductivity characteristics are often different from that of carbon steel.

Internationally, several standards and guidelines have been published to facilitate Structural Engineer's utilise Stainless Steel in structural application as follows:

- AISC (American Institute of Steel Construction) Design Guide 27
- Eurocode 3 – Design of Steel Structures – Part 1-4: General Rules – Supplementary Rules for Stainless Steel
- AS/NZS 4673 – Cold –formed Stainless Steel Structures: Australian and New Zealand standard
- Design Manual for Structural Stainless Steel 4th Edition: SCI Publication P413 – Published by Steel Construction Institute, UK

However, there is yet to be a standard formulated for use of stainless steel in structural applications in India. Therefore, ISSDA as the nodal organization has taken up the task of filling the gap by drafting a design guideline till such time a National Design Code is published. This document is the result of such effort. The infrastructure builders and their designers have been feeling the need for a design guideline for Stainless Steel Structures.

With Design Life expectation of 120 years for structures like bridges, the lifetime cost of such structures with Stainless Steel often work out cheaper particularly in a corrosive environment where traditional carbon steel with coating do not perform admirably.

This document is based in IRC:24 – Steel Bridge Design Code to enable ease of adoption.

2. GRADES AND PROPERTIES OF STAINLESS STEEL

2.1 Different Grades of Stainless Steel and their Application

Stainless steel is the name given to a family of corrosion and heat resistant alloy steels containing a minimum of 10.5% of chromium. In conjunction with low carbon content, chromium imparts remarkable resistance to corrosion and heat.

Just as there are various types of structural and engineering carbon steels meeting different strength, weldability and toughness requirements, there is also a variety of stainless steels with varying levels of corrosion resistance and strength. This array of stainless steel properties is the result of controlled alloying element additions, each affecting mechanical properties and the ability to resist different corrosive environments. It is important to select a stainless steel which is adequate for the application without being overly alloyed and costly.

When an alloy of steel containing chromium in excess of 10.5%, is exposed to air or any other oxidizing environment, a transparent and tightly adherent layer of chromium-rich oxide forms spontaneously on the surface of the steel alloy. If scratching or cutting damages the film, it reforms immediately in the presence of oxygen. Although the film is very thin, about **5×10^{-6} mm**, it is both stable and nonporous. As long as the stainless steel is corrosion resistant enough for the service environment, it will not react further with the atmosphere. For this reason, it is called a passive film. The stability of this passive layer depends on the composition of the stainless steel, its surface treatment and the corrosiveness of its environment. Its stability increases as the chromium content increases and is further enhanced by additions of other elements, such as nickel, molybdenum, titanium, aluminum, niobium, copper, nitrogen, sulphur, phosphorus or selenium, to increase corrosion resistance to specific environments, enhance oxidation resistance, and impart special characteristics.

Stainless steels can be classified into the following five basic types, with each possessing unique properties and a range of different corrosion resistance levels.

Austenitic Type

The most widely used austenitic stainless steels contain 17% to 18% chromium and 8% to11% nickel additions. In comparison to structural carbon steels, which have a body-centred cubic (BCC) atomic (crystal) structure, austenitic stainless steels have a face-centred cubic (FCC) atomic structure. As a result, austenitic stainless steels, in addition to their corrosion resistance, have high ductility, are easily cold formed, and are readily weldable. Compared to structural carbon steels, they also have significantly better toughness over a wide range of temperatures. These steels can be strengthened by cold working, but not by heat treatment. The corrosion performance of austenitic stainless steels can be further enhanced by higher levels of chromium and additions of molybdenum and nitrogen. They are by far the most frequently used stainless steels in buildings and constructions.

Ferritic Type

The chromium content of the most popular ferritic stainless steels is between 10.5%and 18%. Ferritic stainless steels contain either no or very small nickel additions and their body-centred

(BCC) atomic structure is the same as that of structural carbon steels. They are generally less ductile and less weldable than austenitic stainless steels. The forming and machining properties of ferritic stainless steels are similar to those of E350 structural carbon steel. They can be strengthened by cold working, but to a lesser degree than the austenitic stainless steels. Like the austenitic grades, they cannot be strengthened by heat treatment. They have good resistance to stress corrosion cracking and their corrosion performance can be further enhanced by addition of molybdenum. They offer a corrosion resistant alternative to many light gauge galvanized steel applications. Ferritic grades are generally used in thickness gauges of 4 mm and below.

Duplex (austenitic-ferritic) Type

Duplex stainless steels have a mixed microstructure of austenite and ferrite, and so are sometimes called austenitic-ferritic steels. They typically contain 20% to 26% chromium, 1% to 8% nickel, 0.05% to 5% molybdenum, and 0.05% to 0.3% nitrogen. They are about twiceas strong as austenitic steels in the annealed condition which can make section size reduction possible, a property that can be very valuable in weight-sensitive structures like bridges. Duplex grades are suitable for a broad range of corrosive environments. Although duplex stainless steels have good ductility, their higher strength results in more restricted formability, compared to the austenitic alloys. They can also be strengthened by cold working, but not by heat treatment. They have good weldability and good resistance to stress corrosion cracking. This variety of stainless steel can be seen as being complementary to ferritic stainless steels, as they are more likely to be used in heavier gauges.

Martensitic Type

Martensitic stainless steels have a similar body-centred cubic (BCC) structure as ferritic stainless steel and structural carbon steels, but because of their higher carbon content, they can be strengthened by heat treatment. Martensitic stainless steels are generally used in a hardened and tempered condition, which gives them high strength, and provides moderate corrosion resistance. They are suitable for applications that take advantage of their wear and abrasion resistance and hardness, like cutlery, surgical instruments, industrial knives, wear plates and turbine blades. They are less ductile and more notch sensitive than the ferritic, the austenitic and the duplex stainless steels. Although most martensitic stainless steels can be welded, the process may require preheat and postweld heat treatment, which can limit their use in welded components

Precipitation Hardening Type

Precipitation hardening steels can be strengthened by heat treatment to very high strengths. These falls into three microstructure groups depending on the grade: martensitic, semi-austenitic and austenitic. These steels are not normally used in welded fabrications. Their corrosion resistance is generally better than the martensitic stainless steels and similar to the 18% chromium, 8% nickel austenitic stainless steels. Although they are mostly used in the aerospace industry, this grade of stainless steel is also used fortension bars, shafts, bolts and other applications requiring high strength and moderatecorrosion resistance.

2.2 Standards and Nomenclature for different grades of Stainless Steel

Indian Specification for Stainless Steel is IS:6911:2017. Extensive Specifications and Standards are available in both European standards as well as American Codes. Large number of primarily ASTM publications set forth standards for all types of Stainless Steels. UNS (unified numbering system) designation is an identification system for specific metals and alloys; stainless steel alloys are identified in the ASTM standards in accordance with ASTM E527 and SAE J1086. The grade designations in IS:6911 generally follow American standard.

The relevant standard in European code is EN 10088, Stainless Steels. It comprises five parts, of which three are relevant to construction applications.

- Part 1, Lists of stainless steels, shows the chemical compositions and reference data on some physical properties such as modulus of elasticity, E.
- Part 4, Technical delivery conditions for sheet/plate and strip of corrosion resisting steels for construction purposes, gives the technical properties and chemical compositions for the materials used in forming structural sections.
- Part 5, Technical delivery conditions for bars, rods, wire, sections and bright products of corrosion resisting steels for construction purposes, contains the technical properties and chemical compositions for the materials used in long products.

The following table presents a correlation between EN-10088 designations and US designations.

Steel Grade to EN-10088		**US**	
No	*Name*	*ASTM Type*	*UNS*
Austenitic			
1.4301	X5CrNi18-10	304	S30400
1.4306	X2CrNi19-11	304L	S30403
1.4307	X2CrNi18-9	304L	S30403
1.4311	X2CrNin18-10	304LN	S30453
1.4318	X2CrNiN18-7	301LN	S30153
1.4401	X5CrNiMo17-12-2	316	S31600
1.4404	X2CrNiMo17-12-2	316L	S31603
1.4406	X2CrNiMoN17-11-2	316LN	S31653
1.4429	X2CrNiMoN17-13-3	316LN	S31653
1.4432	X2CrNiMo17-12-3	316L	S31603
1.4435	X2CrNiMo18-14-3	316L	-
1.4439	X2CrNiMoN17-13-5	317LMN	S31726
1.4529	X1NiCrMoCuN25-20-7	-	N08926
1.4539	X1NiCrMoCu25-20-5	904 L	N08904
1.4541	X6CrNiTi18-10	321	S32100
1.4547	X1CrNiMoCuN20-18-7	-	S31254
1.4565	X2CrNiMnMoN25-18-6-5	-	S34565
1.4567 *	X3CrNiCu18–9-4		S30430
1.4571	X6CrNiMoTi17-12-2	316Ti	S31635
1.4578 *	X3CrNiCuMo17–11–3-2		-
Duplex			
1.4062 *	X2CrNiN22-2--		S32202
1.4162	X2CrMnNiN21-5-1		S32101

Steel Grade to EN 10088		**US**	
No	*Name*	*ASTM Type*	*UNS*
1.4362	X2CrNiN23-4	2304	S32304
1.4410	X2CrNiMoN25-7-4	2507	S32750
1.4462	X2CrNiMoN22-5-3	2205	S32205
1.4482 *	X2CrMnNiMoN21–5-3		-
1.4501 *	X2CrNiMoCuWN25–7-4		S32760
1.4507 *	X2CrNiMoCuWN25–7-4		S32520
1.4662 *	X2CrNiMnMoCuN24–4-3–2		S82441
Ferritic			
1.4003	X2CrNi12	-	S41003
1.4016	X6Cr17	430	S43000
1.4509	X2CrTiNb18	441	S43940
1.4512	X2CrTi12	409	S40900
1.4521	X2CrMoTi18-2	444	S44400
1.4621 *	X2CrNbCu21	-	S44500

Information on all the above steels are available in EN 10088-4/5 except for those marked with *, which are currently only in EN 10088-2/3.

In addition to IS-6911, RDSO, Indian Railways, have published a specification in the document: BS-S-7.5.3.1-9 [2020]: Specification for Higher Strength Martensitic Stainless Steel for Bridge and Associated Structural Application. It is designated as IRS 350 CR. Its yield strength and ultimate strength is 350 MPa and 485 MPa respectively. The specification lists compatible fasteners and welding consumables. This grade is practically identical to ASTM grade 50CR.

2.3 Grade Suitable for Structural Applications

Austenitic Type

Austenitic stainless steels are generally selected for structural applications which require a combination of good strength, corrosion resistance, formability (including the ability to make tighter bends), excellent field and shop weldability and, for seismic applications, very good elongation prior to fracture.

Grades 1.4301 (widely known as 304) and 1.4307 (304L) are the most commonly used standard austenitic stainless steels and contain 17.5 to 20% chromium and 8 to 11%nickel.

Grades 1.4401 (316) and 1.4404 (316L) contain about 16% to 18% chromium, 10 to 14%nickel and the addition of 2 to 3% molybdenum, which improves corrosion resistance. They will perform well in marine and industrial sites.

The “L” in the designation indicates a low carbon version with reduced risk of sensitisation (of chromium carbide precipitation) and of intergranular corrosion in heat affected zones of welds. Either the “L” grade, or stabilized steel such as grade 1.4541 and1.4571 should be specified for welded sections. Low carbon does not affect corrosion performance beyond the weld areas. When producers use state-of-the-art production methods, commercially produced stainless steels are often low carbon and dual certified to both designations (e.g. 1.4301/1.4307, with the higher strength of 1.4301and the lower carbon content of 1.4307). When less modern technology is used, this cannot be assumed and therefore the low carbon version should be explicitly specified in the documents of projects in which welding is involved.

Grade 1.4318 is a low carbon, high nitrogen stainless steel which work hardens very rapidly when cold worked. It has a long track record of satisfactory performance in the railcar industry and is equally suitable for automotive, aircraft and architectural applications. This has similar corrosion resistance as1.4301 and is most suitable for applications requiring higher strength than 1.4301 where large volumes are concerned.

Duplex Type

Duplex stainless steels are appropriate where high strength, corrosion resistance and/or higher levels of crevice and stress corrosion cracking resistance are required. Grade1.4462 (2205) is an extremely corrosion resistant duplex grade, suitable for use in marine andother aggressive environments. An increasing use of stainless steels for load-bearing applications has led to increasing demand for duplex steels and development of new “lean” duplex grades. These grades are described as lean owing the reduced alloy contents of nickel and molybdenum. Lean grades have comparable mechanical properties of grade1.4462 and a corrosion resistance which is comparable to the standard austenitic grades. This makes them appropriate for use in many onshore exposure conditions.

Ferritic Type

The two “standard” ferritic grades which are suitable for structural applications and commonly available are 1.4003 (a basic ferritic grade containing about 11% chromium) and 1.4016 (containing about 16.5% chromium, with greater resistance to corrosion than 1.4003). Welding impairs the corrosion resistance and toughness of grade 1.4016 (430) substantially.

Modern stabilised ferritic grades, for example 1.4509 (441) and 1.4521(444), contain additional alloying elements such as niobium and titanium which lead to significantly improved welding and forming characteristics. Grade 1.4521 contains 2% molybdenum which improves pitting and crevice corrosion resistance in chloride containing environments (it has similar pitting corrosion resistance to 1.4401). 1.4621 is a recently developed ferritic grade that contains around 20% chromium, with improved polishability compared to 1.4509 and 1.4521.

Applications in the construction industry

Stainless steels have been used in construction ever since they were invented over one hundred years ago. Stainless steel products are attractive and corrosion resistant with low maintenance requirements and have good strength, toughness and fatigue properties. Stainless steels can be fabricated using a range of engineering techniques and are fully recyclable at end-of-life. They are the material of choice for applications in structures situated in aggressive environments including buildings and structures in coastal areas, exposed to de-icing salts and in polluted locations. The high ductility of stainless steel is a useful property where resistance to seismic loading is required since greater energy dissipation is possible. However, seismic applications are outside the scope of this handbook.

Typical applications for austenitic and duplex grades include:

Primary beams and columns, pins, barriers, railings, cable sheathing and expansion joints in bridges

- Beams, columns, platforms and supports in processing plant for the water treatment, pulp and paper, nuclear, biomass, chemical, pharmaceutical, and food and beverage industries
- Seawalls, piers and other coastal structures
- Reinforcing bar in concrete structures
- Curtain walling, roofing, canopies, tunnel lining
- Support systems for curtain walling, masonry, tunnel lining etc.
- Security barriers, hand railing, street furniture
- Fasteners and anchoring systems in wood, stone, masonry or rock
- Structural members and fasteners in swimming pool buildings (special precautions should be taken for structural components in swimming pool atmospheres due to the risk of stress corrosion cracking in areas where condensates may form
- Explosion- and impact- resistant structures such as security walls, gates and bollards
- Fire and Explosion resistant walls, cable ladders and walkways on offshore platforms

Ferritic grades are used for cladding and roofing buildings, as well as for solar water heaters and potable water pipes. They are also used for indoor applications such as elevators and escalators. In the transportation sector, they are used for load-bearing members, such as tubular bus frames. Ferritic type of stainless steel also have a good track record of usage in railway wagons carrying coal, where wet sliding abrasion resistance is important. Although currently they are not widely used for structural members in the construction industry, they have the potential for greater application for strong and moderately durable structural elements with attractive metallic surface. For composite structures where a long service life is required, or where the environmental conditions are moderately corrosive, ferritic decking may provide a more economically viable solution than galvanized decking which may struggle to retain adequate durability for periods greater than 25 years. In addition to composite floor systems, other potential applications where ferritic stainless steel is a suitable substitute for galvanized steel include permanent formwork, roof purlins and supports to services such as cable trays. They could also be used economically in semi-enclosed unheated environments (e.g. railways, grandstands, bicycle sheds) and in cladding support systems, windposts and for masonry supports.

Generally, the following grades of stainless steel may be considered for construction of bridge and related structures. The selection has to be based on strength requirement as well as the environment in which the structure will be built. However, a BIS committee is in the process of drafting a code for specification of stainless steel grades that can be adopted in bridge structures and once the Indian Standard document is published the selection of stainless steel grade has to be done accordingly.

Sl No.	Grade	Other designations	Proof Stress (N/mm^2)	Ultimate Tensile Strength (N/mm^2)	% EL	Coeff of Thermal Expansion (20 to 100 ^0C)
1	IRS 350 CR	X02Cr12Mo, 50CR	350	485	18	10.4×10^{-6}
2	IRS 450 CR		450	550	18	10.8×10^{-6}
3	Duplex 2205	X2CrNiMoN22-5-3 , 1.4462, S32205	450	655	25	13×10^{-6}
4	304	X5CrNi18-10, 1.4301, S30400	205	515	40	16.0×10^{-6}
5	316	X5CrNiMo17-12-2, 1.4401, S31600,	205	515	40	15.9×10^{-6}

2.4 Life cycle costing and environmental impact

There is increasing awareness that life cycle (or whole life) costs, not just initial costs, should be considered when selecting materials. Experience shows that using a corrosion resistant material in order to avoid future maintenance, downtime and replacement can be a more cost-effective solution, even though the initial material costs are higher.

Life cycle costs take account of:

- initial costs,
- maintenance costs,
- diversion from landfills and recycled content,
- service life and environment.

The initial raw material cost of a structural stainless steel product is considerably higher than that of an equivalent carbon steel product, depending on the grade of stainless steel. However, there can be initial cost savings associated with eliminating corrosion resistant coatings. Utilising high strength stainless steels may reduce material requirements by decreasing section size and overall structure weight which cuts initial costs. Additionally, eliminating the need for coating maintenance or component replacement due to corrosion can lead to significant long-term maintenance cost savings.

The excellent corrosion resistance of stainless steel offers reduced inspection frequency and costs, reduced maintenance costs and long service life.

Stainless steel has a high residual scrap value (i.e. value at the end of a structure's life), though this is rarely a deciding factor for a structure with a long projected life (for instance over 50 years). However, because of the high residual scrap value, scrap is diverted from landfills and recycled into new metal and end-of-life (EOL) recycling rates are very high. Stainless steel producers use as much scrap as is available, but the material's overall average 20 to 30 year service life limits scrap availability. Typical recycled content for all types of stainless steel is at least 60%. Stainless steel is 100% recyclable and can be indefinitely recycled into new high quality stainless steel.

Life cycle costing uses the standard accountancy principle of discounted cash flow to reduce all those costs to present day values. The discount rate encompasses inflation, bank interest rates, taxes and, possibly, a risk factor. This allows a realistic comparison to be made of the options available and the potential long term benefits of using stainless steel to be assessed against other material selections.

2.5 Mechanical Properties of Stainless Steel

2.5.1 Basic stress-strain behavior

The stress strain behaviour of stainless steels differs from that of carbon steels in a number of respects. The most important difference is in the shape of the stress strain curve. Whereas carbon steel typically exhibits linear elastic behaviour up to the yield strength and a plateau before strain hardening is encountered, stainless steel has a more rounded response with no well defined yield strength. Fig 2.1 compares the stress-strain characteristics of various stainless steels with carbon steels for strains up to 0.75%. The stress-strain curves shown in the figure are representative of the range of material. Stainless steel "yield" strengths are generally quoted in terms of a proof strength defined for a particular offset permanent strain (conventionally the 0.2% strain). The proportional limit of stainless steels ranges from 40 to 70% of the 0.2% proof strength.

The response of ferritic stainless steel lies somewhere between that of carbon steel and austenitic stainless steel in that it is not quite as rounded or nonlinear as the austenitic grades

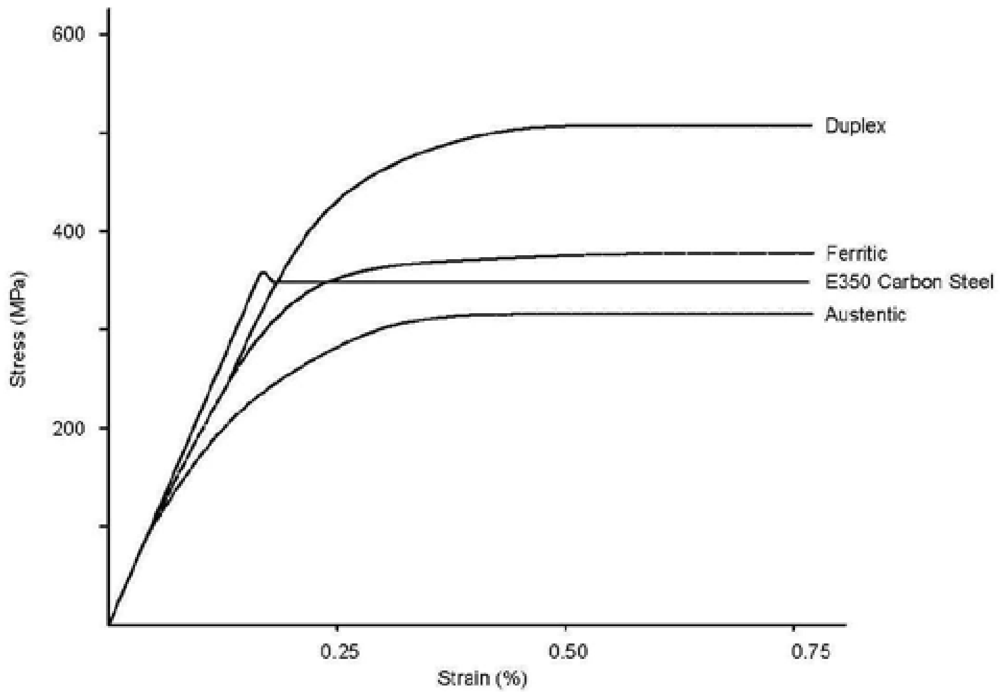


Figure 2.1 : Typical Stress-Strain diagram

but offers more strength than carbon steel. Stainless steels can absorb considerable impact without fracturing due to their excellent ductility (especially the austenitic grades) and their strain hardening characteristics.

2.5.2 Factors Affecting Stress-Strain Behavior

Compared to carbon steels, the metallurgy of stainless steels is more complex and the manufacturing process has a higher impact on their final properties. Certain factors can change the form of the basic stress-strain curve for any given grade of stainless steel and are to some extent independent.

Cold working

Stainless steel is generally available in the “annealed condition”, i.e. it has undergone a heat treatment process in which it was heated up, maintained at that temperature for a time period, and then rapidly quenched. Annealing returns the material to a soft and workable state. Strength levels of stainless steels, especially the austenitic grades, are enhanced by cold working (such as imparted during cold forming operations including roller levelling/flattening and during fabrication). Associated with this enhancement is a reduction in ductility but this normally is of slight consequence due to the initial high values of ductility, especially for the austenitic stainless steels. The price of cold worked stainless steel is slightly higher than the equivalent annealed material, depending on the grade, product form and level of cold working. As stainless steel is cold worked, it tends to exhibit increasing non-symmetry of tensile and compressive behaviour and anisotropy (different stress-strain characteristics parallel and transverse to the rolling directions). The degree of asymmetry and anisotropy depends on the grade, level of cold working and manufacturing route.

Structural sections of thickness above 3mm are not made from heavily cold worked material and the differences in stress strain behaviour for such sections due to non symmetry and anisotropy are not large; the non-linearity has a more significant effect. Anisotropy and non-symmetry are more significant in the design of lighter gauge, heavily worked sections.

For cold worked material, the compression strength in the longitudinal direction is less than the tensile strength in both the transverse and longitudinal directions (the values traditionally given in material standards and reported accordingly by suppliers). Care is therefore needed in the choice of design strength for cold worked material.

During the fabrication of a section by cold forming, plastic deformations occur which result in a significant increase in the 0.2% proof strength. A strength enhancement of about 50% is typical in the cold formed corners of cross sections; the strength of the material in the flat faces also increases. The strength increase can be exploited in design by testing. Subsequent heat treatment or welding of the member will have a partial annealing (softening) effect with a consequential reduction in any enhanced strength properties arising from cold working.

Strain- rate sensitivity is more pronounced in stainless steels than in carbon steels. That is, a proportionally greater strength can be realised at fast strain rates for stainless steel than for carbon steel.

Strength

In design calculations, the characteristic yield strength fy and characteristic ultimate strength fu are taken as the minimum specified values for the 0.2% proof strength and tensile strength given in codes and specifications. These values apply to material in the annealed condition, and hence are conservative for material or sections which have undergone cold working during fabrication. Structural sections are rarely delivered in the annealed condition.

It should be noted that the measured yield strength of austenitic stainless steels may exceed the specified minimum values by a margin varying from 25 to 40%, for plate thicknesses of 25 mm or less. The margin for duplex stainless steels is lower, perhaps from 5 up to 20%. There is an inverse relationship between thickness or diameter, and yield strength; lighter gauges typically have yield strengths that are significantly higher than the minimum requirement whereas at thicknesses of 25 mm and above, the values are usually fairly close to the specified minimum yield strength.

For external, exposed structures in very hot climates, due consideration should be taken of the maximum temperature the stainless steel is likely to reach. While smaller and sheltered components may remain at ambient temperatures, large surface areas of bare stainless steel that are exposed to direct sun can reach temperatures that are about 50% higher than ambient temperature. If the maximum temperature of the stainless steel is likely to reach 60^{o} C, then a 5% reduction should be made to the room temperature yield strength; greater reductions will be necessary for higher temperatures.

For structural design, it is recommended that a value of 200×10^{3} N/mm2 is used for the modulus of elasticity for all stainless steels.

Tables 2.1 to 2.3 provide nominal values of yield strengths of different grades of stainless steels to European and Indian Codes.

Table 2.1: Mechanical Properties of Different Grades of Stainless Steel in Annealed Condition (IS6911:1992)

Grade Designation		Hardness		Yield Strength / 0.2% Proof Stress (Min) MPa	Tensile Strength (Min) MPa	% Elongation (Min)
Letter Symbol	Numerical Symbol	Brinell	Rockwell			
X*04Cr12*	405	187	88	250	440	20
X07Cr17	430	192	88	250	440	18
X12Cr12	410	212	95	410	*590*	16
X20Cr13	420S1	229	-	490	690	14
X 30Cr13	420S2	235	-	*590*	780	11
X 15Cr16Ni2	431	262	-	640	830	10
X 10Cr17Mn6Ni4N20	201	217	*9S*	300	640	40
X07Cr17Mn12Ni4	201A	217	95	260	540	40
X10Cr18Mn9Nl5	202	217	95	310	620	40
X10Cr17Ni7	301	*212*	-	220	590	40
X07Cr18Ni9	302	192	92	210	400	40
X04Cr19Ni9	304S1	192	92	200	400	40
X02Cr19Ni10	304S2	192	88	180	440	40
X15Cr24Ni13	309	217	95	210	490	40
X20Cr25Ni20	310	217	95	210	490	40
X04Cr17Ni12Mo2	316	192	95	210	490	40
X02Crl7Ni12Mo2	316L	192	95	200	440	40
X04Cr17Ni12Mo2Ti	316T2	192	95	210	490	35
X04Cr10Ni10Ti	321	192	95	210	490	35
X04Cr18Ni10Nb	347	192	92	210	490	35

Table 2.2: Mechanical Properties of Austenitic Steels in Work Hardened Condition (IS6911:1992)

Grade Designation		0.2% Proof Stress (Min) MPa	Tensile Strength (Min) MPa	% Elongation (Min)	Applicable To Max Thickness (mm)
Letter Symbol	Numerical Symbol				
X04Cr19Ni9	304 S1	490	830	12	2.8
		740	1030	8	2.4
		910	1180	7	1.8
		960	1270	3	1.4
X07Cr18Ni9	302	490	830	12	2.8
		740	1030	9	2.4
X 10Cr17Ni7	301	490	830	25	3.3
		740	1030	10	2.9
		910	1180	*5*	2.4
		960	1270	4	2.3
X10Cr17Mn6Ni4 N20	201	490	*830*	20	3.3
		740	1030	10	2.9
		910	1180	7	2.4
		980	1270	4	2.3

Table 2.3: Nominal values of the yield strength (fy) and the ultimate strength (fu) for common stainless steels to EN 10088 (MPa)

Grade	Cold Rolled Strip		Hot Rolled Strip		Hot Rolled Plate		Bars, Rods & Sections	
	Nominal Thickness , t							
	t ≤ 8mm		t ≤ 13.5mm		t ≤ 75mm		t or ϕ ≤ 250mm	
	fy	fu	fy	fu	fy	fu	fy	fu
Austenitic								
1.4301	230	540	210	520	210	520	190	500
1.4307	220	520	200	520	200	500	175	500
1.4318	350	650	330	650	330	630	-	-
1.4401	240	530	220	530	220	520	200	500
1.4404	240	530	220	530	220	520	200	500
1.4541	220	520	200	520	200	500	190	500
1.4571	240	540	220	540	220	520	200	500
Duplex								
1.4062	530	700	480	680	450	650	380	650
1.4162	530	700	480	680	450	650	450	650
1.4362	450	650	400	650	400	630	400	600
1.4462	500	700	460	700	460	640	450	650
1.4482	500	700	480	660	450	650	400	650
1.4662	550	750	550	750	480	680	450	650
Ferritic								
1.4003	280	450	280	450	250	450	260	450
1.4016	260	450	240	450	240	430	240	400
1.4509	230	430	-	-	-	-	200	420
1.4521	300	420	280	400	280	420	-	-
1.4621	230	400	230	400	-	-	240	420

2.6 Thermal & Magnetic Properties

The coefficient of thermal expansion for austenitic stainless steels is about 30% higher than that for carbon steel. Where carbon steel and austenitic stainless steel are used together, the effects of differential thermal expansion should be considered in design. The thermal conductivity of austenitic and duplex stainless steels is about 30% of that of carbon steel. Ferritic grades have higher thermal conductivity, which is about 50% of the value for carbon steel. The thermal expansion of ferritic grades is much lower than that of the austenitic grades and approximately equal to that of carbon steels.

Duplex and ferritic grades are magnetic, whereas annealed austenitic stainless steels are essentially not magnetic. In cases where extremely low magnetic permeability is needed, specialist austenitic grades are available and care must be exercised in selecting appropriate welding consumables to eliminate the ferrite content in the weldment. These filler materials give 100% austenitic solidification in the weld metal. Heavy cold working, particularly of the lean alloyed austenitic steels, can also increase magnetic permeability; subsequent annealing would restore the non-magnetic properties.

2.7 Fasteners

The bolts, washers, and nuts shall all have equivalent or greater corrosion resistance than the most corrosion resistant of the metal alloys joined. Austenitic bolts to EN ISO 3506 property class 70 are the most widely available. Reference should be made to EN ISO 3506 for certain size and length restrictions. The ASTM specification for Bolts and Nuts are as under:

(a) Bolts

ASTM A193/A193M

ASTM A320/A320M

ASTM A453/A453M

ASTM A1082/A1082M

ASTM F593

(b) Nuts

ASTM A194/A194M

ASTM A453/A453M

ASTM A962/A962M

ASTM F594

ASTM F836M

Castings and Forgings

Following are the ASTM specifications for Casting and Forging of Stainless Steel.

Castings

ASTM A351/A351M (austenitic stainless steel)

ASTM A747/A747M (precipitation hardening stainless steel)

ASTM A890/A890M (duplex stainless steel)

Forgings

ASTM A182/A182M

ASTM A473

ASTM A705/A705M (precipitation hardening stainless steel)

ASTM A1049/A1049M (duplex stainless steel)

3. DURABILITY OF STAINLESS STEEL

3.1 Durability is defined as the ability of a material to remain serviceable in the surrounding environment during the useful life without damage or unexpected maintenance. In the case of steel material, its durability is directly dependent on its ability to resist corrosion.

Stainless steels are generally highly corrosion resistant and will perform satisfactorily in most environments. The degree of corrosion resistance of a given stainless steel element is predominantly dependent on its constituent elements, which means that each grade has a slightly different response when exposed to a corrosive environment. Care is therefore needed to select the most appropriate grade of stainless steel for a given application. Furthermore, their higher strength may make it possible to reduce section sizes. Austenitic material in the cold worked condition has a similar corrosion resistance to that in the annealed condition.

The most common reasons for a metal to fail to perform as well as expected regarding corrosion resistance are:

- incorrect assessment of the environment or exposure to unexpected conditions, e.g. unsuspected contamination by chloride ions or higher than expected surface accumulations,

- inappropriate stainless steel fabrication techniques (e.g. welding, heat treating and heating during forming), incomplete weld heat tint removal, or surface contamination may increase susceptibility to corrosion,

- too rough or incorrectly orientated finish.

Even when surface staining or corrosion occur, it is unlikely that structural integrity will be compromised. However, the user may still regard unsightly rust staining on external surfaces as a failure. Careful material grade selection, good detailing and workmanship can significantly reduce the likelihood of staining and corrosion. Experience indicates that any serious corrosion problem is most likely to show up in the first two or three years of service.

In certain aggressive environments, some grades of stainless steel will be susceptible to localised attack. Six mechanisms of localized corrosion are described hereinafter.

3.2 Types of Corrosion and Performance of Steel Grades

1.Pitting corrosion

As the name implies, pitting takes the form of localised pits. It occurs as a result of local breakdown of the passive layer, normally by chloride ions although the other halides and other anions can have a similar effect. In a developing pit, corrosion products may create a very corrosive solution, often leading to high propagation rates. In most structural applications, the extent of pitting is likely to be superficial and the reduction in section of a component is negligible. However, corrosion products can stain architectural features. A less tolerant view of pitting should be adopted for services such as ducts, piping and containment structures. Since the chloride ion is by far the most common cause of pitting in exterior applications, coastal areas are rather aggressive. In addition to chloride content, the probability of a service environment causing pitting depends on factors such as the temperature, corrosive pollutants and particulate acidity or alkalinity, the content of oxidizing agents, and the presence or absence of oxygen. The pitting resistance of a stainless steel is dependent on its chemical composition. Chromium, molybdenum and nitrogen all enhance the resistance to pitting.

The Pitting Resistance Equivalent (PRE) gives an approximate empirically derived estimate of pitting resistance and is defined as:

$$PRE = \%\ wt\ Cr + 3.3(\%\ wt\ Mo) + 16(\%\ wt\ N)$$

The PRE of a stainless steel is a useful guide to its corrosion resistance relative to other stainless steels, but should only be used as a rough indicator. Small differences in PRE can easily be overshadowed by other factors that also influence corrosion pitting resistance. Therefore, the PRE should not be the only factor in selection.

2. Crevice corrosion

Crevice corrosion occurs in tight, unsealed crevices where there is a continuous film of water both within and outside the crevice. The crevice must be fine enough to allow entry of water and dissolved chloride yet prevent diffusion of oxygen into the crevice. Crevice corrosion can be avoided by sealing crevices or eliminating them. The severity of a crevice is very dependent on its geometry: the narrower and deeper the crevice, the more severe the corrosion conditions. Joints that are not submerged should be designed to shed moisture. Some stainless steels, including 304 and 316, are susceptible to crevice corrosion when chlorides or salts are present in the environment. More corrosion resistant austenitic and the duplex steels are less susceptible and performance will be dependent on the conditions, especially the temperature.

As in pitting corrosion, the alloying elements chromium, molybdenum and nitrogen enhance the resistance to attack.

3. Bimetallic (galvanic) corrosion

When two dissimilar metals are in electrical contact and are bridged by an electrolyte (i.e. an electrically conducting liquid such as sea water or impure fresh water), a current flows from the anodic metal to the cathodic or nobler metal through the electrolyte. As a result, the less noble metal corrodes.

Stainless steels usually form the cathode in a galvanic couple and therefore do not suffer additional corrosion. Stainless steels and copper alloys are very close in the galvanic series, and when exposed to moderate atmospheric conditions can generally be placed in direct contact without concern.

This form of corrosion is particularly relevant when considering joining stainless steel and carbon or low alloy steels, weathering steel, or aluminium. It is important to ensure the filler metal is at least as noble as the most corrosion-resistant material (usually stainless steel). Likewise, if connected with fasteners, the bolting material should be equivalent to the most corrosion-resistant metal. Galvanic corrosion between different types of stainless steel is hardly ever a concern, and then, only under fully immersed conditions.

Bimetallic corrosion can be prevented by eliminating current flow by:

- insulating dissimilar metals, i.e. breaking the metallic path
- preventing electrolyte bridging, i.e. breaking the electrolytic path by paint or other coating. Where protection is sought by this means and it is impracticable to coat both metals, then it is preferable to coat the more noble one (i.e. stainless steel in the case of a stainless/carbon steel connection).

The risk of deep corrosion attack is greatest if the area of the more noble metal (i.e. stainless steel) is large compared with the area of the less noble metal (i.e. carbon steel). Special attention should be paid to the use of paints or other coatings on the carbon steel. If there are any small pores or pinholes in the coating, the small area of bare carbon steel provides a very large cathode/anode area ratio, and severe pitting of the carbon steel may occur. This is, of course, likely to be most severe under immersed conditions. In these situations, it is preferable to paint the stainless steel also up to a distance of at least 75 mm away from where the metals are in contact so that any pores lead to small area ratios.

Adverse area ratios are likely to occur with fasteners and at joints. Carbon steel bolts in stainless steel members should be avoided because the ratio of the area of the stainless steel to the carbon steel is large and the bolts will be subject to aggressive attack. Conversely, the rate of attack on a carbon steel or aluminium member by a stainless steel bolt is negligible. It is usually helpful to draw on previous experience in similar sites because dissimilar metals can often be safely coupled under conditions of occasional condensation or dampness with no adverse effects, especially when the conductivity of the electrolyte is low.

The prediction of these effects is difficult because the corrosion rate is determined by a number of complex variables.

4. Stress corrosion cracking

The development of stress corrosion cracking (SCC) requires the simultaneous presence of tensile stresses and specific environmental factors unlikely to be encountered in normal atmospheres. The stresses do not need to be very high in relation to the proof stress of the material and may be due to loading, residual effects from manufacturing processes such as welding, or bending. Ferritic stainless steels are not susceptible to SCC. Duplex stainless steels usually have superior resistance to stress corrosion cracking than the austenitic stainless steels.

5. General (uniform) corrosion

Under normal conditions typically encountered in structural applications, stainless steels do not suffer from the general loss of section that is characteristic of corrosion in non-alloyed irons and steels.

6. Intergranular corrosion (sensitisation) and weld decay

When austenitic stainless steels are subject to prolonged heating in the range 450°C to 850°C, the carbon in the steel diffuses to the grain boundaries and precipitates chromium carbide. This removes chromium from the solid solution and leaves a lower chromium content adjacent to the grain boundaries. Steel in this condition is termed “sensitized”. The grain boundaries become prone to preferential attack on subsequent exposure to a corrosive environment. This phenomenon is known as “weld decay” when it occurs in the heat affected zone of a weldment.

There are three ways to avoid intergranular corrosion through the use of:

- steel having a low carbon content,
- steel stabilised with titanium or niobium because these elements combine preferentially with carbon to form stable particles, thereby reducing the risk of forming chromium carbide,
- heat treatment. (However this method is rarely used in practice at present).

Regarding austenitic or duplex stainless steels, a low carbon content (0.03% maximum) stainless steel should be specified when welding sections to avoid sensitisation and intergranular corrosion. Intergranular corrosion is now very uncommon in austenitic or duplex stainless steels because modern steel making practice ensures low carbon contents and thus avoids the problem.

Ferritic stainless steels are more prone to sensitization due to welding than austenitic stainless steels. Therefore, even with a low carbon content, it is still important to use a stabilized ferritic grade for welded sections.

Design for corrosion control

The most important step in preventing corrosion problems is selecting an appropriately resistant stainless steel with suitable fabrication procedures for the given environment. However, after specifying particular steel, much can be achieved in realizing the full potential of the steels resistance by careful attention to detailing. Anti corrosion actio ns should ideally be considered at the planning stage and during detailed design.

4. BASIS OF DESIGN

The design of Stainless Steel bridges shall be carried out as per Limit State Design concept of IS/ IRC Codes, specifically IRC:24.

4.1 General requirements

A structure should be designed and fabricated so that it can:

- remain fit for use during its intended life
- sustain the loads which may occur during construction, installation and usage
- localise damage due to accidental overloads
- have adequate durability in relation to maintenance costs.

The above requirements can be satisfied by using suitable materials, by appropriate design and detailing and by specifying quality control procedures for construction and maintenance. Structures should be designed by considering all relevant limit states.

4.2 Basis of Design

The Limit State Design philosophy as stated in clause 503 of IRC:24 shall be followed. The method is summarized hereinafter for the sake of completeness.

In the limit state design method, the bridge structure shall be designed to withstand safely all loads likely to act on it throughout its design life. Also, the structure shall remain fit for use during its design life. The acceptance limit for safety or serviceability requirements before the failure occurs is called a limit state. In general, the structure shall be designed on the basis of most critical limit state and shall be checked for other limit states. The probability of a limit state being reached during its lifetime should be very low.

4.3 Limit State Design

The design shall be based on the characteristic values of material strengths and applied loads, which takes into account the probability of variations in the material strengths and applied loads. The characteristic values shall be based on statistical data, if available. Where such data are not available, these shall be based on experience. The design values are derived from the characteristic values through the use of partial safety factors both for material strengths and for loads. These factors are dependent on the type of material, the type of load and the limit state being considered. The reliability of the design is ensured when:

Design Load ≤ Design Strength

Limit states are the states beyond which the structure no longer satisfies the specified performance requirements. The Limit States are classified as:

a) Limit State of Strength
b) Limit State of Serviceability
c) Limit State of Fatigue

Limit State of Strength

Limit state of strength is associated with the failure (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and/or property. The limit state of strength includes:

a) Loss of equilibrium of the structure as a whole or any of its parts or components.

b) Loss of stability of the structure (including the effect of overturning)

c) Failure by excessive deformation (including buckling induced deformation), rapture of the structure or any of its parts or components.

d) Brittle Fracture

Limit State of Serviceability

Limit state of serviceability includes:

a) Deformation or deflection, which may adversely affect the appearance or effective use of bridge structure.

b) Vibration of the structure or any of its components causing discomfort to user or damages to the structure or which may limit its functional effectiveness.

c) Corrosion and durability

Limit State of Fatigue

Limit state of fatigue is the state at which stress range due to application of live load reaches a limiting value corresponding to number of load cycles and detail configuration. It is dealt in detail in clause 511 of IRC:24. The same has been summarized under Clause 9.

4.4 Design Loads

The loads and load combinations specified in IRC:6 shall be considered along with the specified load factors.

4.5 Design Strength

The design strength S_d is obtained as given below from ultimate strength, S_u and partial safety factors for material (Table 4.1)

$$S_d = S_u / \gamma_m$$

Partial safety factor for materials (γ_m) account for the possibilities of:

a) unfavourable deviation of material strength from characteristic value

b) unfavourable variation of member sizes

c) unfavourable reduction in member strength due to fabrication and tolerances

d) uncertain calculation of strength of members

Table 4.1 : Safety Factor for Materials, γ_m

Sl No	Definition	Partial Safety Factor	
1)	Resistance governed by yielding γ_{m0}	1.10	
2)	Resistance of member governed by buckling γ_{m0}	1.10	
3)	Resistance governed by ultimate stress γ_{ml}	1.25	
4)	Resistance of connection	Shop Fabrication	Field Fabrication
	a) Bolt – Bearing type γ_{mb}	1.25	1.25
	b) Bolt – Friction type γ_{mf}	1.25	1.25
	c) Welds γ_{mw}	1.25	1.50

Note:

As stainless steel does not show a definite yield point, 0.2% Proof Stress shall be considered as characteristic yield stress, f_y

For grades of stainless steel not specifically included in IS:6911 (or, Table 2.1 of EN 1993-1-4) or National Standards the γ_m factors should be increased by 10%.

4.6 Factors governing Ultimate Strength

4.6.1 Stability - Stability shall be ensured for the structure as a whole and for each of its elements. This should include overall frame stability against overturning given below:

Stability Against Overturning - The structure as a whole or any part of it shall be designed to prevent instability due to overturning, uplift or sliding under factored load as given below:

a) The loads shall be divided into components aiding instability and components resisting instability.

b) The permanent and variable loads and their effects causing instability shall be combined using appropriate load factors as per the Limit States requirements to obtain maximum destabilizing effect.

c) The permanent loads and effects contributing to resistance shall be multiplied by a partial safety factor 0.9 and added together with design resistance (after multiplying by appropriate partial safety factor). Variable loads and their effects contributing to resistance shall be disregarded.

d) The resistance effect shall be greater than or equal to the destabilizing effect. Combination of imposed and dead loads should be such as to cause most severe effect on overall stability.

4.6.2 Fatigue - Fatigue design shall be as per clause 9 which follows Clause 511 of IRC:24. When designing for fatigue the partial safety factor for loads (f) shall be considered as 1.00 for loads causing stress fluctuation and stress range.

5. Cross Section Design

The procedures stated hereinafter essentially follows the stipulations of IRC:24.

5.1 Geometrical Properties

The geometrical properties of the gross and the effective cross-sections of a member or part thereof shall be calculated on the following basis:

a) The properties of the gross cross-section shall be calculated from the specified size of the member or part thereof or read from appropriate table.

b) The properties of the effective cross-section shall be calculated by deducting from the area of the gross cross-section the following:

   i) The sectional area in excess of effective plate width, in case of slender sections (Clause 5.2.2).

   ii) The sectional areas of all holes in the section except for parts in compression. In case of punched holes, hole size 2 mm in excess of the actual diameter may be deducted.

5.2 Classification of Cross-Sections

5.2.1 The local buckling of plate elements of a cross-section can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section, subjected to compression due to axial force, moment or shear.

5.2.1.1 When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling to enable the redistribution of bending moment required before formation of the failure mechanism.

5.2.1.2 When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling.

5.2.2 On the basis of the above, four classes of sections are defined as follows:

Class 1: Plastic - Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The width to thickness ratio of plate elements shall be less than that specified under Class 1 (Plastic) in **Table 5.1**.

Class 2: Compact - Cross-sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 2 (compact), but greater than that specified under Class 1 (Plastic) in **Table 5.1**.

Class 3 : Semi-compact - Cross-sections, in which the extreme fibre in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact) in **Table 5.1**.

Class 4: Slender - Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate element shall be greater than that specified under Class 3 (Semi-compact) in **Table 5.1**. In such cases the effective sections for design shall be calculated by deducting width of compression plate element in excess of the Semi-compact section limit. The design of slender compression element is outside the scope of this code. When different elements of a cross-section fall under different classes, the section shall be classified as governed by the most critical element.

The maximum value of limiting width to thickness ratios of elements for different classifications of sections are given in **Table 5.1.**

Table 5.1 Limiting Width to Thickness Ratio
(Clauses 5.2.2 and 5.2.4)

Compression Element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-Compact
Outstanding Element of Compression Flange	Rolled Section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded Section		b/t_f	8.4ε	9.4ε	13.6ε
Internal Elements of Compression Flange	Compression due to Bending		b/t_f	29.3ε	33.5ε	42ε
	Axial Compression		b/t_f	Not Applicable		
Web of I, H or Box Section	Neutral Axis at Mid Depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative	d/t_w	$84\varepsilon/(1+r_1)$ but $\geq 42\varepsilon$	$105\varepsilon/(1+r_1)$	$126\varepsilon/(1+2r_2)$ but $\geq 42\varepsilon$
		If r_1 is positive	d/t_w		$105\varepsilon/(1+1.5r_1)$ but $\geq 42\varepsilon$	
	Axial Compression		d/t_w	Not Applicable		42ε
Web of a Channel			d/t_w	42ε	42ε	42ε
Angle, Compression due to Bending (both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single Angle or Double Angles with the components separated, Axial Compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not Applicable		15.7ε 15.7ε 25ε
Outstanding leg of an Angle in contact back-to-back in a Double Angle member			d/t	9.4ε	10.5ε	15.7ε

Compression Element	Ratio	Class of Section		
		Class 1 Plastic	Class 2 Compact	Class 3 Semi-Compact
Outstanding leg of an Angle with its back in continuous contact with another component	d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, Rolled or cut from a rolled I or H section	D/t_f	8.4ε	9.4ε	18.9ε
Circular Hollow Tube, including Welded Tube subjected to (a) Moment, (b) Axial Compression	D/t_f	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
	D/t	Not Applicable		$88\varepsilon^2$

NOTE 1 :Elements which exceed semi-compact limits are to be taken as of slender cross-section

NOTE 2:ε= $\sqrt{\frac{235\,E}{f_y\,210000}}$

fy (N/mm^2)	210	220	460
ε	1.03	1.01	0.698

NOTE 3: Webs shall be checked for shear buckling in accordance with Clause 6.6 when d/t > 67ε. where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is outer diameter of the element, Refer **Fig 5.1** &Clauses 5.2.3 and 5.2.4.

NOTE 4: Different elements of a cross-section can be of different classes. In such cases the section is classified based on the least favourable classification.

NOTE 5: The stress ratio r1 and r2 are defined as

r1 = (actual average axial stress (negative, if tensile))/(design compressive stress of web alone)

r2 = (actual average axial stress (negative, if tensile))/(design compressive stress of overall Section)

5.2.3 Types of Elements

a) Internal elements are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners. e.g. web of I-section and flanges and web of box section.
b) Outside elements or Outstands are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane e.g. flange overhang of an I-section, stem of T-section and legs of an angle section.
c) Tapered elements may be treated as flat elements having average thickness defined in SP:6 Part 1 of BIS.

5.2.4 Compound elements in built-up section (**Fig. 5.1**) - In case of compound elements consisting of two or more elements bolted or welded together, the limiting width to thickness ratios as given in **Table 5.1**should be considered as follows :

a) Outstanding width of compound elements (b_e) to its own thickness.
b) The internal width of each added plate between the lines of welds or fasteners connecting it to the original section to its own thickness.
c) Any outstand of the added plates beyond the line of welds or fasteners connecting it to original section to its own thickness.

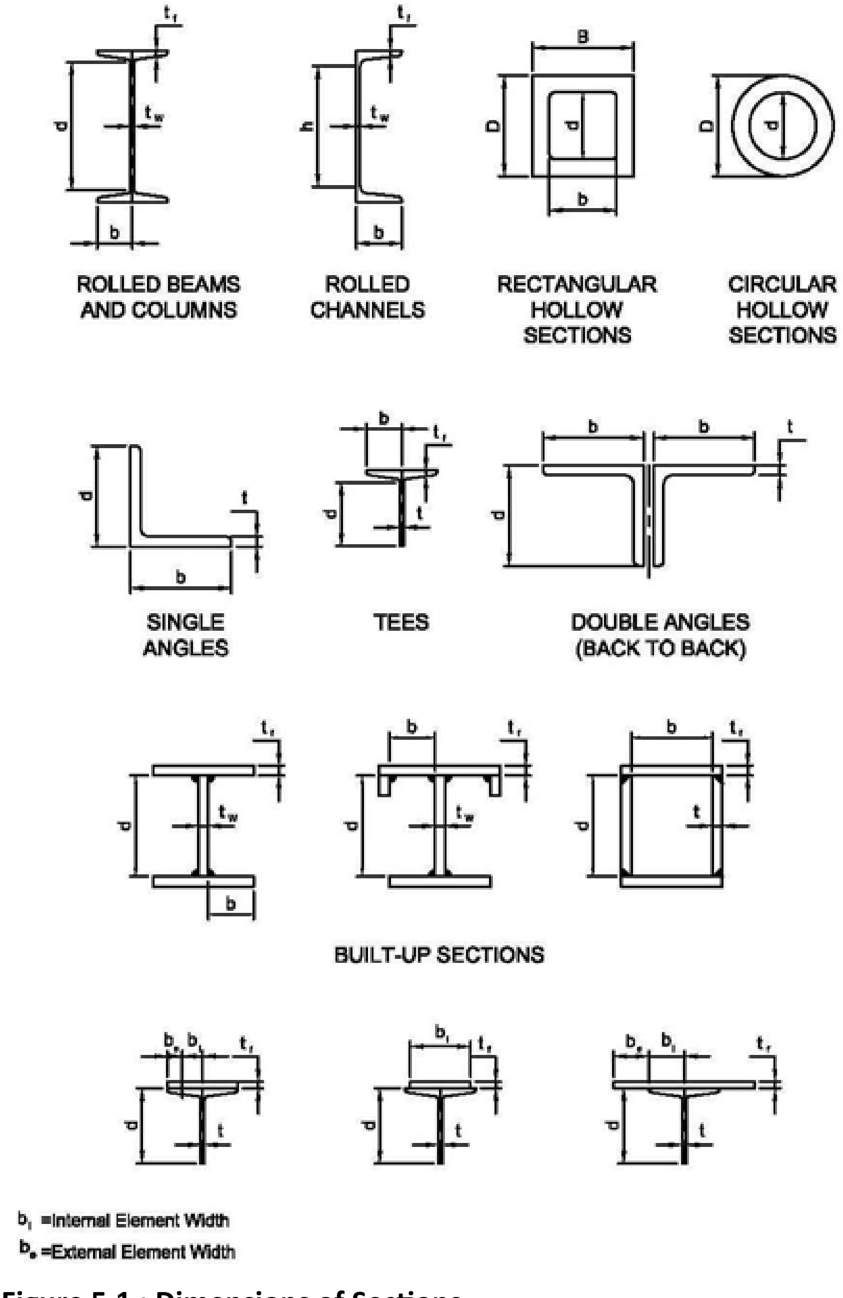


Figure 5.1 : Dimensions of Sections

6. Member Design

The design rules for stainless steel bridges are basically same as those for carbon steel. The rules specified in IRC:24 shall be followed with some modifications due to the difference in behavior of stainless steel.

The rules stated hereinafter after as per the clauses of IRC:24 and are included here for the sake of completeness.

For design of stainless bridges yield strength, f_y, shall be taken as 0.2% proof strength. Modulus of Elasticity, E, shall be taken as $2x10^5$ MPa to take into account non-linearity of stress strain curve while computing the value of ε. Refer IS:6911, International Codes or manufacturer's literature for the specific values of the selected grade.

6.1 GENERAL DESIGN CONSIDERATIONS

6.1.1 Effective Span

The effective span shall be as given below:

a) For main girders - the distance between the centres of bearings
b) For cross girders - the distance between the centres of main girders or trusses
c) For stringers - the distance between the centres of cross girders

NOTE:- Where a cross girder or stringer terminates on an abutment or pier, the centre of bearing thereon shall be taken as one end of the effective span.

d) For pins in bending - the distance between the centres of bearings; but where pins pass through bearing plates having thickness greater than half the diameter of the pins, consideration may be given to the effect of the distribution of bearing pressures on the effective span.

6.1.2 Effective Depth

The effective depth of plate or truss girder should be taken as the distance between the centres of gravity of the upper and lower flanges or chords.

6.1.3 Spacing of Girders

The distance between centres of the main girders shall be sufficient to resist overturning or over stressing due to lateral forces and loading conditions. Otherwise special provisions must be made to prevent this. This distance shall not be less than 1/20 of the span.

6.1.4 Depth of Girders

Minimum depth preferably shall not be less than the following :

a) For trusses : 1/10 of the effective span

b) For girders : 1/25 of the effective span

The effective depth of open web girders shall not be greater than three times the distance between the centres of these girders.

6.1.5 Deflection of Girders

Deflection is to be checked by elastic analysis, using a partial safety factor for loads as 1.0.

6.1.5.1 Rolled steel beams, plate girders or lattice girders, either simple or continuous spans, shall be designed so that the total deflection due to dead load, live load and impact shall not exceed 1/600 of the span. However, this restriction shall not apply if minimum inplace precamber is provided to compensate for all dead and superimposed dead load deflections.

Additionally, the deflection due to live load and impact shall not exceed of 1/800 of the span.

6.1.5.2 The deflection of cantilever arms at the tip due to dead load, live load and impact shall not exceed 1/300 of the cantilever arm and deflection due to live load and impact shall not exceed 1/400 of the cantilever arm.

6.1.5.3 Sidewalk live load may be neglected in calculating deflection.

6.1.5.4 When cross bracings or diaphragms of sufficient depth and strength are provided between beams to ensure the lateral distribution of loads the deflection may be calculated considering all beams acting together.

6.1.5.5 The gross moment of inertia shall be used for calculating the deflection of beams or plate girders. In calculating the deflection of trusses the gross area of each truss member should be used.

6.1.6 Camber

6.1.6.1 Camber, if any, shall be provided as specified by the engineer. Camber may be required to maintain clearance under all conditions of loading or it may be required for the sake of appearance.

6.1.6.2 In the absence of specific guidance, the following principles may be observed.

a) Beams and plate girders up to and including 35 m span need not be cambered.

b) In open web spans the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 percent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design. The camber diagram shall be prepared as indicated in Annex-B of IRC:24.

6.1.7 Minimum Sections

6.1.7.1 For all members of the structure, except parapets and packing plates, the following minimum thicknesses of plates and rolled sections shall apply:

a) 2.5 mm when both sides are accessible for inspection or are in close contact with other plates or rolled sections, or are otherwise adequately protected against corrosion.

b) When one side is not readily accessible for inspection or is not in close contact with another member, or is not otherwise adequately protected and where the thickness required by calculation is less than 5 mm, 1.0 mm shall be added to the calculated thickness subject to the total thickness being not less than 6 mm.

c) 2 mm for box members when the inside of the member is effectively sealed.

d) For rolled steel beams and channels the controlling thickness shall be taken as the mean thickness of the flange, regardless of the web thickness

6.1.7.2 In floor plates and parapets not designed to carry stresses a minimum thickness of 2 mm shall be used if both sides are accessible or 2.5 mm if only one side is accessible. For packing plates the thickness shall not be less than 1.5 mm.

6.1.7.3 No angle less than 65 mm x 45 mm and no flat less than 50 mm wide shall be used in any part of a bridge structure, except for hand railings and shear connectors.

6.1.7.4 Thickness of end angles connecting stringers to cross girders or cross girders to main girders shall be not less in thickness than three quarters of the thickness of the web plates of the stringers and cross girders respectively.

6.1.8 Skew Bridges

For skew bridges, detailed analysis of forces shall be required. However, if the angle of skew is within 15^0, such detailed analysis may not be necessary.

6.1.9 Bearings

6.1.9.1 Provision for jacking of the steel girder for inspection and maintenance of the bearings shall be in-built in the bridge structure and the jacking positions shall be identified and clearly marked.

6.1.9.2 It shall be ensured that, while selecting the bearing type and designing it, the adequacy of the load transfer mechanisms from superstructure to bearing and bearing to sub-structure have been examined and provided for.

6.2 ANALYSIS OF STRUCTURES

6.2.1 General

Effects of design loads on a bridge structure and its members and connections shall be determined by structural analysis using Elastic analysis

6.2.2 Elastic Analysis

6.2.2.1 Assumption- Individual members shall be assumed to remain elastic under the effects of factored design loads for all limit states.

6.2.2.2 The effect of haunching or any variation of the cross-section along the axis of a member shall be considered, and where significant shall be taken into account in the determination of the member stiffness.

6.2.2.3 Appropriate load combinations with corresponding load factors are to be used to find out the maximum values of load effects on members.

6.2.2.4 In a first-order elastic analysis, the equilibrium of the frame in the undeformed geometry is considered, the changes in the geometry of the frame due to the loading are not accounted for, and changes in the effective stiffness of the members due to axial force are neglected. The effect of these on the first order bending moments may be accounted for by carrying out second order elastic analysis.

6.3 DESIGN OF TENSION MEMBERS

6.3.1 Design

Tension members are linear members in which axial forces act causing elongation (stretch). Such members can sustain loads upto ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions (See Clause 6.3.1.3).

The factored design tension T, in the members shall satisfy the following requirement :

$$T < T_d$$

where

T_d = design strength of the member

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of gross section, T_{dg}, rupture of critical section, T_{dn} and block shear T_{db} given in Clauses 6.3.1.1, 6.3.1.2 and 6.3.1.3 respectively

6.3.1.1 Design strength governed by yielding of gross section

The design strength of members under axial tension, T_{dg} as governed by yielding of gross section, is given by

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

f_y = yield stress of the material

A_g = gross area of cross-section

γ_{m0} = partial safety factor for failure in tension by yielding (Table 4.1)

6.3.1.2 Design strength governed by rupture of critical section

6.3.1.2.1 Plates - The design strength in tension of a plate, T_{dn} as governed by rupture of net cross sectional area, A_n at the holes is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

γ_{m1} = partial safety factor for failure at ultimate stress (Table 4.1)

f_u = ultimate stress of the material

A_n = net effective area of the member given by,

$$A_n = \left[b - nd_h + \sum_i \frac{P_{si}^2}{4g_i} \right] t$$

where

b, t = width and thickness of the plate, respectively

d_h = diameter of the hole

g = gauge length between the holes, as shown in Fig. 6.1

p_s = staggered pitch length between line of holes as shown in Fig. 6.1
n = number of holes in the critical section
i = subscript for summation of all the inclined legs

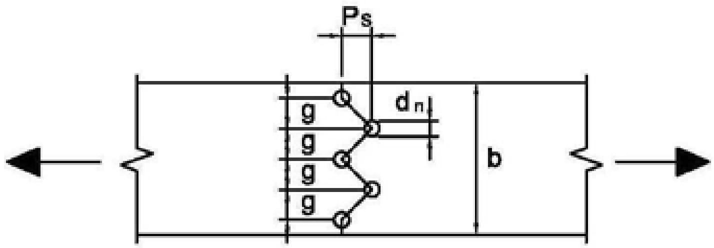


Figure 6.1 : Plates with Holes in Tension

6.3.1.2.2 Threaded rods - The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

$$T_{dn} = 0.9\, A_n f_u / \gamma_{m1}$$

where

A_n = net root area at the threaded section,

6.3.1.2.3 Single angles - The rupture strength of an angle connected through one leg is affected by shear lag. The design strength T_{dn} , as governed by rupture at net section is given by

$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta\, A_{go} f_y / \gamma_{m0}$$

where

$\beta = 1.4 - 0.076\, (w/t)\, (f_y/ f_u)\, (b_s/L_c) \leq (f_u\gamma_{m0}/f_y\, \gamma_{m1}) \geq 0.7$

where

w = outstand leg width

b_s = shear lag width as shown in Fig. 6.2

L_c = length of the end connection, i.e., distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction

For preliminary sizing, the rupture strength of net section may be approximately taken as

$T_{dn} = \alpha\, A_n f_u/\gamma_{m1}$

where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length

A_n = net area of the total cross section

A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg

t = thickness of the leg

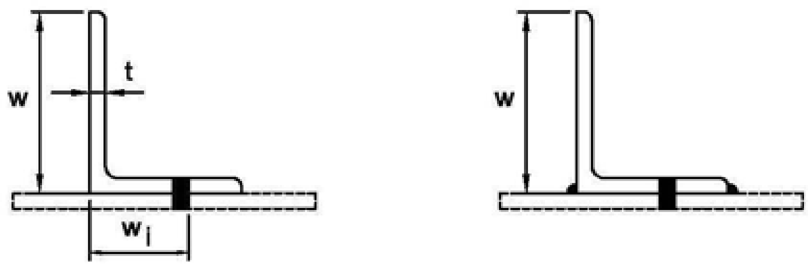


Figure 6.2 : Angles with Single Leg Connections

6.3.1.2.4 Other sections - The rupture strength, T_{dn} of the double angles, channels, I sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in Clause 6.3.1.2.3 where β is calculated based on the shear lag distance, b_s taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross section.

6.3.1.3 Design strength governed by block shear - The strength as governed by block shear at an end connection of plates and angles is calculated as given in Clause 6.3.1.3.1.

6.3.1.3.1 Bolted connections - The block shear strength, T_{db} of connection shall be taken as the smaller of

$$T_{db}= (A_{vg}\, f_y/(S3\ \gamma_{m0}) + 0.9\, A_{tn}\, f_u/\gamma_{m1})$$

or

$$T_{db} = (0.9A_{vn}\, f_u/(S3\ \gamma_{m1}) + A_{tg}\, f_y/\gamma_{m0})$$

where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively [1-2 and 3-4 as shown in Fig. 6.3 (a) and 1-2 as shown in Fig. 6.3 (b)]

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe ofthe angle, end bolt line, perpendicular to the line of force, respectively [2-3 as shown in Fig. 6.3 (b)]

f_u, f_y = ultimate and yield stress of the material, respectively

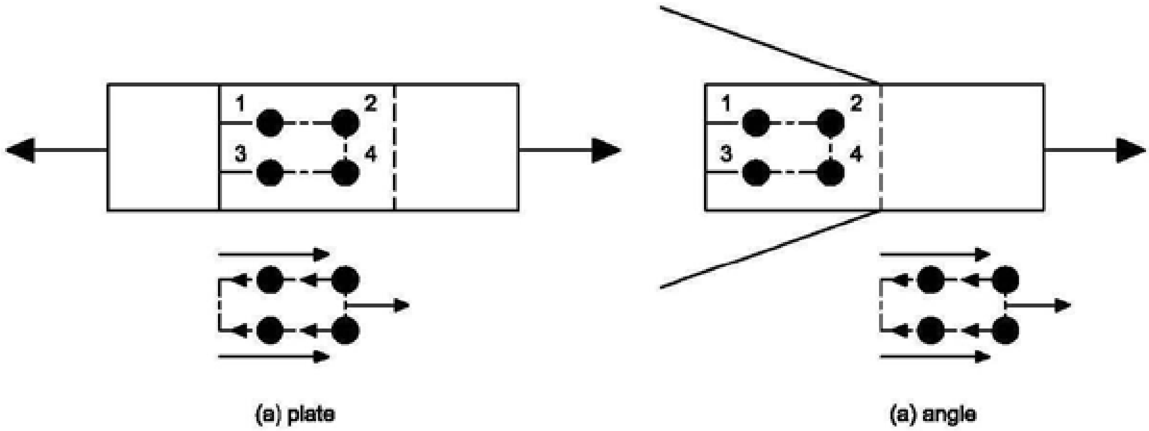


Figure 6.3 : Block Shear Failure

6.3.1.3.2 Welded connection - The block shear strength, T_{db}, shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

6.3.2 Design Details

6.3.2.1 Slenderness ratio

For main members the ratio of unsupported length to the least radius of gyration shall not exceed 300.

6.3.2.2 Configuration

Tension members should preferably be of solid cross section. However, when composed of two or more components these shall be connected as described in Clause 6.3.2.6, 6.3.2.7 and 6.3.2.8.

6.3.2.3 Effective sectional area

When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

6.3.2.4 Lacing and battening

The open sides of built-up tension members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 14 times the mean thickness of the outstand.

6.3.2.5 Lacing and battening shall be designed in accordance with Clause 6.3.2.7 and 6.3.2.8 and shall be proportioned to resist all shear forces due to external forces, if any, in the plane of lacing. The shear shall be considered as divided equally among all transverse systems and plating in parallel planes.

6.3.2.6 Tension members composed of two components back-to-back

6.3.2.6.1 Tension members formed by sections placed back-to-back, either in contact or separated by a distance not exceeding 50 mm shall be connected together in their length at regular intervals by bolting or welding so spaced that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 6.3.2.1.

6.3.2.6.2 Where the components are in contact back-to-back bolting or welding shall be in accordance with clauses applicable.

6.3.2.6.3 When the components are separated they shall be connected through solid washers or packing, bolted or welded.

6.3.2.7 Design of lacing

6.3.2.7.1 As far as practicable the lacing system shall not be varied throughout the length of the tension member.

6.3.2.7.2 Lacing bars shall be inclined at an angle of 40^0 to 70^0 to the axis of the member when a single intersection system is used and at an angle of 40^0 to 50^0 when a double intersection system is used.

6.3.2.7.3 Except for tie as specified in Clause 6.3.2.7.7 double intersection lacing systems shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the member, unless all forces resulting from deformation of the member are calculated and provided for in the lacing and its fastenings.

6.3.2.7.4 Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

6.3.2.7.5 The required section of the lacing bar shall be determined in accordance with the design provisions of lacings of compression members given in Clause 6.4.8. The slenderness ratio of the lacing shall not exceed 140. For this purpose the effective length shall be taken as follows:

i) In bolted construction, the length between the inner end bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

ii) In welded construction, the distance between the inner ends of effective lengths of welds connecting the bars to the components for single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

6.3.2.7.6 Bolting or welding of lacing bars to the main members shall be sufficient to transmit the load to the bars. Where welded lacing bars overlap the main components, the amount of lap shall not be less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less. The welding shall be provided along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between main components, they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

6.3.2.7.7 Laced tension members shall be provided with tie plates at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

6.3.2.7.8 The length of end tie plate parallel to the axis of the member shall not be less than the perpendicular distance between the centroids of the main components and the length of the intermediate tie plates shall not be less than 3/4 of this distance.

6.3.2.7.9 The thickness of all tie plates shall be not less than 1/60 of the distance between the innermost lines of bolts or welds attaching them to the main components, except when effectively stiffened at the edges, in which case the minimum thickness may be 2.5 mm; for this purpose the edge stiffeners shall have a slenderness ratio not less than 170.

6.3.2.7.10 When angles, channels etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The slenderness ratio shall not exceed 140.

6.3.2.8 Design of battens

Battened tension members shall comply with the following requirements.

6.3.2.8.1 The spacing of battens, measured as the distance between the centres of adjacent end pitches of bolts or, for welded construction, the clear distance between the battens, shall be such that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 6.3.2.1

6.3.2.8.2 The effective length of the batten, parallel to the axis of the member, shall be taken as the longitudinal distance between end fastenings.

End battens shall have an effective length of not less than the perpendicular distance between centroids of the main components and the length of the intermediate battens shall have an effective length of not less than one-half of this distance.

6.3.2.8.3 Batten plates shall have a thickness of not less than 1/60 of the minimum distance between the connecting bolts groups or welds except where they are stiffened at their edges.

6.3.2.8.4 Where battens are attached by bolts, not less than two bolts shall be used in each connection. Where battens are attached by welds, the length of welds connecting each longitudinal edge of the batten plate to the component shall, in the aggregate, be not less than half the length of the batten plate, and at least one-third of the weld shall be placed at each end of the longitudinal edge. In addition, welding shall be returned along the ends of the plate for a length of at least four times the thickness of the plate.

Where the tie or batten plates are fitted between main components they shall be connected to each member either by fillet welds on each side of the plate, at least equal in length to that specified in the preceding paragraph or by full penetration butt weld.

6.3.2.9 Splices

Splices in tension members shall have a sectional area 5 percent more than that required to develop the load in the member and, whenever practicable, the cover material shall be disposed to suit the distribution of stress in the various parts of the cross section of the member. Both surfaces of the parts to be spliced shall be covered wherever possible. Bolts or welds shall develop the full strength of the cover material as defined above.

6.4 DESIGN OF COMPRESSION MEMBERS

6.4.1 Design Strength

6.4.1.1 Common hot rolled and built up steel members, used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a,b,c or d as given in Table 6.1 and 6.2.

6.4.1.2 The factored design compression P in the member due to external loads shall satisfy the following requirement:

$P < P_d$

where

P_d = design strength of the member as given below :

= $A_e f_{cd}$

where

A_e = effective sectional area as defined in Clause 6.4.3.2

f_{cd} = design compressive stress obtained as per Clause 6.4.1.2.1

6.4.1.2.1 The design compressive stress, f_{cd} of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{\frac{f_y}{\gamma_{m0}}}{\emptyset+[\phi^2-\lambda^2]^{0.5}} = \frac{\chi f_y}{\gamma_{m0} \leq \frac{f_y}{\gamma_{m0}}}$$

where

φ = 0.5[1 + (- 0.2) + ^{2}]

λ = non-dimensional effective slenderness ratio = $\sqrt{f_y/f_{cc}} = \sqrt{\frac{f_y\left(\frac{KL}{r}\right)^2}{\pi^2 E}}$

f_{cc} = Euler buckling stress = □^{2}E /(KL/r)^2

where

KL/r = Effective slenderness ratio or ratio of effective length KL, to appropriate radius of gyration, r

α = Imperfection factor given in **Table 6.1**

= Stress reduction factor for different buckling class, slenderness ratio and yield stress = $\left[\frac{1}{\left[\emptyset+\left(\emptyset^2-\lambda^2\right)^{0.5}\right]}\right]$

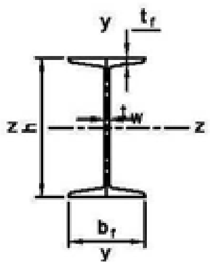
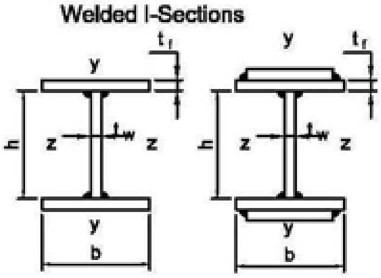
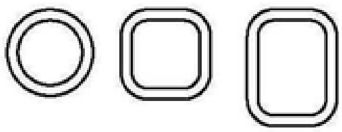
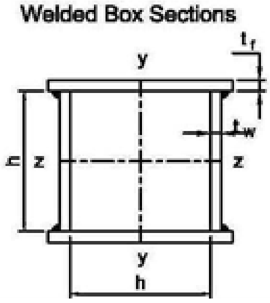
$_{m0}$ = Partial safety factor for material strength

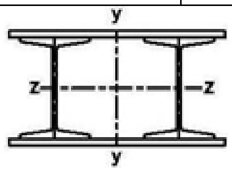
TABLE-6.1 Imperfection Factor,**α**
(Clause 6.1.1 and 6.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

6.4.1.2.2 The classification of different sections under different buckling class a, b, c or d, is given in Table 6.2. The curves corresponding to different buckling class are presented in non-dimensional form, in Fig. 6.4.

TABLE-6.2 Buckling Class of Cross-Sections

Cross-Section	Limits	Buckling about Axis	Buckling Class
Rolled I-Sections 	$h/b_f > 1.2$ $\quad t_f < 40mm$	z-z y-y	a b
	$40mm < t_f \leq 100mm$	z-z y-y	b c
	$h/b_f \leq 1.2$ $\quad t_f \leq 100mm$	z-z y-y	b c
	$t_f > 100mm$	z-z y-y	d d
Welded I-Sections 	$t_f \leq 40mm$	z-z y-y	b c
	$t_f > 40mm$	z-z y-y	c d
Hollow-Sections 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Sections 	Generally (Except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/tw < 30$	y-y	c
Channel,Angle, T and Solid Sections 		Any	c

Cross-Section	Limits	Buckling about Axis	Buckling Class
Built-up Member 		Any	c

6.4.2 Effective Length

6.4.2.1 The effective length, KL is calculated from the member length, L, of the member, considering the rotational and relative translational boundary conditions at the ends. The member length shall be taken as the length from centre to centre of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the centre of the intersecting member at the supported end, shall be taken as the member length.

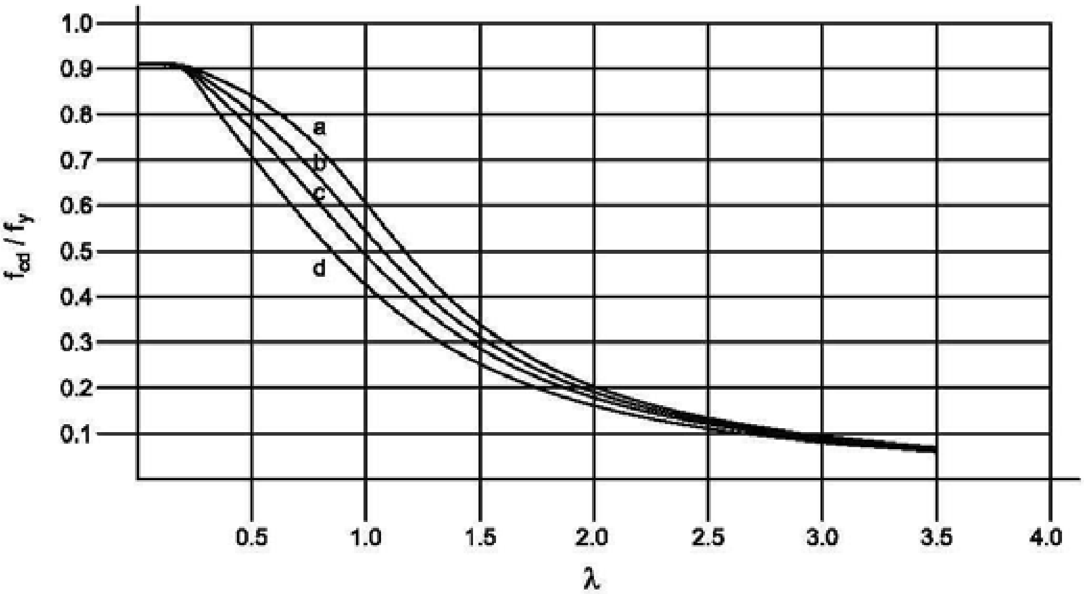
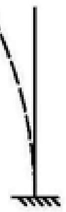
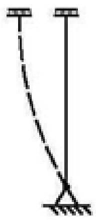
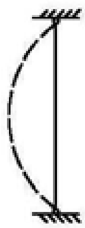
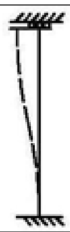
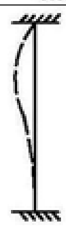


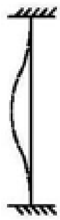
Figure 6.4 : Buckling Curves for Compression Members (Clause 6.4.1.2.1)

6.4.2.2 Where the boundary conditions in the plane of buckling can be assessed, the effective length KL, can be calculated on the basis of Table 6.3.

6.4.2.3 In the case of bolted or welded trusses and braced frames, the effective length, KL of the compression members should generally follow Clause 6.5 under "Design of Trusses or Open-web Girders" of this code. In the case of members of trusses for buckling in the plane perpendicular to the plane of the truss, the effective length, KL shall be taken as the distance between the centres of intersection. The design of angle struts shall be as specified in Clause 6.4.5.

Table 6.3 Effective Length of Prismatic Compression Members
(Clause 6.4.2.2)

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Free	Free		2.0 L
Free	Restrained	Restrained	Free		
Restrained	Free	Restrained	Free		1.0 L
Restrained	Restrained	Free	Restrained		1.2 L
Restrained	Restrained	Restrained	Free		0.8 L

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Restrained	Restrained		0.65 L

6.4.3 Design Details

6.4.3.1 Thickness of plate elements - The classification of members on the basis of thickness of constituent plate elements shall satisfy the width-thickness ratio requirements specified in Table 5.1.

6.4.3.2 Effective sectional area A_e - Except for Class 4 (slender section) in Clause 5.2.2 - the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding or bolting so long as the section is semi-compact or better. Holes not filled with bolts or pins shall be deducted from gross area to calculate effective sectional area.

6.4.3.3 Eccentricity for columns

For the purpose of determining the stress in the section of a column, the reactions from the connecting member or similar loads shall be assumed to be applied at an eccentricity of 100 mm from the face of the section or at the centre of bearing whichever dimension gives the greater eccentricity, and with the exception of the following two cases:

a) In the case of cap connection, the load shall be assumed to be applied at the face of the column section or at the edge of packing, if used towards the span of the beam.
b) In the case of a truss bearing on a cap, no eccentricity needs to be taken for simple bearings without connections, capable of developing any appreciable moment. In case of web member connection with face, actual eccentricity is to be considered.

6.4.3.4 Splices

6.4.3.4.1 Where the ends of compression members are prepared for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist bending or tension, if present. Such splices should maintain the intended member stiffness about each axis. Splices should be located as close to the point of inflection as possible. Otherwise their capacity should be adequate to carry moment (see Clause6.7.3.2.2). The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

6.4.3.4.2 Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which the members are subjected.

6.4.3.4.3 Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members being jointed, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress shall be accounted for.

Wherever possible both surfaces of the parts spliced shall be covered or other means taken to maintain the alignment of the abutting ends.

6.4.3.4.4 Splices in compression members located at or near effectively braced panel points shall be designed to transmit design strength of the member. All other splices in compression members shall have a sectional area 5 percent more than that required to develop the design strength in the member. All cover materials shall, as far as practicable be so disposed with respect to the cross-section of the member so as to transmit the proportional load of the respective parts of the section.

Bolts or welds shall develop the full strength in the cover material as defined above.

6.4.3.4.5 Where flexural tension may occur in the member, the cover material shall be designed to resist such tension.

6.4.4 Base Plates

6.4.4.1 General

6.4.4.1.1 Base plates of compression members should have sufficient stiffness and strength to transmit axial force, bending moments and shear forces at the base of the compression members to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should be provided wherever necessary. Shear resistance at the proper contact surface between steel base and concrete/grout and bearing pressure between the plate and support shall be determined and computed as per provisions of IRC:78.

6.4.4.1.2 If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, c, of the base plate beyond the face of the compression member and gusset may be taken as effective in transferring the column load as given in Fig. 6.5, such that beam pressure at the effective area does not exceed bearing capacity of concrete base.

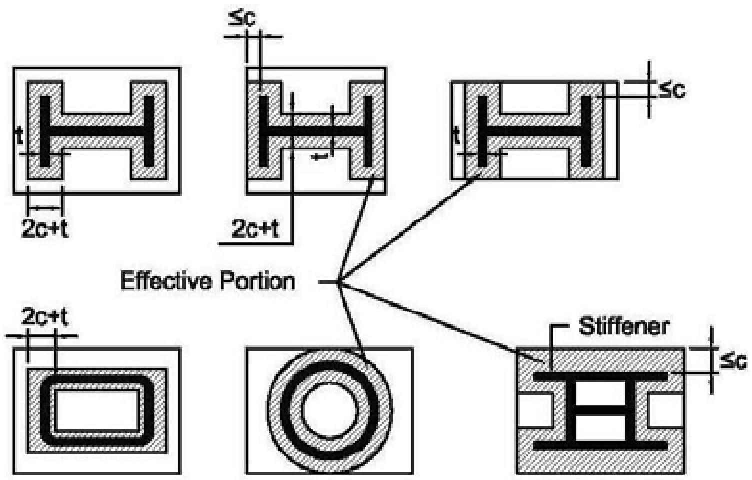


Figure 6.5 : Effective Area of Base Plate

6.4.4.2 Gusseted bases

In gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc. in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces should be machined to ensure perfect contact.

6.4.4.2.1 Where the ends of the compression member and the gusset plates are not faced for complete bearing, the welding, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

6.4.4.2.2 Compression member and base plate connections-Where the end of the compression member is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the compression member is subjected.

6.4.4.3 Slab bases

Compression members with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection.

6.4.4.3.1 The minimum thickness, t_s of rectangular slab bases, supporting members under axial compression shall be

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{m0}}{f_y}} > t_f$$

where

w = uniform pressure from below on the slab base under the factored load axial compression

a,b = larger and smaller projection of the slab base beyond the rectangle circumscribing the compression member respectively

t_f = flange thickness of compression member

When only the effective area of the base plate is used as in Clause 6.4.4.1.1, c^2 may be used in the above equation (See Fig. 6.5) instead of $(a^2 - 0.3b^2)$.

6.4.4.3.2 When the slab base does not distribute the load uniformly, due to eccentricity of the load etc., special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

6.4.4.3.3 Bases for bearing upon concrete or masonry need not be machined on the underside.

6.4.4.3.4 In cases where the cap or base is fillet welded directly to the end of the compression member without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required in Clause 6.4.4.3. Where full strength but welds are provided, machining of contact surfaces is not required.

6.4.5 Angle Struts

6.4.5.1 Single angle struts - The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentrically by connecting one of its legs to a gusset or adjacent member.

6.4.5.1.1 Concentric loading - When a single angle is concentrically loaded in compression the design strength may be evaluated using Clause 6.4.1.2.

6.4.5.1.2 Loaded through one leg - The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e , as given below:

$$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2}$$

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in **Table 6.4**

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2\varepsilon}{250}}} \quad \text{and} \quad \lambda = \frac{\frac{(b_1+b_2)}{2t}}{\varepsilon\sqrt{\frac{\pi^2\varepsilon}{250}}}$$

where

l = centre-centre length of the supporting member

r_{vv} = radius of gyration about the minor axis

b_1, b_2 = width of the two legs of the angle

t = thickness of the leg

TABLE 6.4 Constants k_1, k_2 and k_3
(Clause 6.4.5.1.2)

No of Bolts at each End Connection	Gusset/ Connecting Member Fixity *	k_1	k_2	k_3
2	Fixed	0.20	0.35	20
	Hinged	0.70	0.60	5
1	Fixed	0.75	0.35	20
	Hinged	1.25	0.50	60

* Stiffness of in-plane rotational restraint provided by the gusset/connecting member. For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.

6.4.5.2 Double angle struts

6.4.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraints provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values based on Clause 6.4.1.2. The angles shall be connected together over their lengths so as to satisfy the requirements of Clauses 6.4.6 and 7.

6.4.5.2.2 Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts in each angle, or by the equivalent in welding, shall be designed in accordance with Clause 6.4.5.2 and the angles shall be connected together over their lengths so as to satisfy the requirements of Clauses 6.4.6 and 7.

6.4.5.3 Continuous members: Double angle continuous struts such as those forming the chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with Clause 6.5.

6.4.5.4 Combined stresses: In addition to axial loads, if the struts carry loads, which cause transverse bending, the combined bending and axial stresses shall be checked in accordance with Clause 6.7 of this code.

6.4.6 Compression Members Composed of Two Components Back-to-Back

6.4.6.1 Compression members composed of two angles, channels or tees back to back in contact or separated by a distance not exceeding 50 mm shall be connected together by bolting or welding, so that maximum ratio of the slenderness of each component of the member between such connections is not greater than 40 or 0.6 times the maximum ratio of slenderness of the member as a whole, whichever is less.

The number of connections shall be such that the member is divided into not less than three approximately equal parts.

6.4.6.2 Where the members are separated back-to-back the bolts in these connections shall pass through solid washers or packings, and where the connected angles, legs or tables of tees are 125 mm wide or over or where webs of channels are 150 mm wide or over, not less than two bolts shall be used in each connection, one on the line of each gauge mark.

6.4.6.3 Where these connections are made by welding, solid packing shall be used to effect the jointing unless the members are sufficiently close together to permit butt welding, and the members shall be connected by welding along both pairs of edges of the main components.

6.4.6.4 The bolts or welds in these connections shall be sufficient to carry the shear forces and the moments specified for battened struts and in no case shall the bolts be less than 16 mm dia for members upto and including 10 mm thick; 20 mm diameter for members upto and including 16 mm thick; and 22 mm diameter for members over 16 mm thick.

6.4.6.5 Compression members connected by such bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the bolted or welded surfaces.

6.4.6.6 Where components are in contact back-to-back bolting or intermittent welding shall be done in accordance with applicable clauses.

6.4.7 Lacing and Battening

6.4.7.1 The open sides of built-up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 14 times the mean thickness of the outstand.

6.4.7.2 Lacing and battening plates shall be designed in accordance with Clauses 6.4.8 and 6.4.9 and shall be proportioned to resist a total transverse shear force V_t at any point in the length of the member equal to at least 2.5 percent of the axial force in the member together with all shear due to external forces, if any, in the plane of lacing. The shear force V_t shall be considered as divided equally among all transverse system and plating in parallel planes.

6.4.7.3 Compression members composed of two or more components connected as described in Clauses 6.4.6, 6.4.8 and 6.4.9 may be designed as homogeneous members.

6.4.8 Design of Lacings

6.4.8.1 As far as practicable, the lacing system shall not be varied throughout the length of the compression member.

6.4.8.2 Lacing bars shall be inclined at an angle of 40^0 to 70^0 to the axis of the member where a single intersection system is used, and at an angle of 40^0 to 50^0 where a double intersection system is used.

6.4.8.3 Except for tie plates as specified in Clause 6.4.8.8 below, double intersection lacing systems shall not be combined with members of diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for in the lacing and its fastenings.

6.4.8.4 Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

6.4.8.5 The maximum spacing of lacing bars whether by welding or bolting shall be such that the effective slenderness ratio of the components of the compression member between consecutive connections of the lacing bars to one component is not greater than 50 or 0.7 times the maximum ratio of slenderness of the member as a whole whichever is lesser.

6.4.8.6 The lacing shall be proportioned to resist a total transverse shear, V_t at any point in the member in the manner defined in Clause 6.4.7.2.

6.4.8.7 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the actual shear due to bending, in addition to that specified in Clause 6.4.8.6.

6.4.8.8 The effective slenderness ratio KL/r of the lacing bars shall not exceed 140.

For this purpose the effective length KL shall be taken as follows:

a) In bolted construction - the length between the inner ends of bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersections.

b) In welded construction - the distance between the inner ends of effective lengths of welds connecting the bars to the components in single intersection lacings and 0.7 times this length for double intersection lacing effectively connected at intersections.

6.4.8.9 Lacing bars shall be connected to the main members either bybolting by one or more bolts, in line along the lacings or by welding at each end sufficient to transmit the load to the bars. Any eccentricity of the connection with respect to the centroid of the lacing bar may be ignored and the lacing designed as an axially loaded strut. Where welded lacing bars overlap the main component, the amount of lap shall be not less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less. Welding shall be provided at least along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between the main components they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

6.4.8.10 Laced compression member shall be provided with tie plate at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

6.4.8.11 The length of end tie plates measured between end fastenings along the longitudinal axis of the member shall not be less than (a) the perpendicular distance between the lines of bolts connecting them to the flanges or (b) the distance between vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts and the length of intermediate tie plates shall be not less than three-quarter of (a) above.

6.4.8.12 The thickness of tie plates shall not be less than 1/50 of the distance between the innermost lines of bolts or welds except when effectively stiffened at the free edges in which case the minimum thickness may be 3 mm. For this purpose the edge stiffener shall have a slenderness ratio not greater than 170.

6.4.8.13 Tie plates and their fastenings (calculated in accordance with the method described for battens) shall be capable of carrying the forces for which the lacing system is designed.

6.4.8.14 When angles, channels, etc, are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The effective slenderness ratio shall not exceed 140.

6.4.9 Design of Battens

Battened compression members shall comply with the following requirements:

6.4.9.1 The battens shall be placed opposite each other at each end of the member and at points where the member stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length between centre to centre of connections.

6.4.9.2 In battened compression members when the effective slenderness ratio about the axis perpendicular to the battens is not more than 0.8 times the ratio about the axis parallel to the battens, the spacing of battens between centre to centre of end fastenings shall be such that the slenderness ratio of the lesser main component over this distance shall not be greater than 50 or 0.7 times the ratio of slenderness of the member as a whole about its axis parallel to the battens. In case it is more than 0.8 times the ratio about the axis parallel to the battens, the spacing of battens between centre to centre of end fastenings shall be such that slenderness ratio of the lesser main component over this distance shall not be greater than 50 or 0.7 times of slenderness ratio of the member as a whole about it weaker axis.

6.4.9.3 Battens shall be plates, channels or I sections and shall be bolted or welded to the main components, Battens and their connections shall be so designed that they resist simultaneously a longitudinal shear force equal to V_t D/na and a moment equal to V_t D/2n where

D = the longitudinal distance between centre-to-centre of battens.
a = the minimum transverse distance between the centroids of bolt groups or welding.
V_t = the transverse shear force as defined in Clause 6.4.7.2
n = the number of parallel planes of battens

6.4.9.4 The effective length of a batten parallel to the axis of a member shall be taken as the longitudinal distance between the end fastenings. End battens shall have an effective length of not less than (a) the perpendicular distance between the lines of bolts connecting them to the components, or (b) the distance between the vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts; and intermediate battens shall have an effective length of not less than 3/4 of (a) above, but in no case shall the length (of any batten) be less than twice the width of the smaller component in the plane of the battens.

6.4.9.5 The thickness of batten plates shall not be less than 1/50 of the minimum distance between the innermost lines of connecting bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 3 mm; for the purpose the edge stiffeners shall have a slenderness ratio not greater than 170.

6.4.9.6 Battened compression members not complying with these requirements, or those subjected to bending moments in the plane of the battens shall be designed according to the theory of elastic stability, or empirically with verification by tests, so that they have a load factor of not less than 1.7.

6.4.9.7 Battened compression member composed of two angles forming a cruciform cross-section shall conform to the above requirements except as follows:

a) the battens shall be in pairs placed in contact one against the other, unless these are welded to form cruciform battens.

b) a transverse shear force V_t/S2 shall be taken as occurring separately about each rectangular axis of the whole member.

c) a longitudinal shear force of V_t D/(aS 2) and the moment V_t D/(2 S2) shall be taken in respect of each batten in each of the two planes, except where the effective slenderness ratio can occur about a rectangular axis, in which case each battens shall be designed to resist a shear force of 2.5 percent of the total axial force.

NOTE : V_t , D and a are as defined in Clause 6.4.9.3

6.5 DESIGN OF TRUSSES OR OPEN-WEB GIRDERS

6.5.1 General

Trusses or open web girders are defined as triangulated skeletal girders. The design of individual members and connections should be made in accordance with this Clause in conjunction with Clauses 6.3, 6.4, and 7 as appropriate.

6.5.2 Analysis

For analysis of trusses the following assumption may be made unless rigorous rigid frame analysis is adopted :

a) All members are frictionless pin jointed.

b) All members are straight and free to rotate at the joints.

c) All loads including self weight of members are applied at the joints

Stipulations made in this section are not applicable for design of stiffening trusses of suspension bridges.

6.5.3 Intersection at Joints

For triangulated trusses designed on the assumption of frictionless pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axes meeting at a point, and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections. If, at a joint, the centroidal axes of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account as secondary stress.

Where loads are not applied at truss joints, account should be taken of the following:

a) resulting stresses where load is applied to a member in the plane of a truss other than at a joint

b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross-member, the effect of interaction between the cross-member so loaded and the truss and adjacent cross-member should be taken into account.

6.5.4 Effective Length of Compression Members

6.5.4.1 In bolted or welded trusses the compression members act in a complex manner and the effective lengths KL of such members shall be taken as given in Table 6.5 for computing their design strengths. For battened compression members, all values given in Table 6.5 shall be increased by 10 percent.

Table 6.5 Effective Length of Compression Members

Member	Effective Length KL of Member		
	For Buckling in the Plane of Truss	For Bucking Normal to Plane of Truss	
		Compression Chord or (Compression) member effectively braced by lateral system	Compression Chord or (Compression) member unbraced

Chords		0.85 x distance between centres of intersection with web members	0.85 x distance between centres of intersection with lateral bracing members or rigidly connected cross girder	See clause 6.5.4.4
Web	Single Triangulated System	0.70 x distance between centres of intersection with the main chords	0.85x distance between centres of intersections	Distance between centres of intersections
	Multiple Intersection system where adequate connections are provided at all points of intersection	0.85 x greatest distance between centres of any two adjacent intersection	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersection with the main chords

NOTE : The intersection referred to are those of the centroidal axis of the members.

6.5.4.2 For single angle discontinuous strut connected to gussets or to a section either by bolting by not less than two bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially loaded member provided that the calculated average stress does not exceed the design stress f_{cd} given in Clause 6.4.1.2 in which effective length "KL" is the length of the strut, centre-to-centre of fastenings at each end and 'r' is the minimum radius of gyration.

6.5.4.3 For single angle discontinuous struts intersected by, and effectively connected to another angle in cross bracing, the effective length in the plane of bracings shall be taken as in Table6.7 and normal to the plane of bracing the effective lengths shall be taken as the distance along the bracing members between the points of intersection and the centroids of the main member. In calculating the slenderness ratio, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

6.5.4.4 Effective length of unbraced compression chords

For simply supported trusses with ends restrained at the bearings against torsion, the effective length of the compression chord for buckling normal to the plane of the truss shall be taken as follows:

a) With no lateral support to compression chord; where there is no lateral bracing between compression chords and no cross frames:

KL = span

b) With compression chords supported by U frames, where there is no lateral bracing of the compression chord but where cross-members and verticals forming U frames provide lateral restraints:

$$KL = 2.5\ x\ \sqrt[4]{EIa\delta} \quad but\ not\ less\ than\ "a"$$

where

E =Young's Modulus

I = maximum moment of inertia of compression chord about the axis lying in the plane of the truss.

a = distance between frames, and

δ= the virtual lateral displacement of the compression chord at the frame nearest mid span of the truss, taken as the horizontal deflection. This deflection shall be computed assuming that the cross-member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unrestrained cross member.

When δ is not greater than $a^3/(40EI)$

KL = a

In case of symmetrical U frames, where cross-member and verticals are each of constant moment of inertia throughout their own length; it may be assumed that :

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where

d_1 =distance of the centroid of the compression chord from the top of the cross-member,

d_2 =distance of the centroid of the compression chord from the neutral axis of the cross-member,

b = half the distance between centres of the main trusses,

E =Young's Modulus,

I_1 = moment of inertia of the vertical in its plane of bending, and

I_2 = moment of inertia of the cross-member in its plane of bending

U frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effects of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where:

$$F = \frac{1.4\, x\, 10^{-3} KL}{\delta\left\{\frac{fcc}{fa} - 1.7\right\}}$$

In the above formula:

$$KL = 2.5\, x\, \sqrt[4]{EIa}$$

δ = the deflection of the chord under the action of unit horizontal force F

f_{cc} = Euler buckling stress in chord = $\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

where

E =Young's Modulus

r = radius of gyration

f_a = calculated stress in the chord.

c) with compression chord supporting continuous deck

A compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (e.g., KL = 0) if the friction or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length of 2.5 percent of the maximum force in the chord, in addition to other lateral forces.

6.5.5 Effective Slenderness Ratio of Compression Members

Effective slenderness ratio KL/r of a compression member shall not exceed 120 for main members and 140 for wind bracings and subsidiary members.

6.5.6 Connections at Intersection

6.5.6.1 Connections of members at an intersection shall develop at least the design loads and moments transmitted by the members. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fastenings. All members shall, where possible, be so connected that the load is appropriately distributed over the cross-section; otherwise, consideration shall be given to the distribution of stress through the material to those parts of the section not directly connected, and for this purpose the angle of distribution may be taken as 45^0.

6.5.6.2 Gusset shall be capable of sustaining the design loads and moments transmitted by the members.

6.5.6.3 Gusset plates shall be so shaped and connectors so arranged as to avoid stress concentrations.

6.5.6.4 Bolt and welding groups shall be as compact as practicable.

6.5.7 Lug Angles

6.5.7.1 Lug angles connecting a channel or similar member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

6.5.7.2 In the case of angle members the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the angle and the attachment of the lug angles to the angle member shall be capable of developing a strength 40 percent in excess of that force.

6.5.7.3 In the case of channel or similar members, the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 10 percent in excess of the force not accounted for by the direct connection of the member and the attachment of the lug angles to the member shall be capable of developing a strength 20 percent in excess of that force.

6.5.7.4 In no case shall less than two bolts be used for attaching the lug angle to the gusset or other supporting member.

6.5.7.5 The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset etc.

6.5.8 Section at Pin Holes in Tension Members

In Pin-connected tension members (generally used for erection purpose) the longitudinal net section beyond the pin hole parallel with the axis of the member shall be not less than the required net section of the member. The net section through the pin hole transverse to the axis of the member shall be at least 33 percent greater than the required net section of the member. In the case of members without stiffened edges the ratio of the net width of the members (through the pin hole transverse to the axis of the member) to its thickness shall not be more than 16. Where the thickness of the main material is not sufficient to resist the load from the pin in bearing, or where the net section through the pinhole requires reinforcement, pin plates (see Clause 6.5.9 below) shall be provided and the total thickness shall comply with the above requirements.

6.5.9 Pin Plates

Pin plates shall be of sufficient thickness to make-up the required bearing or cross sectional area and shall be so arranged as to reduce the eccentricity to a minimum.

Their length measured from the centre of the pin to the end (on the reaction side) shall be at least equal to their width and at least one plate on each side shall be as wide as the dimensions of the member will allow. Pin plates shall be connected with enough bolts or welds to transmit the bearing pressure on them and shall be so arranged as to distribute it uniformly over the full section of the member.

6.5.10 Diaphragms in Members

In addition to diaphragms required for the proper functioning of the structure, diaphragms shall be provided as necessary for fabrication, transport and erection.

6.5.11 Lateral Bracings

6.5.11.1 Girders shall be provided with a lateral bracing system extending from end to- end of sufficient strength designed to transmit the effect of wind, seismic and centrifugal forces, if any to the bearings. Bracing system need not be provided if alternative system for lateral load transfer has been catered for e.g., by rigid deck.

6.5.11.2 The bracing on the loaded chord shall be so designed as to transmit to the main girders the longitudinal loads due to tractive effort and/or braking effect in order to relieve the cross girders of horizontal bending stress.

6.5.11.3 Where the depth permits, lateral diagonal bracings shall be fixed between the top chords of main girders of through span, of sufficient rigidity to maintain the chords in line and of sufficient strength to transmit the wind or seismic forces to the portal bracing between end posts.

The floor system may be taken as part of the bracing system provided it is designed for that purpose.

6.5.11.4 The lateral bracings between compression chords shall be designed to resist a transverse shear at any section equal to 2.5 percent of the total compressive force carried by both the chords at the section under consideration. This force should be considered in addition to the wind, and centrifugal forces.

6.5.11.5 Sway bracings

Wherever the depth of girder allows, the intermediate cross bracings or sway bracings between vertical web members shall be proportioned to transmit to the chord supported on bearings through the web members at least 50 percent of the panel lateral load and the vertical members shall be designed to resist the resulting bending moment. The sway bracing so provided shall not be taken as affording any relief to the lateral bracing system or portal system.

6.5.11.6 Portal bracings

Through truss spans shall be provided with suitably designed portal system, as deep as the clearance will allow. The portal system shall be designed to take the full end reaction of the top chord lateral system and the end posts of the portal shall be designed to transfer this reaction to the bearings. In addition, the portal system shall be designed to resist a lateral shear equal to 1.25 percent of the total compressive force in the end posts or in the top chords in the end panel whichever is greater.

6.6 DESIGN OF BEAMS AND PLATE GIRDERS

6.6.1 General

6.6.1.1 Beams are defined as members with solid webs (or with openings in accordance with Clause 6.6.1.4), including members of rolled and hollow section, and plate girders subjected primarily to bending.

6.6.1.2 Beams shall satisfy the deflection limitation presented in Clause 6.1.5.

6.6.1.3 The effective span of a beam shall be taken as the distance between the centres of the supports.

6.6.1.4 Openings

6.6.1.4.1 Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of Clause 6.6.2.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

6.6.1.4.2 Openings in a web may be unstiffened provided that:

a) the overall greatest internal dimension does not exceed one-tenth of the depth of the web, nor, for longitudinally, stiffened webs, one-third of the depth of the panel containing the opening;

b) the longitudinal distance between the boundaries of adjacent openings is at least three times the maximum internal dimension;

c) not more than one opening is provided at any cross-section.

6.6.2 Design Strength in Bending (Flexure)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (Clause 6.6.2.1). When a beam is not adequately supported against lateral buckling (laterally unsupported beams) the design bending strength may be governed by lateral torsional buckling strength (Clause 6.6.2.2).

The factored design moment, M at any section in a beam due to external loads shall satisfy the following requirement :

$$M < M_d$$

where, M_d = design bending strength of the section, calculated as given in Clause 6.6.2.1.2.

6.6.2.1 Laterally supported beam

A beam may be assumed to be adequately supported provided the compression flange has full lateral restraint and nominal torsional restraint at supports suitably imparted by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist, if the frictional or other positive restraint of a structural connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the deck construction is capable of resisting this lateral force.

The design bending strength of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to Clause 6.6.2.1.2.

6.6.2.1.1 Section with web susceptible to shear buckling before yielding - When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding (i.e. $d/t_w \leq 67\varepsilon$), the design bending strength shall be calculated using one of the following methods:

a) The bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (Clause 6.6.4).

b) The bending moment and axial force acting on the section may be assumed to be resisted the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

6.6.2.1.2 When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section (Clause 6.6.4), the design bending strength, M_d, shall be taken as

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than 1.2 $Z_e f_y / \gamma_{m0}$ in case of simply supported and 1.5 $Z_e f_y / \gamma_{m0}$ in cantilever beams.
where

β_b = 1.0 for plastic and compact sections.

$\beta_b = Z_e / Z_p$ for semi-compact sections

Z_p, Z_e = plastic and elastic section moduli of the cross-section, respectively

f_y = yield stress of the material

γ_{m0} = partial safety factor (Clause **4.5**)

6.6.2.1.3 When the design shear force (factored), V_t exceeds 0.6 V_d , where V_d is the design shear strength of the cross-section (Clause 6.6.4), the design bending strength, M_d shall be taken as :

$$M_d = M_{dV}$$

where M_{dV} = design bending strength under high shear as defined in Clause **6.7.2**.

6.6.2.1.4 Holes in the tension zone

a) The effect of holes in the tension flange, on the design bending strength need not be considered if

$$(A_{nf} / A_{gf}) \geq (f_y/f_u) (\gamma_{ml} / \gamma_{m0})/0.9$$

where

A_{nf} / A_{gf} = ratio of net to gross area of the flange in tension

f_y/f_u = ratio of yield and ultimate stress of the material

$\gamma_{ml} / \gamma_{m0}$ = ratio of partial safety factors against ultimate to yield stress (Clause 4.5)

When A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{ef} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in (a) above is satisfied for the complete tension zone of the cross-section, comprising the tension flange and tension region of the web.

c) Fastener holes in the compression zone of the cross-section need not be considered in design bending strength calculation, except for oversize and slotted holes, or holes without any fastener.

6.6.2.1.5 Shear lag Effects - The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge) , $b_0 \leq L_o$ /20

b) For internal elements (supported along two edges), $b_i \leq L_o$ /10

where

L_o = length between points of zero moment (inflection) in the span

b_0 = width of the flange with outstand

b_i = width of the flange as an internal element

When these limits are exceeded, the effective width of flange for design strength may be calculated using specialist literature, or conservatively taken as the value satisfying the limit given above

6.6.2.2 Laterally unsupported beams

Resistance to lateral tortional buckling need not be checked separately (member may be treated as laterally supported (Clause 6.2.1) in the following cases:

a) Bending is about the minor axis of the section;
b) Section is hollow (rectangular/tubular) or solid bars;
c) In case of major axis bending, $_{LT}$ (as defined below) is less than 0.4

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by :

$$M_d = \beta_b Z_p f_{bd}$$

where

β_b = 1.0 for plastic and compact sections.

= Z_e / Z_p for semi-compact sections

Z_p / Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress

$f_{bd} = \chi_{LT} f_y / \lambda_{mo}$

χ_{LT} = bending stress reduction factor to account for lateral torsional buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

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

α_{LT} the imperfection parameter is given by:

α_{LT} = 0.21 for rolled steel section

α_{LT} = 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT} is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p \frac{f_y}{M_{cr}}} \leq \sqrt{1.2 Z_c \frac{f_y}{M_{cr}}}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

where

M_{cr} = elastic critical moment calculated in accordance with Clause 6.6.2.2.1 and

$f_{cr,b}$= extreme fibre bending compressive stress corresponding to elastic lateral buckling moment (Clause 6.6.2.2.1)

6.6.2.2.1 Elastic lateral torsional buckling moment - In case of simply supported, prismatic members with symmetric cross-section, the elastic lateral buckling moment, M_{cr} can be determined from:

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} = \beta_b Z_p f_{cr,b}$$

$f_{cr,b}$of non-slender rolled steel sections in the above equation may be approximately calculated from the following equation :

$$f_{cr,b} = \frac{1.1\pi^2 E}{\left(\frac{L_{LT}}{r_y}\right)^2}\left[1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2\right]^{0.5}$$

The following simplified equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr}.

$$M_{cr} = \frac{\pi^2 E I_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2\right]^{0.5}$$

where

I_t = torsional constant = $\Sigma b_i t_i^3/3$ for open section

I_w = warping constant

I_y r_y = moment of inertia, radius of gyration about the weak axis, respectively

L_{LT} = effective length for lateral torsional buckling (Clause 6.6.3)

h_f = centre-to-centre distance between flanges

t_f = thickness of the flange

M_{cr} for different beam sections, considering loading, support condition and non-symmetric section, shall be calculated using the method given in Annex-C of IRC:24.

Advanced Analysis Methods like Continuous Strength Method (CSM) that takes into effect beneficial effects of strain hardening and Finite Element Method (FEM) that can include all specificities of Ramberg-Osgood model have also been proposed for critical structures.

6.6.3 Effective Length for Lateral Torsional Buckling

6.6.3.1 For simply supported beams and girders of span length, L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length L_{LT} for the lateral buckling to be used in Clause 6.6.2.2.1 shall be taken as in Table 6.6

Table 6.6 Effective Length for Simply Supported Beams, L_{LT}
(Clause 6.6.3.1)

Conditions of Restraint at Support		Loading Condition	
Torsional Restraint	**Warping Restraint**	**Normal**	**Destabilising**
Fully Restrained	Both Flanges fully restrained	0.70 L	0.85 L
Fully Restrained	Compression Flange fully restrained	0.75 L	0.90 L
Fully Restrained	Both Flanges fully restrained	0.80 L	0.95 L
Fully Restrained	Compression Flange partially restrained	0.85 L	1.00 L
Fully Restrained	Warping not restrained in both the Flanges	1.00 L	1.20 L
Partially restrained by Bottom Flange support connection	Warping not restrained in both the Flanges	1.0 L + 2D	1.2 L + 2D
Partially restrained by Bottom Flange Bearing support	Warping not restrained in both the Flanges	1.2 L + 2D	1.4 L + 2D
1) Torsional Restraint prevents rotation about longitudinal axis 2) Warping Restraint prevents rotation of the flange in its plane 3) D is the overall depth of the beam			

In simply supported beams with intermediate lateral restraints against lateral torsional buckling, the effective length for lateral torsional buckling to be used in Clause 6.6.2.2.1, L_{LT}, shall be taken as the length of the relevant segment in between the lateral restraints. The effective length shall be equal to 1.2 times the length of the relevant segment in between the lateral restraints.

Restraints against torsional rotation at supports in these beams may be provided by:

a) web or flange cleats, or

b) bearing stiffeners acting in conjunction with the bearing of the beam, or

c) lateral end frames or external supports providing lateral restraint to the compression flanges at the ends

6.6.3.2 For beams, which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in Clause 6.6.3.1, the effective length for lateral torsional buckling shall be taken as the distance, centre-to-centre, of the restraint members in the relevant segment under normal loading condition and 1.2 times the distance, where the load is not acting on the beam at the shear centre and is acting towards the shear centre so as to have destabilizing effect during lateral torsional buckling deformation.

6.6.3.3 For cantilever beams of projecting length, L, the effective length L_{LT} to be used in Clause 6.6.2.2.1 shall be taken as in Table 6.7 for different support conditions.

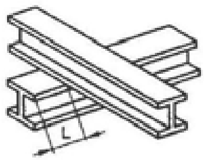
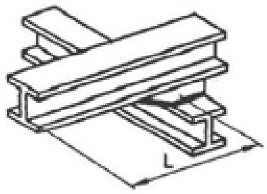
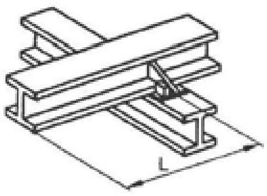
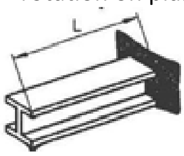
6.6.3.4 Where a member is provided with intermediate lateral supports to improve the lateral buckling strength, these restraints should have sufficient strength and stiffness to prevent lateral movement of the compression flange at the point, relative to the end supports.

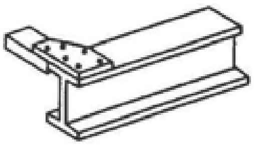
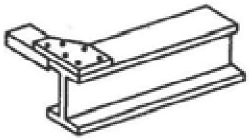
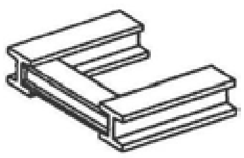
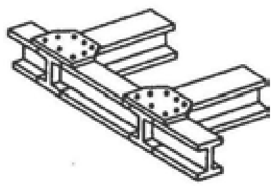
The intermediate lateral restraints should be either connected to an appropriate bracing system capable of transferring the restraint force to the effective lateral support at the ends of the member, or should be connected to an independent robust part of the structure capable of transferring the restraint force. Two or more parallel members requiring such lateral restraint shall not be simply connected together assuming mutual dependence for the lateral restraint.

The intermediate lateral restraints should be connected to the member as close to the compression flange as practicable. Such restraints should be closer to the shear centre of the compression flange than to the shear centre of the section. However, if torsional restraint preventing relative rotation between the two flanges is provided, the intermediate lateral restraint may be connected at any appropriate level.

For beams which are provided with members giving effective lateral restraint at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2.5 percent of the maximum force in the compression flange taken as divided equally between the points at which the restraint members are provided. Further, each restraint point should be capable of resisting one percent of the maximum force in the compression flange.

Table 6.7 : Effective Length LLT for Cantilever of Length L
(Clause 6.6.3.3)

Restraint Condition		Loading Condition	
at Support	at Top	Normal	Destabilising
a) Continuous, with lateral restraint to top flange 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and Torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) Continuous, with partial torsional restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and Torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) Continuous, with lateral and torsional restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and Torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) Restrainted laterally, torsionally and against rotation on plan 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and Torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L

Top restraint conditions			
i) Free 	ii) Lateral restraint to top flange 	iii) Torsional restraint 	iv) Lateral and torsional restraint 

6.6.3.4.1 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum flange force in one beam only.

6.6.3.4.2 In the case of a series of latticed beams or girders, which are connected together by the same system of restraint members, the sum of restraining forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of the force for every member of the series other than the first, upto a maximum total of 7.5 percent.

6.6.3.5 For simply supported beams where there is no lateral bracing of the compression flanges but where cross members and stiffeners forming U-Frames provide lateral restraint.

KL $= 2.5\ x\ \sqrt[4]{(E\ Ic\ \delta}$ but less than "a"

where

E = Young's Modulus

I_c = Maximum moment of inertia of compression flange about its centroidal axis parallel to the web of the girder.

a = distance between frames

δ= the lateral deflection which would occur in the U-Frame at the level of the centroid of the flange being considered when a unit force acts laterally to the U-Frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-Frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-Frame should be taken as fixed in position at each point or intersection between the cross member and a vertical as free and unconnected at all other points.

when δ is not greater than $a^3/(40\ E\ I_c)$

KL = a

In cases of symmetrical U-Frames where cross-members and stiffeners are each of constant moment of inertia throughout their own length.

$$\delta = \frac{{d_1}^3}{3EI_1} + \frac{(d_2)^2\ b}{EI_2}$$

where

d_1 = distance of the centroid of the compression flange from the top of the cross-member

d_2 = distance of the centroid of the compression flange from the neutral axis of the cross-member

b = half the distance between centres of the main girders.

I_1 = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web. A width of web plate upto 16 times the web thickness may be included on each side of centerline of connection.

I_2 = Moment of inertia of the cross member in its plane of bending

6.6.4 Shear

The factored design shear force, V in a beam due to external actions shall satisfy

$V \leq V_d$

where

V_d = design strength

$= V_n / \gamma_{m0}$

where

γ_{m0} = partial safety factor against shear failure (Clause **4.5**)

The nominal shear strength of a cross section, V_n, may be governed by plastic shear resistance (Clause 6.6.4.1) or strength of the web as governed by shear buckling (Clause 6.6.4.2).

6.6.4.1 The nominal plastic shear resistance under pure shear is given by:

$V_n = V_p$

where $V_p = \frac{A_V f_{yW}}{\sqrt{3}}$

A_v = shear area

f_{yw} = yield strength of the web

6.6.4.1.1 The shear area may be calculated as given below

I and channel sections:

Major Axis Bending :

Hot Rolled : $h\ t_w$

Welded : $d\ t_w$

Minor Axis Bending

Hot rolled or welded : $2b\ t_f$

Rectangular hollow sections of uniform thickness :

Loaded parallel to depth (h) : $Ah/(b + h)$

Loaded parallel to width (b) : $Ab/(b + h)$

Circular hollow tubes of uniform thickness : $2A/\pi$

Plates and solid bars : A

Where

A = cross-section area

b = overall breadth of tubular section, breadth of I section flanges

d = clear depth of the web between flanges

h = overall depth of the section

t_f = thickness of the flange

t_w = thickness of the web

NOTE : Fastener holes need not be accounted for in plastic design shear strength calculation provided that :

$A_{vn} \geq (f_y / f_u)(\gamma_{ml} / \gamma_{m0}) A_v / 0.9$

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Clause 6.7 may be referred to for design strength under combined high shear and bending

6.6.4.2 Resistance to shear buckling

6.6.4.2.1 Resistance to Shear buckling shall be verified as specified when

$d/t_w > 67\varepsilon$ for a web without stiffeners and

$> 67\varepsilon\sqrt{\frac{K_v}{5.35}}$

where

K_v = shear buckling coefficient (Clause 6.6.4.2.2)

6.6.4.2.2 Shear buckling design methods

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods :

a) Simple post-critical Method - The simple post critical method based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by :

$V_n = V_{cr}$

where

V_{cr} = shear force corresponding to web buckling

$= A_v . \tau_b$

where

τ_b = shear stress corresponding to web buckling, determined as follows:

a) When $\lambda_w \leq 0.8$

$\tau_b = f_{yw} / \sqrt{3}$

b) When $0.8 < \lambda_w < 1.2$

$\tau_b = [1 - 0.8(\lambda_w - 0.8)](f_{yw} / \sqrt{3})$

c) When $\lambda_w \geq 1.2$

$\tau_b = f_{yw} / (\sqrt{3} \lambda_w^2)$

where

$\lambda_w =$ non-dimensional web slenderness ratio for shear buckling stress, given by

$$\lambda_w = \sqrt{f_{yw}/\left(\sqrt{3}\tau_{cr,e}\right)}$$

The elastic critical shear stress of the web, $\tau_{cr,e}$ is given by :

$$\tau_{cr,e} = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

where

μ = Poisson's ratio

K_v = 5.35 when transverse stiffeners are provided only at supports
= $4.0 + 5.35\,(c/d)^2$ for $c/d < 1.0$
= $5.35 + 4.0\,(c/d)^2$ for $c/d \geq 1.0$

where c, d are the spacing of transverse stiffeners and depth of the web respectively

b) Tension field method - The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and if c/d ≥ 1.0, where c, d are the spacing of transverse stiffeners and depth of the web respectively.

In the tension field method, the nominal shear resistance, V_n is given by

$$V_n = V_{tf}$$

where

$$V_{tf} = \left[A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\emptyset\right] \leq V_p$$

where

τ_b = buckling strength, as obtained from Clause 6.6.4.2.2 (a)

f_v = yield strength of the tension field

$= \left[f_{yw}^2 - 3\tau_b^2 + \psi^2\right]^{0.5} - \psi$

$\psi = 1.5\,\tau_b \sin\sin 2\phi$

ϕ = inclination of the tension field = $\tan^{-1}(d/c)$

w_{rf} = width of the tension field

$= d\cos\phi + (c - S_c - S_t)\sin\phi$

f_{yw} = yield stress of the web

d = depth of the web

c = spacing of the stiffeners of the web

τ_b = shear stress corresponding to buckling of web (Clause 6.6.4.2.2(a))

S_c, S_t= anchorage lengths of tension field along the compression and tension flange respectively, obtained from :

$$s = \frac{2}{sin\phi}\left[\frac{M_{fr}}{f_{yw}t_w}\right]^{0.5} \leq c$$

where

M_{fr} = reduced plastic moment capacity of the respective flange plate (disregarding any edge stiffener) after accounting for the axial force, N_t in the flange, due to overall bending and any external axial force in the cross-section, and is calculated as given below:

$$M_{fr} = 0.25 b_f t_f^2 f_{yf}\left[1 - \left\{\frac{N_f}{\left(\left(\frac{b_f t_f f_{yf}}{\gamma_{m0}}\right)\right)}\right\}^2\right]$$

where

b_f, t_f = width and thickness of the relevant flange respectively

f_{yf}= Yield stress of the flange

6.6.5 Stiffened Web Panels

6.6.5.1 End panels design (Fig. 6.6) -The design of end panels in girders in which the interior panel (panel A) is designed using tension field action shall be carried out in accordance with the provisions given herein. In this case the end panel should be designed using only Simple Post Critical Method, according to Clause 6.6.4.2.2 (a).

Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges to resist a shear force, R_{tf}, and a moment, M_{tf}, due to tension field forces as given in Clause 6.6.5.3. Further, end stiffener should be capable of resisting the reaction plus a compressive force due to moment, equal to M_{tf} , (Fig. 6.6)

NOTES:

1) Panel A is designed utilizing tension field action, as given in Clause 6.6.4.2.2(b)

2) Panel B is designed without utilizing tension filed action, as given in Clause 6.6.4.2.2(a).

3) Bearing stiffener is designed for the compressive force due to bearing plus comprehensive force due to the moment M_{tf} as given in Clause 6.6.5.3.

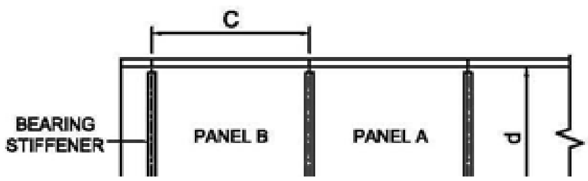


Figure 6.6 : End Panel Designed Not Using Tension Field Action

6.6.5.2 End panels designed using tension field action (Figs. 6.7 and 6.8) - The design of end panels in girders which are designed using tension field action shall be carried out in accordance with the provisions mentioned herein. In this case, the end panel (Panel B) shall be designed according to Clause 6.6.4.2.2(b).

Additionally it should be provided with an end post consisting of a single or double stiffener, (Figs. 6.7 and 6.8), satisfying the following :

a) Single stiffener (Fig. 6.7) - The top of the end post should be rigidly connected to the flange using full strength welds.

   The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to 2/3 M_{tf} due to tension field forces, where M_{tf} is obtained from Clause 6.6.5.3. The width and thickness of the end post are not to exceed the width and thickness of the flange.

b) Double stiffener (Fig. 6.8) - The end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment, M_{tf} due to the tension field forces as given in Clause 6.6.5.3

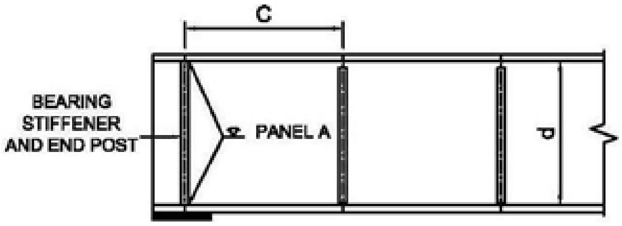


Figure 6.7 : End Panel Designed Using Tension Field Action (Single

NOTES:

1) Panel A is designed utilizing tension field action as given in Clause 6.6.4.2.2(b)
2) Bearing stiffener and end post is designed for combination of compressive loads due to bearing and a moment equal to 2/3 M_{tf} as given in 6.6.5.3.

NOTES:

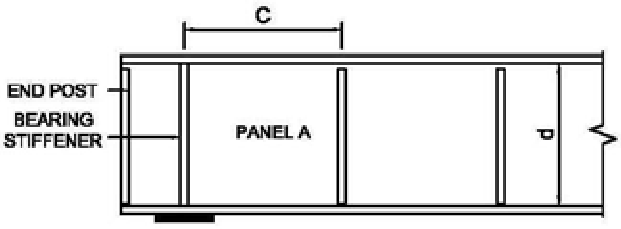


Figure 6.8 : End Panel Designed Using Tension Field Action (Double Stiffener)

1) Panel A is designed utilizing tension field action, as given in Clause 6.6.4.2.2(b)

2) Bearing stiffener is designed for compressive force due to bearing as given in Clause 6.6.4.2.2(a).

3) End post is designed for horizontal shear R_{tf} and moment M_{tf} as given in Clause 6.6.5.3.

6.6.5.3 Anchor forces - The resultant longitudinal shear, R_{tf} and a moment M_{tf} from the anchor of tension field forces are evaluated as given below :

$$R_{if} = \frac{H_q}{2} \ \ and\ M_{tf} = \frac{H_q d}{10}$$

$$where\ H_q = 1.25V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{\frac{1}{2}};\ \ V_p = \frac{dtf_y}{\sqrt{3}}\ ;\ d = web\ depth$$

If the actual factored shear force, V in the panel designed using tension field approach is less than shear strength, V_{tf} (Clause 6.6.4.2.2b), then the values of H_q may be reduced by the ratio;

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}}$$

where

V_{tf} = the basic shear strength for the panel utilizing tension field action as given in Clause 6.6.4.2.2b

V_{cr} = critical shear strength for the panel designed utilizing tension field action as given in Clause 6.6.4.2.2a

6.6.5.4 Panels with openings - Panels with opening of dimension greater than 10 percent of the minimum panel dimension should be designed without using tension field action as given in Clause 6.6.4.2.2(b). The adjacent panels should be designed as an end panel as given in Clause 6.6.5.1 or Clause 6.6.5.2 as appropriate.

6.6.6 Design of Beams and Plate Girders

6.6.6.1 Minimum web thickness - The thickness of the web in a section shall satisfy the following requirements:

6.6.6.1.1 Serviceability requirement

a) When transverse stiffeners are not provided,

$\frac{d}{t_w} \le 200\varepsilon$ (web connected to flanges along both longitudinal edges)

$\frac{d}{t_w} \le 90\varepsilon$ (web connected to flanges along one longitudinal edge only)

b) when only transverse stiffeners are provided (in webs connected to flanges along both longitudinal edges).

i) when $3d \ge c \ge d$,

$\frac{d}{t_w} \le 200\ \varepsilon_w$

ii) when $0.74d \le c < d$,

$\frac{c}{t_w} \le 200\ \varepsilon_w$

iii) when $c < d$

$$\frac{d}{t_w} \leq 270\ \varepsilon_w$$

iv) when c > 3d, the web shall be considered as unstiffened.

c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange) as per Clause 6.6.7.12(a)

i) when 2.4d ≥ c ≥ d,

$$\frac{d}{t_w} \leq 250\ \varepsilon_w$$

ii) when 0.74 d ≤ c ≤ d

$$\frac{c}{t_w} \leq 250\ \varepsilon_w$$

iii) when c < 0.74 d,

$$\frac{d}{t_w} \leq 340\ \varepsilon_w$$

d) when a second longitudinal stiffener (located at neutral axis) is provided.

$$\frac{d}{t_w} \leq 400\ \varepsilon_w$$

where
d = depth of the web
t_w = thickness of the web
c = spacing of transverse stiffener (Fig.6.6, Fig. 6.7)
ε_w = S(250 / f_{yw})
f_{yw} = yield stress of the web

6.6.6.1.2 Compression flange buckling requirement - In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

a) when transverse stiffners are not provided.

$$\frac{d}{t_w} \leq 345\ \varepsilon_f^2$$

b) when transverse stiffeners are provided and

i) when c ≥ 1.5 d,

$$\frac{d}{t_w} \leq 345\ \varepsilon_f^2$$

ii) when c < 1.5 d

$$\frac{d}{t_w} \leq 345\ \varepsilon_f^2$$

where
d = depth of the web
t_w = thickness of the web
c = spacing of transverse stiffener (Figs. 6.6 & 6.7)

$\varepsilon_f = S(250 / f_{yf})$
f_{yf} = yield stress of compression flange

6.6.6.2 Sectional properties

6.6.6.2.1 The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members (Clause 6.4) and for open holes occurring in a plane perpendicular to the direction of stress at the section being considered (Clause 6.6.2.1.4).

The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in Clause 6.6.2.1.4.

The effective sectional area for parts in shear shall be taken as specified in Clause 6.6.4.1.1.

6.6.6.3 Flanges

6.6.6.3.1 In bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than one-third) and the number of flange plates shall be kept to a minimum.

In exposed situations, where flange angles are used, at least one plate of the top flange shall extend over the full length of the girder, unless the top edge of the web is machined flush with the flange angles. Where two or more flange plates are used, tacking welds shall be provided, if necessary to comply with the requirements of Clause**7**.

Each flange plate shall extend beyond its theoretical cut-off point, and the extension shall contain sufficient, bolts or welds to develop in the plate, the load calculated for the bending moment on the girder section (taken to include the curtailed plate) at the theoretical cut-off point.

The outstand of flange plates, that is the projection beyond the outer line of connections to flange angles, channel or joist flanges, or, in the case of welded constructions, their projection beyond the face of the web or tongue plate, shall not exceed the values given in Clause **5.2.2** (Table 5.1).

6.6.6.3.2 In welded construction, the use of curtailed flange plates shall be avoided as far as possible, local strengthening being provided by other means such as inserting by butt welding a thicker and or wider plate. The heavier section plate shall be suitably tapered to the lighter plate. If, in welded construction the use of curtailed flange plates cannot be avoided, the end of the plate shall be tapered in plan to a rounded end and all welds shall be continuous round the ends.

6.6.6.3.3 Flange splices

Flange splices preferably should not be located at points of maximum stress. Where splice plates are used, their area shall not be less than 5 percent in excess of the area of the flange element spliced; their centre of gravity shall coincide, as nearly possible, with that of the element spliced. There shall be enough bolts or welds on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. In welded construction, flange plates shall be joined by complete penetration butt welds, wherever possible. These butt welds shall develop the full strength of the plates.

6.6.6.3.4 Connection of flanges to web

The flanges of plate girders shall be connected to the web by sufficient bolts or welds to transmit the maximum horizontal shear force resulting from the bending moment gradient in the girder, combined with any vertical loads which are directly applied to the flange. If the web is designed using tension field method as given in Clause 6.6.4.2.2(b), then the weld should be able to transfer the tension field stress, f_{yw}, acting on the web.

6.6.6.3.5 Bolted construction

For girders in exposed situations and which do not have flange plates for their entire length, the top edge of the web plate shall be flush with or above the angles, and the bottom edge of the web plate shall be flush with or set back from the angles

6.6.6.3.6 Welded construction

The gap between the web plates and flange plates shall be kept to a minimum and for fillet welds, shall not exceed 1 mm at any point before welding.

6.6.6.4 Webs

6.6.6.4.1 Effective sectional area of web of plate girder - The effective cross-sectional area shall be taken as the full depth of the web plate multiplied by the thickness.

NOTE : Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like, or where the proportion of the web included in the flange area is 25 percent or more of the overall depth, the above approximation is not permissible and the maximum shear stress shall be computed based on theory.

6.6.6.4.2 Splices in webs

Splices and cut-outs for service ducts in the webs preferably should not be located at points of maximum shear force and heavy concentrated loads.

Splices in the webs of the plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In bolted construction, splice plate shall be provided on each side of the web. In welded construction, web splices shall preferably be made with complete penetration butt welds. Where this is not possible, splice plates on both sides should be used (Refer to Clause 7).

6.6.6.4.3 Where additional plates are required to augment the strength of the web, they shall be placed on each side of the web and shall be equal in thickness. The proportion of shear force, assumed to be resisted by these plates shall be limited by the amount of horizontal shear which they can transmit to the flanges through their fastenings, and such reinforcing plates and their fastenings shall be carried up to the points at which the flange without the additional plates is adequate (Refer to Clause7).

6.6.6.5 Design of Steel Concrete Composite Girders

Steel Concrete Composite Girders shall be designed and detailed as per the provisions of IRC:22 – "Standard Specifications and Code of Practice for Road Bridges : Section VI : Composite Construction". The shear connectors shall be of same grade of stainless steel as the girder material and the slab reinforcement bars should preferably be epoxy coated or stainless steel.

6.6.7 Design of Stiffeners

6.6.7.1 General

6.6.7.1.1 When the web of a member acting alone (that is without stiffeners) proves inadequate, stiffeners for meeting the following requirements should be provided.

a) Intermediate transverse web stiffeners - To improve the buckling strength of a slender web due to shear (Clause 6.6.7.2).

b) Load carrying stiffener - To prevent local buckling of the web due to concentrated loading (Clauses 6.6.7.3 and 6.6.7.5).

c) Bearing stiffener - To prevent local crushing of the web due to concentrated loading (Clauses 6.6.7.4 and 6.6.7.6).

d) Diagonal stiffener - To provide local reinforcement to a web under shear and bearing (Clause 6.6.7.7).

e) Tension stiffener - To transmit tensile forces applied to a web through a flange (Clause 6.6.7.8).

f) Torsion stiffener - To provide torsional restraint to beams and girders at supports (Clause 6.6.7.9).

The same stiffeners may perform more than one function and their design should comply with the requirements of all the functions designed for.

6.6.7.1.2 Outstand of web stiffeners - Unless the outer edge is continuously stiffened,the outstand from the face of the web should not exceed $20t_q\varepsilon$.

When the outstand is between $14t_q\varepsilon$ and $20t_q\varepsilon$, then the stiffener design should be on the basis of a core section with an outstand of $14t_q\varepsilon$.

where

t_q = thickness of the stiffener

6.6.7.1.3 Stiff bearing length - The stiff bearing length of any element b_1, is that length which cannot deform appreciably in bending. To determine b_1 the dispersion of load through a steel bearing element should be taken as 45^0 through solid material, such as bearing plates, flange plates etc. (Fig. 6.9)

6.6.7.1.4 Eccentricity - Where a load or reaction is applied eccentric to the centre line of the web or where the centroid of the stiffener does not lie on the centre line of the web, the resulting eccentricity of loading should be accounted for in the design of the stiffener.

6.6.7.1.5 Buckling resistance of stiffener - The buckling resistance F_{qd}, should be based on the design compressive stress f_{cd} (Clause 6.4.1.2.1) of a strut (curve c), the radius of gyration being taken about the axis parallel to the web. The effective section is the full area or core area of the stiffener (Clause 6.6.7.1.2) together with an effective length of web on each side of the centre line of the stiffeners, limited to 20 times the web thickness. The design strength used should be the minimum value obtained for buckling about the web or the stiffener.

The effective length of intermediate transverse stiffeners used in calculating the buckling resistance, F_{qd} should be taken as 0.7 times the length, L, of the stiffener.

The effective length for load carrying web stiffeners used in calculating the buckling

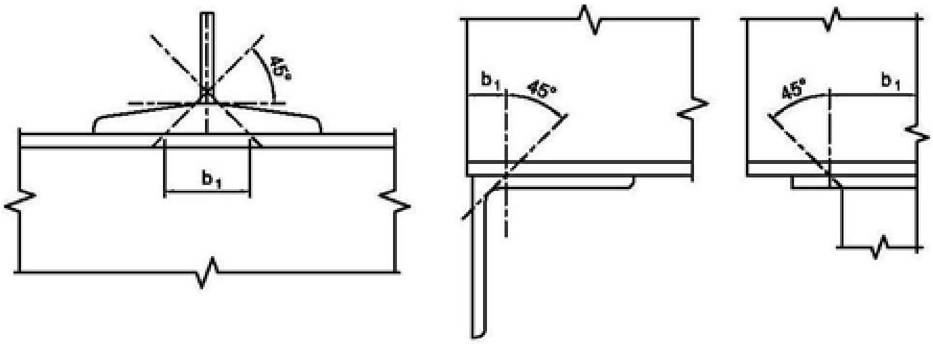


Figure 6.9 : Stiff Bearing Length, b_1

resistance, F_{xd}, assumes that the flange through which the load or reaction is applied is effectively restrained against lateral movement relative to the other flange, and length should be taken as:

a) KL = 0.7 L when flange restrained against rotation in the plane of the stiffener (by other structural elements)

b) KL = L when flange not so restrained

where

L = length of the stiffener

If the load or reaction is applied to the flange by a compression member, then unless effective lateral restraint is provided at that point, the stiffener should be designed as part of the compression member applying the load, and the connection between the compression member and the beam flange shall be checked for the effects of the strut action.

6.6.7.2 Design of intermediate transverse web stiffeners

6.6.7.2.1 General - Intermediate transverse stiffeners may be provided on one or both sides of the web.

6.6.7.2.2 Spacing - Spacing of intermediate stiffeners, where they are provided, shall comply with Clause 6.6.6.1 depending on the thickness of the web.

6.6.7.2.3 Outstand of Stifferers - The outstand of the stiffeners should comply with Clause 6.6.7.1.2.

6.6.7.2.4 Minimum stiffeners - Transverse web stiffeners not subject to external loads or moments should have a second moment of area, I_s about the centreline of the web (if stiffeners are on both sides of the web) and about the face of the web (if single stiffener on only one side of the web is used) such that:

if $c/d \geq S2$ $\qquad$ $I_s \geq 0.75\ d\ t_w^3$

and if $c/d < \sqrt{2}$ $I_s \geq 1.5\, d^3\, t_w^3 / c^2$

where

d = depth of the web

t_w = minimum required web thickness for spacing using tension field action, as given in Clause 6.6.4.2.1.

c = actual stiffener spacing

6.6.7.2.5 Buckling check on intermediate transverse web stiffeners - Stiffeners not subjected to external loads of moments should be checked for a stiffener force:

$F_q = V - V_{cr} / \gamma_{m0} \leq F_{qd}$

where

F_{qd} = design resistance of the intermediate stiffeners

V = factored shear force adjacent to the stiffener

V_{cr} = shear buckling resistance of the web panel designed without using tension field action Clause 6.6.4.2.2(a).

Stiffeners subject to external loads and moments should meet the conditions for load carrying web stiffeners in Clause 6.6.7.3. In addition they should satisfy the following interaction expression.

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

If $F_q < F_x$ then $(F_q - F_x)$ should be taken as zero.

where

F_q = stiffener force given above

F_{qd}= design resistance of an intermediate web stiffener corresponding to buckling about an axis parallel to the web (Clause 6.6.7.1.5).

F_x= external load or reaction at the stiffener

F_{xd}= design resistance of a load carrying stiffener corresponding to buckling about axis parallel to the web (Clause 6.6.7.1.5).

M_q = moment on the stiffener due to eccentrically applied load and transverse load, if any.

M_{yq} = yield moment capacity of the stiffener based on its elastic modulus about its centroidal axis parallel to the web.

6.6.7.2.6 Connection of intermediate stiffeners to web - Intermediate transverse stiffeners not subject to external loading should be connected to the web so as to withstand a shear between each component of the stiffener and the web (in kN/mm) of not less than

$t_w^2 / (5b_s)$

where

t_w = web thickness (in mm)

b_s = outstand width of the stiffener (in mm)

For stiffeners subject to external loading, the shear between the web and the stiffener due to such loading has to be added to the above value.

Stiffeners not subject to external loads or moments may terminate clear of the tension flange and in such a situation the distance cut short from the line of the weld should not be more than $4t_w$.

6.6.7.3 Load carrying stiffeners

6.6.7.3.1 Web Check - Load carrying web stiffeners should be provided where compressive forces applied through a flange by loads or reactions exceed the buckling strength, F_{cdw}, of the unstiffened web, calculated using the following:

The effective length of the web for evaluating the slenderness ratios is calculated as in Clause 6.6.7.1.5. The area of cross section is taken as

$(b_1 + n_1)t_w$

where

b_1 = width of stiff bearing on the flange (Clause 6.6.7.1.3).

n_1 = dispersion of the load through the web at 45^0, to the level of half the depth of the cross section.

The buckling strength of this web about axis parallel to the web is calculated as given in Clause 6.4.1.2.1 using curve 'c'.

6.6.7.4 Bearing stiffeners

Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceed the local capacity of the web at its connection to the flange, F_w as given below:

$F_w = (b_1 + n_2)\, t_w f_{yw} / \gamma_{m0}$

where

b_1 = stiff bearing length (Clause 6.6.7.1.3)

n_2 = length obtained by dispersion through the flange to the web junction at a slope of 1:2.5 to the plane of the flange.

t_w = thickness of the web

f_{yw} = yield stress of the web

6.6.7.5 Design of load carrying stiffeners

6.6.7.5.1 Buckling check - The external load or reaction, F_x, on a stiffener should not exceed the buckling resistance, F_{xd}, of the stiffener as given in Clause 6.6.7.1.5.

Where the stiffener also acts as an intermediate stiffener it should be checked for the effect of combined loads in accordance with Clause 6.6.7.2.5.

6.6.7.5.2 Bearing check - Load carrying web stiffeners should also be of sufficient size that the bearing strength of the stiffener, F_{psd}, given below is not less than the load transferred, F_x.

$$F_{psd} = A_q f_{yq} / (0.8 \lambda_{m0}) \geq F_x$$

where

F_x = external load or reaction

A_q = area of the stiffener in contact with the flange

f_{yq} = yield stress of the stiffener

6.6.7.6 Design of bearing stiffeners

Bearing stiffeners should be designed for the applied load or reaction less the local capacity of the web as given in Clause 6.6.7.4. Where the web and the stiffener material are of different strengths the lesser value should be assumed to calculate the capacity of the web and the stiffener. Bearing stiffeners should project nearly as much as the overhang of the flange through which load is transferred.

6.6.7.7 Design of diagonal stiffeners

Diagonal stiffeners should be designed to carry the portion of the applied shear and bearing that exceeds the capacity of the web.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in Clause 6.6.7.6.

6.6.7.8 Design of tension stiffeners

Tension stiffeners should be designed to carry the portion of the applied load or reaction less the capacity of the web as given in Clause 6.6.7.4 for bearing stiffeners.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in Clause 6.6.7.6.

6.6.7.9 Torsional stiffeners

Where bearing stiffeners are required to provide torsional restraint at the supports of the beam, they should meet the following criteria:

a) Conditions of Clause 6.6.7.4

b) Second moment of area of the stiffener section about the centreline of the web, I_s, should be such that :

$$I_s \geq 0.34 \alpha_s D^3 T_{cf}$$

where

α_s = 0.006 for $L_{LT}/ r_y \leq 50$

= 0.3 /(L_{LT}/r_y) for 50 < L_{LT}/r_y = 100

= $30/(L_{LT}/r_y)^2$ for $L_{LT}/ r_y > 100$

D = overall depth of beam at support

T_{cf} = maximum thickness of compression flange in the span under consideration

KL = laterally unsupported effective length of the compression flange of the beam

r_y = radius of gyration of the beam about the minor axis

6.6.7.10 Connection of load carrying and bearing stiffeners to web - Stiffeners, which resist loads or reactions applied through a flange, should be connected to web by sufficient welds or fasteners to transmit a design force equal to the lesser of:

a) The tension capacity of the stiffener
b) The sum, of the forces applied at the two ends of the stiffener when they act in the same direction or the larger of the forces when they act in opposite directions.

Stiffeners, which do not extend right across the web, should be of such length that the shear stress in the web due to the design force transmitted by the stiffener does not exceed the shear strength of the web. In addition, the capacity of the web beyond the end of the stiffener should be sufficient to resist the applied force.

6.6.7.11 Connection to flanges

6.6.7.11.1 In tension - Stiffeners required to resist tension should be connected to the flange transmitting the load by continuous welds or non-slip fasteners.

6.6.7.11.2 In compression - Stiffeners required to resist compression should either be fitted against the loaded flange or connected by continuous welds or non-slip fasteners.

The stiffener should be fitted against or connected to both flanges when:

a) a load is applied directly over a support; or
b) it forms the end stiffener of a stiffened web; or
c) it acts as a torsion stiffener

6.6.7.12 Hollow sections - Where concentrated loads are applied to hollow sections, consideration should be given to local stresses and deformations and the section reinforced as necessary.

6.6.7.13 Longitudinal stiffeners - Where longitudinal stiffeners are used in addition to transverse stiffeners, they shall be as follows :

a) One longitudinal stiffener shall be placed on the web at a distance from the compression flange equal to 1/5 of the distance from the compression flange angle, plate or tongue plate to the neutral axis when the thickness of the web is less than the limit specified in Clause 6.6.6.1. The stiffener shall be designed so that I_s is not less that $4ct_w^3$ where I_s and t_w are as defined in Clause 6.6.7.2.4 and c is the actual distance between the transverse stiffeners.

b) A second longitudinal stiffener (single or double) shall be placed at the neutral axis of the girder when the thickness of the web is less than the limit specified in Clause 6.6.6.1. This stiffener shall be designed so that I_s is not less that $d_2t_w^3$ where I_s and t_w are as defined in Clause 6.6.7.2.4 and d_2 is twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis.

c) Longitudinal web stiffeners shall extend between vertical stiffeners, but need not be continuous over them.

d) Longitudinal stiffeners may be in pairs arranged on each side of the web, or single located on one side of the web.

6.6.7.14 Detailing requirements

a) Load bearing stiffeners should be in pairs (that is two legs of plates, angles etc.) placed symmetrically at both sides of the web. When the condition is not met the effect of the resulting eccentricity should be considered.

b) The ends of the load bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web.

c) Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

d) Outstanding legs of each pair of load bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds, does not exceed the design bearing strength.

e) Load bearing stiffeners consisting of two legs shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to twenty times the web thickness (but limited to the edge distance of the web and half the distance of the adjacent stiffener).

In case of bearing stiffeners consisting of four or more legs, the effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs and a portion of web plate not exceeding the length of the web as specified for single leg stiffeners on the outer sides of the outer legs.

f) The load bearing stiffeners shall be provided with sufficient bolts or welds to transmit to the web the whole of the load in the stiffeners.

g) In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t where t is the thickness of the web plate.

h) Where transverse stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than 1.5 d_1 and not less than 0.33 d_1, where d_1 is the depth as defined in clause 6.6.8. Where longitudinal stiffeners are provided d_1 shall be taken as the clear distance between the horizontal stiffener and the farthest flange ignoring fillets.

NOTE : If the thickness of the web is made greater, or the spacing of stiffener made smaller than that required by the standard, the moment of inertia of the stiffener need not be correspondingly increased.

Intermediate transverse stiffeners, when not acting as load bearing stiffeners, may be joggl ed and may be single or in pairs placed one on each side of the web. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange. They can be connected or fitted to, or kept well clear of the flanges.

6.6.8 Lateral Bracings

All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings all lateral forces due to wind, seismic effect etc, as applicable.

6.6.9 Expansion and Contraction

In all bridges, provision shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provision shall also be made for changes in length of span resulting from live loads.

6.7 MEMBERS SUBJECTED TO COMBINED FORCES

6.7.1 General

This clause governs the design of members subjected to combined forces such as shear and bending, axial force and bending, or shear force, axial force and bending.

6.7.2 Combined Shear and Bending

6.7.2.1 No reduction in moment capacity of the section is necessary as long as the cross-section is not subjected to high shear force (factored value of applied shear force is less than or equal to 60 percent of the shear strength of the section as given in Clause 6.6.4. The moment capacity may be taken as, M_d (Clause 6.6.2) without any reduction.

6.7.2.2 When the factored value of the applied shear force is high (exceeds the limit in Clause 6.7.2.1) the factored moment of the section should be less than the moment capacity of the section under higher shear force M_{dv} calculated as given below:

a) Plastic or compact section

$$M_{dv} = M_d - \beta\,(M_d - M_{fd}) \leq 1.2\, Z_e f_y/\gamma_{m0}$$

where

$\beta = (2V/V_d - 1)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect (Clause 6.6.2.1.2) considering web buckling effects (Clause 6.6.2.1.1)

V = factored applied shear forces as governed by web yielding or web buckling.

V_d = design shear strength as governed by web yielding or web buckling (Clause 6.6.4.1 or 6.6.4.2)

M_{fd} = plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{m0}

Z_e= elastic section modulus of the whole section

b) Semi-compact Section

$M_{dv} = Z_e f_y / \gamma_{m0}$

6.7.3 Combined Axial Force and Bending Moment

Under combined axial force and bending moment section strength as governed by material failure and member strength as governed by buckling failure shall to be checked in accordance with Clauses 6.7.3.1 and 6.7.3.2., respectively.

6.7.3.1 Section strength

6.7.3.1.1 Plastic and compact sections - In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied.

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha 1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha 2} \leq 1.0$$

Conservatively, the following equation may be used under combined axial force and bending moment.

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

Where

M_y, M_z = factored applied moments about the minor and major axis of the cross section, respectively

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, (Clause 6.7.3.1.2)

N = factored applied axial force (Tension T, or Compression P)

N_d = design strength in tension (T_d) as obtained from Clause 6.3 or in compression due to yielding given by : $N_d = A_g f_y / \gamma_{m0}$

M_{dy}, M_{dz} = design strength under corresponding moment acting alone, (Clause 6.6.2)

A_g = gross area of the cross section

α_1 , α_2= constants as given in Table 6.8

λ_{m0} = partial factor of safety in yielding

Table-6.8 Constants α_1 and α_2

Section	**α_1**	**α_2**
I and Channel	$5\ n \geq 1$	2
Circular Tubes	2	2
Rectangular Tubes	$1.66 / (1 - 1.13\ n^2) \leq 6$	$1.66 / (1 - 1.13\ n^2) \leq 6$
Solid Rectangles	$1.73 + 1.8\ n^3$	$1.73 + 1.8\ n^3$

6.7.3.1.2 For plastic and compact sections without bolt holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}

a) Plates

$M_{nd} = M_d\ (1 - n2)$

b) Welded I or H section

$$M_{ndy} = M_{dy}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy}\ where\ n \geq a$$

$$M_{ndz} = M_{dz}\frac{(1-n)}{(1-0.5a) \leq M_{dz}}$$

where $n = N/N_d$ and $a = (A - 2\ b\ t_f) / A \leq 0.5$

c) For standard I or H sections

for $n \leq 0.2$ $M_{ndy} = M_{dy}$

for $n > 0.2$ $M_{ndy} = 1.56\ M_{dy}\ (1 - n)(n + 0.6)$

$M_{ndz} = 1.11\ M_{dz}\ (1 - n) \leq M_{dz}$

d) For rectangular hollow sections and welded box sections – When the section is symmetric about both axes and without bolt holes

$M_{ndy} = M_{dy}\ (1-n) / (1-0.5\ a_f) \leq M_{dy}$

$M_{ndz} = M_{dz}\ (1-n) / (1-0.5\ a_w) \leq M_{dz}$

where $a_w = (A-2\ b\ t_f) / A \leq 0.5$

$a_f = (A-2\ h\ t_w) / A \leq 0.5$

e) Circular hollow tubes without bolt holes

$M_{nd} = 1.04\ M_d\ (1 - n^{1.7}) \leq M_d$

6.7.3.1.3 Semi-compact section - In the absence of high shear force (Clause 6.7.2.1) semi-compact section design is satisfactory under combined axial force and bending, if the maximum longitudinal stress under combined axial force and bending, fx, satisfies the following criteria.

$f_x \leq f_y / \gamma_{m0}$

For cross section without holes, the above criteria reduces to

$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 0.5$

where

N_d, M_{dy}, M_{dz} are defined in Clause 6.7.3.1.1

6.7.3.2 Overall member strength

Members subjected to combined axial force and bending moment shall be checked for overall buckling failure as given in this section.

6.7.3.2.1 Bending and axial tension - The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d (Clause 6.6.2.2)

$M_{eff} = [M - \psi\, T\, Z_{ec} / A] \leq M_d$

where

M, T = factored applied moment and tension, respectively

A = area of cross-section

Z_{ec} = elastic section modulus of the section with respect to extreme compression fibre

ψ = 0.8 if T and M vary independently or otherwise

= 1.0

6.7.3.2.2 Bending and axial compression - Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships.

$$\frac{P}{P_{dy}} + k_y \frac{C_{my} M_y}{M_{dy}} + k_{LT} \frac{M_z}{M_{dz}} \leq 1.0$$

$$\frac{P}{P_{dz}} + 0.6 k_y \frac{C_{my} M_y}{M_{dy}} + k_z \frac{C_{mz} M_z}{M_{dz}} \leq 1.0$$

where

C_{my}, C_{mz} = equivalent uniform moment factor as per Table6.9

P = applied axial compression under factored load

M_y , M_z = maximum factored applied bending moments about y and z-axis of the member, respectively.

P_{dy} , P_{dz} = design strength under axial compression as governed by buckling about minor (y) and major (z) axis respectively.

M_{dy} , M_{dz} = design bending strength about y (minor) or z (major) axis of the cross section (Clause 6.6)

$$k_y = 1 + (\lambda_y - 0.2)n_y \leq 1 + 0.8\, n_y$$

$$k_z = 1 + (\lambda_z - 0.2)n_z \leq 1 + 0.8\, n_z$$

$$k_{LT} = 1 - \frac{0.1\lambda_{LT} n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1 n_y}{(C_{mLT} - 0.25)}$$

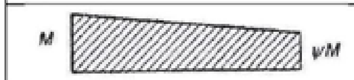
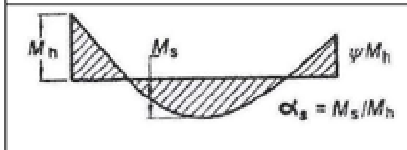
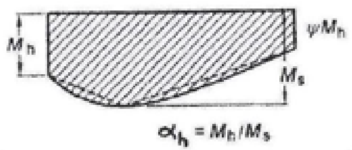
where

n_y , n_z = ratio of actual applied axial force to the design axial strength for buckling about the y and z axis, respectively and

C_{mLT} = equivalent uniform moment factor for lateral torsional buckling as per Table 6.9 corresponding to the actual moment gradient between lateral supports against torsional deformation in the critical region under consideration.

TABLE 6.9 : Equivalent Uniform Moment Factor

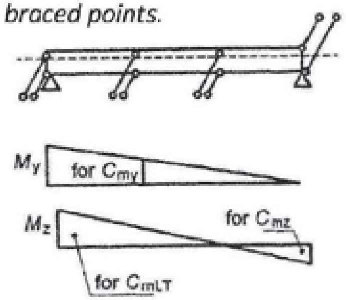
(Clause 6.7.3.2.2)

Bending moment diagram	Range		C_{my} , C_{mz}, C_{mLT}	
			Uniform loading	Concentrated load
	$-1 < \psi < 1$		$0.6 + 0.4\ \psi > 0.4$	
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0.2 + 0.8\ \alpha_s \geq 0.4$	$0.2 + 0.8\ \alpha_s \geq 0.4$
	$-1 \leq \alpha_s \leq 0$	$0 \leq \psi \leq 1$	$0.1 - 0.8\ \alpha_s \geq 0.4$	$-0.8\ \alpha_s \geq 0.4$
		$-1 \leq \psi \leq 0$	$0.1(1-\psi) - 0.8\ \alpha_s \geq 0.4$	$0.2(1-\psi) - 0.8\ \alpha_s \geq 0.4$
	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0.095 - 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
	$-1 \leq \alpha_h \leq 0$	$0 \leq \psi \leq 1$	$0.095 + 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
		$-1 \leq \psi \leq 0$	$0.95 + 0.05\ \alpha_h\ (1 + 2\ \psi)$	$0.95 + 0.05\ \alpha_h\ (1 + 2\ \psi)$

For members with sway buckling the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$

C_{my} , C_{mz}, C_{mLT} *shall be obtained according to the bending moment diagram between the relevant braced points.*

Moment factor	Bending axis	Points braced in direction
C_{my}	z-z	y-y
C_{mz}	y-y	z-z
C_{mLT}	z-z	z-z



7. Connections

7.1 General

7.1.1 The term "connection" applies to all joints between different components of a structural member, joints between separate structural members and splices in members. The term "fasteners" applies to bolts and pins.

7.2 Basis of Design

7.2.1 All connections should satisfy the provisions of Clause 7for the ultimate limit state.

7.2.2 The fatigue consideration should be in accordance with the recommendations of Clause **9** of this Code.

7.2.3 The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure and comply with the requirement specified in this Clause. Connections shall be capable of transmitting the calculated design loads and moments communicated by the members.

7.2.4 Where members are connected to the surface of a web or flange of a section, the ability of the web or flange to transfer the applied forces locally should be checked and local stiffening provided where necessary.

7.2.5 Ease of fabrication and erection, as also subsequent inspection and maintenance should be considered in the design of connections. The following may be considered in this respect.

- use of standardised details
- the clearances necessary for safe erection
- the clearances needed for tightening fasteners,
- the need for access for welding,
- the requirements of welding procedures,
- the effects of angular and length tolerances on fit-up.

7.2.6 In general, use of different forms of fasteners to transfer the same force shall be avoided. Sufficient number of one type of fastening shall be provided to transmit the entire load for which the connection is designed.

7.2.7 The partial safety factor in the evaluation of design strength of connections shall be taken as given in **Table 4.1**.

7.2.8 The design of joints, in particular, needs the most careful attention to maintain optimum corrosion resistance. This is especially so for joints that may become wet from the weather, spray, immersion, or condensation, etc. The possibility of avoiding or reducing associated corrosion problems by locating joints away from the source of dampness should be investigated. Alternatively, it may be possible to remove the source of dampness; for

instance, in the case of condensation, by adequate ventilation or by ensuring that the ambient temperature within the structure lies above the dew point temperature.
Where it is not possible to prevent a joint involving carbon steel and stainless steel from becoming wet, consideration should be given to preventing bimetallic corrosion. The use of carbon steel bolts with stainless steel structural elements should always be avoided. In bolted joints that would be prone to an unacceptable degree of corrosion, provision should be made to isolate electrically the carbon steel and stainless steel elements. This entails the use of insulating washers and possibly bushes; typical suitable details are shown in Figure 7.1 for

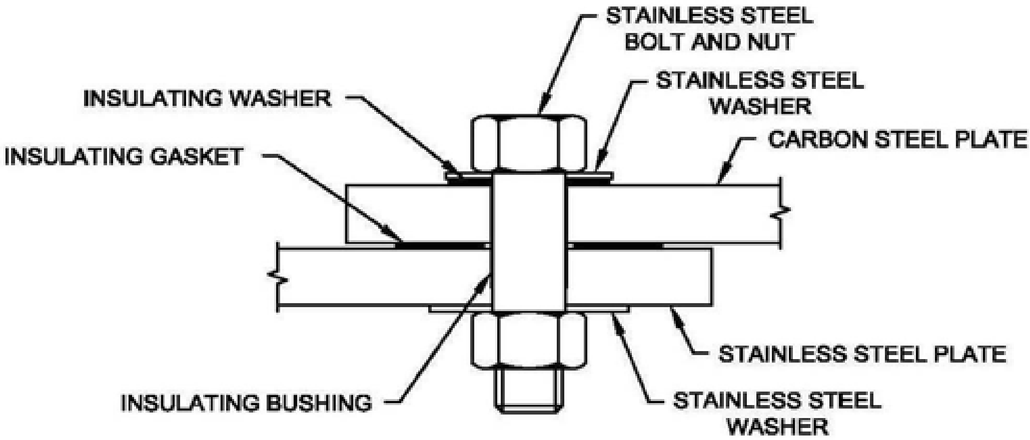


Figure 7.1 : Typical Bolted Connection with Carbon Steel Elements

bolts installed in the snug-tight condition. The insulating washers and bushings should be made of a thermoset polymer such as neoprene (synthetic rubber), which is flexible enough to seal the joint when adequate pressure is applied and long lasting to provide permanent metal separation. Sealing the joint is important to prevent moisture infiltration which would lead to crevice corrosion. Note also that the insulating washer should not extend beyond the stainless steel washer in case a crevice is created. In atmospheric conditions with chloride exposure, an additional strategy to protect against crevice corrosion is to insert an insulating, flexible washer directly under the bolt head, or to cover the area with clear silicone sealant.

With respect to welded joints involving carbon and stainless steels, it is generally recommended that any paint system applied to the carbon steel should extend over the weldment onto the stainless steel up to a distance of about 75 mm. Care should be taken in selecting appropriate materials for the environment to avoid crevice corrosion in bolted joints.

7.3 Alignment of Members

The centroidal axes of members meeting at a joint or at a splice should preferably meet at a point. When this is not the case, the moment on the connection due to any eccentricity should be taken into account.

7.4 Welded Connections

7.4.1 Welds shall conform to IS 816 and IS 9595 as appropriate.

The heating and cooling cycle involved in welding affects the microstructure of all stainless steels, and this is of particular importance for duplex stainless steels. It is essential that suitable welding procedures and compatible consumables are used and that qualified welders undertake the work. This is important not only to ensure the strength of the weld and to achieve a defined weld profile but also to maintain corrosion resistance of the weld and surrounding material.

Compatible consumables should be used, such that the specified yield strength, tensile strength, elongation at failure and minimum Charpy V- notch energy value of the filler metal should be equivalent to, or better than that specified for the parent material.

For welding stainless steel to carbon steel, the filler metal should be over-alloyed to ensure adequate mechanical properties and corrosion resistance of the joint.

Over- alloying avoids dilution of the joined elements in the fusion zone of the base stainless steel. When welding stainless steel to galvanized steel, the zinc coating around the area to be joined needs to be removed before welding. The inclusion of zinc can result in embrittlement or reduced corrosion resistance of the finished weld and the fumes given off when attempting to weld through the galvanized layer are a significant health hazard. Once the galvanizing has been removed, welding requirements are as for welding stainless steel to ordinary carbon steel.

7.4.2 Types of welds

The following types of welds can be used :

a) Continuous full penetration or partial penetration butt welds.

b) Continuous or intermittent fillet welds,

c) Plug welds

Intermittent butt welds shall not be used.

Partial penetration butt welds shall not be used for transmitting tensile forces or bending moments along longitudinal axis of the welds. Plug welds shall not be used for transmitting loads or moments and shall be used only to prevent the buckling or separation of lapped parts or to joint components of built-up members.

7.4.3 Strength of weld

7.4.3.1 Butt weld

The strength of a full penetration butt weld shall be taken as equal to the strength of the weaker of the parts joined provided the yield stress of the weld metal is atleast equal to that of the parent metal.

The strength of a partial penetration butt weld together with its reinforcing fillet weld, if any, shall be calculated as for a full penetration fillet weld. The throat thickness shall be taken as

a) the depth of weld preparation where this is of the J or U type.

b) the depth of weld preparation minus 3 mm where the preparation is the V or bevel type.

7.4.3.2 Fillet weld

The strength of a fillet weld shall be based on the effective throat thickness and the effective length.

The effective throat thickness shall be considered as the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side.

The effective length shall be considered as the actual length minus twice the leg length. In case of fillet welds with end returns as per Clause **7.4.4.1** the effective length shall be considered as the actual length.

7.4.4 General requirements of welds

7.4.4.1 Fillet welds

Maximum leg length of a fillet weld shall be 1 mm less than the thickness of the connected parts at the edge.

Minimum leg length of a fillet weld shall be in accordance with IS 9595 Intermittent fillet welds. Intermittent fillet weld should not be used at locations where they could result in the possible formation of rust pockets. Where the connection is protected from weather, e.g. in the interior of box sections, intermittent fillet welds are permitted.

The clear unconnected gap between the ends of the welds whether in line or staggered shall not be more than 200 mm and also shall not be more than -

a) 12 times the thickness of the thinner part when the part is in compression
b) 16 times the thickness of the thinner part when the part is in tension
c) One-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subject to compression or shear.

In a line of intermittent welds, there shall be a weld at each end of the part connected.

In built-up members in which plates are connected by intermittent welds, continuous side fillet welds shall be provided at the ends of each side of the plate for a length at least equal to three quarter of the width of the narrower plate concerned. In exceptional cases, where this is not possible, the intermittent plug or slot weld shall be provided to prevent separation.

End returns

The fillet weld shall be returned continuously around the corner at the end of the side of a part for a length beyond the corner of not less than twice the leg length of the weld.

End connections by side fillets

If side fillets alone are used in end connections, both sides of the part shall be welded and the length of the weld on each side shall not be less than the distance between the welds nor less than 4 times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of thinnest part connected, intermediate plug or slot welds shall be used to prevent separation.

End connections by transverse welds

The overlap between the connected parts shall not be less than four times thickness of the thinnest part and the parts shall be connected by two transverse lines of welds. Where the distance between the weld exceeds 16 times the thickness of the thinnest part connected intermediate slot or plug welds shall be used to prevent separation.

Welds with packings

Where two parts connected by welding are separated by packing having thickness less than the leg length of a weld necessary to transmit the force, the required leg length will be increased by thickness of the packing. The packing shall be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to or greater than the leg length of weld necessary to transmit force, each of the parts shall be connected to the packing by a weld capable of transmitting the design force.

Welds in holes and slots

Fillets welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of the lapped parts or to join components of built-up members.

7.4.4.2 T butt joints

Butt welds in T joints shall be completed by means of fillet welds each having a size of not less than 25 percent of the thickness of the outstanding part.

7.4.4.3 Plug welds

The entire area of the hole or slot shall be filled with weld metal having a thickness

a) equal to the thickness of the holed or slotted part where it is 16 mm or less.
b) In other cases, not less than any of the following :
    1) 16 mm
    2) 0.45 times the diameter of the hole or the width of the slot.
    3) One-tenth of the length of slot but not greater than the thickness of the holed or slotted part.

The diameter of the hole or the width of a slot shall not be less than the thickness of the hole or slotted parts plus 8 mm.

The distance between centres of holes or between the centre lines of slots shall not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of the slots measured in the direction of their length shall not be less than double the length of the slot.

The ends of the slot shall be semicircular except where the slot terminates at the edges of the part where it can be square.

7.4.4.4 Welding procedure

The welding procedure and details shall be in accordance with IS 9595 unless otherwise stipulated in this Clause. Reference should also be made to Manufacturers recommendations.

7.4.5 Design stresses in welds

7.4.5.1 Shop welds

7.4.5.1.1 Fillet welds - Design strength of a fillet weld, *fwd* shall be based on its throat area.

$$f_{wd} = f_{wn} / \gamma_{mw}$$

where

$f_{wn} = f_u /$ S3

f_u = smaller of the ultimate stress of the weld or the parent metal

λ_{mw} = partial safety factor (**Table 4.1**)

7.4.5.1.2 Butt welds - Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.
7.4.5.1.3 Slot or plug welds - The design shear stress of slot or plug welds shall be as per Clause **7.4.5.1.1.**

7.4.5.2 Site welds - The design strength in shear and tension for site welds made during erection of structural members shall be calculated as per Clause **7.4.5.1**, using appropriate partial safety factor as per **Table 4.1**.

7.4.5.3 Long joints - When the length of the welded joint, l_j, of a splice or end connection in a compression or tension element is greater than 150 t_t, the design capacity of weld (Clause **7.4.5.1.1**), f_{wd} shall be reduced by the factor

$$\beta_{tw} = 1.2 - \frac{0.2 l_f}{150\, t_t} \leq 1.0$$

where

l_f = length of the joint in the direction of the force transfer

t_t = throat size of the weld

7.4.6 Fillet weld applied to the edge of a plate or section

7.4.6.1 Where a fillet weld is applied to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness in order to avoid washing down of the exposed areas (**Fig. 7.2a**).

7.4.6.2 Where a fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed ¾ of the thickness of the section at the toe (Fig. **7.2b**).

Figure 7.2 : Fillet Welds on Square Edge of Plate or Round Toe of Rolled Section

7.4.6.3 Where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (Fig.7.3).

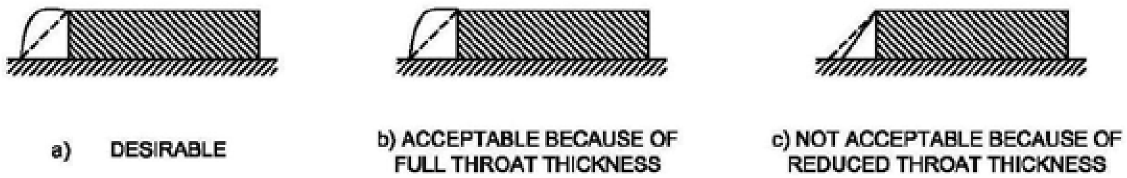


Figure 7.3 : Full Size Fillet Welds Applied to the Edge of a Plate or Section

7.4.6.4 When fillet welds are applied to the edges of a plate or section in members subject to dynamic loading, the fillet weld shall be of full size, that is, with its leg length equal to the thickness of the plate or section, with the limitations specified in Clause **7.4.6.3**.

7.4.6.5 End fillet weld normal to the direction of force shall be of unequal size with a throat thickness not less than $0.5t$ where t is the thickness of the part as shown in **Fig. 7.4**.

The difference in thickness of the welds shall be negotiated at a uniform slope.

7.4.7 Stresses due to individual forces - When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by:

$$f_a \text{ or } q = \frac{P}{t_t l_w}$$

where

f_a= calculated normal stress due to axial force in N/mm^2

q = shear stress in N/mm^2

P = force transmitted (axial force N or the shear force Q)

t_t= effective throat thickness of weld in mm

l_w= effective length of weld in mm

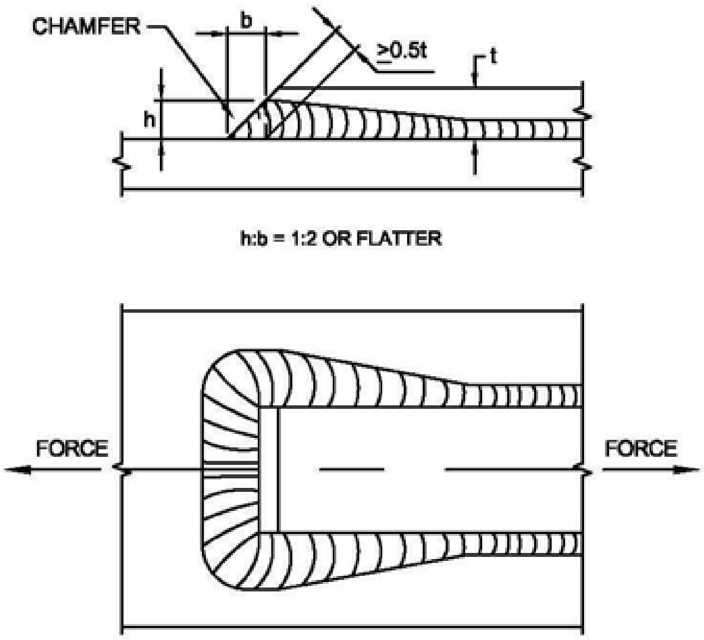


Figure 7.4 : End Fillet Welds Normal to Direction of Force

7.4.8 Combination of stresses

7.4.8.1 Fillet welds

7.4.8.1.1 When subjected to a combination of normal and shear stress, the equivalent stress f_e shall satisfy following

$$f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

where

f_a= normal stresses, compression or tension, due to axial force or bending moment (Clause **7.4.7**)

q = shear stress due to shear force or tension (Clause **7.4.7**)

7.4.8.1.2 Check for the combination of stresses need not be done for :

a) Side fillet welds joining cover plates and flange plates, and
b) Fillet welds where sum of normal and shear stresses does not exceed *fwd* (Clause **7.4.5.1.1**)

7.4.8.2 Butt welds

7.4.8.2.1 Check for the combination of stresses in butt welds need not be carried out provided that :

a) Butt welds are axially loaded, and
b) In single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 percent of the design shear stress.

7.4.8.2.2 Combined bearing, bending and shear - Where bearing stress, f_{br}, is combined with bending (tensile or compressive), f_b, and shear stresses, q, under the most unfavorable conditions of loading in butt welds, the equivalent stress, f_e, as obtained from the following formula shall not exceed the values allowed for the parent metal.

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

where

f_e = equivalent stress

f_b= calculated stress due to bending in N/mm^2

f_{br}= calculated stress due to bearing in N/mm^2

q = shear stress in N/mm^2

7.5 Connections made with Bearing Type Bolts or Pins

7.5.1 General

Connections and splices in all members may be made by the use of fasteners (bolts or pins). The arrangement of plates, rolled sections and other connecting elements shall be such as to make proper provision for all axial, flexural, shear and/or torsional forces in the members being connected.

Bolted splices in all compression members shall be located as near as practicable to points of effective lateral support.

A member carrying a calculated stress shall not have a splice or connection with a single bolt.

Connections and splices for minor members, such as light bracing members, railings, etc. may have single bolted connections.

Minimum diameter of fasteners used in load bearing members shall be 16 mm diameter.

7.5.2 Connections and splices in flexural members

a) The connection between a flange and a web of a built-up girder shall be designed to transmit the longitudinal shear force in the flange combined with any vertical loads which are directly applied to the flanges.

b) Flange splices

   1) General

      Flange splices to join flange components are to be made from the same grade of steel but may be of different cross-sections.

   2) Bolted splices

      Where bolted splice plates are used to obtain a splice in a flange the sum of their areas shall be at least equal to the area of the flanges as spliced. The centres of gravity of the sections on either side of the splice shall coincide as far as practicable. The splice plates and connections on each side of the splice shall be capable of transmitting at least the greater of -

      i) 1.10 times the force in the flange at the splice point computed from factored loads.

      ii) 0.80 times the maximum capacity of the weaker flange, considering appropriate safety factor **(Table 4.1)**, the net section being used for tension flanges and the gross section for compression flanges.

c) Web splice

   A splice in the web of a plate girder or rolled section used as a beam shall be designed to resist the shearing forces and the portion of the design moment in the web, and for the moment due to the eccentricity of shear introduced by the splice connection, computed from factored loads.

   Web plates shall be spliced symmetrically by plates on each side. The splice plates shall extend as far as practicable for the full depth of the web. There shall not be less than at least two rows of bolts on each side of the joint.

7.5.3 Connections in triangulated structures

a) Eccentric connections :

   Axially stressed members meeting at a joint shall have their gravity axes intersect at a point if practicable; if not, provision shall be made for bending stresses due to the eccentricity

b) Connections at intersections :

Connection of members at an intersection shall develop at least 1.10 times the design loads and moments in the members computed from factored loads. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fasteners.

All members shall, where possible, be so connected that the load is appropriately distributed over their cross-section.

If this is impracticable, consideration shall be given to the way in which the stresses at the joint are distributed to those parts of the cross-section of the member which are not directly connected at the joint. For this purpose the angle of distribution of stress may be taken as 45^0.

Gusset plates shall be capable of sustaining 1.05 times the design loads and moments transmitted by the members. If an unsupported edge of a gusset plate is in compression and if the length of such edge exceeds 50 times the thickness of the gusset plate, the edge shall be suitably stiffened.

c) Splices in tension members and compression members of non bearing type

Such splices shall be made symmetrical about the gravity axes of the members as far as is practicable.

Bolted splices shall be designed for any applied moment computed from factored loads and the greater of

1) 1.10 times the computed forces in the member, and
2) 0.80 times the capacity of the member, considering appropriate partial safety factor (**Table 4.1**).

The ends of the members need not be in close contact.

d) Splices in compression members of bearing type

In bearing type splices in a compression member, the ends of the members shall be machined and assembled to be in close contact with each other.

For a bearing splice it may be assumed that the machined faces transmit 50 percent of the compressive force in the member. The splice plates and connection shall be however designed to transmit 60 percent of the factored compressive force in the member and the factored moment, if any.

NOTE : Before specifying bearing splices the designer shall, however, satisfy that such facilities for machining are available in the particular project.

7.5.4 Details of bolted connections

a) Diameter of bolt holes

The diameter of a bolt hole shall generally be taken as the nominal diameter of the bolt plus 1.5 mm unless otherwise specified

b) Edge distances

1) In case of rolled, machine flame cut, sawn or plane edges the distance between the centre of the bolt hole to such edge shall not be less than 1.5 times the diameter of the hole.
2) In case of sheared or hand flame cut edges the edge distance shall be 1.75 times the diameter of the hole.
3) The maximum edge distance to the nearest line of fasteners from an edge of any unstiffened part should not exceed 12t where t is the thickness of the thinner outer plate (This rule does not apply to fasteners interconnecting the components of back-to-back tension members). Where the members are exposed to corrosive influences the maximum edge distance shall not exceed 40 mm plus 4t where t is the thickness of thinner connected plate.

c) Pitch of bolts

1) The minimum distance between the centres of any two adjacent bolts shall not be less than 2.5 times the diameter of the shank of the bolt.
2) The maximum distance between the centres of any two adjacent bolts connecting members either in tension or in compression shall not exceed either 32t or 300 mm, where t is thickness of the thinner outside element.
3) The distance between centres of two consecutive bolts in a line along the direction of stress shall not exceed 16t or 200 mm in tension members, and 12t or 200 mm in compression members. In the case of compression members transferring forces through butting faces the pitch shall not exceed 4.5 times the diameter of bolt from the abutting faces. This pitch will apply for a distance equal to 1.5 times the width of the member.
4) When bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distance as specified in (2) and (3) above between centre of adjacent connectors may be increased by 50 percent.
5) Except as noted in (4) above, the distance between centre of two consecutive bolts in a line adjacent and parallel to an edge of an outside connected part should not be greater than (100 mm +4t) or 200 mm, whichever is lesser, where t is the thickness of the thinner outside plate.

7.5.5 Bearing type bolts

7.5.5.1 Effective areas of bolts

Since threads can occur in the shear plane, the effective area A_{eb} for resisting shear should normally be taken as the net tensile stress area, A_{nb} of the bolts. For bolts where the net tensile stress area is not defined, A_{nb} shall be taken as the area at the root of the threads.

Where it can be shown that the threads do not occur in the shear plane. A_{eb}may be taken as the cross section area, A_{sb}at the shank.

The net sectional area of a bolt or screwed tension rod A_{nb} shall be taken as the tension area for the particular diameter of bolt as given in the table below :

Nominal Bolt Diameter (mm)	12	14	16	18	20	22	24	27	30	33
Nominal Stress Area (mm^2)	84	115	157	192	245	303	353	459	561	694

7.5.5.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition

$$V_{sb} \leq V_{db}$$

Where V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (Clause **7.5.5.3**) and bearing, V_{dpb} (Clause **7.5.5.4**)

7.5.5.3 Shear capacity of bolt – The design strength of the bolt, V_{dsb}, as governed shear strength is given by

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows :

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

where

f_{ub} = ultimate tensile strength of a bolt

n_n = number of shear planes with threads intercepting the shear plane

n_s = number of shear planes without threads intercepting the shear plane

A_{sb} = cross-sectional area of the bolt at the shank

A_{nb} = net shear area of the bolt taken as the area corresponding to root diameter at the thread.

7.5.5.4 Bearing capacity of bolt – The design strength of a bolt on any plate, V_{dpb}, as governed by bearing is given by

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

V_{npb} = nominal bearing strength of a bolt, calculated as follows :

$= 2.5 \, k_b \, d \, t \, f^1_u$

where

k_b is smallest of $e/3d_0$; $p/3d_0 - 0.25$; f_{ub}/f_u; 1.0

e, p = end and pitch distances of the fastener along bearing direction

d_o= diameter of the hole

f^1_u= smaller of f_{ubr}, fu

f_{ubr}= 0.5f_{yb} + 0.6f_{ub}<f_{ub}

f_{yb}= yield stress of the bolt

f_{ub}= ultimate stress of the bolt

f_u= ultimate tensile stress of the plate

d = nominal diameter of the bolt

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or, if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking

NOTE: The block shear of the edge distance due to bearing force may be checked as given in Clause **6.3.1.3**

7.5.5.5 Tension capacity of bolt - A bolt subjected to a factored tensile force (T_b) shall satisfy

$T_b \leq T_{db}$

where

$T_{db} = T_{nb}/\lambda_{mb}$

and Tnb = nominal tensile capacity of the bolt, calculated as follows :

T_{nb}= 0.90 f_{ub} A_{nb} < f_{yb} A_{sb}(λ_{mb}/ λ_{mo})

where

f_{ub}= ultimate tensile stress of the bolt

f_{yb}= yield stress of the bolt

A_{nb}= net tensile stress area as specified in Clause **7.5.1**

A_{sb}= shank area of the bolt

7.5.6 Bolt subjected to combined shear and tension – A bolt required to resist both design shear force (V_{sb}) and design tensile force (T_b) at the same time shall satisfy

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

where

V_{sb}= factored shear force acting on the bolt

V_{db}= design shear capacity (Clauses **7.5.5.3 & 7.5.6.3**)

T_b= Factored tensile force acting on the bolt

T_{db}= design tension capacity (Clauses **7.5.5.5 &7.5.6.5**)

7.5.7 Long joints - When the length of the joint, l_j, of a splice or end connection in a compression or tension element containing more than two fasteners (i.e. the distance between the first and last rows of fasteners in the joint, measured in the direction of the load transfer) exceeds 15d in the direction of load, the nominal shear capacity of the fastener (Clauses **7.5.5.2 & 7.5.6.2**) shall be reduced by the factor, β_{lj}, given by

$$\beta_{lj} = 1.075 - l_j/(200d) \quad \text{but } 0.75 \leq \beta_{lj} \leq 1.0$$

$$= 1.075 - 0.005(l_j/d)$$

where

d = nominal diameter of the fastener

NOTE: This provision does not apply when the distribution of shear over the length of joint is uniform as in the connection of web of a section to the flanges.

7.5.8 Large grip lengths - When the grip length, l_g(equal to the total thickness of the connected plates) exceeds 5 times the diameter, d, of the fastener the design shear capacity shall be reduced by a factor β_{lg}, given by

$$\beta_{lg} = 8d / (3d + l_g)$$

$$= 8/(3 + l_g/d)$$

β_{lg} shall not be more than β_{ij} given in Clause **7.5.7**. The grip length, l_g in no case shall be greater than 8d

Packing Plates: The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by

$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$

where

t_{pk} = thickness of the thicker packing in mm

7.6 Connections made with High Strength Friction Grip (HSFG) Bolts

7.6.1 In high strength friction grip bolting, initial pretension in bolt develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to IS 3757. Their installation procedures shall conform to IS 4000.

It may be noted that some International Codes initially did not recommend connections with HSFG bolts in Stainless Steel structures. However, recent experimental research has demonstrated that behavior of HSFG bolts in most grades of Stainless Steel are identical to that in carbon steel elements. It is recommended that manufacturers advice/ test results should be consulted before adopting HSFG bolt connection in Stainless Steel Bridges.

7.6.2 Long joints - The provision for the long joints in Clause **7.5.7** shall apply to friction grip connections also.

7.6.3 Capacity after slipping - When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection (Clause **7.5.5**).

The block shear resistance of the edge distance due to bearing force may be checked as given in Clause **6.1.3.**

7.6.4 Tension resistance – A friction bolt subjected to a factored tension force (**T_f**) shall satisfy

$T_f \leq T_{df}$

where

$T_{df} = T_{nf}/\gamma_{mf}$

T_{nf}= nominal tensile strength of the friction bolt, calculated as follows:

$T_{nf} = 0.9\, f_{ub} A_n \leq f_{yb}\, A_{sb}\, (\gamma_{mf} / \gamma_m)$

where

f_{ub}= ultimate tensile stress of the bolt

A_n= net tensile stress area as specified in IS 1367. (For bolts where the tensile stress area is not defined, A_n shall be taken as the area at the root of the threads)

A_{sb}= shank area of the bolt

γ_{mf}= partial factor of safety

7.6.5 Combined shear and tension Bolts in a connection for which slip in the serviceability limit state shall be limited and which are subjected to a tension force, T, and shear force, V, shall satisfy

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

where

V_{sf}= applied factored shear at design load

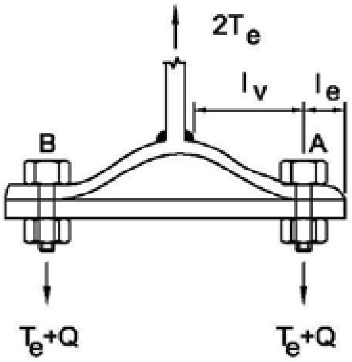
V_{df}= design shear strength

T_f= externally applied factored tension at design load

T_{df}= design tension strength

7.7 Prying Forces

Where prying force, Q, is significant, prying force shall be calculated as given below and added to the tension in the bolt.



$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_0 b_e t^4}{27 l_e l_v^2}\right]$$

where

l_v= distance from the bolt centreline to the toe of the fillet weld or to half the root radius for a rolled section;

l_e= distance between prying force and bolt centerline and is the minimum of, either the end distance or the value given by

$$l_e = 1.1\, t\, \sqrt{\frac{\beta f_0}{f_y}}$$

β = 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt

η = 1.5

b_e= effective width of flange per pair of bolts

f_o= proof stress in consistent units

t = thickness of the end plate

8. FIRE SAFETY

8.1 Adequate provision may be made as far as possible for fire fighting equipment to access all parts of the bridge.

8.2 Austenitic stainless steels generally retain a higher proportion of their room temperature strength than carbon steel above temperatures of about 550°C. All stainless steels retain a higher proportion of their stiffness than carbon steel over the entire temperature range.

8.3 In case of accidental occurrence of fire in a bridge it should be mandatory for the authorities to have the bridge inspected by competent experts in order to ascertain the health of the structure before it can be declared safe for use.

9. FATIGUE

9.1 General

This section applies to the design of structures and structural elements subject to loading which could lead to fatigue failure. This shall, however, not cover the following:

a) Corrosion fatigue

b) Low cycle (high stress) fatigue

c) Thermal fatigue

d) Stress corrosion cracking

e) Effects of high temperature (>150°C)

f) Effects of low temperature (< brittle transition temperature)

9.1.1 For the purpose of design against fatigue, different details (of members and connections) are classified under different fatigue class. The design stress ranges corresponding to various numbers of cycles, are given for each fatigue class. The requirements of this Clause shall be satisfied with, at each critical location of the structure subjected to cyclic loading, considering relevant number of cycles and magnitudes of stress range expected to be experienced during the life of the structure.

9.2 Design

9.2.1 Reference design conditions

The Standard S-N curves for each detail category are given for the following conditions:

a) The detail is located in a redundant load path, wherein local failure at that detail alone will not lead to overall collapse of the structure.

b) The nominal stress history at the local point in the detail is estimated/ evaluated by a conventional method without taking into account the local stress concentration effects due to the detail.

c) The load cycles are not highly irregular.

d) The details are accessible for and subject to regular inspection.

e) The structure is exposed to only mildly corrosive environment as in normal atmospheric condition and suitably protected against corrosion (pit depth <1 mm).

f) The transverse fillet or butt weld connects plates of thickness not greater than 25 mm.

g) As far as possible, holes should preferably be avoided in members and connections subjected to fatigue.

Fatigue need not be investigated if condition in Clauses 9.2.2.3, 9.5.1, or 9.6 is satisfied.

The values obtained from the standard S-N curve shall be modified by a capacity reduction factor μ_r, when plates greater than 25 mm in thickness are joined together by transverse fillet or butt welding, as given by :

$$\mu_r = (25/t_p)^{0.25} \leq 1.0$$

where

t_p = actual thickness in mm of the thicker plate being joined.

No thickness correction is necessary when full penetration butt weld reinforcements are machined flush and proved free of defect through non-destructive testing.

9.2.2 Design spectrum

9.2.2.1 Stress evaluation - The design stress shall be determined by elastic analysis of the structure to obtain stress resultants and the local stresses may be obtained by a conventional stress analysis method. The normal and shear stresses shall be determined considering all design loads on the members, but excluding stress concentration due to the geometry of the detail. The stress concentration effect is accounted for in detail category classification (Table 9.4). The stress concentration, however, not characteristic of the detail shall be accounted for separately in the stress calculation.

In the fatigue design of trusses made of members with open sections in which the end connections are not pinned, the stresses due to secondary bending moments shall be taken into account unless the slenderness ratio, (KL/r), of the member is greater than 40.

In the determination of stress range at the end connections between hollow sections, the effect of connection stiffness and eccentricities may be disregarded, provided:

a) The calculated stress range is multiplied by appropriate factor given in Table 9.1 in the case of circular hollow section connections and Table 9.2 in the case of rectangular hollow section connections.

b) The design throat thickness of fillet welds in the joints is greater than the wall thickness of the connected member.

9.2.2.2 Design stress spectrum - In the case of loading events producing non-uniform stress range cycle, the stress spectrum may be obtained by a rational method.

Table 9.1 Multiplying Factors for Calculated Stress Range
(Circular Hollow Sections)

Types of Connection		Chords	Verticals	Diagonals
Gap Connections	K type	1.5	1.0	1.3
	N type	1.5	1.8	1.4
Overlap Connections	K type	1.5	1.0	1.2
	N type	1.5	1.65	1.25

Table 9.2 Multiplying Factors for Calculated Stress Range
(Rectangular Hollow Sections)

Types of Connection		Chords	Verticals	Diagonals
Gap Connections	K type	1.5	1.0	1.5
	N type	1.5	2.2	1.6
Overlap Connections	K type	1.5	1.0	1.3
	N type	1.5	2.0	1.4

9.2.2.3 Low fatigue - Fatigue assessment is not required for a member, connection or detail, if normal and shear design stress ranges, f, satisfy the following conditions:

$f \leq 27/\gamma_{mft}$

Or if the actual number of stress cycles, N_{SC}, satisfies

$$N_{SC} < 5\ x\ 10^6 \left(\frac{\frac{27}{\gamma_{mft}}}{\gamma_{fft} f} \right)^3$$

where

γ_{mft} ,γ_{fft} = partial safety factors for strength and load, respectively (See Clause 9.2.3)

f = actual fatigue stress range for the detail

9.2.3 Partial safety factors

9.2.3.1 Partial safety factor for loads and their effects (γ_{fft}) - Unless and otherwise the uncertainty in the estimation of the applied loads and their effects demand a higher value, the partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0.

9.2.3.2 Partial safety factor for fatigue strength (γ_{mft}) - Partial safety factor for strength is influenced by consequences of fatigue damage and level of inspection capabilities.

9.2.3.3 Based on consequences of fatigue failure, component details have been classified as given in the Table 9.3 and the corresponding partial safety factor for fatigue strength shall be used.

a) Fail-safe structural component/detail is the one where local failure of one component due to fatigue crack does not result in the failure of the structure due to availability of alternative load path (redundant system).

b) Non-fail-safe structural component/detail is the one where local failure of one component leads rapidly to failure of the structure due to its non-redundant nature.

Table 9.3 Partial Safety Factors for Fatigue Strength (γ_{mft})

Inspection and Access	Consequence of Failure	
	Fail-safe	Non-Fail-safe
Periodic Inspection, maintenance and accessibility to detail is good	1.00	1.25
Periodic Inspection, maintenance and accessibility to detail is poor	1.15	1.35

9.3 Detail Category

Tables 9.4 (a) to (d) indicate the classification of different details into various categories for the purpose of assessing fatigue strength. Details not classified in the table may be treated as the lowest detail category of a similar detail, unless superior fatigue strength is proved by testing and/or analysis.

Holes in members and connections subjected to fatigue loading shall not be made:

a) Using punching in plates having thickness greater than 12 mm unless the holes are sub-punched and subsequently reamed to remove the affected material around the punched hole, and

b) Using gas cutting unless the holes are reamed to remove the material in the heat affected zone.

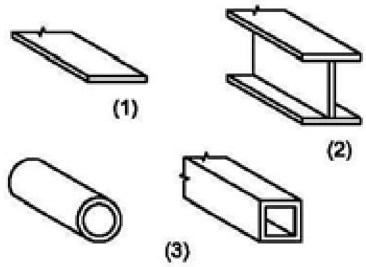
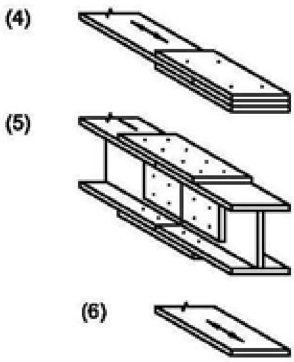
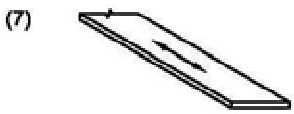
9.3.1 Members subjected to tensile forces under repetitive loading cycles are primarily susceptible to fatigue failure.In common with carbon steel structures, the combination of stress concentrations and defects at welded joints leads to locations being almost invariably more prone to fatigue failure than other parts of the structure. Much can be done to reduce the susceptibility of a structure to fatigue by adopting good design practice. This involves judiciously selecting the overall structural configuration and carefully choosing constructional details (i.e the Detail Categories) that are fatigue resistant. The key to fatigue resistant design is a rational consideration of fatigue early in the design process. A fatigue assessment performed only after other design criteria have been satisfied may result in an inadequate or costly structure.

It may be possible to eliminate potential fatigue problems by giving due regard to constructional details and avoiding:

- sharp changes in cross-section and stress concentrations in general,
- misalignments and eccentricities,
- small discontinuities such as scratches and grinding marks,
- unnecessary welding of secondary attachments, e.g. lifting lugs,
- partial penetration welds, fillet welds, intermittent welding, and backing strips,
- arc strikes.

Table 9.4 (a) Detail Category Classification: Group 1: Non-welded Details

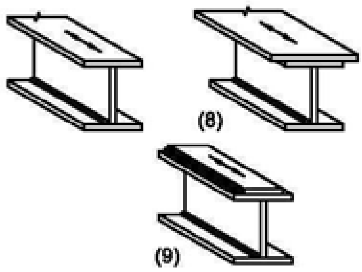
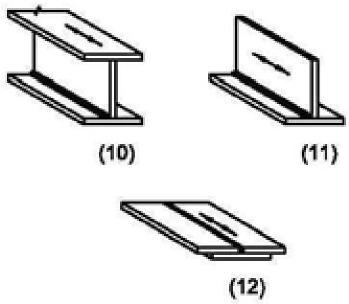
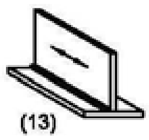
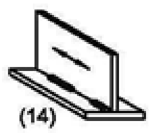
(Clauses 9.2.2.1 and 9.3)

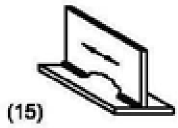
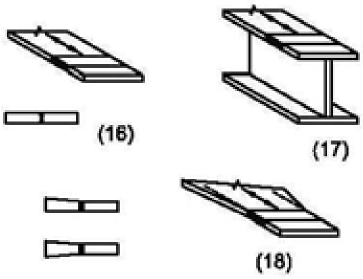
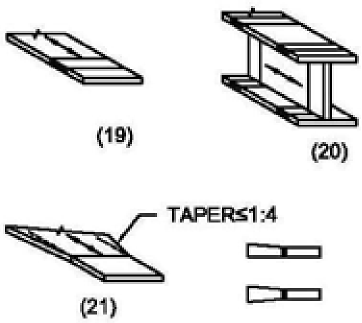
Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
i)	118		Rolled and Extruded Products i) Plates and Flats (1) ii) Rolled Sections (2) iii) Seamless Tubes (3) Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress
ii)	103		Bolted Connections (4) & (5): Stress range calculated on the gross section and on net section. Unsupported one-sided cover plate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses. Material with gas-cut or sheared edges with no draglines. (6): All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress
iii)	92		Material with machine gas-cut edges with draglines or manual gas-cut material (7): Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress

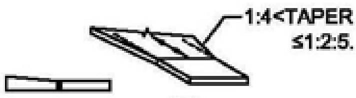
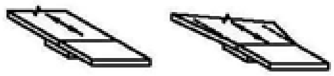
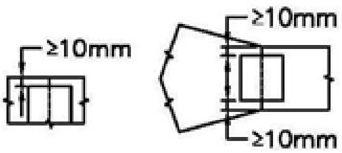
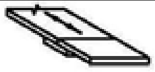
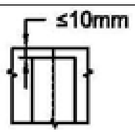
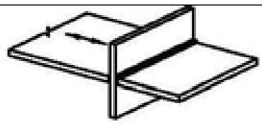
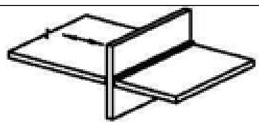
NOTE: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

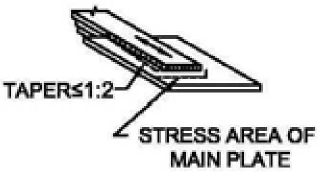
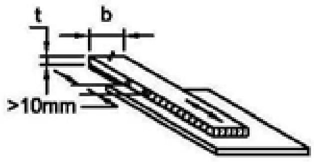
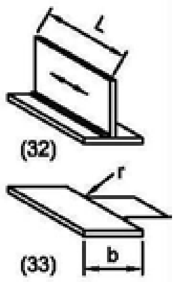
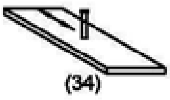
Table 9.4 (b) Detail Category Classification: Group 2: Welded Details – not in Hollow Sections

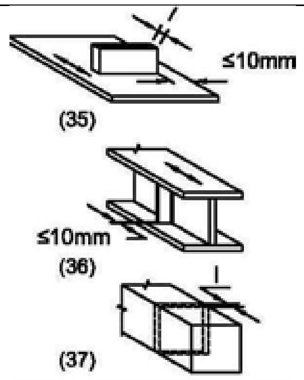
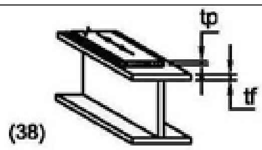
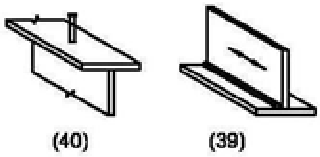
(Clauses 9.2.2.1 and 9.3)

Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
i)	92		Welded plate I-sections and box girders with continuous longitudinal welds (8) & (9) : Zones of continuous automatic fillet or butt welds carried out from both sides and all welds having un-repaired stop-start position.
ii)	83		Welded plate I-section and box girders with continuous longitudinal welds (10) & (11) : Zones of continuous automatic butt welds from one side only with a continuous backing bar and all welds not having un-repaired stop-start positions. (12) : Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides use Detail Category 92).
iii)	66		Welded plate I-section and box girders with continuous longitudinal welds (13) : Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.
iv)	59		Intermittent longitudinal welds (14) : Zones of intermittent longitudinal welds

Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
v)	52		Intermittent longitudinal welds (15) : Zones containing cope holes in longitudinally welded T-joints. Cope hole not to be filled with weld.
vi)	83		Transverse butt welds (complete penetration) Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (16) : Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 percent NDT inspection and weld surface to be free of exposed porosity in the weld metal. (17) : Plate girders welded as in, (16) before assembly. (18) : Transverse splices as in (16) with reduced or tapered transition with taper $\leq$ 1:4
vii)	66		Transverse butt welds (complete penetration) Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (19) : Transverse splice of plates, rolled sections or plate girders (20) : Transverse splice of rolled section or welded plate girders, without cope hole. With cope hole use Detail Category 52, as in (15) (21) : Transverse splices in plates or flats being tapered in width or in thickness where the taper is 1$\leq$4.

<table>
<tr><th rowspan="2">Sl No</th><th rowspan="2" colspan="2">Detail Category</th><th colspan="2">Constructional Detail</th></tr>
<tr><th>Illustration (See Note)</th><th>Description</th></tr>
<tr><td>viii)</td><td colspan="2">59</td><td>
(22)</td><td>Transverse butt welds (complete penetration)
Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides.
(22) : Transverse splices as in (21) with taper in width or thickness 1:4 but ≤1:2.5</td></tr>
<tr><td>ix)</td><td colspan="2">52</td><td>
(23) (24)
</td><td>Transverse butt welds (complete penetration)
(23) : Transverse butt-welded splices made on a backing bar. The end of the fillet weld of the backing strip shall stop short by more than 10mm from the edges of the stressed plate.
(24) : Transverse butt welds as per (23) with taper on width or thickness <1:2.5</td></tr>
<tr><td>x)</td><td colspan="2">37</td><td>
(25)
</td><td>Transverse butt welds (complete penetration)
(25) : Transverse butt welds as in (23) where fillet welds end closer than 10mm to plate edge</td></tr>
<tr><td>xi)</td><td colspan="2">52</td><td>
(26)</td><td>Cruciform joints with load-carrying welds
(26) : Full penetration welds with intermediate plate NDT inspected and free of defects.
Maximum alignment of plates either side of joint to be <0.15 times the thickness of intermediate plate.</td></tr>
<tr><td rowspan="2">xii)</td><td>41</td><td>(27)</td><td rowspan="2">
(27) & (28)</td><td rowspan="2">(27) : Partial penetration or fillet welds with stress range calculated on plate area.
(28) : Partial penetration or fillet weld with stress range calculated on throat area of weld.</td></tr>
<tr><td>27</td><td>(28)</td></tr>
</table>

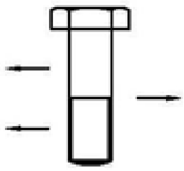
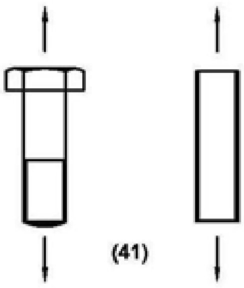
<table>
<tr><th rowspan="2">Sl No</th><th rowspan="2" colspan="2">Detail Category</th><th colspan="4">Constructional Detail</th></tr>
<tr><th colspan="3">Illustration (See Note)</th><th>Description</th></tr>
<tr><td>xiii)</td><td colspan="2">46</td><td colspan="3">
(29)</td><td>Overlapped welded joints
(29) : Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration</td></tr>
<tr><td>xiv)</td><td>41</td><td>(30)</td><td colspan="3" rowspan="2">
(30 & 31)</td><td>Overlapped welded joints
(30) : Fillet welded lap joint, with welds and main plate both having design capacity greater than the overlapping elements</td></tr>
<tr><td>xv)</td><td>33</td><td>(31)</td><td>(31) : Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld.</td></tr>
<tr><td rowspan="6">xvi)</td><td colspan="2" rowspan="2">66</td><td>(32)</td><td>(33)</td><td rowspan="6"></td><td rowspan="6">Welded attachments (non-load carrying) – Longitudinal welds
(32) : Longitudinal fillet welds. Class of detail varies according to length of attachment weld as noted
(33) : Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r), formed by machining or flame cutting plus grinding. Class of detail varies according to r/b ratio as noted.</td></tr>
<tr><td>_</td><td>1/3≤r/b</td></tr>
<tr><td colspan="2">59</td><td>l≤50mm</td><td>_</td></tr>
<tr><td colspan="2">52</td><td>50<l≤100</td><td>1/6≤r/b<1/3</td></tr>
<tr><td colspan="2">37</td><td>100mm<l</td><td>_</td></tr>
<tr><td colspan="2">33</td><td>_</td><td>r/b < 1/6</td></tr>
<tr><td>xvii)</td><td colspan="2">59</td><td colspan="3">
(34)</td><td>Welded attachments
(34): Shear connectors on base material (failure in base material)</td></tr>
<tr><td>xviii)</td><td colspan="2">59</td><td>t≤ 12mm</td><td colspan="2"></td><td>Transverse welds</td></tr>
</table>

Sl No	Detail Category	Constructional Detail		
		Illustration (See Note)		Description
	52	t >12mm		(35): Transverse fillet welds with the end of the weld $\geq$ 10mm from the edge of the plate (36) : Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses. (37) : Diaphragm of both girders welded to the flange or web by continuous or intermittent welds.
xix)	37	t_f or t_p $\leq$ 25mm		Cover plates in beams and plate girders (38) : End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential.
	27	t_f or t_p > 25mm		
xx)	67			Welds loaded in shear (39) : Fillet welds transmitting shear. Stress range to be calculated on weld throat area. (40) : Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud).

NOTE: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 9.4 (c) Detail Category Classification: Group 3: Bolts

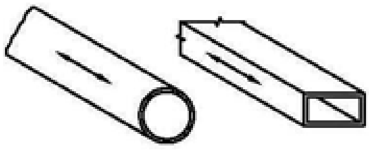
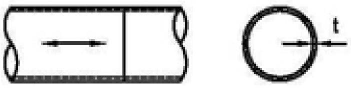
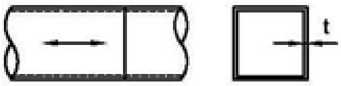
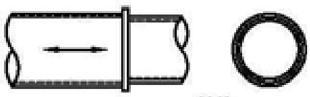
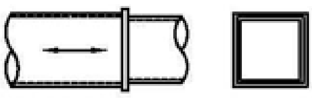
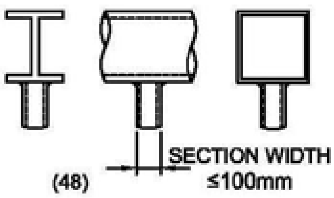
(Clauses 9.2.2.1 and 9.3)

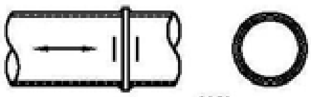
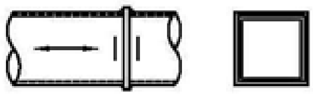
Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
i)	83	 (41)	Bolts in shear (8.8/TB bolting category only) (41) : Shear stress range calculated on the minor diameter of the bolt (A_c) NOTE – If the shear on the joint is insufficient to cause slip of the joint the shear in the bolt need not be considered in fatigue
ii)	27	 (41)	Bolts and threaded rods in tension (tensile stress to be calculated on the tensile stress area, A_t) (42) : Additional forces due to prying effects shall be taken into account. For tensional bolts, the stress range depends on the connection geometry. NOTE – In connections with tensioned bolts, the change in the forces in the bolts is often less than applied forces, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections.

NOTE: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 9.4 (d) Detail Category Classification: Group 4 : Welded Details – in Hollow Sections

(Clauses 9.2.2.1 and 9.3)

Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
i)	103	 (43)	Continuous Automatic Longitudinal Welds (43) : No stop-starts or as manufactured, proven free to detachable discotinuities
ii)	66 t $\geq$ 8mm 52 t < 8mm	 (44)	Transverse Butt Welds (44) : Butt-welded end-to-end connection of circular hollow sections NOTE – Height of weld reinforcement less than 10 percent of weld with smooth transition to the plate surface. Welds made in flat position and proven free to detachable discontinuities
iii)	52 t $\geq$ 8mm 41 t < 8mm	 (45)	(45) : Butt-welded end-to-end connection of rectangular hollow section
iv)	41 t $\geq$ 8mm 37 t < 8mm	 (46)	Butt Welds to intermediate plate (46) : Circular hollow sections, end-to-end butt-welded with an intermediate plate
v)	37 t $\geq$ 8mm 30 t < 8mm	 (47)	(47) : Rectangular hollow sections, end-to-end butt-welded with an intermediate plate
vi)	52		Welded attachments (non load carrying) (48) : Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction $\leq$ 100mm

Sl No	Detail Category	Constructional Detail	
		Illustration (See Note)	Description
vii)	33 $t \geq 8$mm	 (49)	Fillet welds to intermediate plate (49) : Circular hollow sections, end-to-end fillet welded with an intermediate plate
	29 $t < 8$mm		
viii)	29 $t \geq 8$mm	 (50)	(50) : Rectangular hollow sections, end-to-end fillet welded with an intermediate plate
	27 $t < 8$mm		

NOTE: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

9.4 Fatigue Strength

The fatigue strength of the standard detail for the normal or shear fatigue stress range, not corrected for effects discussed in Clause 9.2.1 is given below (Figs. 9.1&9.2)

a) Normal stress range

when $N_{SC} \leq 5x10^6$

$$f_f = f_{fn}\sqrt[3]{\frac{5x10^6}{N_{SC}}}$$

when $5x10^6 \leq N_{SC} \leq 10^8$

$$f_f = f_{fn}\sqrt[5]{\frac{5x10^6}{N_{SC}}}$$

b) Shear stress

$$\tau_f = \tau_{fn}\sqrt[5]{\frac{5x10^6}{N_{SC}}}$$

where

f_f , τ_f = design normal and shear fatigue range of the detail, respectively, for life cycle of N_{SC}

f_{fn} , τ_{fn} = normal and shear fatigue strength of the detail for 5 x 10^6 cycles, for the detail category

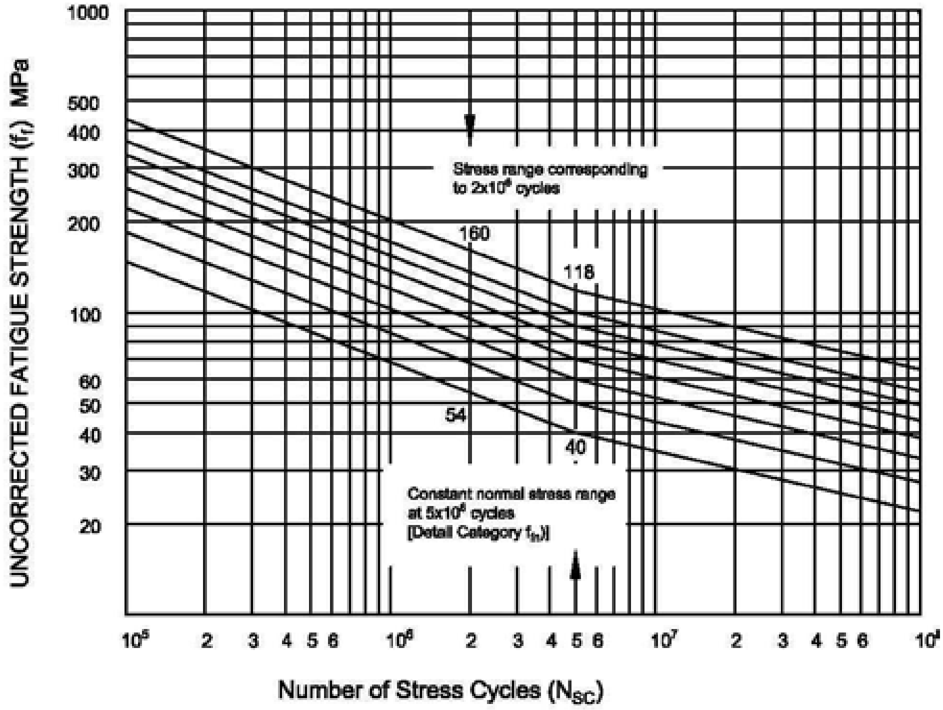


Figure 9.1 : S-N Curve for Normal Stress (Clause 9.4)

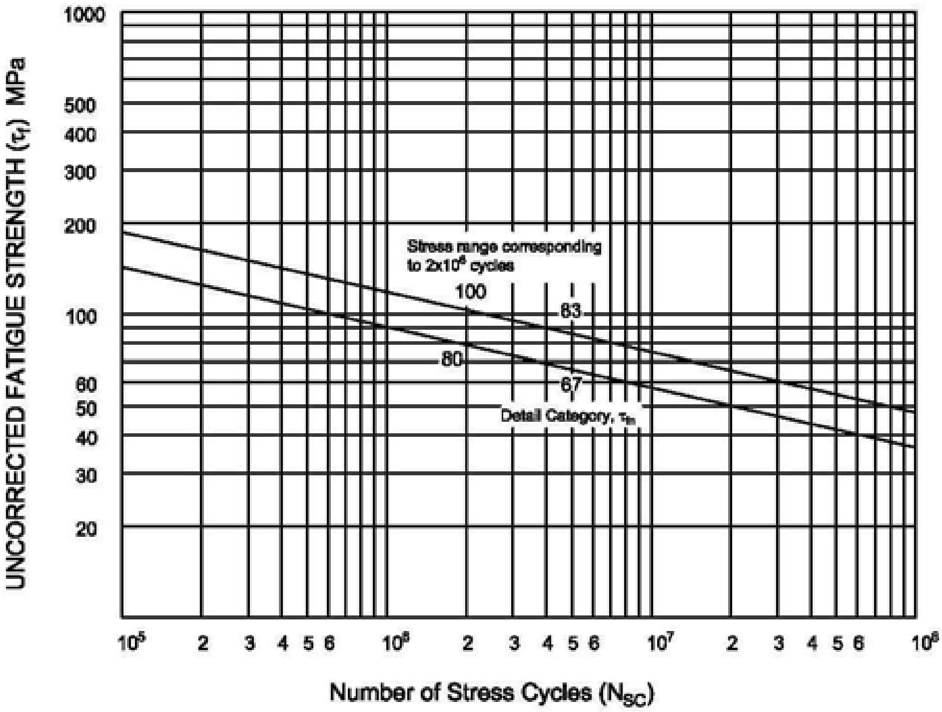


Figure 9.2 : S-N Curve for Shear Stress (Clause 9.4)

9.5 Fatigue Assessment

The design fatigue strength for N_{SC} life cycles (f_{fd} , τ_{fd}) may be obtained from the standard fatigue strength for N_{SC} cycles by multiplying with correction factor, μ_r, for inspection level and thickness, as discussed in Clause 9.2.1 and dividing by partial safety factor given in Table 9.3.

9.5.1 Exemptions - At any point in a structure if the actual normal and shear stress range f, are less than the design fatigue strength range corresponding to 5 x 10^6 cycles, with appropriate partial safety factor, no further assessment for fatigue is necessary at that point.

9.5.2 Stress limitations

9.5.2.1 The (absolute) maximum value of the normal and shear stresses shall never exceed the elastic limit (f_y , τ_y) for the material under cyclic loading.

9.5.2.2 The maximum stress range shall not exceed 1.5 f_y for normal stresses and 1.5 f_y / S3 for the shear stresses under any circumstance.

9.5.2.3 Constant stress range - The actual normal and shear stress range f and τ at a point of the structure subjected to N_{SC} cycles in life shall satisfy.

$$f \leq f_{fd} = \mu_r f_f / \gamma_{mft}$$

$$\tau \leq \tau_{fd} = \mu_r \tau_f / \gamma_{mft}$$

where

μ_r = correction factor (Clause 9.2.1)

γ_{mft} = partial safety factor against fatigue failure, given in Table 9.3

f_f , τ_f = normal and shear fatigue strength ranges for the actual life cycle, N_{SC}, obtained from Clause 9.4.

9.5.2.4 Variable stress range - Fatigue assessment at any point in a structure, wherein variable stress ranges f_{ft} or τ_{ft} for n_i number of cycles (i = 1 to r) are encountered, shall satisfy the following:

a) For normal stress (f)

$$\frac{\sum_{i=1}^{r5} \quad n_i f_i^3}{5\times10^6(\mu f_{fn}/\gamma_{mft})^3} + \frac{\sum_{j=r5}^{r} \quad f_j^5}{5\times10^6(\mu f_{fn}/\gamma_{mft})^5} \leq 1.0$$

b) For shear stresses (τ)

$$\sum_{j=1}^{r} \quad n_i \tau_{ft}^5 \leq 5 \times 10^6 (\mu \tau_{fn}/\gamma_{mft})^5$$

where γ_5 is the summation upper limit of all the normal stress ranges (f_i) having magnitude lesser than (μ_c f_{fn} /γ_{mft}) for that detail and the lower limit of all the normal stress ranges (f_j)

having magnitude greater than ($\mu_c f_{fn} / \gamma_{mft}$)) for the detail. In the above summation all normal stress ranges, f_i and τ_i having magnitude less than $0.55\mu_c f_{fn}$ and $0.55 \mu_c \tau_{fn}$ may be disregarded.

9.6 Necessity for Fatigue Assessment

No fatigue assessment is necessary if any of the following conditions is satisfied.

a) The highest normal stress range f_{fmax} satisfies

$$f_{fmax} \leq 27\mu_c / \gamma_{mft}$$

b) The highest shear stress range τ_{fmax} satisfies

$$\tau_{fmax} \leq 67 \mu_c / \gamma_{mft}$$

c) The total number of actual stress cycles N_{SC}, satisfies

$$N_{SC} \leq 5 \; x \; 10^6 \left(\frac{27\mu_c}{\gamma_{mft} f_{feq}} \right)^3$$

where f_{feq} = equivalent constant amplitude stress range in Mpa given by

$$f_{feq} = \left[\frac{\sum_{i=1}^{\gamma_5} \quad n_i \, f_{ft}^3 + \sum_{j=\gamma_5}^{r} \quad n_i f_{ft}^5}{n} \right]^{\frac{1}{3}}$$

where

n = $\sum_{j=1}^{\gamma} \quad n_i$

f_{ft}, f_{fj} = stress ranges falling above and below the f_{fn} the stress range corresponding to the detail at 5 x 10^6 number of life cycles.

10. FABRICATION ASPECTS

In general, clauses 513 and 514 of IRC:24 along with the IRC Guide Book should be followed for Fabrication, Inspection, Transportation, Handling and Erection of Steel Structures. For stainless steel structure fabrication, special care is required for welding operation including selection of welding consumables in order to ensure that the fabricated structure does not have any potential corrosion problem.

It is more important in stainless steel than carbon steel to reduce locations at which crevice corrosion may initiate. Welding deficiencies such as undercut, lack of penetration, weld spatter, slag and stray arc strikes are all potential sites and should thus be minimised. Stray arc strikes or arcing at loose earth connections also damage the passive layer, and possibly give rise to crevice corrosion, thereby ruining the appearance of a fabrication.

The following aspects should be carefully considered while planning and executing welded joints.

- Welded joints with sharp edges and abrupt change in cross-section or profile should be avoided. The steep change in lines of stresses should be as undisturbed as possible.

- If possible, the center lines of the welded parts should coincide in one point. Weld seams should be avoided in high stress areas. If this is not possible, higher requirements of inspection shall be planned.

- If required, to make a decision in the process of development an evidence of the calculated thickness of the weld a can be proved by production weld test ,with respect to the weldability of the parent materials and the welding consumables the requirements and recommendations information of their manufacturers shall be observed.

- For steel components with stress in thickness direction, suitable design measures and shall be taken and material with the required reduction of material in thickness direction shall be selected.

- Corrosion protection should be ensured by suitable welding design, e.g. full penetration weld. Partial penetration welds or intermittent welds should have sufficient corrosion protection.

- Assemblies shall be designed so as to offer the best access possible when welding or inspecting them.

- The accumulation of joints should be avoided. If necessary, forged pieces or castings can be used.

- Welding secondary parts onto tension flanges by transverse beads should be avoided.

- In the heat-affected zone of cold deformed steel alloys the decrease of strength shall be considered in calculation. Designs with mixed assemblies combining welded joints with bolted or riveted joints should be avoided.
- Minimize the amount of weld metal , Do not over weld
- Use intermittent welding in preference to a continuous weld pass
- Place welds above the neutral axis, Balance the welding about the middle of the joint by using a double-V joint
- Weld-deposition sequence shall be adjusted to minimize the residual stresses and distortion to obtain the desired dimensional stability & mechanical attributes.
- If possible, welding shall be accomplished in PA or PB positions by using simple tilting or rotating device.
- The welding shop will be protected against the harmful wind & weather (e.g. wind, rain, snow and air-draughts) while welding. General cleanliness and the absence of contamination are important for attaining good weld quality. Oils or other hydrocarbons, dirt and other debris, strippable plastic film, and wax crayon marks should be removed to avoid their decomposition and the risk of carbon pick up and weld surface contamination. The weld should be free from zinc, including that arising from galvanised products, and from copper and its alloys.
- Preferably the arc current return cable shall be attached directly to the substrate to enable ample electrical contact with least resistance in circuit. It is recommended to attach the welding current return cable as close as possible to the arc spots.
- The substrate or work piece temperature, just before the arc strikes, shall always be maintained 15 °C (Min) & inter-run temperature be ≤ 180 °C (Max).
- If the tack-welds are merged into the final weld, then those tack-welds too will be subjected to the same degree of inspection, as required for the final weld assembly. Tack welds becoming the part of final weld shall be accomplished in a manner that they are melted & merged precisely during welding.
- After removing the notches & ridges caused by the finishing operations, as grinding or cutting, the wall-thicknesses shall be maintained at least 98 % of that designed nominal thicknesses. If the nominal wall thickness reduced beyond 2% a repair shall be warranted by welding. Deviation shall only be allowed with the prior agreements between the customer & manufacturer.
- If the drawings specify notch free surfaces for the obvious reason of fatigue- strength then that weld surfaces shall be ground flush in that direction of loading.
- For repairs, only, the qualified welding procedure & welder suitable for that type and class of welding shall be used. If the systematic damage or deviations observed from

the blueprint drawing the prior agreement of the customer shall be inevitable before welding.

- Stainless steels have a rate of thermal expansion 50% greater than carbon steels. The lower thermal conductivity than carbon steel leads to steeper temperature gradients. Use frequent tacks, or skip welding to reduce stresses. Minimize weaving techniques which result in slower travel speeds and higher heat input. Stringer beads are most desired when welding on stainless steel or nickel base alloys.

- Post-Weld Cleaning is a very important step. The purpose of post weld cleaning is to ensure a properly formed chrome oxide film on the surface for optimum corrosion resistance: the smoother the finish, the higher the corrosion resistance. The heat from welding is capable of depleting chrome at the surface which can result in corrosion. To avoid rust, it is very important to remove the chrome depleted zone by chemical or mechanical post weld cleaning.

- Use of stainless steel brushes and other tools are highly recommended to avoid impinging iron particles into the surface which will cause rust.

- Do not mix stainless steel and carbon steel fabrications to avoid iron contamination. Iron particles serve to initiate localized corrosion.

11. WORKED OUT EXAMPLE

In IRC:SP:120 – the Explanatory Handbook to IRC:22-2015 – the IRC design code for Steel Concrete Composite Construction, a worked out example of a 30m two lane Steel Concrete Composite bridge has been presented. A same design with identical loading etc. is presented here with Stainless Steel of grade IRS 350 CR. The example problem in SP:120 considered Steel Grade E-350 B0.

The example bridge is having a span length of 32.0 m (c/c of expansion joint) with effective span of 30.0 m (c/c of bearings). The bridge carries a 2-lane single carriageway road . The overall width of the deck is 12 m comprising of 7.5 m carriageway with 1.5 m wide footway on either side of the carriageway. The carriageway and footway is separated by 0.45 m concrete crash barriers. Structural scheme for the superstructure comprise of steel concrete composite plate girders, four numbers spaced @ 3 m c/c, with 200 mm thick in-situ RCC deck slab on top. The deck cantilevers 1.5 m outside the web of the outer girders.. Corrugated Profiled Sheet is proposed to be used for the casting of deck slab

1.0 Design Data for Simply Supported Stainless Steel Composite Girder

1.1Structural Geometry

Total Length of Span =	L	32.0	m	Distance b/w centers of the expansion joint
Effective span (C/C of Bearing) =	Lo	30.0	m	
Deck width =	decw	12.00	m	
Number of Plate Girders =		4		
Edge cantilever distance =	B0	1.500	m	
Outer Girder spacing =	B1	3.000	m	(C/C spacing of Girders)
Inner Girder spacing =	B2	3.000	m	

1.2Material Property

Steel Plate Girder Properties

Material Properties: -
Steel Grade Designation (Chara. Yield Strength) :**IRS350CR**

Yield Strength	fsyk.1	350	MPa	*For Plate < 16mm*	*RDSO Doc. No. BS-S-7.5.3.1-9, Ver-10*
	fsyk.2	350	MPa	*For Plate 16 - 40mm*	*Table 2, IS 2062: 2011*
	fsyk.3	350	MPa	*For Plate 40- 50mm*	*Table 2, IS 2062: 2011*
Elastic Modulus of Steel Plate Girder - E_{sk}		2.00E+05	MPa		
Coefficient of thermal expansion of steel - α		1.20E-05	/°C		
Unit Weight of Steel	u_{ws}	78.50	KN/m^3		*SP120-2018*

Cast-in-Situ Concrete:

Material Properties: -				
Concrete Grade		M40		
Compressive Strength of Concrete	f_{ck}	40		
Elastic Modulus of Concrete	E_{ck}	3.30E+04	Mpa	*SP120-2018*
Unit Weight of Concrete	u_{wc}	25.00	kN/m^3	*SP120-2018*
Creep Factor	Kc	0.50		*IRC:22 C; 604.3*
Coefficient of thermal expansion of concrete	α	1.17E-05		*SP120-2018*
Thickness of slab	slh	0.22	m	
Grade of steel		Fe500		
Modular Ratio: -				
Es/Ec for Transient Loading	m.1	7.50		*IRC:22 C; 604.3*
Es/Ec for Permanent Loading	m.2	15.00		

1.3 Girder Geometry

Steel Girder :		*Midspan*	*Support*	*At Splice*		(considering reduction due
Width of Top Flange =	w_t	350	350	350	mm	to bolt area on tension flange)
Thickness of Top Flange =	t_{ft}	20	20	20	mm	
Width of Bottom Flange =	w_b	450	450	356	mm	
Thickness of Bottom Flange =	t_{fb}	20	20	20	mm	
Width of Bottom cover plate =	w_{b1}	0	-	269	mm	
Thickness of Bottom cover plate =	t_{fb1}	0	-	0		
Depth of Web =	d_w	1700	1700	1700	mm	(Clear Depth of Web)
Thickness of Web =	t_w	12	12	12	mm	
					m	
Top Flange Outstand =	b	169.0	169.0	169.0	mm	
Bottom Flange Outstand =	b_t	219.0	219.0	172.2	m	

Depth of Girder only section

Min. Overall depth required - (Leff/25)	dech.r	1.20	m	Clause– 6.1.4
Min. Overall depth provided	dech.p	1.74	m	OK

Spacing of Girders

Min. spacing of girders required - Leff/20	gsp.r	1.50	m		*Clause – 6.1.3*
Min. spacing of girders provided	gsp.p	3.00	m	**OK**	

Check for Minimum Section

Minimum Web Thickness:

Ratio of Depth to Thickness =d/tw	141.67	**OK**
Horizontal stiffeners		**NOT REQUIRED**
Depth of web, d	1700	mm
	ε_f	0.845
	ε_w	0.845
Distance between two stiffners,c		1500 mm
minimum tw1		8.87 mm *Cl. 6.6.6.1.1,b,ii*
minimum tw2		5.83 mm *Cl. 6.6.6.1.2,b,ii*
Minimum thickness of Web =	tw.1	8.87 mm
Provided Thickness of Web =	tw	12.00 **OK**

Notes:

1) The structure chosen is identical to the structure in the standard worked-out example in Appendix-IV of IRC:SP:120-2018.

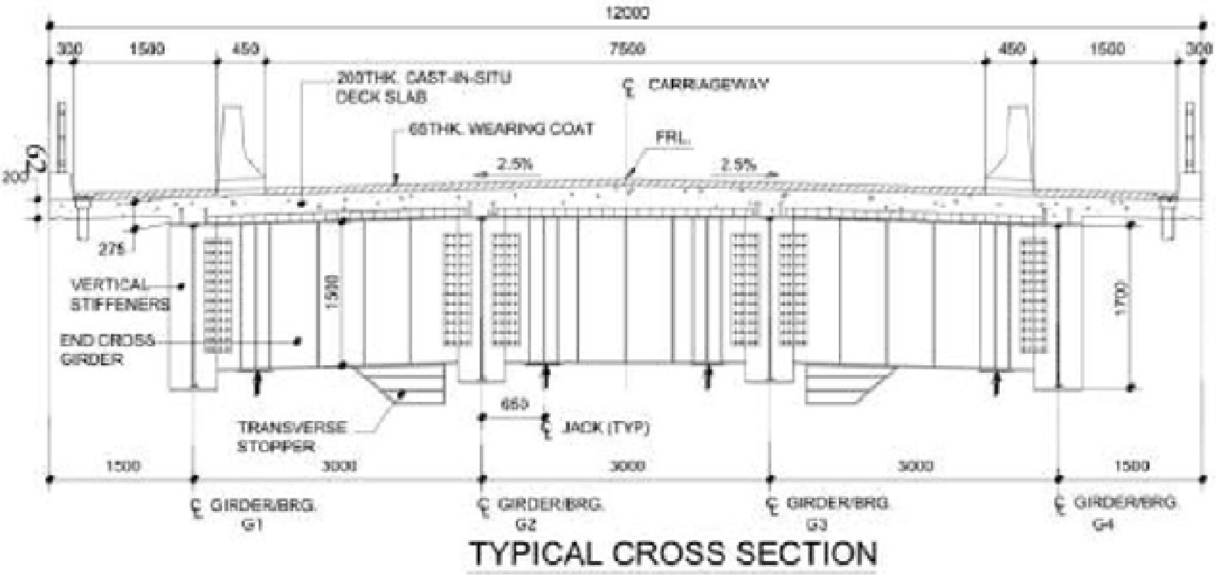
2) As in the above example, the design calculations are presented for the external main girder (marked 'G1').

3) The design calculation is presented for critical bending moment at midspan and shear force at support, being representative section checks for academic interest & for purpose of this assignment.

4) The analysis results for SIDL and live loads are taken from values in the Worked-out example, as the span, carriageway configuration and structural arrangements are identical.

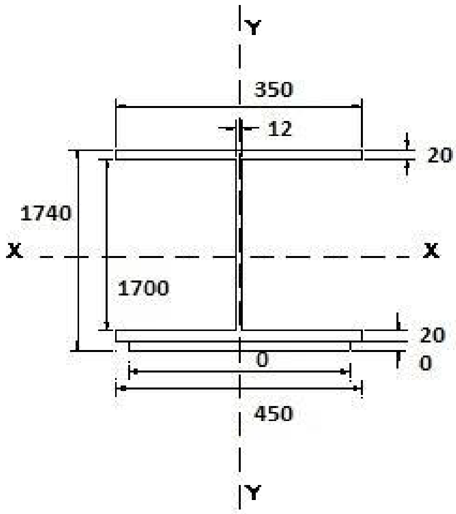
5) The steel section chosen is different from the IRC:SP:120-2018 to justify the efficiency in using stainless steel over carbon steel. The bending moment, shear force and deflection for self weight of steel sections are therefore recalculated and presented in section-9. The associated effects from temperature gradient and differential shrinkage are also recalculated due to change in steelwork geometry and dimensions.

2.0 Gross Girder Section Property Calculation



2.1 External Girder (G1) -

2.1.1 SteelOnly Property at Mid Span:



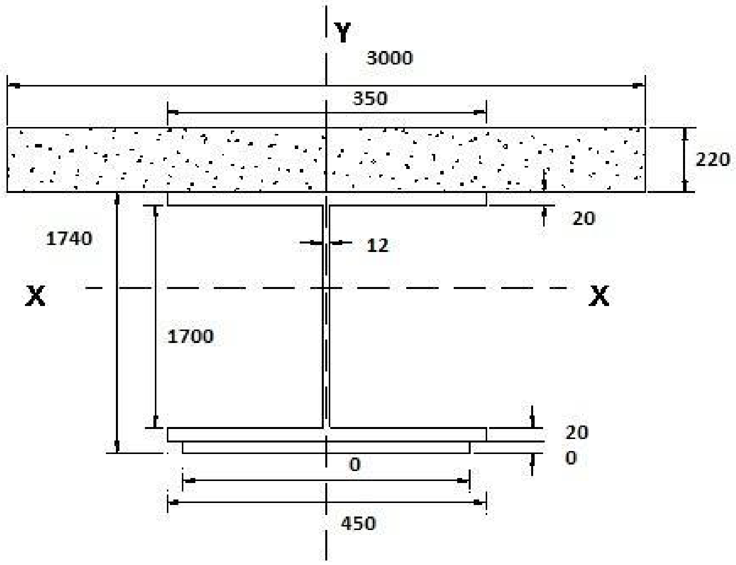
	At Midspan	At Support	
tfb	350	350	mm
tfh	20	20	mm
bfb	450	450	mm
bfh	20	20	mm
bcb	450	-	mm
bch	20	-	mm
wdep	1700	1700	mm
wth	12	12	mm

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Top Flange	350	20	7000	1730	12110000	907.25	2.333E+05	5.76E+09	7.15E+07
Web	12	1700	20400	870	17748000	47.25	4.913E+09	4.96E+09	2.45E+05
Bottom Flange	450	20	9000	10	90000	812.75	3.000E+05	5.95E+09	1.52E+08
Cover Plate	0	0	0	0	0	822.75	0.000E+00	0	0.00E+00
							Σ	1.67E+10	2.24E+08
C/S area of Steel Girder : -			A.sg =	36400	mm^2 =	0.0364	m^2		

Center of Gravity: -	(From Base)	Y_b =	822.75	mm		
	(From Top)	Y_t =	917.25	mm		
Moment of Inertia: -		I_{xx} =	1.67E+10	mm^4 =	0.0167	m^4
Section Modulus: -	At Top Fibre	Z_t =	1.817E+07	mm^3 =	0.0182	m^3
	At Bottom Fibre	Z_b =	2.026E+07	mm^3 =	0.0203	m^3
Moment of Inertia about Y - Axis		Iyy =	2.236E+08	mm^4 =	0.0002	m^3
Plastic Section Modulus: -	(From Top)	Y_t =	953.33	mm		
	At Top Fibre	Z_p =	2.235E+07	mm^3 =	0.0223	m^3

2.1.2 Composite for Transient Loading (For Live Loads/Short Term) at Mid Span:

Effective Width of slab:-	$\frac{L_o}{8} + X \leq \frac{B1}{2} + X$	=	3.00	m	*Cl.603.2.1 of IRC:22*
Avg. depth of slab:-		=	0.22	m	
Slab Eccentricity from girder vertical axis		=	0.00	m	



Equi. Beff 400.0

tfb	350	mm
tfh	20	mm
bfb	450	mm
bfh	20	mm
bcb	0	mm
bch	0	mm
wdep	1700	mm
wth	12	mm

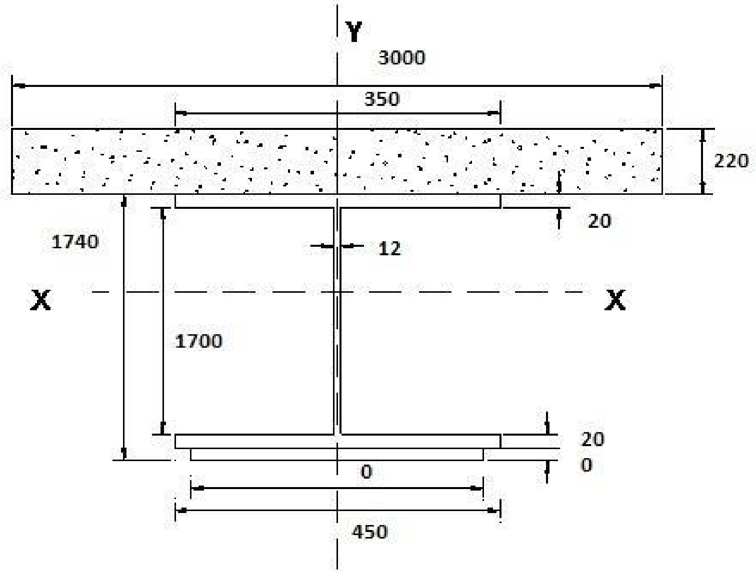
	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Slab	400.0	220.00	88000	1850	162800000	300.58	3.549E+08	8.31E+09	6.60E+10
Top Flange	350	20	7000	1730	12110000	180.58	2.333E+05	2.28E+08	7.15E+07
Web	12	1700	20400	870	17748000	679.42	4.913E+09	1.43E+10	2.45E+05
Bottom Flange	450	20	9000	10	90000	1539.42	3.000E+05	2.13E+10	1.52E+08
Cover Plate	0	0	0	0	0	1549.42	0.000E+00	0	0.00E+00
							Σ	4.42E+10	6.62E+10

C/S Area of Composite Section: -	A.cp1 =	124400	mm^2 =	0.1244	m^2

Center of Gravity: -	(From Base)	$Y_{b.cp1}$ =	1549.4	mm		
	(From Top)	$Y_{t.cp1}$ =	410.6	mm		
Moment of Inertia: -		$I_{xx.cp1}$ =	4.419E+10	mm^4 =	0.0442	m^4
Section Modulus: -	At Slab Top	$Z_{t.sl1}$ =	1.076E+08	mm^3 =	0.1076	m^3
	At Slab Bottom/Top Fibre	$Z_{t.cp1}$ =	2.319E+08	mm^3 =	0.2319	m^3
	At Bottom Fibre	$Z_{b.cp1}$ =	2.852E+07	mm^3 =	0.0285	m^3
Moment of Inertia about Y - axis: -		$I_{yy.cp1}$ =	6.622E+10	mm^4 =	0.0662	m^4

2.1.3 Composite for Permanent Loading (For SIDL/Long term)

Effective Width of slab:-	=	3.000	m
Avg. depth of slab:-	=	0.220	m



Equiv Beff 200.0

tfb	350	mm
tfh	20	mm
bfb	450	mm
bfh	20	mm
bcb	0	mm
bch	0	mm
wdep	1700	mm
wth	12	mm

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Slab	200.0	220.00	44000	1850	81400000	465.07	1.775E+08	9.69E+09	3.30E+10
Top Flange	350	20	7000	1730	12110000	345.07	2.333E+05	8.34E+08	7.15E+07
Web	12	1700	20400	870	17748000	514.93	4.913E+09	1.03E+10	2.45E+05
Bottom Flange	450	20	9000	10	90000	1374.93	3.000E+05	1.7E+10	1.52E+08
Cover Plate	0	0	0	0	0	1384.93	0.000E+00	0	0.00E+00
							Σ	3.79E+10	3.32E+10

C/S Area of Composite Section: - A.cp2 = 80400 mm^2 = 0.0804 m^2

Center of Gravity: - (From Base) $Y_{b.cp2}$ = 1384.9 mm

(From Top) $Y_{t.cp2}$ = 575.1 mm

Moment of Inertia: - $I_{xx.cp2}$ = 3.786E+10 mm^4 = 0.0379 m^4

Section Modulus: - At Slab Top $Z_{t.sl2}$ = 6.584E+07 mm^3 = 0.0658 m^3

At Slab Bottom/Top Fibre $Z_{t.cp2}$ = 1.066E+08 mm^3 = 0.1066 m^3

At Bottom Fibre $Z_{b.cp2}$ = 2.734E+07 mm^3 = 0.0273 m^3

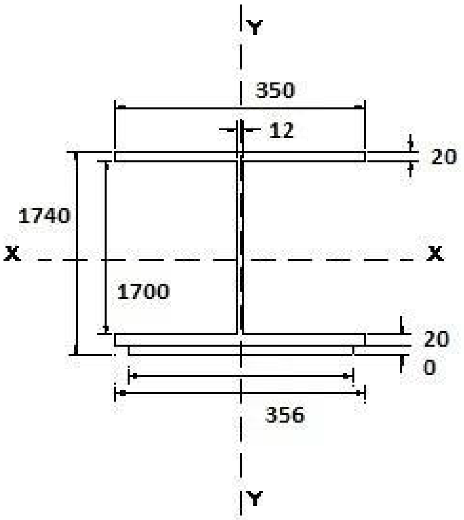
Moment of Inertia about Y - axis: - $I_{yy.cp2}$ = 3.322E+10 mm^4 = 0.0332 m^4

2.1.4 Summary of Section Properties at Mid Span:

	Steel Only for Dead Load	Composite 1 for Live Load	Composite 2 for SIDL
Area (mm^2)	36400.0	124400.0	80400.0
Y*(From Base) (mm)	822.7	1549.4	1384.9
I_{xx} (mm^4)	1.67E+10	4.42E+10	3.79E+10
I_{yy} (mm^4)	2.24E+08	6.62E+10	3.32E+10
Z_p (mm^3)	2.23E+07	-	-
$Z_{1(A-A)}$ (mm^3)-Slab Top	-	1.08E+08	6.58E+07
$Z_{2(B-B)}$ (mm^3)-Girder Top	1.82E+07	2.32E+08	1.07E+08
$Z_{3(C-C)}$ (mm^3)-Girder Bottom	2.03E+07	2.85E+07	2.73E+07

2.2.1Steel Only Property at Splice

At Splice



tfb	350	mm
tfh	20	mm
bfb	356	mm
bfh	20	mm
bcb	269	mm
bch	0	mm
wdep	1700	mm
wth	12	mm

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Top Flange	350	20	7000	1730	12110000	863.19	233333.333	5.22E+09	7.15E+07
Web	12	1700	20400	870	17748000	3.19	4913000000	4.91E+09	2.45E+05
Bottom Flange	356	20	7128	10	71280	856.81	237600	5.23E+09	7.55E+07
Cover Plate	269	0	0	0	0	866.81	0	0	0.00E+00
							Σ	1.54E+10	1.47E+08

C/S area of steel girder A.sg = 34528 mm^2 = 0.0345 m^2

Center of Gravity: - (From Base) Y_b = 866.81 mm

(From Top) Y_t = 873.19 mm

Moment of Inertia: - I_{xx} = 1.5362E+10 mm^4 = 0.0154 m^4

Section Modulus: - At Top Fibre Z_t = 1.759E+07 mm^3 = 0.0176 m^3

At Bottom Fibre Z_b = 1.772E+07 mm^3 = 0.0177 m^3

Moment of Inertia about Y - Axis Iyy = 1.472E+08 mm^4

	B	D	Area	Y_T-Y	AY_T
Top Flange	350	20	7000	865.33	6.057E+06

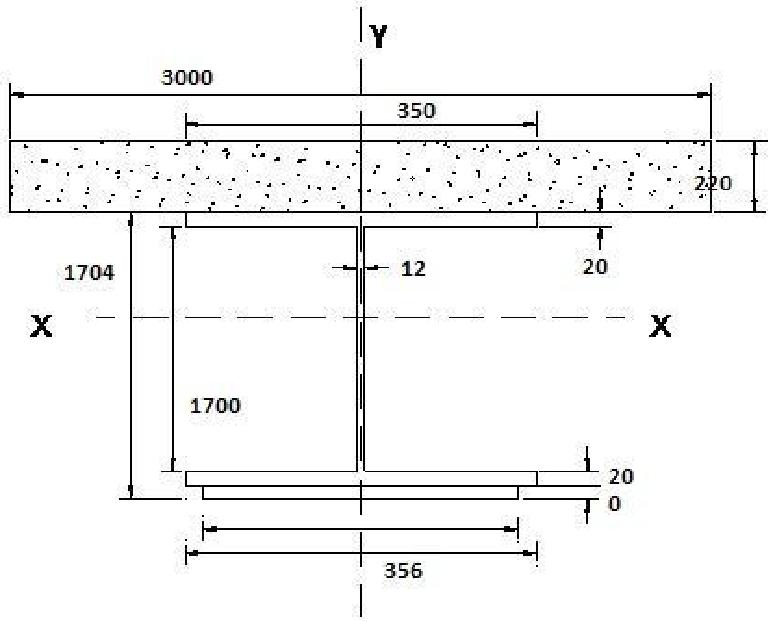
Web	12	855.33	10264	427.67	4.390E+06	PNA
Web	12	844.67	10136	422.33	4.281E+06	
Bottom Flange	356	20	7128	854.67	6.092E+06	
Cover Plate	269	0	0	864.67	0.000E+00	

Plastic Section Modulus: -	(From Top)	Y_t =	875.33	mm		
	At Top Fibre	Z_p =	2.042E+07	mm^3 =	0.0204	m^3

2.2.2 Composite for Transient Loading (For Live Loads) at splice

Effective Width of slab:- $\frac{L_o}{8} + X \leq \frac{B1}{2} + X$	=	3.00	m
Avg. depth of slab:-	=	0.22	m
Slab Eccentricity from girder vertical axis		0.00	m

Equi. Beff 400 mm



tfb	350	mm
tfh	20	mm
bfb	356	mm
bfh	20	mm
bcb	269	mm
bch	0	mm
wdep	1700	mm
wth	12	mm

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Slab	400	220	88000	1850	162800000	277.06	3.549E+08	7.11E+09	3.55E+08
Top Flange	350	20	7000	1730	12110000	157.06	2.333E+05	1.73E+08	7.15E+07
Web	12	1700	20400	870	17748000	702.94	4.913E+09	1.5E+10	2.45E+05

Bottom Flange	356	20	7128	10	71280	1562.94	2.376E+05	1.74E+10	7.55E+07
Cover Plate	269	0	0	0	0	1572.94	0.000E+00	0	0.00E+00
							Σ	3.97E+10	5.02E+08

C/S Area of Composite Section: - A.cp1 = 122528 mm^2 = 0.1225 m^2

Center of Gravity: - (From Base) $Y_{b.cp1}$ = 1572.94 mm

(From Top) $Y_{t.cp1}$ = 387.06 mm

Moment of Inertia: - $I_{xx.cp1}$ = 3.9688E+10 mm^4 = 0.0397 m^4

Section Modulus: - At Slab Top $Z_{t.sl1}$ = 1.025E+08 mm^3 = 0.1025 m^3

At Slab Bottom/Top Fibre $Z_{t.cp1}$ = 2.376E+08 mm^3 = 0.2376 m^3

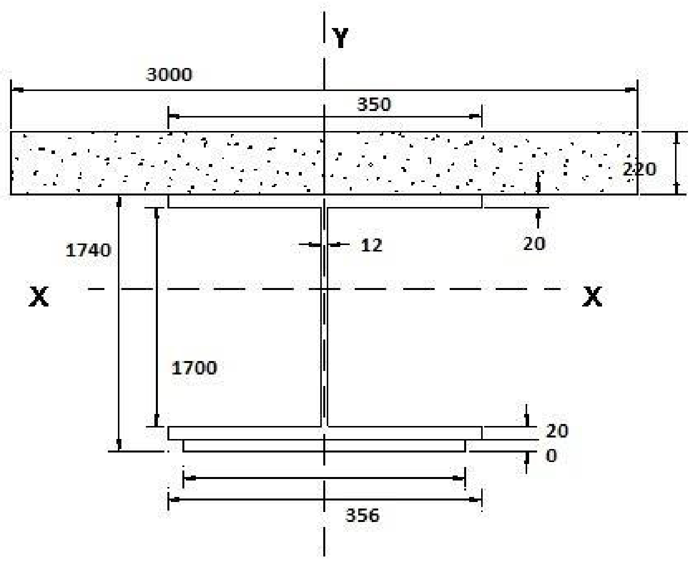
At Bottom Fibre $Z_{b.cp1}$ = 2.523E+07 mm^3 = 0.0252 m^3

Moment of Inertia about Y - axis: - $I_{yy.cp1}$ = 5.021E+08 mm^4 = 0.0005 m^4

2.2.3 Composite for Permanent Loading (For Live Loads) at splice

Effective Width of slab:- $\frac{L_o}{8} + X \leq \frac{B1}{2} + X$ = 3.00 m

Avg. depth of slab:- = 0.22 m



Equi. Beff 200

tfb	350	mm
tfh	20	mm
bfb	356	mm

bfh	20	mm
bcb	269	mm
bch	0	mm
wdep	1700	mm

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I_{XX}	I_{NA}	I_{yy}
Slab	200	220	44000	1850	81400000	432.30	1.775E+08	8.4E+09	3.30E+10
Top Flange	350	20	7000	1730	12110000	312.30	2.333E+05	6.83E+08	7.15E+07
Web	12	1700	20400	870	17748000	547.70	4.913E+09	1.1E+10	2.45E+05
Bottom Flange	356	20	7128	10	71280	1407.70	2.376E+05	1.41E+10	7.55E+07
Cover Plate	269	0	0	0	0	1417.70	0.000E+00	0	0.00E+00
							Σ	3.42E+10	3.31E+10

C/S Area of Composite Section: -A.cp1 = 78528 mm^2 = 0.078528 m^2

Center of Gravity: - (From Base) $Y_{b.cp1}$ = 1417.70 mm

(From Top) $Y_{t.cp1}$ = 542.30 mm

Moment of Inertia: - $I_{xx.cp1}$ = 3.4241E+10 mm^4 = 0.0342 m^4

Section Modulus: - At Slab Top $Z_{t.sl1}$ = 6.314E+07 mm^3 = 0.0631 m^3

At Slab Bottom/Top Fibre $Z_{t.cp1}$ = 1.062E+08 mm^3 = 0.1062 m^3

At Bottom Fibre $Z_{b.cp1}$ = 2.415E+07 mm^3 = 0.0242 m^3

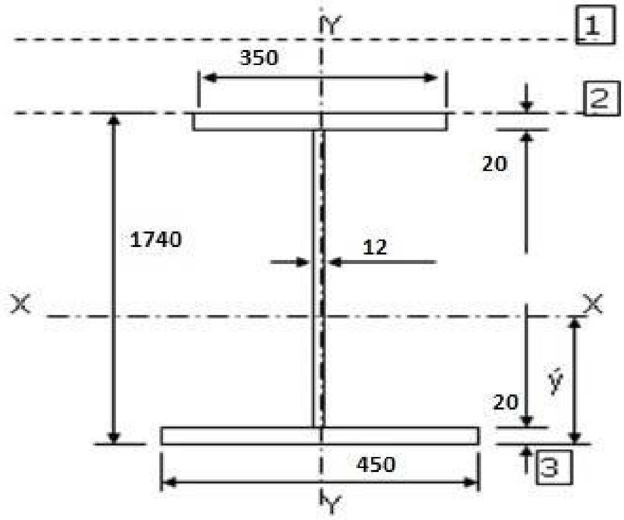
Moment of Inertia about Y - axis: - $I_{yy.cp1}$ = 3.315E+10 mm^4 = 3.315E+07 m^4

2.2.4 Summary of Section Properties at splice:

	Steel Only for Dead Load	Composite 1 for Live Load	Composite 2 for SIDL
Area (mm^2)	34528.0	122528.0	78528.0
Y*(From Base) (mm)	866.8	1572.9	1417.7
I_{xx} (mm^4)	1.54E+10	3.97E+10	3.42E+10
I_{yy} (mm^4)	1.47E+08	5.02E+08	3.31E+10

Z_p (mm^3)	2.04E+07	-	-
$Z_{1(A-A)}$ (mm^3)-Slab Top	-	1.03E+08	6.31E+07
$Z_{2(B-B)}$ (mm^3)-Girder Top	1.76E+07	2.38E+08	1.06E+08
$Z_{3(C-C)}$ (mm^3)-Girder Bottom	1.77E+07	2.52E+07	2.42E+07

2.3.1 Steel Only Property at Support:



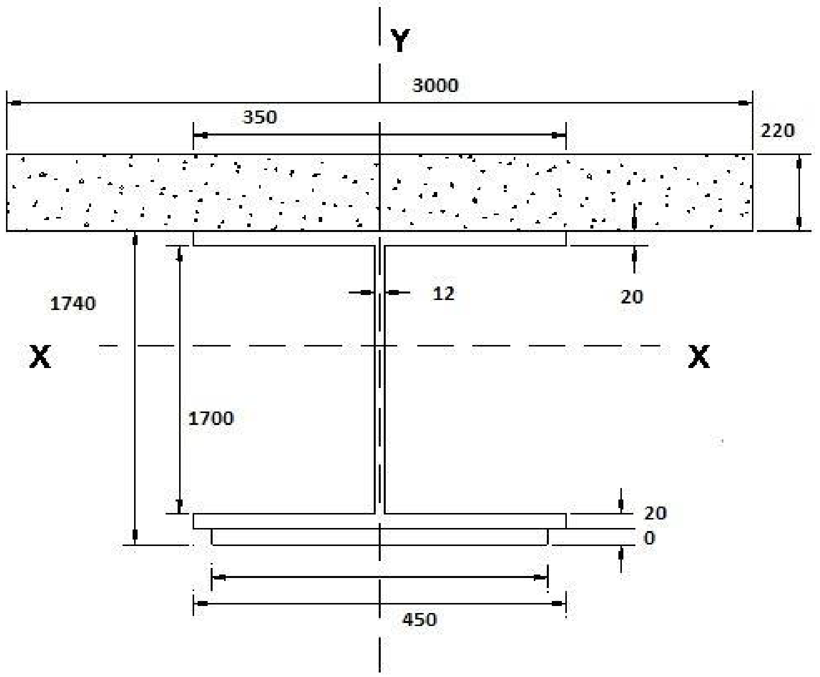
At Support		
tfb	350	mm
tfh	20	mm
bfb	450	mm
bfh	20	mm
wdep	1700	mm
wth	12	mm

C/S area of Steel Girder : -		A.sg =	36400	mm^2 =	0.0364	m^2
Center of Gravity: -	(From Base)	Y_b =	822.75	mm		
	(From Top)	Y_t =	917.25	mm		
Moment of Inertia: -		I_{xx} =	1.67E+10	mm^4 =	0.0167	m^4
Section Modulus: -	At Top Fibre	Z_t =	1.817E+07	mm^3 =	0.0182	m^3
	At Bottom Fibre	Z_b =	2.026E+07	mm^3 =	0.0203	m^3
Moment of Inertia about Y - Axis		Iyy =	2.236E+08	mm^4		
Plastic Section Modulus: -	(From Top)	Y_t =	953.33	mm		
	At Top Fibre	Z_p =	2.235E+07	mm^3 =	0.0223	m^3

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Top Flange	350	20	7000	1730	12110000	907.25	2.333E+05	5.76E+09	7.15E+07
Web	12	1700	20400	870	17748000	47.25	4.913E+09	4.96E+09	2.45E+05
Bottom Flange	450	20	9000	10	90000	-812.75	3.000E+05	5.95E+09	1.52E+08
							Σ	1.67E+10	2.24E+08

2.3.2 Composite for Transient Loading (For Live Loads) at Support:

Effective Width of slab:-	=	3.000	m
Avg Depth of slab	=	0.220	m
Slab Eccentricity from girder vertical axis	=	0.000	m



Equiv Beff 400

C/S Area of Composite Section: -		A.cp1 =	124400	mm^2 =	0.1244	m^2
Center of Gravity: -	(From Base)	$Y_{b.cp1}$ =	1549.42	mm		
	(From Top)	$Y_{t.cp1}$ =	410.58	mm		
Moment of Inertia: -		$I_{xx.cp1}$ =	4.419E+10	mm^4 =	0.0442	m^4
Section Modulus: -	At Slab Top	$Z_{t.sl1}$ =	1.076E+08	mm^3 =	0.1076	m^3
	At Slab Bottom/Top Fibre	$Z_{t.cp1}$ =	2.319E+08	mm^3 =	0.2319	m^3
	At Bottom Fibre	$Z_{b.cp1}$ =	2.852E+07	mm^3 =	0.0285	m^3

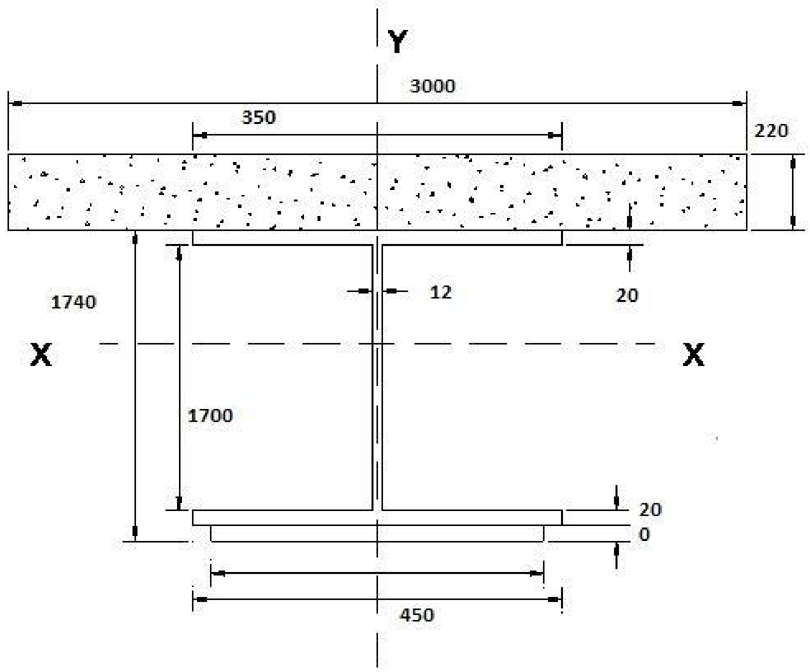
Moment of Inertia about Y - axis: - $I_{yy.cp1}$ = 6.622E+10 mm^4 = 0.0662 m^4

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Slab	400.0	220.00	88000	1850	162800000	300.58	3.549E+08	8.31E+09	6.60E+10
Top Flange	350	20	7000	1730	12110000	180.58	2.333E+05	2.28E+08	7.15E+07
Web	12	1700	20400	870	17748000	-679.42	4.913E+09	1.43E+10	2.45E+05
Bottom Flange	450	20	9000	10	90000	-1539.42	3.000E+05	2.13E+10	1.52E+08
							Σ	4.42E+10	6.62E+10

2.3.3 Composite for Permanent Loading (For SIDL):

Effective Width of slab:- = 3.000 m

Avg. depth of slab:- = 0.220 m



Equiv Beff 200.0

C/S Area of Composite Section: - A.cp2 = 80400 mm^2 = 0.0804 m^2

Center of Gravity: - (From Base) $Y_{b.cp2}$ = 1384.93 mm

(From Top) $Y_{t.cp2}$ = 575.07 mm

Moment of Inertia: - $I_{xx.cp2}$ = 3.788E+10 mm^4 = 0.0379 m^4

Section Modulus: - At Slab Top $Z_{t.sl2}$ = 6.584E+07 mm^3 = 0.0658 m^3

At Slab Bottom/Top Fibre	$Z_{t.cp2}$ =	1.066E+08	mm^3 =	0.1066	m^3
At Bottom Fibre	$Z_{b.cp2}$ =	2.734E+07	mm^3 =	0.0273	m^3

Moment of Inertia about Y - axis: -	$I_{yy.cp2}$ =	3.322E+10	mm^4 =	3.322E-02	m^4

	B	D	Area	Y_B	AY_B	Y_B-NA_B	I	I_{ZZ}	I_{yy}
Slab	200.0	220.0	44000	1850	81400000	465.07	1.775E+08	9.69E+09	3.30E+10
Top Flange	350.0	20.0	7000	1730	12110000	345.07	2.333E+05	8.34E+08	7.15E+07
Web	12.0	1700.0	20400	870	17748000	-514.93	4.913E+09	1.03E+10	2.45E+05
Bottom Flange	450.0	20.0	9000	10	90000	-1374.93	3.000E+05	1.7E+10	1.52E+08
							Σ	3.79E+10	3.32E+10

2.3.4 Summary of Section Properties at Support:

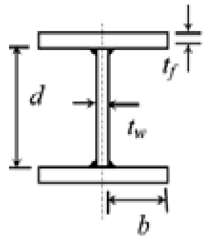
	Steel Only for Dead Load	Composite 1 for Live Load	Composite 2 for SIDL
Area (mm^2)	36400	124400	80400
Y*(From Base) (mm)	822.747	1549.42	1384.93
I_{xx} (mm^4)	1.667E+10	4.419E+10	3.786E+10
I_{yy} (mm^4)	2.24E+08	6.622E+10	3.322E+10
Z_p (mm^3)	2.23E+07	-	-
$Z_{1(A-A)}$ (mm^3)	-	1.076E+08	6.584E+07
$Z_{2(B-B)}$ (mm^3)	1.817E+07	2.319E+08	1.066E+08
$Z_{3(C-C)}$ (mm^3)	2.026E+07	2.852E+07	2.734E+07

Effective Girder Section Properties

3.1 Check for local buckling – calculation of effective section properties

Top flange plate:	**b**	**t**
Outstand welded with web	169	20
Actual member bw/tw	8.45	
Limiting Class 2 - bw/tw, 9.4ε	7.94	
Hence, class 3		
The section of top flange is considered as plastic after deck slab is hardened in accordance with clause 603.1.3 (1) of IRC:22-2015.		
Web plate:	**d**	**t**
Web plate	1700	12
Actual member bw/tw	141.67	
Limiting Class 3 - bw/tw, 126ε	106.49	
Hence, class 4		
Ineffective excess width of web	422	
Effective width	1278	

Table 2 & Fig 5 : IRC-22



CALCULATION OF EFFECTIVE WIDTH OF WEB

Calculation of beff for midspan section with gross cross section properties:

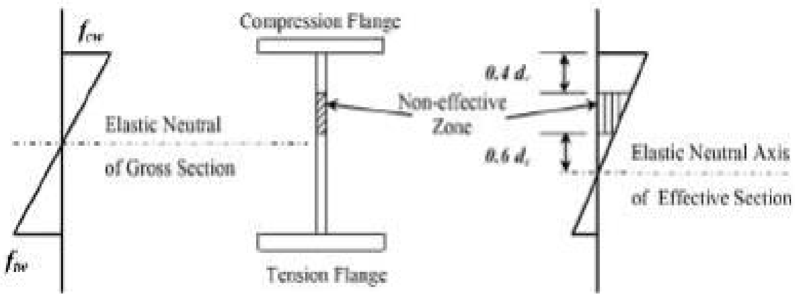


Fig. 4b Effective Width of Slender Web

Distribution of effective width:

The distribution of the effective width is done in accordance with Table 4.1 of EN 1993-1-5:2006. The effective width is divided in two parts i.e., a) 0.4 dc from the top of the web & b) 0.6 dc from the elastic neutral axis.

Ineffective excess width of web = 422 mm

Effective width of web in compression zone, dc = 475.25 mm
Start of ineffective web from top of web, 0.4dc = 190.10 mm
End of ineffective web from top of web = 612.10 mm

Hence, Plate girder section is Class - 3 : Semi-compact (after deducting the excess width of web)

Calculation of Ix-x:

Sl. No.	No.	A (mm^2)	y (mm)	Ay (mm^3)	Ay^2 (mm^4)	Io (mm^4)	Io +Ay2 (mm^4)
Deductions for ineffective web :							
1. Web plate	1	5064	1318.9	6.68E+06	8.81E+09	7.52E+07	8.88E+09
Composite Section (T-Loading)	Σ	119336		1.86E+08	3.29E+11	5.19E+09	3.34E+11
Composite Section (P-Loading)	Σ	75336		1.05E+08	1.78E+11	5.02E+09	1.83E+11
Girder only	Σ	31336		2.33E+07	2.76E+10	4.84E+09	3.24E+10

3.2 Summary of reduced Section Properties at Mid Span:

	Girder Only	Composite (T-Loading)	Composite (P-Loading)
Area (mm^2)	31336.00	119336.00	75336.00
Y*(From Base) (mm)	742.57	1559.20	1389.36
I_{zz} (mm^4)	1.51E+10	4.38E+10	3.78E+10
I_{yy} (mm^4)	2.24E+08	6.62E+10	3.32E+10
y_t- slab top (mm)	-	400.80	570.64
y_i-girder top (mm)	997.43	180.80	350.64
y_b-girder bottom (mm)	742.57	1559.20	1389.36
$Z_{t(A-A)}$ (mm^3)-Slab Top	-	1.09E+08	6.62E+07
$Z_{i(B-B)}$ (mm^3)-Girder Top	1.52E+07	2.42E+08	1.08E+08
$Z_{b(C-C)}$ (mm^3)-Girder Bottom	2.04E+07	2.81E+07	2.72E+07

3.3 Calculation of reduced Section Properties at Support:

Sl. No.	No.	A (mm^2)	y (mm)	Ay (mm^3)	Ay^2 (mm^4)	Io (mm4)	Io +Ay^2 (mm4)
Deductions for ineffective web :							
1. Web plate	1	5064	1318.90	6.68E+06	8.81E+09	7.52E+07	8.88E+09

Composite Section (T-Loading)	Σ	119336		1.86E+08	3.29E+11	5.19E+09	3.34E+11
Composite Section (P-Loading)	Σ	75336		1.05E+08	1.78E+11	5.02E+09	1.83E+11
Girder only	Σ	31336		2.33E+07	2.76E+10	4.84E+09	3.24E+10

	Girder Only	Composite (T-Loading)	Composite (P-Loading)
Area (mm^2)	31336.00	119336.00	75336.00
Y*(From Base) (mm)	742.57	1559.20	1389.36
I_{zz} (mm^4)	1.51E+10	4.38E+10	3.78E+10
I_{yy} (mm^4)	2.24E+08	6.62E+10	3.32E+10
y_t- slab top	-	400.80	570.64
y_i-girder top	997.43	180.80	350.64
y_b-girder bottom	742.57	1559.20	1389.36
$Z_{t(A-A)}$ (mm^3)-Slab Top	-	1.09E+08	6.62E+07
$Z_{i(B-B)}$ (mm^3)-Girder Top	1.52E+07	2.42E+08	1.08E+08
$Z_{b(C-C)}$ (mm^3)-Girder Bottom	2.04E+07	2.81E+07	2.72E+07

3.4 Summary of reduced Section Properties at Splice:

Sl. No.	No.	A (mm^2)	y (mm)	Ay (mm^3)	Ay2 (mm^4)	Io (mm^4)	Io +Ay2 (mm4)
Deductions for ineffective web :							
1. Web plate	1	5064	1318.9	6.68E+06	8.81E+09	7.52E+07	8.88E+09
Composite Section (T-Loading)	Σ	117464		1.86E+08	3.29E+11	5.19E+09	3.34E+11
Composite Section (P-Loading)	Σ	73464		1.05E+08	1.78E+11	5.02E+09	1.83E+11
Girder only	Σ	29464		2.33E+07	2.76E+10	4.84E+09	3.24E+10

	Girder Only	Composite (T-Loading)	Composite (P-Loading)
Area (mm^2)	29464.00	117464.00	73464.00
Y*(From Base) (mm)	789.11	1583.89	1424.51
I_{zz} (mm^4)	1.41E+10	3.93E+10	3.41E+10
I_{yy} (mm^4)	1.47E+08	5.02E+08	3.31E+10

y_t- slab top	-	376.11	495.49
y_i-girder top	950.89	156.11	315.49
y_b-girder bottom	789.11	1583.89	1424.51
$Z_{t(A-A)}$(mm^3)-Slab Top	-	1.04E+08	6.88E+07
$Z_{i(B-B)}$(mm^3)-Girder Top	1.48E+07	2.52E+08	1.08E+08
$Z_{b(C-C)}$(mm^3)-Girder Bottom	1.78E+07	2.48E+07	2.39E+07

4.0 Design of Shear Connector

Shear studs and transverse reinforcement for the composite girders are calculated as per cl.606 of IRC:22-2015.

4.1 ULS Capacity -

[cl.606.4.1 of IRC:22-2015]

	G1-Support				
	V	A_{ec}	Y	I_{xx}	$VA_{ec}Y/I_{xx}$
	t	m^2	m	m^4	t/m
SIDL	29.9	0.044	0.465	0.038	16.1
LL & FPLL	40.1	0.088	0.301	0.0442	24.0
	G1-Splice				
	V	A_{ec}	Y	I_{xx}	$VA_{ec}Y/I_{xx}$
	t	m^2	m	m^4	t/m
SIDL	27.8	0.044	0.43	0.0342	15.4
LL & FPLL	29.6	0.088	0.28	0.0397	18.2
	G1-Midspan				
	V	A_{ec}	Y	I_{xx}	$VA_{ec}Y/I_{xx}$
	t	m^2	m	m^4	t/m
SIDL	6.8	0.044	0.47	0.0378	3.7
LL & FPLL	27.9	0.088	0.30	0.0438	16.8

Max V_L	=	40.1	t/m
Heigh of the Stud	=	100.00	mm
Taking Stud dia	=	22.00	mm
Grade of concrete	=	40.00	Mpa
Ultimate strength	=	115.00	kN
No. of Stud	=	2	
S_{L1}, Spacing	=	573	mm

<u>For Full Shear Connection -</u> *[cl.606.4.1.1 of IRC:22-2015]*

Grade of Steel reinforcement in deck slab — Fe 500

Grade of concrete of deck slab and haunch — M 40

Longitudinal Reinf. In deck slab	ϕ 12	@ 100 c/c	@ Support location
	ϕ 12	@ 200 c/c	@ Midspan - at L/2 location symmetrically from both end
Transverse Reinf. In deck	ϕ 20	@ 125 c/c	

Transverse Reinforcement In haunch	ϕ 0	@ 0 c/c

	G1-Support	G1-Splice	G1-midspan	
Area of logitudinal reinf., A_{sl} (mm^2)	3392.9	1696.5	1696.5	
Eff. area of concrete, A_{ec} (mm^2)	660000	660000	660000	
f_y (MPa)	500.0	500.0	500.0	
f_{ck} (MPa)	40.0	40.0	40.0	
γ_m	1.25	1.25	1.25	*[Table 1: IRC:22-2015]*
H_1 (t)	135.7	67.9	67.9	
H_2(kN)	950.4	950.4	950.4	

Min H	=	135.7	t	at support	Min H	=	67.9	at splice/ midspan
Ultimate strength	=	11.5	t		Ultimate strength	=	11.5	
No. of Stud	=	2			No. of Stud	=	2	
Zero moment to Maximum moment		15000	mm					
S_{L2}, Spacing	=	169.5	mm	at support		=	338.9	at splice/ midspan

4.2 SLS Fatigue Capacity –

[cl.606.4.2 of IRC:22-2015]

	G1-Support					G1-Splice				
	V_R	A_{ec}	Y	I_{xx}	$VA_{ec}Y/I_{xx}$	V_R	A_{ec}	Y	I_{xx}	$VA_{ec}Y/I_{xx}$
	t	m^2	m	m^4	t	t	m^2	m	m^4	t
LL	22	0.088	0.301	0.0442	13.2	19.4	0.088	0.277	0.0397	11.92

Max V_r	=	13.2	t/m	
Assuming No. of Cycle	=	1E+07		
Ultimate strength	=	28	kN	*[Table 8 of IRC:22-2015]*
No. of Stud	=	2		
S_R, Spacing	=	425	mm	
Max Spacing allowed	=	355	mm	*[cl.606.9 of IRC:22-2015]*
Max Spacing allowed	=	400	mm	*[cl.606.9 of IRC:22-2015]*
Min Spacing allowed	=	75	mm	*[cl.606.9 of IRC:22-2015]*

Clear distance from edge of compression to c/l of nearest stud should not be greater than = *50* mm *[cl.606.9 of IRC:22-2015]*

Shear Stud Provided: φ22 of 100mm ht. @ 169mm c/c in 2 rows

4.3 Transverse Shear Check

[cl.606.10 of IRC:22-2015]

L		=	450	mm	
A_{st}		=	50.3	cm^2/m	(Assumed similar reinforcement in top and bottom layers)
V_L=	401 kN/m	<	1799	kN/m	*OK*
		<	5687	kN/m	*OK*
Min A_{st} Required		=	2.0	cm^2/m	OK

Main Girder Design-ULS

5.0 ULS Bending Capacity Check of Composite Girder

5.1 Check for External Girder (G1)

5.1.1 Section Compactness — *Cl.603.1.1 of IRC:22-2015*

	Mid Span	
For top flange,		
b/t_{ft}=	8.45	
	7.94	
	> 9.4ε	
As shear studs are connected, Top flange =	*Plastic*	*Cl.603.1.3.1 of IRC:22-2015*
For web,		
d_w/t_w =	106.50	
	106.49	
	> 126ε	
	Slender	
Section is	*Slender*	

5.1.2 Bending Moment Capacity: Ultimate Stage

	Mid Span	*Annex-1_Cl.I.1.1 & I.2 of IRC:22*
η =	1.00	
γ_m =	1.10	
γ_c =	1.50	
λ =	0.80	
α_{cc} =	0.67	
f_y =	350.00	MPa
f_{ck} =	40.00	MPa
α =	22.26	
A_s =	31336	mm^2
A_f=	7000	mm^2
b_{eff} =	3000	mm
b_f =	350	mm
d_s =	220	mm
CG of steel beam section from top of girder =	997.43	mm
d_c =	1107	mm
Plastic Neutral Axis Position =	*In Steel top Flange*	

Neutral Axis Depth, x_u=	222	mm	
Moment Capacity , M_p =	10648	kN-m	**Pass**
Ratio M/Mcap	**0.98**		

5.1.3 Lateral Torsional Buckling Resistance Moment Check for Steel Girder (Construction stage)

	Mid Span		*Cl.I.5 of IRC:22*
Elastic modulus, E =	200000	MPa	
Poisson's ratio, μ =	0.30		
Shear modulus, G =	76923	MPa	
Effective length for lateral torsional buckling, L_{LT} =	2500	mm	*Spacing of top plan bracing (temporary during construction)*
I_y =	2.24E+08	mm^4	
Torsional constant, I_t =	2.82E+06	mm^4	
β_f=	0.282		
h_y=	1720	mm	
Warping constant, I_w =	1.34E+14	mm^6	
k =	0.75	*Considered partially fixed*	
k_w=	1		
Average thickness of bottom flange, t_2	20		
α	0.72		
y_g=	-247	mm	
y_j=	-374	mm	
c_1 =	1.052	*Table C1 of IRC:24-2010*	
c_2 =	0.382	*Table C1 of IRC:24-2010*	
c_3 =	0.753	*Table C1 of IRC:24-2010*	
M_{cr}	31592	kN-m	
Z_e =	1.52E+07	mm^3	
Z_p =	1.91E+07	mm^3	
β_b=	0.79		
f_y =	350	MPa	

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} = 0.41$$

α_{LT}	=	0.49	*(for welded section)*
$\phi_{LT} = 0.5[1+\alpha_{LT}(\overline{\lambda}_{LT}-0.2)+\overline{\lambda}_{LT}^{\,2}]$	=	0.64	
$\chi_{LT} = \dfrac{1}{[\phi_{LT}+(\phi_{LT}^{\,2}-\overline{\lambda}_{LT}^{\,2})^{\frac{1}{2}}]}$	=	0.89	
Design buckling resistance, $M_{el\ (buckling)}$ =	$M_{el\ (buck)} = \chi_{LT}.M_{el}$	4309	kN-m**PASS**
	<u>**Ratio M/Mcap**</u>	<u>**0.90**</u>	

6.0 ULS Shear Capacity of Steel Girder

6.1 Check for External Girder (G1) *Cl.603.3.3.2 (1) of IRC:22-2015*

6.1.1Plastic Shear Resistance

Shear Area, A_v =	20400	mm^2
f_{yw} =	350	MPa
γ_{m0} =	1.10	
$V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}}$ =	4122	kN

6.1.2 Shear Buckling resistance: Simple post critical method *Cl.603.3.3.2.2 of IRC:22-2015*

$67\,\varepsilon_w\sqrt{\frac{K_v}{5.35}}$ 68.22 *Cl.509.4.2.1 of IRC:24-2010*

Resistance to shear buckling shall be verified

Spacing of transverse stiffeners, c =		1500	mm
d/t_w	=	141.67	
c/d	=	0.88	
k_v	=	10.87	
$\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(d/t_w\right)^2}$	=	97.92	MPa
$\lambda_w = \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})}$	=	1.44	
$\tau_b = f_{yw}/\left(\sqrt{3}\lambda_w^{\,2}\right)$	=	99	MPa
$V_n = V_{cr} = A_v \cdot \tau_b$	=	2023	kN

6.3 External Girder (G1)

V_d =	$V_n/\gamma_{m,0}$	1839	kN
Design Shear, V		1221	kN
Ratio V/Vd		**0.66**	

Reduce Moment Resistance

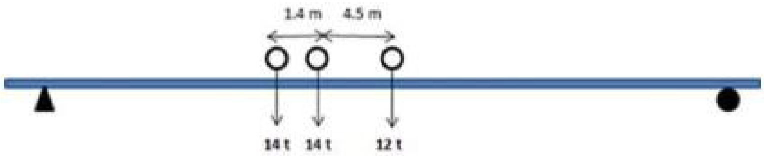
Cl.603.3.3.3 of IRC:22-2015

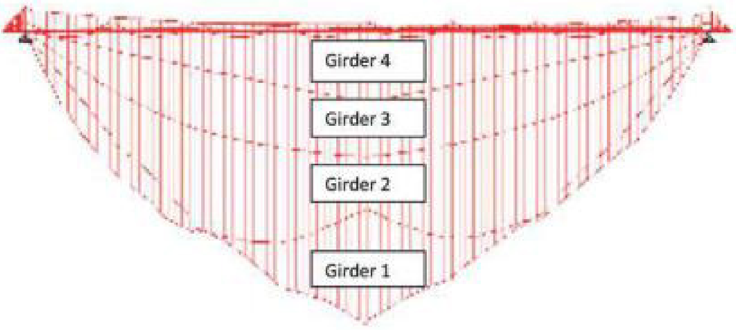
Since the span is simply supported. Therefore location of maximum shear and moment are non-coexistent. At the location of max. BM, the ratio V/Vd <0.6. Thus, no reduction of moment capacity is envisaged.

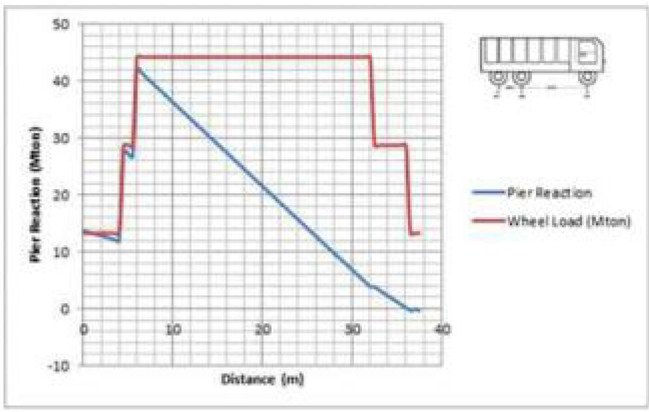
Summary:	At Service	During erection
Mcap (kNm)	10648	4309
Vcap (kN)	1839	

6 Limit State of Fatigue

For fatigue assessment, fatigue Load as given in Clause 204.6 of IRC:6-2014 is used. The 40T fatigue vehicle is moved over the girders and the envelope of bending moment for various girders are shown below :







Check for low fatigue

Fatigue assessment is not required if following conditions are satisfied *Cl.605.5 of IRC 22.2015*

i) $f < 27/\gamma_{mft}$

ii) $N_{SC} < 5\text{x}10^6 \left[\frac{27/\gamma_{mft}}{f\,\gamma_{fft}}\right]^3$

iii) $\tau_{fmax} \leq 67\mu_r/\gamma_{mft}$

f	=	51.86		
N_{sc}	=	2000000		*Cl. IRC:6-2017*
Y_{fft}	=	1		
Y_{mft}	=	1.25		*Table 3 of IRC:22.2015*
$M_{midspan}$	=	1.458E+09	N-mm	

Detailed fatigue assessment is required

Nominal fatigue stress

t_f	=	20	mm	
f_{fn}	=	92		*Table 5 of IRC:22.2015*

Assuming continuouslongitudinal fillet weld with no stop start and good maintenance and inspection to details

f_f	=	124.86	N/mm^2	*Cl.605.3 of IRC 22*
μ_r	=	1.06	>1	*Cl.605.2 of IRC 22*
$f_{fd} = \mu_r f_f/\gamma_{mft}$	=	99.89	N/mm^2	*Cl.605.4 of IRC 22*
Z_e	=	2.81E+07	mm^3	
M_d	=	2.808E+09	N-mm	
		Adequate	from fatigue consideration	

Shear fatigue stress

Maximum shear force	=	312.00	kN	
τ_{max}	=	15.29	N/mm^2	
τ_f	=	110.50	N/mm^2	*Cl.605.3 of IRC 22*
$\tau_{fd} = \mu_r \tau_f / \gamma_{mft}$	=	88.40	N/mm^2	*Cl.605.4 of IRC 22*
	Section provided is	**Safe**	from fatigue consideration	

7.0 Design of Stiffeners

7.1 Summary of stiffeners

Stiffeners	Width (mm)	Thickness (mm)
Intermediate	75	10
Bearing	100	10
	100	10

7.2 Intermediate web stiffener: Longitiudnal Girder G1

f_y	=	350		
c	=	1500	mm	
d	=	1700	mm	
ε				
	=	0.845		
k_V	=	10.87		
Minimum web thickness required, t_w	=	9	mm	*Cl.6.1.7.1*
c/d	=	0.88	<	1.41
I_S	≥	2.289E+06	mm^4	
Trying intermediate stiffener of 75 X 10 mm				
t	=	10	mm	
d	=	75	mm	
$I_{sprovided}$	=	3.542E+06	mm^4	**Hence OK**

7.2.1 Check for outstand

Outstand of stiffener	=	75	mm
ε missible limit, 14t_q	=	118.322	mm
		Pass	

7.2.2 Check for buckling

Stiffeners not subjected to external loads or moments should be checked for stiffener force: $F_q = V - V_{cr} / \gamma_{m0} \leq F_{qd}$ *Cl.6.6.7.2.5*

V < V_{cr} No force in stiffener

7.3 Load bearing stiffener: at bearing location *Cl.6.6.5*

End is panel is checked as a beam spanning between the two flanges

7.3.1 Check for shear capacity of end panel

Under pure shear,$V_n=V_p$ = 4122.28 kN *Cl.6.6.5.3*

V_{cr} = 2023 kN

H_q $= 1.25\, V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$

= 3677.10 kN

R_{tf} = 1838.55 kN

V_d = 3306.64 kN

Pass *End panel can carry the shear force from anchor of tension field forces*

7.3.2 Check for moment capacity of end panel

Assumed that end post consist of single stiffener

$M_{tf} = H_q.d/10$ = 625.11 kN-m

y = 750 mm

I = 3.375E+09 mm^4

Mq = 1431.82 kN-m

Pass *End panel can carry moment from anchor of tension field forces*

Force F_m due to M_{tf} = 416.74 kN

Total compressive at bearing = 1637.67 kN

7.3.3 Bearing check

Area of stiffener, Aq should be > 4117.56 mm^2 *Cl.6.6.7.5.2*

Using	Count	B (mm)	t (mm)
	4	100	10
	2	100	10

600 mm

Area = 0 2

7.3.4 Check for outstand

Count	B (mm)	t (mm)	$14t_q$	
4	100	10	118.32	**Pass**
2	100	10	118.32	**Pass**

7.3.5 Buckling Check

Area of Effective section	=	11760		*Cl.6.6.7.1.5*
MOI of effective section	=	2.387E+07	mm^4	
Radius of gyration, r_x	=	45.05	mm	
L_e	=	1190	mm	*Table 6.3*
(Flange is restrained against rotation and lateral deflection)				
lambda	=	26.42		
Euler's buckling stress, f_{cc}	=	2829	N/mm^2	
Effective slenderness ratio, λ	=	0.35		
Imperfection factor, α	=	0.49		
$\phi = 0.5*[1+\alpha(\lambda-0.2)+\lambda^2]$	=	0.60		
f_{cd}	=	293.5	N/mm^2	
(for buckling curve "c" & f_y = 350 N/mm^2)				
Buckling resistance	=	3452.06	kN	
	Section	**Pass**	in buckling	

7.3.6 Check stiffener as load bearing stiffener

			Cl.6.6.7.4
b1	=	0	*(Assumed for simplicity, conservative approach)*
n2	=	50	mm
tw	=	12	mm
fyw	=	350	N/mm^2
γ_{m0}	=	1.10	
$F_w = (b_1 + n_2)t_w \frac{f_{yw}}{\gamma_{m0}}$	=	190.91	kN
Designed force for stiffener	=	1446.76	kN
Bearing capacity of stiffener	=	1909.09	kN
		Pass	

8.0 Serviceability limit state- design checks

Stress Limitation

(Stresses in N/mm^2 and Moment in KN.m)

Stress	Self wt	Deck slab	SIDL	SURF.	CWLL+FPLL	Wind	Diff. Shrink	Positive Temp. Gradient	Negative Temp. Grad	Σ	Stress ratio
M	439.8	2292.3	569	786	2971	230	-				
σt Slab			0.57	0.79	3.62	0.28	-0.12	3.25	-1.04	8.40	0.44
σb slab			0.35	0.49	1.63	0.13	0.59	-9.19	3.70	6.89	0.36
σtgirder	28.97	150.99	5.28	7.30	12.25	0.95	8.90	-9.19	3.70	218.34	0.70
σb girde	-21.57	-112.41	-20.93	-28.92	-105.67	-8.17	-2.40	6.34	-8.47	-308.55	0.95

Stress	Unit	Self wt	Deck slab	SIDL	SURF.	CWLL+FPLL	Wind	Diff. Shrink	Σ
V	kN	4	0	37	8.4	186	2.4	-	
$\sigma_{b\ girder}$	N/mm^2	0.20	0.00	1.81	0.41	9.12	0.12	-	11.66

Stresses in N/mm2	f_{bc}	f_{bt}	l_b	f_{ec}	f_{et}	*Cl. 604.3.1 of IRC 22*
Girder Top	218.34	-9.19	11.66	219.28	22.19	**Safe**
Girder Bottom	8.47	-300.07	11.66	21.90	300.75	

Limiting Stress (Cl. 604.3.1 of IRC 22-2015)		
	Compressive N/mm^2	Tensile N/mm^2
Slab/Reinf.	19.2	400
Girder	315	315

8.1 Deflection Checks/Precamber Requirement

Allowable deflection for total load = 50 mm
Allowable deflection for live load and impact = 37.5 mm

S.No	Component	Deflection mm
1	SW Girder	10.31
2	Deck slab weight	64.47
3	SIDL	6.4
4	Surfacing	5.6
5	CWLL	16.6
6	FPLL	2

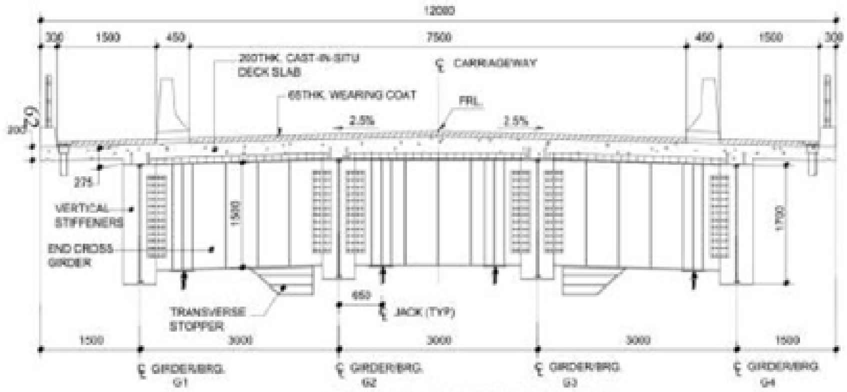
Deflection due to total loads = 105.382
3 mm **Pre-Camber is required**
Deflection due to live load and impact = 18.6 mm **Live load deflection is ok**

Pre-camber calculation

S.No	Component	Deflection mm
1	SW Girder	10.3
2	Deck slab weight	64.5
3	SIDL	6.4
4	Surfacing	5.6

Pre-camber to be provided = 86.7
8 mm

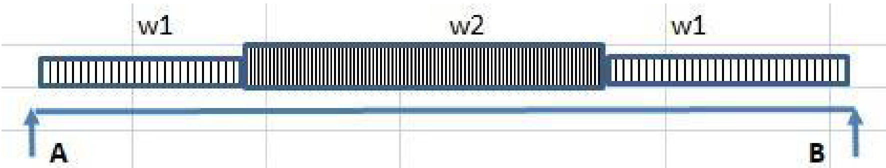
9.0 EFFECTS DUE TO SELF WEIGHT OF GIRDER AND DECK SLAB



TYPICAL CROSS SECTION

C/S Area of cross girder =	0.034	m^2
No. of Cross Girder per span =	3	
Additional UDL on girder due to cross girder =	0.40	kN/m
UDL of steel girder midspan section (w2)	2.86	kN/m
UDL of steel girder support section (w1)	2.86	kN/m
Unit weight of green concrete on ext girder (G1)= w3	18.88	kN/m
Weight of 40mm thk. permanent formwork on G1 =w4	1.5	kN/m

9.1A BM/SF Calcs due to self weight of steel girder + cross girder:



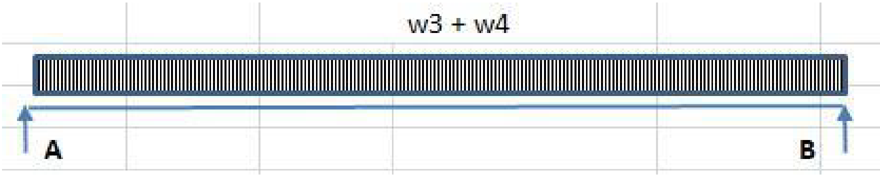
Length of doubler plate =	0	**m**
Effective Span =	30	**m**

Max SF =	58.64	kN	Including additional weight for stiffeners/connections/bracings =	20%
Max BM =	439.80	kNm		

9.1B Deflection due to self weight of steel girder + cross girder:

Weighted Average UDL =	3.26	kN/m
Deflection =	10.31	mm

9.1C BM/SF Calcs due to self weight of green concrete and permanent formwork:



Effective Span =	30	**m**
Max SF =	305.64	kN
Max BM =	2292.30	kNm
Deflection=	64.47	mm

9.2 Force due to Uniform Temperature Difference (TU) -

NOTE: +ve Stress is tension
-ve moment is sagging

Uniform temperature difference = ±35 ^{o}C

The structure being simply supported, uniform temperature will not have any effects on design of the superstructure.

9.3 Force due to Temperature Gradient (TG) -

9.3.1 For External Girder (G1)

9.3.1.1 Girder Inputs

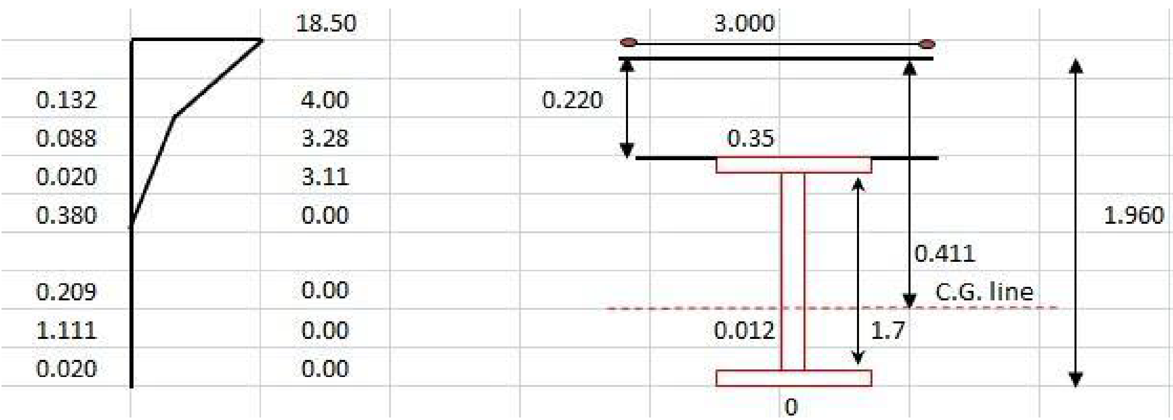
m= 7.5

Deck Slab Width		=	3.00	m
Deck Slab Thickness		=	0.22	m
Total Height of the section	h	=	0.22	m
C.G. from top	Yt	=	0.41	m
C.G. from bottom	Yb	=	1.55	m
M.O.I. of the Section	I	=	0.04	m^4
Area of the Section	A	=	0.12	m^2
Modulus of Elasticity	Es	=	2.00E+08	kN/m^2
Modulus of Elasticity of Concrete	Ec	=	1.65E+07	kN/m^2
Coefficient of thermal expansion of steel	α	=	1.20E-05	/^{o}C
Section Modulus at the top of slab	Z_S	=	0.11	m^3
Section Modulus at the bottom of slab	Z_T	=	0.23	m^3
Section Modulus at the bottom of girder	Z_B	=	0.03	m^3

9.3.1.2 Stress due to Rise in Temperature

Cl. 215.3 , IRC-6:2010

h	T^{o}C				
0.2	18.0	T_1	18.50	h_1	0.13
0.3	20.5	T_2	4.0	$h-h_1$	0.09

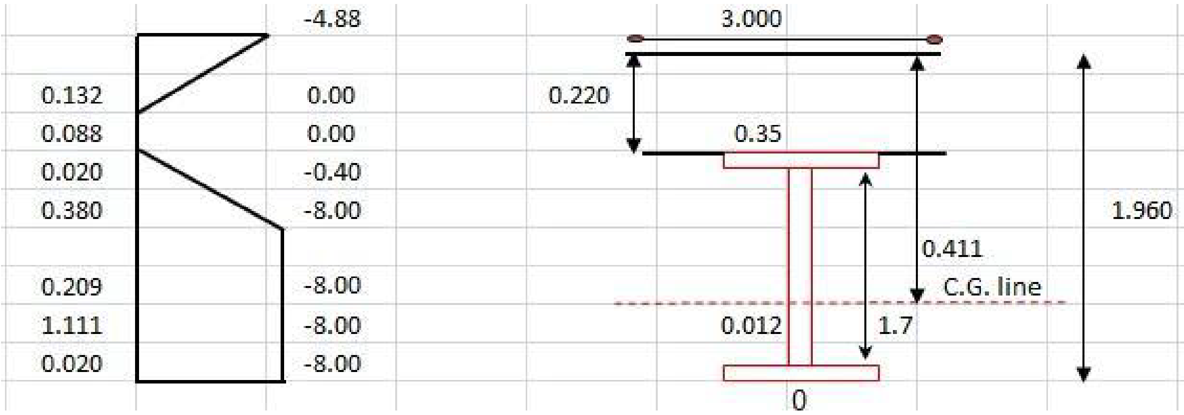


Sl No	b (m)	d (m)	e from top (m)	Area (m^2)	T_{top} (Degree)	T_{bot} (Degree)	T_{Avg} (Degree)	stress by Assuming End Restraint E_s*□*T (kN/m^2)
1	0.400	0.132	0.052	0.05	18.50	4.00	11.3	27000.0
2	0.400	0.088	0.043	0.04	4.00	3.28	3.6	8734.4
3	0.350	0.020	0.010	0.01	3.28	3.11	3.2	7672.1
4	0.012	0.380	0.127	0.00	3.11	0.00	1.6	3737.7
5	0.012	0.209	0.105	0.00	0.00	0.00	0.0	0.0
6	0.012	1.111	0.555	0.01	0.00	0.00	0.0	0.0
7	0.000	0.020	0.010	0.00	0.00	0.00	0.0	0.0

Sl No	Restrained Force (kN)	Distance of CG of force from CG of Girder, e (m)	Restrained moment M_T(kNM)	Stress due to axial force Release P/A(kN/m_2)	Stress due to BM Release M/Z(kN/m_2)	Final Stress (MPa)
1	1425.6	0.4	511.4	-14500.0	-5523.0	3.3
2	307.5	0.2	72.6			
3	53.7	0.2	9.7	-14500.0	-2563.6	-9.2
4	17.0	0.0	0.7			
5	0.0	-0.3	0.0			
6	0.0	-1.0	0.0			
7	0.0	-1.5	0.0	-14500.0	20842.3	6.3
	1803.8		594.5			

9.3.1.3 **Stress due to Fall in Temperature** *Cl. 215.3 , IRC-6:2010*

h	**T^{o}C**				
0.2	4.4	T_1	-4.88	h_1	0.132
0.3	6.8	T_2	-8.0	$h-h_1$	0.088



Sl No	**b (m)**	**d (m)**	**e from top (m)**	**Area (m2)**	**Ttop (Degree)**	**Tbot (Degr ee)**	**TAvg (Degr ee)**	**stress by Assuming End Restraint Es*α*T (kN/m2)**
1	0.400	0.132	0.044	0.05	-4.88	0.00	-2.4	-5856.0
2	0.400	0.088	0.044	0.04	0.00	0.00	0.0	0.0
3	0.350	0.020	0.013	0.01	0.00	-0.40	-0.2	-480.0
4	0.012	0.380	0.247	0.00	-0.40	-8.00	-4.2	-10080.0
5	0.012	0.209	0.105	0.00	-8.00	-8.00	-8.0	-19200.0
6	0.012	1.111	0.555	0.01	-8.00	-8.00	-8.0	-19200.0
7	0.000	0.020	0.010	0.00	-8.00	-8.00	-8.0	-19200.0

	Sl No	Restrained Force(kN)	Distance of CG of force from CG of Girder, e (m)	Restrained moment MT (kNM)	Stress due to axial force Release P/A(kN/m2)	Stress due to BM Release M/Z(kN/m2)	Final Stress (MPa)
	1	-309.2	0.4	-113.3	5326.8	-1430.8	-1.0
	2	0.0	0.2	0.0			
	3	-3.4	0.2	-0.6	5326.8	-664.1	3.7
	4	-46.0	-0.1	3.5			
	5	-48.3	-0.3	15.2			
	6	-255.9	-1.0	249.3			
	7	0.0	-1.5	0.0	5326.8	5399.4	-8.5
		-662.6		154.0			

9.4 Differential Shrinkage Strain (SC) -

NOTE: +ve Stress is tension
-ve moment is sagging

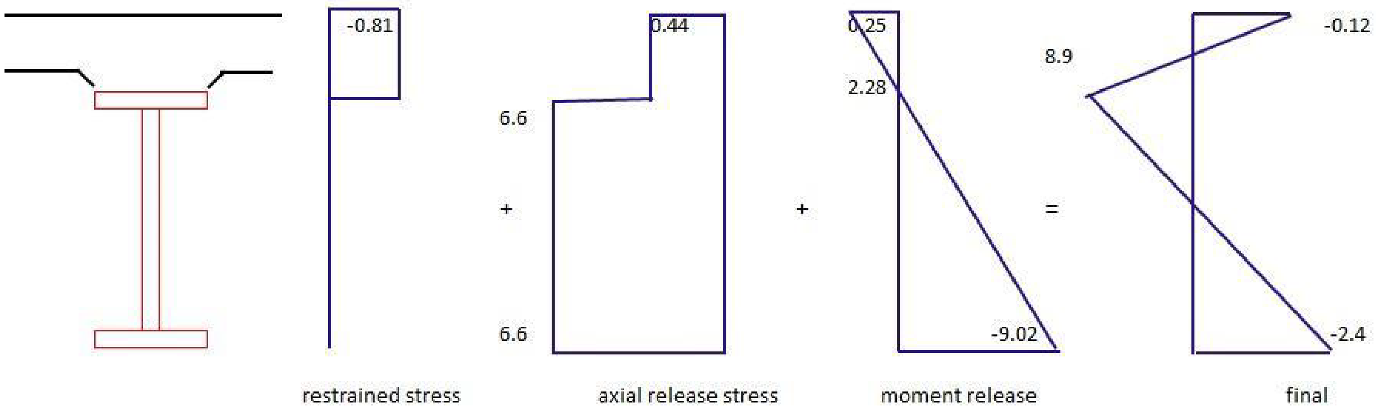
9.4.1.1 External Girder Inputs at Mid Span

m = 15

Total Height of the section (slab thk)	h	=	0.22	m	
Differential shrinkage strain	ε_{DS}	=	**0.00006**		*cl.6.4.2.6- IRC 112*
Grade of Concrete		=	M40		
Aslab =c/s area Ac		=	0.044	m^2	
Area of section in terms of steel		=	0.08	m^2	
Modulus of Elasticity of Concrete (for long term)	Ec	=	1.65E+07	kN/m^2	
Modulus of Elasticity of Steel	Es	=	2.00E+08	kN/m^2	
Creep Factor	φ	=	**1.10**		*Assumed conservatively*
C.G from top	Ycg	=	0.57	m	
Eccentricity	e_{cc}	=	0.46	m	
Section Modulus at the top	Z_S	=	0.07	m^3	
Section Modulus at the bottom	Z_T	=	0.11	m^3	
Section Modulus at the bottom	Z_B	=	0.03	m^3	

9.4.1.2 Design forces due to Diff. Shrinkage Strain at Mid Span

Restrained stresses (in terms of steel)		ε_{DS} x Es x f	=	12100.0	kN/m^2
Restrained force	P =	ε_{DS} x Ec x A_{slab} x f	=	532.4	kN
Restrained moment	M =	P x ecc	=	245.2	kN-m
Force release stress	Pre,s		=	0.44	MPa
Force release stress	Pre,t		=	6.62	MPa
Force release stress	Pre,b		=	6.62	MPa
Moment release stress	Mre,s		=	0.25	MPa
Moment release stress	Mre,t		=	2.28	MPa
Moment release stress	Mre,b		=	-9.02	MPa



9.4.1.3 Check for combine stresses at Mid Span

Top Deck	=	-0.8	+	0.4	+	0.2	=	-0.1	MPa
Top Girder	=	0.0	+	6.6	+	2.3	=	8.9	MPa
Bottom of Girder	=	0.0	+	6.6	+	-9.0	=	-2.4	MPa

9.5 Analysis Result

9.5.1 Unfactored primary forces from the analysis

			Mid- span	Support
Self Wt		BM (kN-m)	439.80	0
		SF(kN)	4	58.64
Deck Slab		BM (kN-m)	2292.30	0
		SF(kN)	0	305.64
SIDL		BM (kN-m)	569	0
		SF(kN)	37	112
Surfacing		BM (kN-m)	655	0
		SF(kN)	7	81
CWLL	**Hog**	BM (kN-m)	-38	-39
	Sag	BM (kN-m)	2775	0
		SF(kN)	167	217
FPLL		BM (kN-m)	196	0
		SF(kN)	19	45
Wind with LL		BM(kN-m)	383	0
		SF(kN)	4	48
Wind w/o LL		BM(kN-m)	461	1
		SF(kN)	5	57
Construction Load		BM(kN-m)	504	-2
		SF(kN)	0	68

Note : *Other than the results for self weight of steel elements and deck slab, all other results are considered identical to the IRC:SP:120-2018 worked out example*

9.5.2 Ultimate Limit State

Factor for critical load case

Loads	Basic					Cons. Stage	
		Mid Span		Support		Mid Span	
	Factor	*BM (kN-m)*	*SF (kN)*	*BM (kN-m)*	*SF (kN)*	*Factor*	*BM (kN-m)*
Self Weight	1.35	593.7	5.4	0.0	79.2	1.05	461.8
Deck Slab	1.35	3094.6	0.0	0.0	412.6	1.05	2406.9
SIDL	1.35	768.2	50.0	0.0	151.2	0.00	0.0
Surfacing	1.75	1146.3	12.3	0.0	141.8	0.00	0.0
CWLL+FPLL	1.50	4456.5	279.0	0.0	393.0	0.00	0.0
Wind With LL	0.90	344.7	3.6	0.0	43.2	0.00	0.0
Wind w/o LL	0.00	0.0	0.0	0.0	0.0	1.05	484.1
Temp Differences	0.00	0.0	0.0	0.0	0.0	0.00	0.0

Construction Load	0.00	0.0	0.0	0.0	0.0	1.05	529.2
Σ		10403.9	350.2	0.0	1220.9		3882.0

9.5.3 Serviceability limit state- design checks

Loads	**At midspan**		
	Factor	***BM** (kN-m)*	***SF (kN)***
Self Weight	1	439.8	4.0
Deck Slab	1	2292.3	0.0
SIDL	1	569.0	37.0
Surfacing	1.2	786.0	8.4
CWLL+FPLL	1	2971.0	186.0
Wind With LL	0.6	229.8	2.4
Wind w/o LL	0	0.0	0.0
Temp Difference	0	0.0	0.0
Σ		7287.9	237.8

9.0 Comparison with Design with Carbon Steel

The make-up of midspan section for design with E 350 grade Carbon Steel is compared with the same for design with two grades of Ferritic grades of Stainless Steel, namely IRS 350 CR and IRS 450 CR.

Grade of Steel	**E 350 (IS 2062)**	**IRS 350 CR**	**IRS 450 CR**
Yield Stress	350 MPa	350 MPa	450 MPa
Web	1700 x 12	1700 x 12	1600 x 12
Top Flange	500 x 20	350 x 20	300 x 20
Bottom Flange	500 x 20 & 450 x 20	450 x 20	350 x 20
Cross-sectional Area (mm^2)	49400	36400	32200
Spacing of Top Lateral Bracing (mm)	4280	2500	2500
Approx Tonnage of the Span	71	59	54

12. LIFE CYCLE COSTING

A bridge represents a long-term, multi- year investment. Following its planning, design, and construction, a bridge requires periodic maintenance and possibly repair or rehabilitation actions to ensure its continued function and safety. It is logical to consider the whole life cost instead of the initial cost to evaluate a particular bridge option. Life cycle costing (LCC) presents a rational method for carrying out the same.

Eventually the owner has to decide that a bridge must be replaced, effectively designating the end of its useful life. This end typically comes decades and sometimes even centuries after the initial construction was completed. In simplest terms, the time between a bridge's construction and its replacement or removal from service is its service life. The sequence of actions and events and their outcomes–e.g., construction, usage, aging, damage, repair, renewal–that lead to the end of the service life and the condition of the bridge during its life compose the life cycle. Owners must make decisions about what management strategy to follow, what materials and designs to use, what repairs to make and when they should be made, based on their expectations about what the subsequent costs and outcomes will be. LCC is a set of economic principles and computational procedures for comparing initial and future costs to arrive at the most economical strategy for ensuring that a bridge will provide the services for which it was intended

LCC is essentially a technique for considering the economic efficiency of expenditures. Given a certain set of requirements that a bridge must meet–e.g., traffic volumes to be carried, maximum vehicle loads, geotechnical and climate conditions – the lowest-cost set of actions meeting those requirements is preferable to other sets of actions. The bridge resulting from those actions represents a more efficient use of scarce resources – i.e., public funds and time– than other alternatives. It is this consideration of all resources used to produce the bridge's services that distinguishes LCC from discounted cash-flow analysis, a computationally similar technique used by financial analysts to compare streams of revenue and expenditure.

LCC is a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment. US NCHRP Report 483 provides some guideline to work out LCC of a project. It is users' discretion to consider the costs involved with activities (e.g. dismantling at the end of service life) along with the initial and periodic maintenance cost.

A simplified calculation is presented with the worked out example with Stainless Steel. It is compared with a design with Carbon Steel presented in IRC:SP:120. The rates considered are the prevailing rates during the first quarter of 2023.

30m Two lane Steel-Concrete Composite Superstructure with IRS 350 CR grade Stainless Steel, the worked out example in this document.

The basic quantities are:

Steel (IRS 350 CR Grade)	= 59 MT
M40 Grade Concrete	= 135 CuM
S.S Reinforcement Bars	= 16.5 MT

The initial cost of Construction

Sl no	Item	Unit	Quantity	Rate	Amount
1	Structural Steel	MT	59	2,42,500	1,43,07,500
2	Concrete	CuM	135	7,800	10,53,000
3	Reinforcement	MT	16.5	1,25,000	20,62,500
	TOTAL				1,74,23,000
				==	174.23 lakh

Yearly O&M cost is considered as 1% of construction cost increasing at 3% per annum

Thorough Inspection and Minor repair is considered every 10 years at 3% of construction cost escalated at 3% per annum.

Discounting Rate for Future Expenses = 10%

The 50 years and 100 years NPV works out to 202.52 and 203.57 lakh respectively

30m Two lane Steel-Concrete Composite Superstructure with IS:2062 E-350 BO grade Carbon Steel, the worked out example in IRC SP:120.

The basic quantities are:

Steel (E 350 BO Grade) = 71 MT

M40 Grade Concrete = 135 CuM

Reinforcement Bars (1786) = 16.5 MT

Considering the bridge is located in a coastal region, coating system type 4 (as per Annexe D of IRC:24 is envisaged. After 15 years, it is considered that two coats of Aluminium Paint will be applied every 5 years.

The initial cost of Construction

Sl no	Item	Unit	Quantity	Rate	Amount
1	Structural Steel	MT	71	2,08,000	1,47,68,000
2	Concrete	CuM	135	7,800	10,53,000
3	Reinforcement	MT	16.5	89,000	14,68,500
4	Painting	MT	71	9,500	6,74,500
	TOTAL				1,79,64,000
				==	179.64 lakh

Yearly O&M cost is considered as 1% of construction cost increasing at 3% per annum

Thorough Inspection and repair is considered every 5 years at 5% of construction cost escalated at 3% per annum.

Painting is considered every 5 Years starting from year 15. Painting rate at 00 year is considered as Rs 5,500 per MT escalating at 8% per annum.

Discounting Rate for Future Expenses = 10%

The 50 years and 100 years NPV works out to 240.45 and 251.18 lakh respectively.

The calculations are done for the superstructure only as the intent of this exercise is to examine the difference between carbon steel and stainless steel. The results are summarized in the following table.

Steel Grade	Tonnage	Initial Cost (Rs Lakh)	50 years NPV	100 years NPV	50 years NPV to Initial Cost	100 years NPV to Initial Cost
SS 350 CR	59	174	203	204	1.16	1.17
SS 450 CR	54	172	199	201	1.16	1.17
MS E350	71	180	240	251	1.34	1.40

It can be observed that after 100 years of service, the O&M expenses for the Carbon Steel bridge works out to 40% of the construction cost at present prices. It is also expected that it will be the end of service life and a replacement have to be procured.

On the other hand for a Stainless Steel bridge the same O&M expense works out to only about 17% of the construction cost. The bridge will remain operational for another twenty to fifty years. At the end of its service 90 to 95% of the Stainless Steel will be recycled.

This exercise demonstrates the economic advantage of adopting stainless steel if one considers the whole life of the structure.

With the present concern for environment primarily Carbon Emission through the life cycle of any constructed facility, Life Cycle Analysis (LCA) is being considered as more appropriate while evaluating different materials of construction, design, construction process. LCA takes three factors:

- Economic
- Environmental
- Social

LCA can be simple to extremely complex. ISO 14040 series provides basic guidelines. Essentially in LCA of a bridge or any project, the economic, environmental as well as social impact of the following

- Extraction of the relevant raw materials, e.g. quarrying, mining
- Refinement and conversion to process materials, e.g. steelmaking or cement production
- Manufacturing and packaging processes, e.g. steelwork fabrication or making precast concrete products

- Transportation and distribution between each stage
- Waste at each stage
- On-site construction impacts, e.g. water and energy use, temporary works, shuttering, worker commuting, etc
- Operation during the lifetime including maintenance, refurbishment, replacement, etc.
- At the end of its useful life, demolition, final transportation, waste treatment and disposal.Any recycling or recovery operations built into the life cycle should lead to a proportionate reduction in the adverse environmental impact and should be accounted for.

Structural Steel because of its strength, durability and recyclability stands out a construction material for sustainable construction. Carbon Steel is however susceptible to corrosion. Stainless Steel being corrosion resistant ensures long service life of bridges and a suitable material for sustainable construction.

12. REFERENCES

1) IRC:24-2010 : Standard Specifications and Code of Practice for Road Bridges Section V: STEEL ROAD BRIDGES (LIMIT STATE METHOD)
2) IRC:22-2015 : Standard Specifications and Code of Practice for Road Bridges Section VI: Composite Construction (Limit States Design)
3) IS 800:2007 : Indian Standard: GENERAL CONSTRUCTION IN STEEL — CODE OF PRACTICE
4) EN 1993-1-4-2006 : Eurocode 3 - Design of steel structures - Part 1-4: General rules - Supplementary rules for stainless steels
5) IS 6911 : 2017 : Indian Standard: STAINLESS STEEL PLATE, SHEET AND STRIP SPECIFICATION
6) EN 10088 Stainless steels - Part 1-4