IRC:SP:120-2018

EXPLANATORY HANDBOOK TO IRC:22-2015

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES, SECTION VI–COMPOSITE CONSTRUCTION

INDIAN ROADS CONGRESS
2018

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Published by:

INDIAN ROADS CONGRESS

Kama Koti Marg,
Sector-6, R.K. Puram,
New Delhi-110 022

NOVEMBER, 2018

Price : ₹ 1000/-
(Plus Packing & Postage)

First Published : November, 2018

Printed at India Offset Press, Delhi - 110 064
500 Copies

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EXPLANATORY HANDBOOK TO IRC:22-2015

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BACKGROUND AND OVERVIEW OF THE CODE

The principal aim of this explanatory handbook is to provide the users with guidance on the interpretation of various clauses of IRC: 22-2015 and to present one worked-out example. The example covers, topics that are in line with the codal clauses which are likely to be encountered in a typical bridge designs.

The work of preparing the explanatory handbook document on IRC:22 was entrusted to Shri Arijit Guha and Team of M/s Institute for Steel Development & Growth (INSDAG). The draft prepared by the Consultant was discussed by the Steel and Composite Structures Committee (B-5), IRC in its several Meetings.

The B-5 Committee discussed and approved the document in its meeting held on 16th March, 2018 for placing it in the Bridges Specifications and Standards (BSS) Committee. The BSS Committee approved the document in its meeting held on 25th April, 2018. Subsequently, the Executive Committee approved the document on 3rd May, 2018 for placing it before the IRC Council. Finally, the document was considered by the IRC Council in its meeting held on 4th May, 2018 during the 215th Mid-term Council Meeting of IRC held at Aizawl (Mizoram) and was approved for publication.

The Steel and Composite Structures Committee (B-5) of the Indian Roads congress was reconstituted in 2018 with the following personnel:

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IRC thanks all the committee members of the B-5 Committee for their support in bringing out this handbook.

INTRODUCTION

CHAPTER 1
GENERAL OVERVIEW ON IRC:22-2015

C.1.0 General Overview

C.1.1 Introduction

The existing IRC:22, which was published in 1986 by the Indian Roads Congress, is the document which gives the standard specifications and code of practices for design of road bridges so far as composite construction is concerned. The design methodology as have been in practice when this code was last published up to recently, mainly suggested the use of Working Stress or Allowable Stress Method of design as far as steel construction is concerned.

It is a well-known fact that, design of concrete structures in India had started to use the state-of-the-art Limit States Method since 1978. The usefulness of Limit states method as far as rationality of design as well as overall economy is concerned, has not been taken advantage of, in design of steel structure both in general steel construction as well as bridge construction, till recently. It is worthwhile to mention, that as far as international scenario is concerned, the Limit States Method have been in use for steel design in most of the developed and developing countries of the world since the 1970's. The basic code of design of steel structure for general construction in India, which is published by the Bureau of Indian Standards (i.e., IS 800) had been using the Allowable Stress Design (ASD) method till the year 2008, until it was modified to the Limit States Method (LSM) of design and was subsequently published in that year. Likewise most of the codes pertaining to bridge designs were practicing the Allowable Stress Design method in line with the earlier version of IS 800, which was published in 1984.

While IS:800 (whose previous edition was published in 1984 in ASD) was being modified to the recently published version (i.e. IS:800 – 2007) of LSM design, parallel efforts were being taken up under the purview of the Indian Roads Congress, to modify various codes like IRC:22, IRC:24, etc. to this latest design technology. IRC:24 deals with design of steel bridges, while IRC:22 provides specification of design of composite construction. Finally IRC:22 and IRC:24 have been modified by incorporating Limit States Method and have been published by the Indian Roads Congress recently. Also various other codes are being modified simultaneously to cater to the changes that have taken place over the years to make the design of bridges in India at par with the state-of-the-art designs practiced internationally.

This explanatory handbook mainly provides explanations of various provisions as laid down in the revised IRC:22-2015 for easy understanding of the practicing design engineers and also provides design example to help the engineer in designing steel-concrete composite road bridges based on the revised LSM draft of IRC:22.

The loadings, load combinations and various safety factors have been based on IRC:6. The nature of load for which the bridge is designed has been considered as per recommendations of MoRTH (Ministry of Road transport & Highways).

C.1.2 Steel in Bridges – General Overview

History of mankind suggests that human communities have always felt the necessity of development of infrastructure since long time back in order to aid itself, in its endeavour to move for its own development. The primary and most important part of infrastructure has been roadways, and one of the most important component of roadways are ‘bridges’, which provides link across hindrances like rivers, small water bodies, cross roads etc.

General affinity of Bridge engineers in India has long been to go for steel bridges. The Howrah Bridge (Rabindra Setu) over river Hooghly is testament to the capabilities of Indian engineers. Other bridges, which tell the story of the competence level of Indian engineers, include Road cum Rail Bridge over river Ganga near Mokameh in Bihar, road-cum-rail Bridge across river Brahmaputra in Guwahati and numerous others.

C.1.3 Advantage of Steel as Construction Material

While studying the advantages of steel bridges over other materials of constructions like mainly the concrete ones (both RCC and PSC), the two main aspects to be kept in mind as usual are strength and economy. The lesser depth of steel girder in bridge means lower finished height of the working level. This will lead to lesser length and height of the approach way, leading to significant savings in overall cost of the entire system. Moreover steel usually has a higher strength-to-weight ratio leading to lesser dead to live load ratio especially important in longer span lengths. A list of advantages of steel intensive bridges has been given below:

1) As discussed above steel intensive bridges have lesser dead load compared to concrete bridges, thus requiring smaller foundations (fewer piles) and having lesser seismic demand leading to faster construction and enhanced performance during earthquakes, especially longer spans.
2) Steel bridges are constructed with extensive prefabrication leading to higher quality control and hence better long-term performance.
3) Steel intensive construction in general reduces site operations that lead to better and integrated planning between factory and site operations and reduced space requirements at site causing faster construction and lesser imposition on the site environment.
4) Use of high strength steel in bridges further reduces dead load, depth of structure and eases transportation and erection causing lesser overall cost of facilities
5) Structural systems for continuous steel structures like continuous girders are easier to accomplish. This will reduce the number of bearings and increase durability.

6) With the advent of modern durable corrosion resisting paints, the painting cycle is fixed or predictable which helps in better planning and resource allocation.

7) Finally and not the least important is that, elements of a steel intensive system are re-usable and can be easily dismantled and steel re-use leading to lesser wastage.

C.1.4 Steel – Concrete Composite Construction

The global resurgence of steel in construction both in bridges and in other structures has in no way lessened the importance of concrete as a construction material. It has been widely acknowledged by civil engineering experts across the world that appropriate combinations of the steel and concrete, utilizing the distinctly different merits of the two materials, is the best possible solution for a structural problem. As a result bridge engineering mostly in road bridges and flyovers, is moving towards more extensive use of steel-concrete composite structural systems utilizing the strength of the two materials synergistically, especially in urban areas, as for grade separators in roads. Although the concept, which utilizes the high tensile strength to weight of steel and compressive strength and cost of concrete has been extensively used in many parts of the world, it is being used only recently in India, especially in the urban roadway bridges.

In conventional construction, concrete slabs rest over steel beams and are supported by them. Under load these two components act independently and a relative slip occurs at the interface if there is no connection between them. With the help of a deliberate and appropriate connection provided between the beam and the concrete slab, the slip between them can be eliminated. In this case the steel beam and the slab act as a ***“composite beam”*** and their action is similar to that of a monolithic Tee beam. Though steel and concrete are the most commonly used materials for composite beams, other materials such as pre-stressed concrete and timber can also be used.

Various sectional configurations including I-sections and box sections have been found to be suitable for various types of Bridge spans. In urban areas, for flyovers acting as grade separators, under various considerations it is found that box girders are more appropriate vis-à-vis I-girders for the obligatory span(s) and the reverse is true for the shorter approach spans. Live examples for this are flyovers at Mayapuri and Andrewsganj intersections along the Ring Road in Delhi, Bridges on Metro Rail line at Delhi and flyovers in Kolkata under the Kolkata Urban Development Projects.

C.1.5 Advantage of Steel-Concrete Composite Construction

There are many advantages associated with steel concrete composite construction. Some of these are listed below:

1) The most synergetic utilisation of steel and concrete is achieved.

2) Keeping the span and loading unaltered; a more economical steel section (in terms of depth and weight) is adequate in composite

construction compared with conventional non-composite construction.

3) As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom, reduced approach span lengths.

4) Because of its larger stiffness, composite beams have less deflection than steel beams acting alone.

5) Composite construction provides efficient arrangement to cover large column free space.

6) Composite construction is amenable to “fast-track” construction because of using rolled steel and pre-fabricated components, rather than cast-in-situ concrete.

7) Steel beam sections encased in concrete have improved fire resistance and corrosion a lot and are very important in Bridges.

C.1.6 Design Approach for Steel – Concrete Composite Girders

Steel-concrete composite girders has been widely adopted for moderately longer spans in urban flyovers, such as the mandatory or obligatory spans in the form of box girders and other approach spans in the form of plated I–girders, as discussed earlier. Though there is no dedicated code of practice for design of steel-concrete composite box girder in India, it is possible to design the same using the provisions of the Indian composite codes and theory of the behaviour of box girders. It is relevant to note here that appropriate codes/design guidelines for steel-concrete composite box girders are under preparation by the Indian Roads Congress.

The central spans of these flyovers or bridges may be designed as simply supported, continuous or continuous under live load. Component lengths for shop fabrication should be the maximum possible, consistent with the delivery and site restrictions to reduce the amount of on-site assembly. The available length of the plates/rolled sections normally defines the maximum length for road delivery without restrictions, although longer lengths can be transported by special arrangements.

A minimum number of shop butt welds should be used consistent with plate sizes available. The decision whether to introduce thickness changes within a fabricated length should take into account the cost of butt welds compared with the potential for cost of material saving. Design procedure should ensure maximum shop welding and minimum site welding.

Curved bridges in plan may readily be formed using straight fabricated girders, with direction changes introduced at each site splice. If required however, steel girders can also be curved in plan. For smaller radii, curved girders are necessary to avoid the effects of long cantilevers. Skew and plan tapered bridges may also be built in steel-concrete composite. Ideally, plan layout should be as simple as possible.

For road bridges and flyovers, various options of Plated I-girders or box girders can be adopted depending on the length or span, the nature of the span i.e. whether spans are continuous or simply supported the number of lanes, and the loading.

These options further vary, in terms of transverse spacing of girders or complete separation of two adjacent lanes by means of expansion joints and providing two different sets of transverse beams for the two adjacent lanes of roads.

C.1.6.1 *Design methodology*

The design loading of Steel-Concrete Composite Girders follows the sequence of construction. The girders are designed for both construction stage loading considering wet concrete at which stage only steel section carries the load and final composite stage loading with hardened concrete. Since hardened concrete increases the bending and torsional stiffness of the girder, the sequence of construction becomes vital. As mentioned earlier, till date only IRC:22 is available in India to design standard I-girders with concrete deck as composite bridge and there is no specific guideline or code for design for box girders in India. As mentioned earlier one such guideline is now under preparation by the Indian Road Congress or IRC. For the time being the Euro Codes and British Codes of Practice are generally followed for the design of these types of box girders.

The current Indian code IRC:22 uses Limit States Method (LSM) of design. As per this method, the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) must be satisfied by the design. In practice ULS generally governs the design, exception being the checking at SLS for slip of HSFG bolted splices, deflection check, design of shear connectors and design for fatigue.

IRC:22 encourages the use of steel limit states design because:

1. It is more rational than the existing working stress method which can be explained by the following few examples:
    a) Unlike WSM, LSM allows different buckling curves for direct compression in a section (i.e. curve a, b, c, d) for different classification of sections, such as Plastic, Compact, Semi-compact and Slender sections.
    b) For sections under bending, the reduction in permissible bending moment due to lateral torsional buckling is different for rolled and welded section, unlike the existing working stress method.
    c) Usually plated girders are used for Bridges, which can be so selected, that the girder is either “Plastic” or “Compact” and consequently, limit states design results in the use of lighter members both for plated as well as rolled sections, since plastic moment capacity is usually the governing design capacity, for the ULS check, unlike the working stress method.
    d) Limit states method allows different partial loads factor individually for different types of dead loads, live loads, etc., under different load combinations which result in the most probable load action on the section under consideration.
    e) In LSM, for different materials of construction, i.e. structural steel, reinforced concrete, reinforcements, etc., different partial safety

factors against each material are attributed to calculate the design strength of a member in terms of axial force, bending moment, shear force, etc. thus rendering it more correct and rational.

f) For serviceability design consideration, separate design checks against stress, deflection, etc. under different types of load combinations are specified in LSM.

2) Workmanship requirements, including tolerances, are rationalized.

To determine the design forces and stresses, elastic analysis is generally done using the gross section (i.e. not considering the effect of shear lag or effective width). The stresses generated in the concrete deck are in proportion to the modulus of elasticity of concrete to steel (i.e. modular ratio). For sustained loading like imposed loads due to finishes/bridge furnishings, the effects of creep is generally taken into account in calculation of stresses. This is generally taken care of by increasing the modular ratio by creep factor. Since, it is quite evident that the steel section alone (before hardening of the concrete) is much weaker compared to the composite section (after hardening of concrete), it needs to be checked for stresses due to erection and construction loadings. Therefore, as stated earlier, construction sequence plays an important role in the design process of the steel-concrete composite girder, since the contribution of concrete slab is available only after hardening. There are two types of construction of steel-concrete composite girders:

- ***Propped Construction:*** - The steel girder is propped at regular intervals along their lengths from below during casting of concrete deck slab. This propping is not removed till the hardening of the concrete. All the load coming on the bridge super-structure, which includes self-weight of the steel sections are resisted by the composite section.
- ***Un-propped Construction:*** - In this type of construction the steel structure after erection is not propped from below during the casting of the concrete deck slab. Usually, the deck shuttering is supported from the erected steel structure itself. Hence, the steel structure alone has to take all the dead load, including its self-weight, weight of wet concrete, services loads and erection loads. To reduce the effective length of the top flanges of the girders against lateral torsional buckling, lateral bracing systems are provided at the top flange level. These bracings also help in transferring all horizontal loads on the top flange during RCC casting to the end support of the steel girders. Composite action is available after hardening of the concrete (to resist the live loads and other imposed loads). However, at the Limit States, due to redistribution on the inelastic range of stresses, the total dead and live loads are taken to be resisted by composite sections.

Continuous Composite Girders are widely used for girders across the globe. Two-span continuous units are not the most efficient system because of very high negative moments. Three-span and four-span units are more preferable, but not always possible. Units over four-spans are not recommended. A good span arrangement for three-span or four-span units is to have the interior spans 40 to 60 per cent longer than the end spans. If the end spans become too short compared to the interior spans, uplift can be a problem at the girder ends. On the other hand, if the end spans are too long in relation to the interior spans a disproportionate amount of steel will be required for the end spans.

C.1.6.2 *Code provisions*

There are various code provisions including loading and design procedures, which provides the basic guidelines for the design of a bridge girder for all bridges made of concrete, steel or composite. As indicated earlier, IRC:6 (Standard Specifications and Code of Practice for Road Bridges, Section: II – Loads and Load Combinations deals with highway loading.

Other than IRC:6, Indian Roads Congress or IRC has different other codes on highway bridges such as IRC:24, providing stipulations for steel bridges IRC:112 for concrete bridges and IRC:22 as being discussed in this book, for composite bridges.

C.1.6.3 *Transportation and erection of composite box girders*

A very interesting aspect of composite girder design is the fact that sometimes some of its design parameters are determined or considered based on its actual handling requirements or facilities available in the fabrication shop and erection site. This comes out of the fact that the steel girder needs to be transported from the fabrication facility and erected to final position. Many a times, proportioning of the girders is governed by erection methodologies and stresses arising out of them. Some usual practices are worth discussing in this context.

- ***Splicing Length:*** - The splicing length of a steel girder for large spans are dependent on factors like the length of plates commonly available with the manufacturers and suppliers, on the transportable length of the girder through the route from the fabrication yard to the erection site and the capacity of available cranes for lifting at site. Usually in Indian conditions maximum available length of plates and transportable length of the girders match each other quite closely (about 13 m). The capacity of cranes available for handling at sites plays an important role and can become a limiting factor.
- ***Permanent Shuttering:*** - Casting of the deck slab is an important stage in the construction of the composite girder, and for casting of these slabs for bridges over rivers, or even over road crossings, proper formwork is absolutely necessary. To provide separate staging to support the formwork over flowing water bodies or rivers is a very difficult and costly proposition. Similarly for bridges over road crossings staging from bottom will lead to obstruction to traffic flow below. For these cases precast concrete planks, steel

deck plates, etc. may be resorted to as a left-in-place shuttering. Precast planks tend to increase the total thickness of the deck with considerable thickness of non-participating concrete deck. For road bridges the most suitable formwork is in the form of removable stiffened steel plate supported between the webs of two parallel longitudinal girders.

C.1.6.4 *Durability consideration*

Natural weathering or corrosion of steel is the most common phenomenon observed, which leads ultimately to reduction in the thickness of the steel elements. Painting the steel with corrosion resistant paints can protect against corrosion most effectively.

Zinc based primers are known to provide good corrosion resistance and these primers are necessarily provided after shop fabrication. During transportation and erection, proper care needs to be taken so that this coat of primers does not get damaged. If high strength friction grip bolts are used for site splicing, the location of the same needs to be left un-painted for development of adequate friction between the overlapping plates.

If there are minor scratches, which are visible only after erection, they may be corrected by touch-up with the same primer. Micacious Iron Oxide based paints are now available in India and it is said that these paints are the most effective solution for corrosion protection under humid and acidic conditions. This type of paint may be used as an intermediate coat to the structure at site. As the final coat, polymer based acrylic paint or high build chlorinated rubber paint of required shade are generally prescribed which help to increase the weathering resistance.

Composition of the painting also includes deciding upon the Dry Film Thickness (DFT) of individual coats as well as the total paint coat. This depends on the exact condition of the environment the structure will be exposed to as well as desired duration of protection. Usually the primers are 20μm to 25μm thick while the intermediate coats are 75μm to 85μm thick. The final coat should be around 75μm.

C.1.6.5 *Vibration*

The effect of excessive vibration of a bridge superstructure due to imposed loading is a serviceability limit states. Composite decks are usually too stiff to be susceptible to wind-excited oscillations, which are not considered in the design of steel-concrete composite design.

The dynamic effects of highway loading are assumed to be adequately covered by the impact factors that are included in the normal loads specified in IRC:6. This is consistent with the conclusion reached from a study of steel bridges in the United States which states that “there is no evidence of bridge motions producing discomfort of occupants of moving vehicles, so there appears to be no need for limits on deflections or accelerations of bridges which do not carry pedestrian traffic under normal conditions.

CHAPTER 2
DEFINITIONS PERTAINING TO STEEL-CONCRETE COMPOSITE BRIDGE GIRDERS AND LIMIT STATES DESIGN METHODOLOGY

C.2.0 General — Cl. 600.0

C.2.1 Scope of IRC:22 — Cl. 600.1 & Cl. 600.2

This code applies to all constructions in bridges which are mainly steel-concrete composite in nature as have been emphasized in the first two clauses of the code, i.e. clause 600.1 and 600.2. These clauses clearly defines the scope of the code along with the definition of the scope as mentioned in the above two clauses.

This code is not applicable to box girder bridges. It is pertinent to mention that a separate design guideline for box girder is under formulation, which will totally cater to the design of steel-concrete composite box girder bridges. The present revision of IRC:22 include simply supported and continuous bridges and is based on the limit states method of design. Clause 600.2 gives a definition of the type of bridge under consideration, which can be classified as steel-concrete composite girders. It clearly states that a steel-concrete composite construction is one in which steel girders are used as the primary members and cast-in-situ reinforced concrete and/ or pre-cast concrete slab with necessary grouting as the deck.

C.2.2 Terminology & Definitions and Symbols — Cl. 600.3 & Cl. 600.4

The clause relevant for definitions of terminologies which are associated with design of steel-concrete composite girders as well as Limit States Method of Design have been represented in Clause 600.3 for easy understanding of the designer.

All the symbols used in the code other than those used for load categorization as per clause 601.3 has been illustrated in Clause 600.4.

C.2.3 Limit States Method – Definitions & Applicability — Cl. 601

According to this method a structure or part of it is considered unfit for use when it exceeds the limit states, beyond which it infringes one of the criteria governing its performance or use. Thus it may be stated that, the probability of operating conditions not reaching failure conditions forms the basis of “Limit States Design”.

A Civil Engineering Design must ensure that the structures and facilities which have been designed are (i) fit for their purpose (ii) safe and (iii) economical and durable. Thus, safety is one of the paramount factors governing a design. However, it is difficult to assess at the design stage how safe a proposed design will actually be consistent with economy. There is, in fact, a great deal of uncertainty about the many factors, which influence both safety and economy. Firstly, there is a natural variability in the material strengths and secondly, it is impossible to predict the maximum loading, which a bridge may be subjected to during its life. Thus uncertainties affecting the safety of a structure are due to

- Uncertainty about loading.
- Uncertainty about member strength variability of material strength and member dimension.
- Uncertainty of behaviour and design equations.

These uncertainties together make it impossible for a designer to guarantee that a structure will be absolutely safe. All that the designer could ensure is that the risk of failure is extremely small, despite the uncertainties.

An illustration of the statistical meaning of safety is given in **Fig. 2.1**. Let us consider a structural component (say, a beam) designed to carry a given nominal load. The actual resistance (R.M.) itself is not a fixed quantity, due to variations in material strengths and member dimensions that might occur between nominally same elements. The actual resistance of these elements can be expected to vary as a consequence. The statistical distribution of these member strengths (or resistances) can be as sketched in curve (a). The design strength is often designated as the strength at the lower tail with 5% exclusion limit. The characteristic design strength is taken as the highest load in the upper tail with 5% exclusion limit.

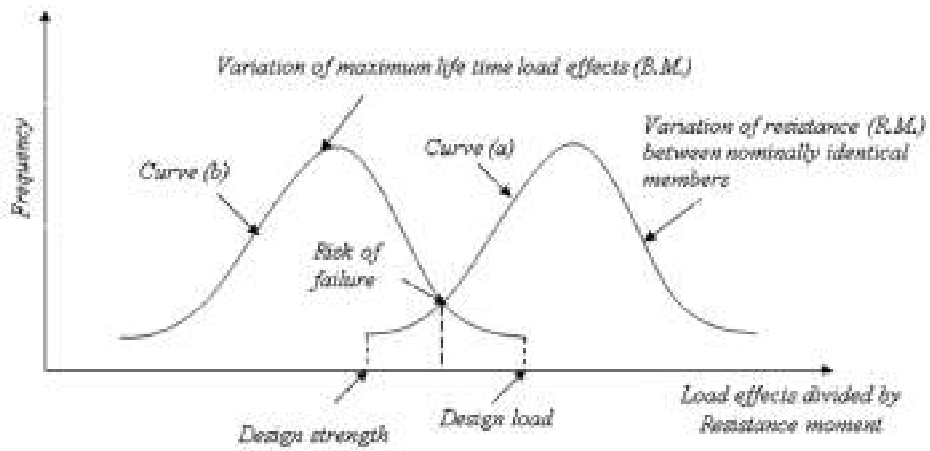


Fig. 2.1 Statistical Meaning of Safety

Similarly, the variation in the maximum loads and therefore load effects (such as bending moment) which different structural elements (all nominally the same) might encounter in their service life can have a distribution shown in (b). The uncertainty here is both due to variability of the loads applied to the structure, and also due to the variability of the load distribution through the structure.

Clearly the abscissa values over which the line curves overlap are possible cases of failure. In order to ensure reliability and safety, the design strength is kept larger than the design load by increasing the loads with load factor to obtain design load and decreasing the resistance using strength factor to obtain design strength and ensuring that the design strength is greater than the design load.

C.2.3.1 *General methodology of analysis* ***Cl. 601.1***

As clearly stated normal elastic method is valid for analysis of the structure after considering load history, sequence of concrete casting and development of composite strength. In case of either propped or un-propped construction, the load sharing takes place as described in section C.1.6.1.

C.2.3.2 *Limit states* ***Cl. 601.2***

Limit States are the various conditions in which a structure would be considered to have failed to fulfil the purpose for which it is built. In general two cases of limit states are considered at the design stage and these are listed in **Table 2.1**.

Table 2.1 Types of Limit States

Ultimate Limit State	**Serviceability Limit State**
• Strength (yield, buckling)	• Stress in Steel and Concrete and Steel
• Stability against overturning and sway	• Deflection
• Fracture due to fatigue	• Concrete crack width
• Brittle Fracture	• Vibration
	• Slip at the interface between steel and concrete
	• Fatigue checks (including reparable damage due to fatigue
	• Corrosion

"Ultimate Limit States" (ULS) are those catastrophic failure states, which require a larger reliability in order to reduce the probability of its occurrence to a very low level.

"Serviceability Limit State" (SLS) refers to the limits on acceptable performance of the structure. Not all these limits can be covered by structural calculations. For example, corrosion is covered by specifying forms of protection (like painting) and brittle fracture is covered by material specifications, which ensure that steel is sufficiently ductile. Serviceability Limit States are the states at which, either the stress in structural steel, or the deflection of the structure, or concrete crack width reaches the prescribed limit. SLS also refers to the states when the slip at the interface between steel and concrete becomes excessive or even when vibration becomes excessive specially at overhanging foot or cycle paths.

"Fatigue Limit States" is the state at which stress range due to application of fatigue vehicle load, reaches prescribed limit corresponding to the number of design load cycles and configuration of fabrication detail.

C.2.3.3 *Design loads (design actions and their combinations)* ***Cl. 601.3***

For load factors and load combinations references have to be made to IRC:6.

C.2.3.4 *Material strength & partial safety factor for material* *Cl. 601.4*

In Limit States Method of Design, the factored loads, in different combinations, are applied to the structure to determine the load effects. The latter are then compared with the design strength of the elements. This is expressed mathematically as:

The effects of

$$\gamma_L . Q_k . \leq \left[\frac{1}{\gamma_m} \right] \{R_m\}$$

Where,

γ_L = partial factor for loads that takes account of inaccuracies in assessment of loads, stress distribution and construction corresponding to the limit states (Refer. Tables of IRC:6 as indicated above)

γ_m = factors that take into account, uncertainty in material strength and quality, and manufacturing tolerances.

Q_k = specified nominal load or load effects.

R_m = member/material strength.

The partial load factor γ_L for a particular load varies depending upon the load combination. These factors have been indicated in IRC:6. The design strength of an element is reduced by the factor γ_m. Yield strength f_y of steel and characteristic strength f_{ck} of concrete with appropriate partial safety factors as mentioned below in **Table – 2.2** (Table – 1 of IRC:22) are to be used for assessment of strength:

Table 2.2: Material safety factors (γ_m)

Material	**Partial Safety Factor (γ_m)**
	Ultimate Limit
Structural Steel against Yield Stress	1.10
Structural Steel against Ultimate stress	1.25
Steel Reinforcement (γ_s) against yield stress	1.15
Shear Connectors against yield stress	1.25
Bolts & Rivets for Shop and Site Fabrication	1.25
Welds for hop Fabrication	1.25
Welds for Site Fabrication	1.50
Concrete (γ_c) For Basic and Seismic Combinations	1.50
Concrete (γ_c) For Accidental Combinations	1.20
Note: Partial safety factors are not only given for f_y and f_{ck} but also for f_u.	

C.2.4 **Materials and Properties** **Cl. 602**

The above clause gives the materials and their properties, which have discussed in details in **Appendix-III**.

CHAPTER 3
MAjOR DESIGN PROVISIONS WITH EXPLANATIONS

C.3.0 Design for Ultimate Limit States — Cl.603

A typical composite girder section is as given in **Fig. 1** of the code. The location of the neutral axis and the permissible moment corresponding to various locations of NA has been discussed in **Annexure - I** of the code.

C.3.1 General Conditions of Design for Ultimate Limit States — Cl. 603.1

As clearly explained in the stipulation itself, elastic theory assuming concrete to be un-cracked and un-reinforced shall be used for analysis of the bridge.

a) **Negative Moments:**

Negative moments over internal supports should be checked against section strength assuming steel girder acting integrally with concrete (considering uncracked and un-reinforced).

For calculating stresses, when concrete in tension has cracked due to negative moment, mainly at the internal supports of continuous girders, the composite sections for both long-term and short-term composite moments shall consist of the steel section and the longitudinal reinforcement within an effective width as discussed in the code (Cl. 603.2) of the slab.

b) **Positive Moments:**

For continuous girders, provided adjacent spans do not differ appreciably, positive maximum moments in the adjacent spans should be increased by 40 f_{ct} / f_{ck} per cent for checking of strength without decreasing support moment. This provision gives partial recognition to the philosophy of plastic design.

For calculating stresses due to positive bending moments, the composite section shall consist of the steel section and the transformed area of the effective width of the concrete slab. The modular ratio to be considered for conversion from effective concrete area to equivalent steel area is to be taken different for long-term and short-term forces and is discussed later.

C.3.1.1 *Section classifications* — ***Cl. 603.1.1***

The plate elements of a cross section may buckle locally due to compressive stresses. The local buckling 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.

When elastic analysis is used along with elastic design, the member shall be capable of developing the yield stress under compression without local buckling. When section is designed to resist moments due to elastic analysis by reaching plastic moment then the compressive yield stress shall be maintained until the full section plasticise. On the above basis, four classes of sections are defined as follows:

Class 1: *Plastic* – Plastic cross-sections are those which can develop their full-plastic moment M_p and allow sufficient rotation at or above this moment so that redistribution of bending moments can take place in the structure until complete failure mechanism is formed. The width to thickness ratio of plate elements shall be less than that specified under class 1 (Plastic), in Fig. 5 & Table 2 of IRC:22.

Class 2: *Compact* – Cross sections, which can develop plastic moment of resistance, before local buckling under compression initiates are compact sections. The width to thickness ratio of plate elements shall be less than that specified under class 2 (Compact), in Fig. 5 & Table 2 of IRC:22.

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 Fig. 5 & Table 2 of IRC:22.

Class 4: *Slender* – Cross sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under class 3 (Semi - Compact), in Fig. 5 & Table 2 of IRC:22. In such cases, the effective sections for design shall be calculated either by following the provisions of IS:801 to account for the post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

Section classifications can be best illustrated by the Moment-Rotation curve as given below:

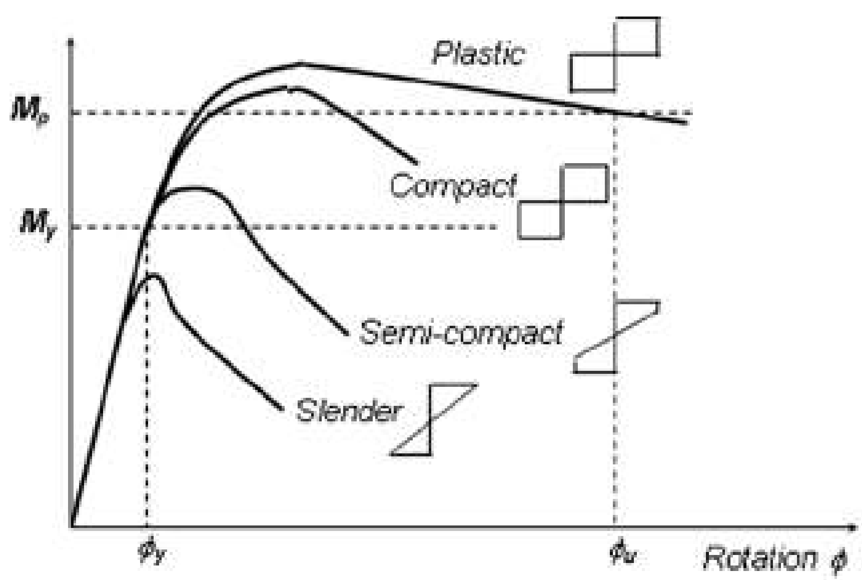


Fig. 3.1 Section Classification based on Moment-Rotation Characteristics

C.3.1.2 *General rules of composite sections classifications* ***Cl. 603.1.2***

1) *A composite section should be classified according to the least favourable class of steel elements in compression.*

   -----This takes into account the first possible mode of buckling due to compressive forces in any internal element of the section.

2) *A steel compression element restrained by its connection to a reinforced concrete element may be placed in a more favourable class.*

   -----With provision of proper shear connector between the steel element and the concrete, buckling of the steel compression flange element may be avoided by the restraint given by the more solid concrete connection.

3) *Plastic stress distribution should be used for section classification except at the boundary between class 2 and 3 where the elastic stress distribution should be used taking into account sequence of construction and the effects of creep and shrinkage.*

   -----Since the boundary condition of these two classes suggest that after attaining the elastic stress the member would start to buckle internally.

4) *For classification, design values of strength of materials should be taken. Concrete in tension should be neglected. The stress distribution should be established for the gross cross-section of the steel web and the effective flanges.*

   -----This is self-explanatory, since the stress distribution diagram of the gross section considering equivalent area of the concrete flange will give the actual idea as to the location of the Neutral Axis and the total moment carrying capacity of the cross section.

5) *Welded mesh under tension should not be included in the effective section unless it has sufficient ductility to withstand fracture when embedded in concrete.*

   -----This provision caters to the requirement of negative bending moment of a composite section mostly at the internal support of a continuous girder.

6) *In global analysis for stages of construction, account should be taken of the class of steel section at the particular stage considered.*

   -----For example, the top flange of a composite girder acts as compression element with no restraint during construction stage but acts as a compressive element with full restraint from concrete cover slab after composite action has set in.

C.3.1.3 *Classification of composite section without concrete encasement* ***Cl. 603.1.3***

1) *A steel compression flange, which is restrained against buckling by effective attachment to a concrete slab by shear connectors, may be assumed to be in class 1 if the spacing of the connector is in accordance with 606.9.*

-----Provisions for maximum spacing of shear connectors are maintained to ensure shear connection between steel and concrete and thus proper composite action of the cross section.

2) *Other steel flanges and webs in compression in composite girders should be classified on the basis of width to thickness ratios (width and thickness of individual elements shown in* ***Fig. 5*** *of IRC:22) and proneness to local buckling. Accordingly sections are categorized in three groups as indicated in* ***Table 2*** *of IRC:22.*

3) *Cross-sections with webs in Class 3 and flanges in Class 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance with* ***Fig. 4a*** *of IRC:22.*

4) *For Slender Webs* ***Fig. 4b*** *of IRC:22 gives the effective section.*

C.3.1.4 *Types of elements* *Cl. 603.1.4*

The elements of a section have been classified into three type's namely internal elements, outstand elements and tapered elements. These have been defined individually and explained based on the local buckling criteria of each element. The definitions along with the examples given in the code are self-explanatory with respect to the limiting width to thickness ratio of each element as given in Fig. 5 and Table 2 of IRC:22.

C.3.2 Effective Width of Concrete Slab **Cl. 603.2**

A composite beam acts with the concrete slab as its flange. The bending stress in the concrete flange is found to vary along the breadth of the flange as in **Fig. 3.2**, due to the shear lag effect. This phenomenon is taken into account by replacing the actual breadth of flange (B) with an effective breadth (b_{eff}), such that the area FGHIJ nearly equals the area ACDE. Research based on elastic theory has shown that the ratio of the effective breadth of slab to actual breadth (b_{eff}/B) is a function of the type of loading, support condition, and the section under consideration. For design purpose a portion of the beam span (as given in Clause 603.2.1 and 603.2.2) is taken as the effective breadth of the slab in the composite beam.

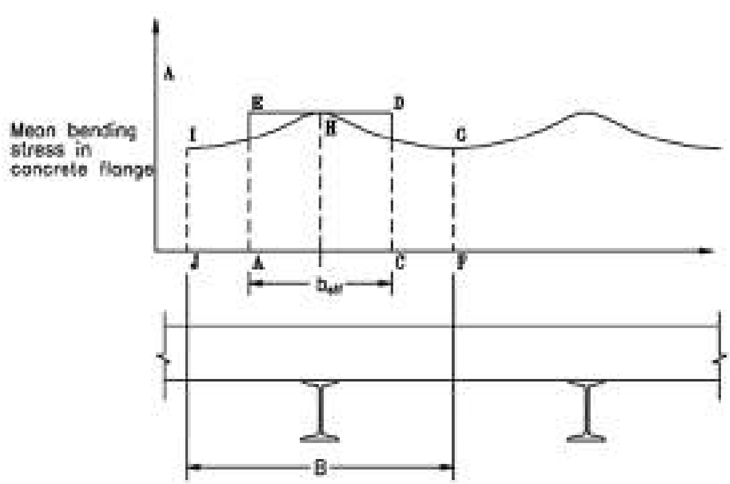


Fig. 3.2 Use of Effective Width to allow for Shear Lag

As per IRC:22, the effective width on either side of the steel web is given as (Ref: Fig. 3.3 below),

$$b_{eff\,1} \leq \frac{L_o}{8} \leq \frac{B_1}{2}\,or\,\frac{B_2}{2}$$

and the total width of the slab which is effective is as given in clause 603.2.1.

C.3.2.1 *Effective width of simply supported girder* ***Cl. 603.2.1***

1. Inner Beams

Eqn. 3.2 of IRC:22 gives the equation for simply supported beams for inner beams where the spacing of beams are unequal and Eqn. 3.3 gives the spacing for the same where the inner beams are equally spaced, where

Equation 3.2 ---- $b_{eff} = \frac{L_o}{4} \leq \frac{B_1 + B_2}{2}$

Equation 3.3 ---- $b_{eff} = \frac{L_o}{4} \leq B$ [$B_1 = B_2 = B$]

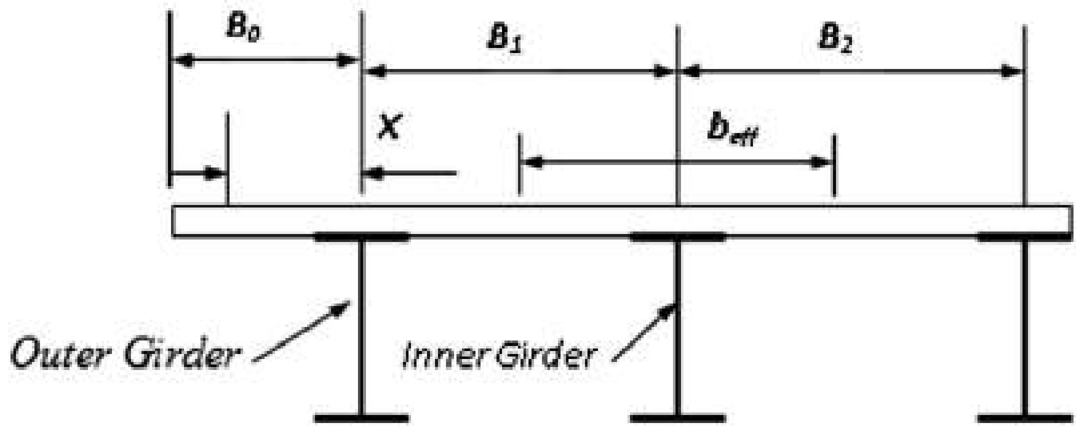


Fig. 3.3 Effective Widths for Composite Beams

2. Outer Edge Beams

Equation for outer edge beams is

$$b_{eff} = \frac{L_o}{8} + X \leq \frac{B_1}{2} + X$$

Where, $\frac{L_o}{8} \leq \frac{B_1}{2}$ and $X = B_0 \leq \frac{L_0}{8}$

C.3.2.2 *Effective width of continuous girder* ***Cl. 603.2.2***

Effective width for continuous girders is same as that for simply supported girders. The only difference is that the effective length required for width calculation varies between end span and interior span and also for continuous supports. Effective length required for width calculation varies between end span and interior span and also for continuous supports. This is as per standard elastic theory of moment distribution. The effective length to be considered is as given in **Fig. 3.4** given below.

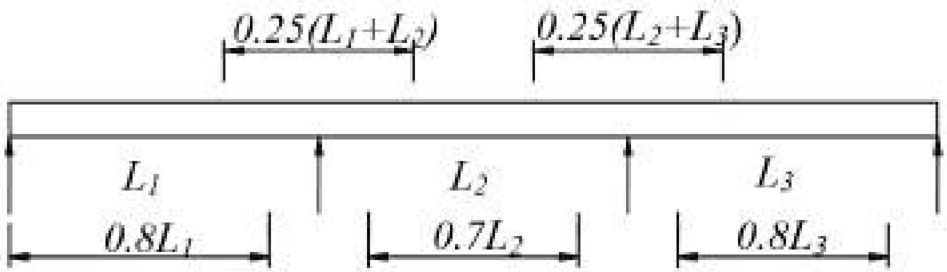


Fig. 3.4 Value of L_0 for Continuous Beam

C.3.2.3 *Effective cross section for strength calculation* *Cl. 603.2.3*

Since concrete is strong in compression and steel in tension, for positive moment under simply supported condition, full compression in concrete flange at top and full tension in bottom steel flange is considered. The contribution of longitudinal reinforcement in slab is neglected since concrete itself contributes enormously to compression.

Similarly, in continuous girders at supports, the concrete at slab top is neglected since it is weak and cracks in tension. Contribution of longitudinal reinforcement within the effective width in concrete slab along with the structural steel section may be considered in determining the design moment capacity of the section at the support.

C.3.3 Analysis of Structures **Cl. 603.3**

C.3.3.1 *Elastic analysis* *Cl. 603.3.1*

Individual members shall be assumed to remain elastic under the action of the factored design loads for all limit states. The effect of haunch 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.

In the first-order elastic analysis, the equilibrium of the frame in the un-deformed 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 effects of these on the first-order bending moments shall be considered for design using methods of moment amplification using appropriate amplification factor. However, for stability of a bridge as a whole, ultimate limit states as per provision or IRC:6 need to be followed.

As indicated in the clause the design strength of a section under factored load may be determined as per the stipulation given in Annexure I or II of IRC:22.

C.3.3.2 *Plastic analysis* ***Cl. 603.3.2***

The clause itself is self-explanatory.

C.3.3.3 *Design of structure* *Cl. 603.3.3*

The sections of a girder as discussed earlier (section, C.3.1.1) need to be classified as plastic, compact, or semi-compact based on the local buckling criteria. This depends on the *b/t* or *d/t* ratio of the individual elements of the section.

- The development of the composite action depends on the load history of the structure, which is also a function of the nature of all the types of load that may occur on the structure during its design period. Thus, an appropriate value of Modular Ratio, m, needs to be considered at each stage of loading and stresses and deflections are to be taken as the summation of values of successive stage.
- The effective width for each stage needs to be calculated based on the appropriate modular ratio as applicable for that stage of loading as per section C.4.3 (Clause 604.3 of IRC:22).

C.3.3.3.1 Design of structure [effect of lateral buckling on moment] *Cl. 603.3.3.1*

1) At construction stage, the top flange may buckle laterally closer to mid span in both simply supported and continuous girders. This is due to the fact that top flange is not laterally supported during construction stage before hardening of concrete, unless separate lateral bracings are provided. After the concrete hardens, the top flange becomes laterally supported by the hardened concrete.
2) At construction and composite stage, in the bottom flange closer to support in continuous girders has the same issue as mid span as discussed above.

At the construction stage the effect of lateral buckling on the bottom flange in a continuous girder shall be taken care of by considering the girder as a cantilever up to the point of inflection from the support.

C.3.3.3.2 Design against vertical shear *Cl. 603.3.3.2*

Design against vertical shear should ensure basically that the factored design shear force, V should be less than the design shear strength of the girder, which has been given by the equation,

$V \leq$ V_d, where V_d = Design shear strength = V_n/γ_{mo}

V_n = Nominal shear strength which can be governed by plastic shear resistance or resistance to shear buckling

Plastic Shear Strength

V_n = Nominal shear strength = V_p = Plastic shear strength

Where, $V_p = \frac{A_v \cdot f_{yw}}{\sqrt{3}}$

A_v is the shear area, which is discussed in detail in this clause and needs no further elaboration, and f_{yw} is the yield strength of weld.

Shear Buckling Resistance

There are two methods for determination of shear buckling resistance.

1) *Simple post critical method:* This method can be used for I-sections with or without transverse stiffeners but having transverse stiffeners of the web at the supports

   V_n = Nominal shear strength = V_{cr} = shear force corresponding top web buckling

   Where, $V_{cr} = A_v . \tau_b$

   The shear stress corresponding to web buckling, τ_b is determined as discussed elaborately in this particular clause of the code.

2) *Tension field method:* This method may be used for webs with intermediate transverse stiffeners in addition to transverse stiffeners at the supports, provided that adjacent panels or end posts provide anchorage for the tension fields. However it should be used only when $c/d \geq 1.0$,

   Where, *c* is the clear spacing between transverse stiffeners and *d* is the depth of the web.

   However, when the transverse stiffeners are widely spaced, the tension field method becomes over-conservative. Therefore, it is not recommended for use where, $c/d > 3.0$.

   V_n = *Nominal shear strength* = V_{tf}

   Where, $V_{tf} = [A_v . \tau_b + 0.9 w_{tf} . t_w . f_y . \sin ø] \leq V_p$

   The steps for determining the value of V_{tf}, has been discussed in detail in the code and it needs no further elaboration.

C.3.3.3.3 Reduction of bending stress under high shear force *Cl. 603.3.3.3*

The external shear *'V'* varies along the longitudinal axis *'x'* of the beam with bending moment as $V = dM/dX$. While the beam is in the elastic stage, the internal shear stresses τ, which resist the external shear V, can be written as,

$$\tau = \frac{VQ}{It}$$

Where, *V* is the shear force at the section, *I* is the moment of inertia of the entire cross section about the neutral axis, *Q* is the moment about neutral axis of the area that is beyond the fibre at which τ is calculated and *'t'* is the thickness of the portion at which τ is calculated.

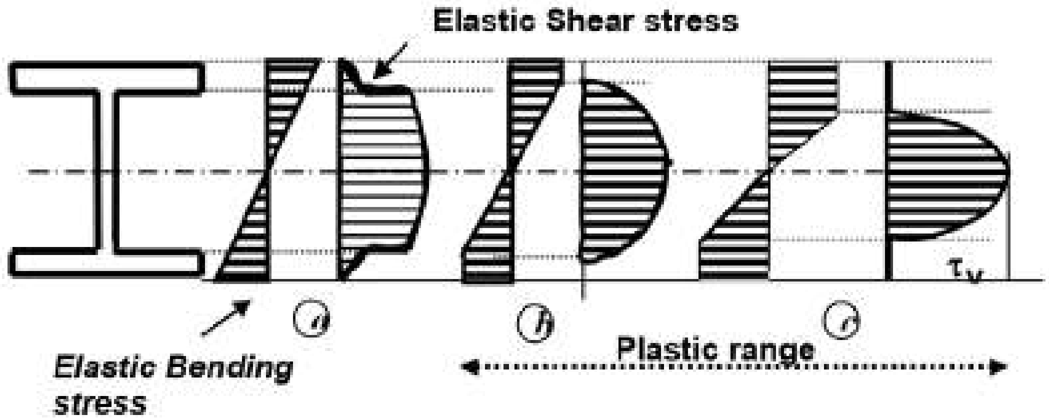


Fig. 3.5 Combined Bending and Shear in Beams

1. *Plastic or Compact Section*

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

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

M_d = plastic design moment of the whole section disregarding high shear force effect considering web buckling effects.

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

V_d = design shear strength as governed by web yielding or web buckling

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

2. *Semi-compact Section*

$M_{dv} = Z_e . f_y / \gamma_{mo}$ Where Z_e = elastic section modulus of the whole section

C.3.3.3.4 Design against longitudinal shear *Cl. 603.3.3.4*

Longitudinal shear between steel and concrete is taken up by the shear connectors and as such this need to be referred to the stipulations laid down in Clause 606 of the code.

C.3.4 Hybrid Sections **Cl. 603.4**

This clause is self-explanatory.

CHAPTER: 4
DESIGN FOR SERVICEABILITY LIMIT STATES

C.4.0 Design for Serviceability Limit States — Cl.604

C.4.1 General Conditions for Serviceability Check — Cl.604.1

As discussed earlier, deflection, vibration, and stress check, etc. are the major serviceability checks. Normal elastic analysis to be adopted for finding out design moments and stresses under various load combinations and load factors as indicated through Clause 601.3. Concrete is to be assumed as unreinforced and un-cracked, when under compression. Concrete under tension at supports of continuous beams have to be investigated for state of stress to decide whether cracked or uncracked conditions need to be assumed

C.4.1.1 *Method of construction* — *Cl.604.1.1*

Method of construction controls the stress and strain in the constituents of composite construction.

1. Un-propped construction
2. Propped construction

Though the difference in the above two methods of construction does not, affect the ultimate limit load, but the intermediate stresses in individual constituents of the composite construction varies for different loading conditions which have been elaborated in the above clause.

C.4.2 Negative Moments — Cl.604.2

Check for negative moment needs to be done as mentioned in section C.3.1, where it has been discussed in detail.

C.4.3 Stresses and Deflections — Cl.604.3

For composite girders, to check for serviceability against stresses and deflection the width of concrete effective as compression flange is a function of the modular ratio, m, which is different for short term loading effect (live and impact load) and permanent loading (dead Load and super imposed dead load). The value of m, for short term and long term effect has been elaborated in the clause.

The long-term values include the effect of creep, shrinkage for which no separate calculation may be needed. Final stresses and deflection is to be worked out separately at each stage of load history with relevant modular ratios and section modulus and then added together.

C.4.3.1 *Limiting stresses for serviceability check* — *Cl.604.3.1*

For Structural Steel : -- The various nominal stresses shall be followed as indicated below based on general practices:

The nominal Stress due to bending Moment

$$f_b \leq \frac{f_y}{\gamma_{m,ser}}$$

The Nominal Stress due to Shear

$$\tau_b \leq \frac{f_y}{\sqrt{3}\gamma_{m,ser}}$$

Combined Stress

$$\sqrt{{f_b}^2 + 3{\tau_b}^2} \leq \frac{f_y}{\gamma_{m,ser}}$$

a) Where relevant the above checks should include additional stresses from transverse load.

b) The value of γ_m shall be taken as 1.0.

c) Plate buckling effect may be ignored.

The nominal stress range due to frequent load combination should be limited to 1.5 $f_y/\gamma_{m,ser}$.

C.4.3.2 *Limit of deflection and camber* *604.3.2*

Limiting deflection under the effect of short-term loading like live and impact load only is 1/800 of the span. In case of all loads taken together including dead load, super-imposed dead load, live load and Impact load, the total deflection of the girder shall not exceed 1/600 of span. Provision of suitable camber to offset the deflection against permanent loadings is always admissible as mentioned in the above clause. If camber is suitably provided then deflection should be limited considering the live and impact load only.

C.4.4 Control of Cracking in Concrete **604.4**

As required in the clause, the measures for control of cracking of concrete slab in the negative moment region should be suitably adopted. Minimum reinforcement for concrete in tension for crack control shall be adopted as per clause 12.3.3 of IRC:112. Crack width calculation as well as limiting crack width shall be as per clause 12.3.4 and 12.3.2 of IRC:112.

C.4.5 Temperature Effect **604.5**

The clause reference is self-explanatory.

CHAPTER 5
DESIGN FOR FATIGUE LIMIT STATES

C.5.0 Design for Fatigue Limit 605.0

The concept of stress range has been introduced for the first time in this code in tune with whatever has been introduced in code of practice or general steel construction (i.e. IS:800). The existing practice of adopting stress ratio concept as per IS:1024 thus become null and void as per this concept.

C.5.1 General 605.1

The term FATIGUE is inherently associated with structures, which experience variation in stresses in their cross section due to loads, which occur intermittently like in bridge girders. These loads may occur on a regular or fixed pattern like vibratory loads or irregular pattern like moving vehicular loads on roads and rail bridges. The static loads induce a constant stress in the structure. The variable loads on the other hand induce stresses which occur with the application of the loads and which go away with the removal of the loads. This additional stress due to the variable loads like the live load and the associated impact load is called Fatigue Stress.

The process of checking the safety of a structure at locations critical for fatigue failure under a given loading for a fixed number of loading cycles during its design life has been discussed in detail in the clause 605.0. The various definitions given in this clause have already been discussed in Section C.2.2.

C.5.2 Fatigue Design 605.2

Curve, defining the relationship between the numbers of stress cycles to failure (N_{sc}) at a constant stress range (S_c), during fatigue loading on a specific detail category of a structure is the standard S-N Curve. The conditions for carrying out fatigue design are as presented in the clause itself. Only due consideration should be given to welded joints involving plates greater than 25 mm to provide additional factor of safety against welding. Thus, 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, which is 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.

Determination of stress

Elastic analysis shall be done to determine stress resultants. Any secondary stress concentration due to geometric detail is not considered since this is taken care of by the Detail Category be considered. The stress concentration, however, not characteristic of the detail shall be accounted for in the stress calculation.

Low Fatigue

A structure is considered to be under low fatigue when the fatigue stress range or the number of design load cycles is very low and the two conditions are given as,

$$f \leq 27/\gamma_{mft} \quad \text{.. (1)}$$

$$N_{SC} \leq 5x10^6\left(\frac{27/\gamma_{mft}}{\gamma_{fft}.f}\right) \quad \text{.................................. (2)}$$

Where,

γ_{fft} = partial safety factors for strength and load, respectively = 1.0

γ_{mft} = partial safety factor for fatigue strength, which is influenced by consequences of damage and level of inspection capabilities and shall be obtained from Table 3 of IRC:22. For this the conditions of Fail-safe structural components and non-fail-safe components need to be considered which have been discussed in detail in the clause in the code itself.

f = actual fatigue stress range for the detail.

C.5.3 Fatigue Strength Calculation **605.3**

The fatigue strength of a detail category for both normal and shear stress range shall be obtained from the equation as given below and shall be corrected by capacity reduction factor as applicable.

For Normal Stress range,

When N_{SC} ≤ 5 x 10^6; $f_f = f_{fn}\sqrt[3]{5x10^6/N_{SC}}$

(Slope of the curve is 1 in 3 as is evident in S-N curve (Fig. 7a of IRC:22)

When 5 x 10^6 ≤ N_{SC} ≤ 108; $f_f = f_{fn}\sqrt[5]{5x10^6/N_{SC}}$

(Slope of the curve is 1 in 5 as is evident in S-N curve (Fig. 7a of IRC:22)

For Shear Stress range,

$$\tau_f = \tau_{fn}\sqrt[5]{5x10^6/N_{SC}}$$

(Slope of the curve is 1 in 5 as is evident in S-N curve (Fig. 7b of IRC:22).

Where,

f_f, τ_f = design normal and shear fatigue stress range of the detail, respectively, for life cycle of N_{SC}

f_{fn}, τ_{fn} = design normal and shear fatigue stress range respectively of the detail for 5 x10^6 cycles as given in Table 4, 5 & 6 of IRC:22.

The standard S-N Curves for normal and shear stress are given below in Fig. 7a and 7b of IRC:22.

C.5.4 Fatigue Assessment 605.4

The design fatigue load shall be lesser than the fatigue strength and thus,

$$f_{fd} = \mu_r \; f_f / \gamma_{mft}$$

$$\tau_{fd} = \mu_r \; \tau_f / \gamma_{mft}$$

Where,

μ_r = correction factor (discussed in Sec C.5.2)

γ_{mft} = partial safety factor against fatigue failure, given in Table 3 of IRC:22.

f_f, τ_f = normal and shear fatigue strength ranges for the actual life cycle, NSC obtained from Section C.5.3 for the detail.

Stress Limitations

The two sets of limitations in the stress as put down in the code takes care of the possibility of ultimate failure against Limit States of Collapse due to Fatigue. Hence, it is mandatory that,

1) The absolute maximum normal and sear stress shall not exceed the elastic limit (f_y, τ_y) for the material under cyclic loading.
2) The maximum stress range shall not exceed 1.5 f_y for normal stress range and $1.5 f_y / \sqrt{3}$ for shear stress under any circumstances.

Constant Stress Range

Constant stress range signifies the condition where the member or the detail under consideration is subjected to same stress range throughout its design life. In short it refers to member or details where the intermittent design load causing fatigue is constant. A very common example is the crane gantry girder in workshops. Fatigue assessment for constant stress range has been described thus far as mentioned above.

Variable Stress Range

Variable stress range signifies the condition where the member or the detail under consideration is subjected to variable stress range during its design life. In short it refers to member or details where the intermittent load causing fatigue is not the same in each cycle of loading. Though all Railway and Road bridges are designed for a constant stress range, they are good examples of structures where the members or details under considerations are subjected to variable stress range. Fatigue assessment for variable stress range has been discussed in detail in clause 605.4 of IRC:22.

C.5.5 Exemption from Fatigue Assessment 605.5

The above clause which is self-explanatory, lays down the conditions under which no fatigue assessment is required.

CHAPTER 6
SHEAR CONNECTORS

C.6.0 Shear Conector 606.0

C.6.1 Provisions and Design of Shear Connectors 606.1

Shear connectors are required at the interface of steel and concrete for all types of Composite members, including slab to steel beam connection between structural steel and concrete in composite columns. The requirement varies depending upon the nature of the force to be transferred at the interface. The shear connector should be capable of transmitting the design stress or loads at the interface. The clause lays down basic guidelines for choosing and designing the shear connectors.

Shear connector can be of both mild steel or high tensile steel and shall be designed to transfer the full longitudinal shear. Shear connector strength and spacing are to be checked separately for all the limit states using appropriate factored load combinations and factored strength. All shear connectors should be capable of resisting uplift. Channel and stud shear connectors provide adequate safety against uplift.

Flexible shear connectors are preferred because of their better performance. Channel, Angle and Tee shear connectors may be of mild steel, whereas, the shear studs may be made of high tensile steel also.

C.6.2 Longitudinal Shear 606.2

Fatigue assessment is a limit state of serviceability; hence stresses should be calculated on the basis of elastic theory using appropriate sectional properties based on effective widths and modular ratios as per the load history and development of composite action.

C.6.3 Design Strength of Shear Connectors 606.3

Shear connectors are to be checked for ultimate limit states and fatigue limit states. Strength of flexible shear connectors for ultimate limit state and fatigue limit states have been given in Table 7 and 8 of IRC:22. Strength of other types of shear connectors is to be determined by standard tests.

C.6.3.1 *Ultimate strength of shear connectors* *606.3.1*

The ultimate strength of shear connectors, both static load and for fatigue has been discussed in this clause.

Ultimate strength of shear connectors

Design ultimate strengths of flexible shear connectors mainly stud connectors and channel connectors can be determined by equation 6.1 and 6.2 of IRC:22 respectively.

The recommendation in terms of height, width, length etc., for use of channel connectors is mainly due to the structural shape of the connectors and to ensure proper bondage with the concrete.

C.6.3.2 *Fatigue strength of shear connectors* *606.3.2*

As discussed earlier the fatigue strength of shear connectors need to be determined from Fig. 7b of the code using the relevant S-N curve. The strength shall be determined from the formula as given below.

$$\tau_f = \tau_{fn} \sqrt[5]{5x10^6 / N_{SC}}$$

The nominal fatigue strengths of some standard shear connectors have been indicated in Table 8 of IRC:22.

C.6.4 Spacing and Design of Shear Connectors 606.4

The spacing of shear connectors for a steel-concrete composite beam is governed by the minimum of the spacing calculated separately considering ultimate limit sates and fatigue limit states.

C.6.4.1 *Ultimate limit strength* *606.4.1*

The longitudinal shear at the interface of steel and concrete is given as,

$$V_L = \sum \left[\frac{V.A_{ec}.Y}{I} \right]_{dl,ll}$$

Spacing of Shear connectors is given as, $S_{L1} = \frac{\sum Q_u}{V_L}$

[the notation and symbols are as explained in the code]

Q_u is the Ultimate static strength of one shear connector which is to be taken from Table 7 (IRC:22) or calculated as per Eqn. 6.1 or 6.2 and the summation is over the number of shear studs at one section.

C.6.4.1.1 Full shear connection *606.4.1.1*

Force in the shear connectors also arise at the interface shear between steel and concrete due to bending. The maximum longitudinal shear force at steel concrete interface due to bending moment is to be calculated over the shear span L, from zero moment to maximum moment section and is given by Eqn. 6.5 and 6.6 of IRC:22

The maximum compressive force in concrete in the composite beam will be governed by H, which is the smaller of H_1 and H_2. This compressive force from concrete has to be transferred through shear connectors to equilibrate the tensile force in steel caused by flexure. This should be accomplished over the shear span length, L. Sufficient connectors should be provided to resist the longitudinal shear force H.

Spacing of Shear connectors is given as $S_{L2} = \frac{\sum Q_u}{H}.L$

C.6.4.2 *Serviceability limit strength (limit state of fatigue)* *606.4.2*

Longitudinal shear due to fatigue is given as,

$V_r = \sum\left[\frac{V_R.A_{ec}.Y}{I}\right]$ {V_r, A_{ec}, Y and I, are as explained in the code}

V_R = The difference between the maximum and minimum shear envelop due to live load and impact

Spacing of Shear connectors from fatigue consideration is given as

$$S_R = \frac{\sum Q_r}{V_r}$$

Q_r is the Nominal fatigue Strength one shear connector, which is to be taken from Table 8 (IRC:22). For Full shear connection the least of S_{L1}, S_{L2} and S_R is to be provided as the actual spacing of the shear connectors.

C.6.5 Partial Shear Connection 606.5

Sometimes due to problem of accommodating shear connectors uniformly or to achieve economy, partial shear connections are provided between steel beam and concrete slab. Then the shear force that can be resisted by the connectors is taken as their total capacity *($F_c < F_{cf}$)* between points of zero and maximum moment. Here F_{cf} refers to the total force under full composite action. Partial Shear Connection can be used for plastic and compact sections. If n_f and n_p are the number of shear connectors required for full interaction and the number of actual shear connectors for partial interaction respectively, then the degree of shear connection is defined as, n_p/n_f. Therefore,

Degree of shear connection, $S_c = \frac{n_p}{n_f} = \frac{F_c}{F_{cf}}$

{The limiting values for S_c for various types of steel section have been elaborated in the clause}.

C.6.6 Detailing of Shear Connectors 606.6

The dimensional detail of shear connectors that may be used for ensuring composite action has been shown in detail in Fig. 8 of IRC:22 and has been explained below:

1) The diameter of the stud connector welded to the flange plate shall not exceed twice the flange thickness to which it is attached.
2) The height of the stud connectors shall not be less than four times their diameter or 100 mm.
3) The diameter of the head of the stud shall not be less than one and a half times the diameter of the stud (shank).

4) The leg length of the weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate.

5) Channel and angle connectors shall have at least 6 mm fillet welds placed along the heel and toe of the channels/angles.

6) The clear distance between the edge of the flange and the edge of the shear connectors shall not be less than 25 mm.

7) To resist separation with the steel girder, top flange of stud and channel shear connectors shall extend into the deck slab at least 40 mm above bottom of transverse reinforcements and also a minimum of 40 mm into the compression zone of concrete flange.

8) Where a concrete haunch is used, between the steel flange and the soffit of the slab, top flange of the stud or channel shear connectors shall extend up to at least 40 mm above the transverse reinforcements in the haunches, provided the reinforcements are sufficient to transfer longitudinal shear.

9) Where shear connectors are placed adjacent to the longitudinal edge of the slab, transverse reinforcement provided in accordance with Clause 606.11 of IRC:22 shall be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

10) The overall height of a connector including any hoop, which is an integral part of the connector, shall be at least 100 mm with a minimum clear cover of 25 mm.

C.6.7 Detailing of Haunches **606.7**

The dimensions of reinforced concrete haunches that may be provided between top of steel girder and soffit of concrete slab shall be as given in Fig. 9 of IRC:22.

C.6.8 Cover to Shear Connectors **606.8**

The detail of the clear cover that shall be provided for shear connectors within concrete slab has been presented in Fig. 10 of IRC:22.

C.6.9 Limiting Criteria for Spacing of Shear Connectors **606.9**

In a composite beam since the concrete slab is in contact with the steel member below, the compression flange of semi-compact section (class 3) used as the steel member may be considered to be in class 1 or class 2 because of the restraint provided by the shear connectors.

The following criteria pertaining to the spacing of shear connectors need to be followed to ensure proper load transfer as required for composite action:

1) Where the slab is in contact over the full length (e.g. solid slab), maximum spacing is governed by eqn. 6.11 of IRC:22 as given below.

$S_L \leq 21 \times t_f \sqrt{250 / f_y}$

2) Where the slab is not in contact over the full length (e.g. slab with ribs transverse to the beam), maximum is governed by eqn. 6.12 of IRC:22 as given below.

$S_L \leq 14 \times t_f \sqrt{250 / f_y}$

3) The clear distance from the edge of the compression flange to the nearest line of shear connectors should not be greater $9.t_f \sqrt{250 / f_y}$ than or 50 mm which whichever is less.

The other criteria are for spacing has been elaborated further in the code.

C.6.10 Transverse Shear Check 606.10

Transverse shear strength of concrete slab along with the transverse reinforcement in slab ensures load transfer between concrete slab and steel beam through the shear connectors. The minimum transverse reinforcement required to ensure the above load transfer mechanism has been given in detail in clause 606.10 and clause 606.11 of IRC:22. Fig. 11 of IRC:22 gives the various shear planes for different type of slab to beam connection in composite construction.

C.6.11 Transverse Reinforcements 606.11

Planes which are critical for longitudinal shear failure, in the process of transfer of longitudinal shear between slab and steel girder, are of four main types, as shown in Fig.12 of IRC:22. To ensure the load transfer mechanism between steel and concrete following provisions shall be satisfied:

1) If the concrete by itself is insufficient to take the longitudinal shear, sufficient transverse reinforcements shall be provided to transfer longitudinal shear force from the girder to the effective width of the slab.

2) The area of transverse reinforcement per unit length of beam will be the sum total of all the reinforcement (A_t, A_h or A_b as shown in **Fig. 12a, Fig. 12b** and **Fig. 12c**), which are intersected by the shear plane and are fully anchored on both the sides of the shear plane considered.

The transverse reinforcements shall be placed at locations as shown in Fig. 13 of IRC:22.

CHAPTER 7
COMPOSITE COLUMNS

C.7.0 Composite Columns 607.0

C.7.1 General Overview 607.1

A steel-concrete composite column is a compression member, comprising of either a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally used as a load-bearing member in a composite framed structure. Typical cross-sections of composite columns with fully and partially concrete encased steel sections are in Fig. 14(a) of IRC:22. Meanwhile, Fig. 14(b) shows three typical cross-sections of concrete filled tubular sections. Note that there is no requirement to provide additional reinforcing steel for composite concrete filled tubular sections, except for meeting the requirements of fire resistance where appropriate.

In composite construction, the bare steel sections support the initial construction loads, including the weight of structure during construction. Concrete is later cast around the steel section, or filled inside the tubular sections. The concrete and steel are combined in such a fashion that the advantages of both the materials are utilized effectively. The lighter weight and higher strength of steel permit the use of smaller and lighter foundations. The subsequent concrete addition enables the building frame to easily limit the sway and lateral deflections. While adopting composite columns the following points need to be considered:

1) The material grade for adopting composite columns or compression members have been limited to IS:2062 for steel and normal weight concrete of strength M25 to M90.
2) Isolated columns and compression members in framed structures where the other structural members are either composite or steel members can be adopted as composite members.
3) The effects of local buckling may be neglected for a steel section fully encased wherein reinforcements and stirrups are provided such that they effectively hold the concrete, and for other types of cross-section where the maximum width to thickness ratio given in section 607.4 are not exceeded.
4) The influence of local buckling of the steel section on the resistance of the composite section as a whole shall be considered for design.
5) The steel contribution ratio, δ (i.e. the ratio of the load taken up by the steel member alone to the total load on the composite section) should fulfil the criteria: $0.2 \le \delta \le 0.9$.

C.7.2 Construction Particular 607.2

In fully encased steel sections, the steel section shall be unpainted. It ensures adequate friction between concrete and steel section to generate composite action through bond between steel and concrete. Calculated shear stress at interface shall be kept limited in accordance with Table 13 of IRC:22, beyond which mechanical shear connectors are to be provided

Concrete shall have an all-round cover of at least 40 mm or one-sixth of the breadth b of the flange, at abutting surface to ensure protection against corrosion and spalling of concrete. The cover to reinforcement should be in accordance with IRC:112.

C.7.3 Members Under Axial Compression 607.3

Typical cross section of Composite columns is as already indicated in Fig. 14a and 14b above. The design methodology is different for column types shown in Fig. 14a from that shown in Fig. 14b.

C.7.3.1 *General design philosophy* *607.3.1*

This clause has been explained in detail in the code itself.

C.7.3.2 *Design guidelines* *607.3.2*

1) The maximum cover to longitudinal reinforcements has been stipulated as indicated in the clause above to ensure proper composite behaviour.
2) The maximum reinforcement has also been restricted to allow for proper concreting and avoid formation of voids and honeycombs.

C.7.4 Local Buckling of Steel Sections 607.4

The equations for determining the plastic resistance of an encased steel section or concrete filled rectangular/square or circular tubular section P_p is valid provided that local buckling in the steel sections does not occur before, during or immediately after pouring of concrete.

Since slender elements of steel sections buckle locally more easily with the application of loads, stability of such elements against buckling need to be ensured by stipulating an appropriate upper limit on width to thickness ratio. To prevent premature local buckling before failure governs by yielding, this clause gives the permissible width to thickness ratio of the steel sections in compression.

C.7.5 Short Compression Members 607.5

Short compression members are those whose slenderness ratios are lesser than the prescribed limit (i.e. $\bar{\lambda} \leq 0.2$ within which no lateral buckling will take place with the application of axial force. A short compression member is liable to fail by squashing due to axial load.

The non-dimensional slenderness ratio, $\bar{\lambda}$ of a short member is given as,

$$\bar{\lambda} = \sqrt{\frac{P_{pu}}{P_{cr}}}$$

The plastic resistance of the cross section to compression for a short member is given as,

$P_{pu} = A_s.f_y + \alpha_c.A_c.\,[0.80f_{ck}] + A_{st}.f_{st}$,

For circular column, the ratio of the volume of helical reinforcement to the volume of the core concrete shall not be less than 0.36 $(A_g / A_{co} - 1)\, f_{ck} / f_{st}$.

The elastic critical buckling load of axial member is given as,

$$P_{cr} = \frac{\pi^2 (EI)_e}{l^2}$$

The design plastic res istance of steel encased concrete column and concrete filled rectangular and circular column are given in clause 607.5.1 and 607.5.2.

C.7.6 Effective Flexural Stiffness 607.6

The effective flexural stiffness for slender columns, which fail due to lateral buckling, varies with the nature of the load. For short term loading, the effect of load eccentricity is insignificant since the load acts for a short period of time, whereas for long term or permanent loading the eccentricity of the load causes increase in the actual design bending moment due to the continuous action of the eccentric load and the consequent increasing lateral deformation with time at constant load.

The effective flexural stiffness for short term loading is given by adding the individual stiffness of each element of a cross section.

$(EI)_e = E_s\, I_s + 0.6\, E_{cm}\, I_c + E_{st}.I_{st}$

Where,

E_s and E_{st} are the modulus of elasticity of the steel section and the reinforcement respectively

E_{cm} is the secant modulus of the concrete

For slender columns under long-term loading, the creep and shrinkage of concrete will cause a reduction in the effective elastic flexural stiffness of the composite column, thereby reducing the buckling resistance. However, this effect is significant only for slender columns. As a simple rule, the effect of long term loading should be considered if $\bar{\lambda}$ >0.2.

For long term loading the amount of eccentricity plays an important role. If the eccentricity of loading is more than twice the cross-section dimension, the effect on the additional bending moment (second order effect) caused by axial force action on the increased deflections due to creep and shrinkage of concrete will be very small compared to externally applied normal forces from the linear analysis. Consequently, it may be neglected and no special provision for long-term loading is necessary. Moreover, no provision is also necessary if the non-dimensional slenderness, $\bar{\lambda}$ of the composite column is less than the limiting values given in Table 10 of IRC:22, since in that case also the second order bending moments are small.

However, when $\bar{\lambda}$ the limit exceeds prescribed above and the e/D < 2, the effect of creep and shrinkage of concrete should be accounted for by adopting the long-term loading modulus of elasticity of concrete E_{cs} instead of E_{cm} where E_{cs} which have been is defined in the clause.

C.7.7 Resistance of Member Subjected to Axial Compression 607.7

The limiting conditions for which the non-sway columns need not be checked for lateral buckling are as follows:

1. The axial force in the column is less than $0.1\ P_{cr}$ where P_{cr} is the elastic buckling strength of the column.
2. The non-dimensional slenderness $\bar{\lambda}$ is less than 0.2.

To check the safety of a compression member, check for buckling about each principal axis of the composite section need to be done as laid down in detail in this clause. The buckling curve to be adopted has been given in Table 11 depending on the type of column.

C.7.8 Combined Compression and Bending 607.8

The interaction curve for uni-axial bending moment and axial force is as given in **Fig. 7.1** (Fig. 15 of IRC:22).

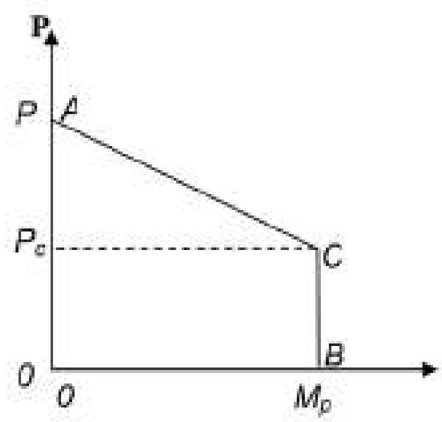


Fig. 7.1 Interaction Curve for Compression and Uni-axial Bending

a) Corresponding to point A in the above curve for M = 0, the maximum axial force is as discussed in clause 607.5 & 607.7.

b) For P = 0, (Point B), the plastic moment of resistance of the composite cross section is as given below.

$$M_p = (Z_{ps} - Z_{psn})\ f_y/\gamma_m + (Z_{pr} - Z_{prn})\ f_{st}/\gamma_{st} + \alpha_c.0.8.(Z_{pc} - Z_{pcn})\ f_{ck}/\gamma_c$$

Z_{ps}, Z_{pr}, and Z_{pc}	plastic section moduli of the steel section, reinforcement and concrete about their own centroids respectively
Z_{psn}, Z_{prn} & Z_{pcn}	plastic section moduli of the steel section, reinforcement and concrete about neutral axis of gross cross section respectively.

Since the effect of transverse shear force on the section plays an important role in determining the plastic resistance of a section a reduction in the steel strength i.e. (1 - β) f_y / γ_m is considered. It is considered only the actual factored shear force in the section is greater than or equal to 0.5 times the design shear strength of the section.

Actual shear force, V may be distributed into V_s acting on the structural steel and V_c, acting on the reinforced concrete section by:

$$V_s = V.\frac{M_{ps}}{M_p}$$

$$V_c = V - V_s$$

M_{ps}, is the plastic moment of resistance of steel section alone and M_p is the plastic moment of resistance of the entire composite section.

C.7.8.1 *Second order effects on bending moment* *607.8.1*

For non-sway columns second order effects need to be considered, when

a) $\frac{P}{P_{cr}} > 0.1$

b) $\bar{\lambda}$ >0.2

In order to take into account the second order effects a correction factor, k, is used to determine the maximum first order bending moment, which has been discussed in the above clause.

C.7.8.2 *Resistance of members subjected to combined compression and uni-axial bending moment* *607.8.2*

The basic criteria for design under combined axial force and uni-axial bending moments are given below which need to be satisfied for safe design. The criteria take into account the buckling of the column under axial load and also on the interaction effect between axial force and bending moment as indicated in **Fig. 7.2**. The design conditions to be satisfied have been elaborated in this clause vide eqn. 7.10 and 7.11.

The expressions are obtained from geometry consideration of the simplified interaction curve illustrated in **Fig. 7.2**.

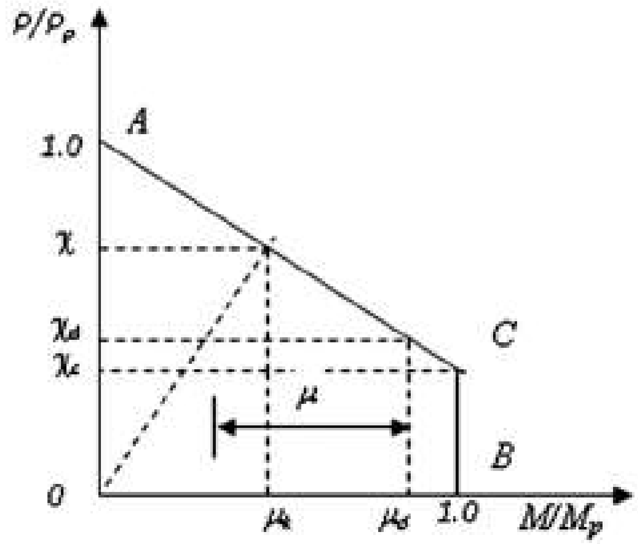


Fig. 7.2 Interaction Curve for Compression and Uni-Axial Bending

C.7.8.3 *Resistance of members subjected to combined compression and biaxial bending moment* *607.8.3*

For the design of a composite column under combined compression and bi-axial bending, the axial resistance of the column in the presence of bending moment for each axis has to be evaluated separately. Thereafter the moment of resistance of the composite column is checked in the presence of applied moment about each axis, with the relevant non-dimensional slenderness of the composite column. Imperfections have to be considered only for that axis along which the failure is more likely. If it is not evident which plane is more critical, checks should be made for failure due to flexure about both the axes.

The design conditions to be satisfied have been elaborated in this clause vide eqn. 7.12 to 7.16 of IRC:22.

The expressions are obtained from geometry consideration of the simplified interaction curve illustrated in Fig. 7.3.

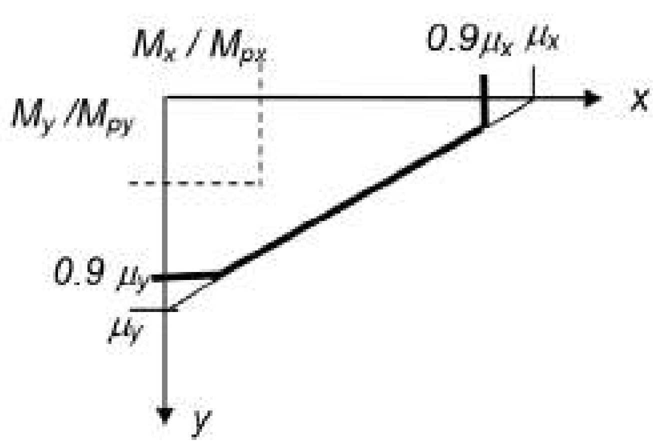


Fig. 7.3 Moment Interaction Curve for Bi-Axial Bending

C.7.9 Mechanical Shear Connection and Load Introduction 607.9

The four important criteria governing the provision of mechanical shear connector in a compression member are:

a) Proper sharing of loads between steel section and concrete should be ensured at points of introduction of actual load and moments coming from members connected to the column anywhere over its length, considering the shear resistance at the interface between steel and concrete.

b) Proper transfer and sharing of loads between steel section and concrete due to any transverse shear force in the compression member.

c) To allow for load sharing due to longitudinal shear stress between the steel and concrete at the interface due to the effect of moment gradient in the columns.

d) For axially loaded columns and compression members, longitudinal shear outside the local areas of load introduction need not be considered.

C.7.9.1 *Load introduction* *607.9.1*

1) Shear connectors should be provided at regions of load introduction and at regions with change in cross section, if the designs shear strength, τ, given in the Table 13 of IRC:22 is exceeded at the interface between steel and concrete.

2) The shear forces should be determined from the change of sectional forces within the introduction length.

3) If the loads are introduced into the concrete cross section only, an elastic analysis considering creep and shrinkage should be used. Otherwise, the forces at the interface should be determined by elastic theory or plastic theory, to determine the more severe case.

4) For all practical purposes, the introduction length should not exceed *2d* or *L/3*, where d is the minimum transverse dimension of the column and L is the column length.

No shear connection is required for composite columns to accommodate creep or shrinkage, if the load application is by endplates, since the full interface between steel and concrete is permanently under compression. Otherwise the load application/introduction should be verified as explained for partial loading condition in the above clause.

Shear connection through mechanical shear connectors have been discussed in detail in clause 607.9.1.1.

C.7.9.2 *Longitudinal shear outside the area of load introduction* *607.9.2*

a) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified for transverse loads and/or end moments. Shear connectors should be provided based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength τ.

b) In the absence of a more accurate method, elastic analysis, considering long term effects and cracking of concrete, may be used to determine the longitudinal shear at the interface.

c) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in Table 13 (IRC:22) may be assumed for τ. The values given in Table 13 applies to sections with a minimum concrete cover of 40 mm. For greater concrete cover and adequate reinforcement, higher values of τ may be used.

d) Unless otherwise verified, shear connectors should always be provided for partially encased I-sections with transverse shear as discussed in the above clause.

C.7.10 Shear Check **607.10**

Reference in the above clause has been given to IRC:112 (Code of Practice for Concrete Road Bridges) for calculation of shear resistance given by the concrete section along with steel reinforcements. The total shear resistance of a composite section is the sum of the shear resistance given by concrete section along with steel reinforcements and the shear resistance given by the steel section as per clause section 3.3.3.2. The shear force shall be distributed in the steel section and the concrete section in accordance with Clause 607.9.

CHAPTER 8
FILLER BEAM DECKS FOR BRIDGES

C.8.0 Filler Beam Decks for Bridges 608.0

Filler beam decks have been introduced in Indian bridge codes for the first time. Though these types of girders are very rare in Indian sub-continent, these have been included in this code for future construction practices as and when undertaken by any bridge authority. Filler beam decks shall be used with its own restrictions as far as geometrical dimensions, spacing of steel girder, section classifications and detailing.

C.8.1 Specific Requirements 608.1

The basic requirements of filler beam decks in terms of structural configuration, geometry, boundary dimensions and material specifications have been indicated extensively. The explanations to important specifications may be as indicated below:

1) The girders are capable of taking tensile bending stresses both at top and bottom due to provision of steel flanges of filler beams. Hence, these girders can be both simply supported as well as continuous.
2) The clause states that both rolled and welded beams may be used as filler beams. The straightness and the restrictions in skew angle up to 30° ensure the uniform stress distribution along the width of the girder.
3) The depth limitations between 250 mm to 1100 mm have been specified from considering deflection as well as dead load restriction. The web to web distance between the steel beams has been restricted to ensure uniform and equal distribution of applied loads along the width of the girder system. The minimum concrete cover has been specified from corrosion and abrasion point-of-view.
4) The clear distance between edges of steel flanges have been restricted to 150 mm to ensure proper pouring of concrete.
5) The steel beam surface need to be de-scaled to ensure proper bonding between steel and concrete as no additional shear connectors are provided in filler beam decks.

C.8.2 Global analysis 608.2

The conditions and criteria for carrying out global analysis of these decks have been specified. In case of non-uniform application of load in the transverse direction of the filler beam deck, after concrete hardening; the analysis shall take into account the difference between the deflections between adjacent filler beams. However, if it is verified that sufficient accuracy is obtained by a simplified analysis considering one rigid cross section, the same can be done based on the designer's discretion.

The influence of shrinkage in concrete, effect of slip between steel and concrete may be ignored since the entire system consisting of the concrete and the series of steel beams are together taken as one unit for design.

In transverse bending, the presence of steel beams shall be ignored as it does not contribute additionally to the resistance against transverse bending or shear.

C.8.3 Section Classification **608.3**

This section indicates the method of classification of steel outstand flange of a composite section. Table – 14 of IRC:22 gives section classification for steel flange of filler beams.

An effective web of the same cross-section in Class 2 may represent a web in Class 3, which is encased in concrete. The entire web is considered to be active as class 2 since the encasing concrete is assumed to prevent local buckling of the web.

C.8.4, C.8.5 & C.8.6 Bending Moments, Shear Forces & Minimum Reinforcements **608.4, 608.5 & 608.6**

Moments and shear strength to be calculated as per **Annexure–I** and clause 603.3.3.2. Contribution from concrete as well as for minimum reinforcement in concrete reference to be made to relevant clauses in IRC:112.

CHAPTER 9
PRECAST SLAB ON STEEL BEAMS

C.9.0 Precast Slab on Steel Beams 609.0

The use of pre-cast slab shall be made in lieu of cast-in-situ slab as and where required with same design philosophy.

C.9.1 Full Depth Precast Slab 609.1

The minimum requirements for use of full-depth pre-cast slab shall be maintained to ensure efficient function of the slab.

1) The minimum depth criteria of 200 mm need to be maintained from handling point of view, mainly transportation and lifting-cum-placement at location.
2) To ensure composite action the panels need to be connected to the steel girder through shear studs connected to the girders. Slab panels with rectangular pockets shall be placed on girders at stud locations which would be later filled with grouts.
3) Provision of wearing course similar to for cast-in-situ slab as mentioned in the clause is required to ensure proper grip for running vehicles as well as for smooth surface finish.

C.9.2 Partial Depth Pre-cast Slab 609.2

A simple concept, wherein the slab consist of thin precast slab over which cast-in-situ RCC is placed to form a the total slab which acts as the concrete portion for the composite girders as well as the base slab. The precast slab acts as a permanent formwork for the cast-in-situ slab and no additional shuttering is required.

The minimum depth criteria of 75 mm need to be maintained not only from handling point of view, like transportation and lifting-cum-placement at location but also as structural component to act as shuttering for the cast-in-situ RCC slab above it. The length and width of these slabs are also maintained based on the handling process available for the site. Proper bonding between the cast-in-situ concrete and precast slab along with the steel beams has to be ensured for proper composite actions.

C.9.3 General Design Principles 609.3

General design principles for both full-depth and partial depth precast slab including calculation of effective width, bending moment, shear forces, deflection, design and requirement of shear stud are similar to that of cast-in-situ RCC slab.

The precast slab together with any in-situ concrete (for partial depth slab) should be designed as continuous in both the longitudinal and the transverse direction. The joints between slabs should be designed to transmit membrane forces as well as bending moment and shear forces.

C.9.4 Joints Between Steel Beam and Precast Concrete Slab 609.4

1) Vertical tolerances need to be maintained for slab directly resting on steel girders to ensure proper surface levelling and slope (as and where required).

2) The shear transfer mechanism shall be as per clause 606. The following precautions need to be maintained to ensure proper transfer of forces through the shear studs.

   a) The concrete which fills the pockets in the precast slab through which shear studs are placed shall be properly designed and detailed such that it can be properly cast.

   b) The minimum infill of 25 mm all around the stud is required for easy placement of slab at the location on steel beam so that they can pass through the pockets kept in the slab.

   c) For shear studs arranged in group, proper and sufficient reinforcements at the location of the group of shear studs need to be placed to ensure group action without premature local failure either in precast or cast-in-situ slab.

   d) Protection against corrosion of shear studs and steel girder under the slab without bedding shall be ensured as explained in the clause itself.

C.9.5 Joints between Precast Members 609.5

The clause explains the requirement for proper transfer of forces when there is a joint between two consecutive precast slabs. This clause of have been discussed in detail in the code itself and requires no further explanations.

C.9.6 Structural Connections at joints 609.6

This clause is self-explanatory. Along with its sub-clauses this clause gives specifications that are required from various conditions as indicated below:

1) Design procedure at location of joints.

2) General consideration for deign at location of joints such as, protection form corrosion and weather, maintaining proper final appearance, manufacturing, erection and assembly of slabs.

3) Maintaining proper design details while assembly and fixing of slabs.

4) Reinforcement continuity at joints.

C.9.7 Construction & Erection 610.0

References to relevant codes have been summarized in this clause to ensure proper modern construction practices.

C.9.8 Testing Methods 611.0

For test specifications of steel and concrete references to relevant codes have been made in this clause.

C.9.8.1 *Testing of Stud Shear Connectors* 611.1

Push-out tests as indicated in the clause itself has to be followed. The test procedure as described is a standard procedure which is in practice all over the world to determine the strength of the shear connectors.

C.9.9 Fire Resistance 612.0

The clause is self-explanatory.

C.9.10 Maintennance 613.0

The clause itself is quite explanatory as to the requirement of specialized guidebook and codes for maintenance of bridges.

CHAPTER 10
APPENDICES

APPENDIX – I

MOMENT OF RESISTANCE

C.I.1 **Moment of Resistance of Composite Sections with Plastic and Compact Sections (Positive Moments)** **I.1**

C.I.1.1 *Bending Moment with Full Shear Connection* *I.1.1*

This Code has adopted a rectangular stress block in concrete slab in tune with IRC:112 for the ultimate strength evaluation of the section.

Here a stress factor $a = \dfrac{f_y / \gamma_m}{\dfrac{\alpha_{cc}}{\gamma_c}.\eta.(f_{ck})}$ is applied to convert the concrete section into steel.

The additional assumptions made in this code are given below:

- The maximum strain in concrete at outermost compression fibre is taken as 0.0035 in bending.
- The total compressive force in concrete is given by $F_{cc} = (\alpha_{cc}/\gamma_c).\eta.\ (f_{ck}).\ b_{eff}.\ \lambda.x_u$ and this act at a depth of $\lambda.\ x_u/2$, not exceeding d_s.
- The stress strain curve for reinforcing steel and concrete are as per IRC:112.

The three cases that may arise are given below with the corresponding M_p:

Case I: Plastic neutral axis within the slab (Ref. Fig.I.1 of IRC:22)

Case II: Plastic neutral axis within the top flange of steel section (Ref. Fig.I.2, IRC:22).

Case III: Plastic neutral axis lies within web (Ref. Fig. I.3, IRC:22).

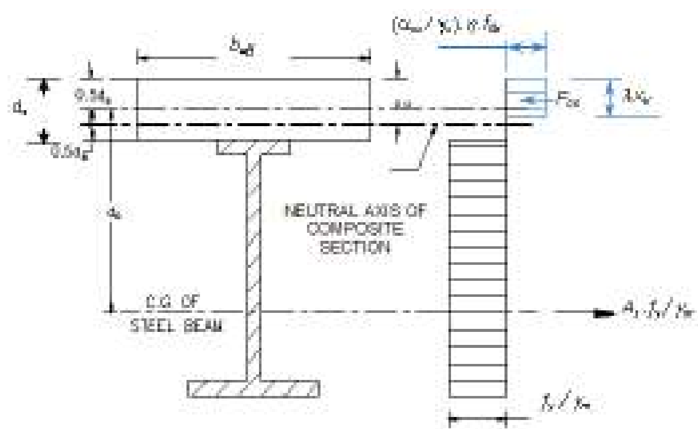


Fig. I.1 Stress Distribution in a Composite Beam with Neutral Axis within Concrete Slab

Case I: Plastic Neutral Axis within the Slab (Ref. Fig. I.1 of IRC:22)

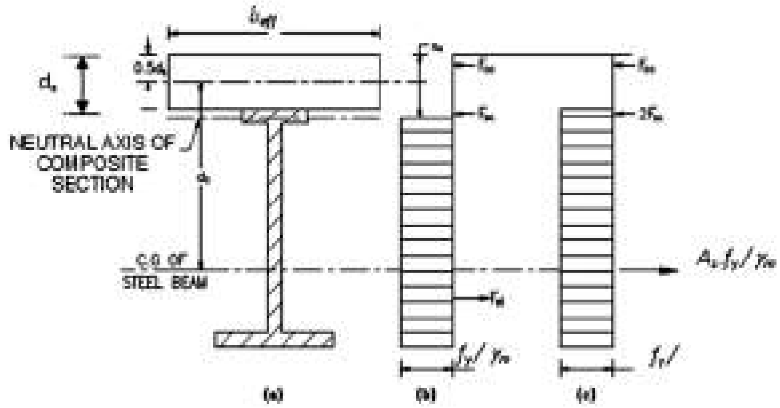


Fig. I.2 Stress Distribution in a Composite Beam with Neutral Axis within Flange of Steel Beam

Case II: Plastic Neutral Axis within the Top Flange of Steel Section (Refer Fig. I.2, IRC:22).

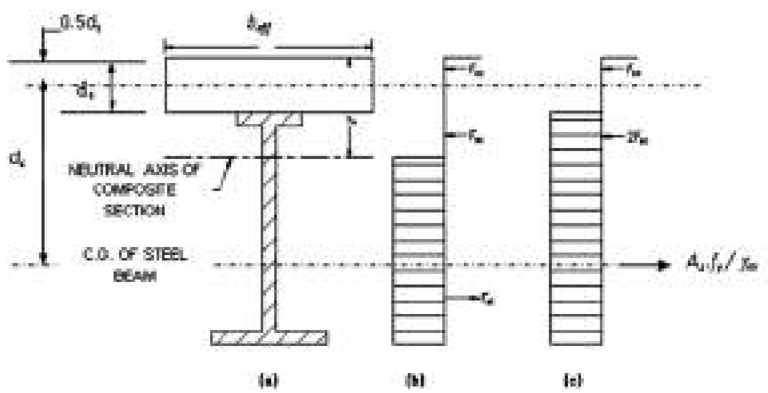


Fig. I.3 Stress Distribution in a Composite Beam with Neutral Axis within the Web of the Steel Beam

Case III: Plastic Neutral Axis Lies within Web (Refer Fig. I.3, IRC:22).

The positive design bending strength of composite beams has been summarized in Table I.1 of IRC:22.

C.I.1.2 *Bending Moment with Partial Shear Connection* *I.1.2*

Sometimes due to the problem of accommodating shear connectors uniformly or to achieve economy, partial shear connections are provided between steel beam and concrete slab. The force resisted by the connectors is taken as their total shear capacity between points of zero and maximum moment which is less than the actual shear force ($F_{cp} < F_{cf}$). Thus, design moment of resistance reduces due to partial shear connection. The relation between M/M_p with degree of shear connection F_{cp}/F_{cf} is shown in **Fig. I.4** below. The curve *ABC* is not valid for very low value of shear connection (hence shown in dotted line for lower values). It can be seen from the curve that at $F_{cp} / F_{cf} = 0.7$, the bending resistance is only slightly below M_p. Using this a considerable saving in the cost of shear connectors can be achieved without unduly sacrificing the moment capacity. However for design purpose the curve *ABC* can be replaced by a straight-line A_C. Degree of shear connection S_c is given by

$$S_c = \frac{n_p}{n_f} = \frac{F_{cp}}{F_{cf}} = \frac{M - M_{ps}}{M_p - M_{ps}}$$

n_p = Number of shear connectors provided for partial shear connection

n_f = Number of shear connectors required for full shear connection

F_{cp} = Capacity of shear connectors in partial shear connection with n_p no. of connectors

F_{cf} = Capacity of shear connectors in full shear connection with n_f no. of connectors

M = Reduced bending resistance of the partially shear connected section

M_p = Plastic moment of resistance of the composite section

M_{ps} = Plastic moment of resistance of steel section alone

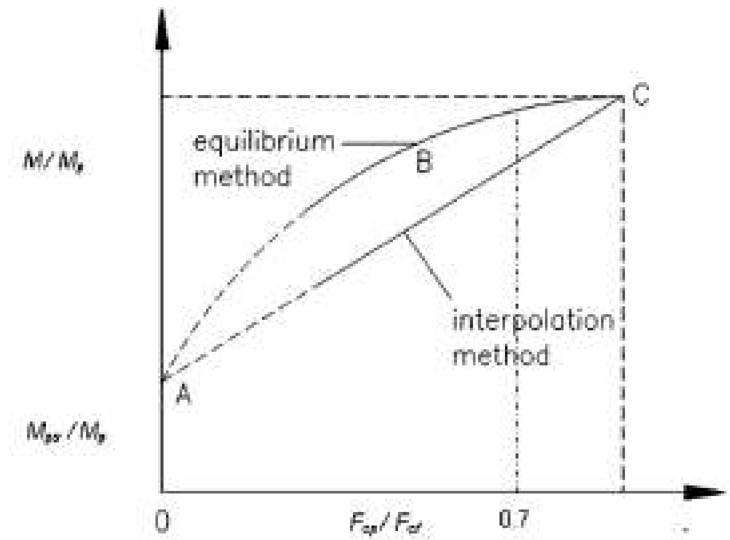


Fig. I.4 Design methods of Partial Shear Connection

Therefore, to obtain the required bending resistance M, the number of shear connectors required (assuming all connectors to have equal capacity) is given as

$$n_p = \frac{M - M_{ps}}{M_p - M_{ps}} n_f$$

C.I.2 Moment of Resistance of Composite Section with Semi-Compact and Slender Structural Steel Section (Positive moments) I.2

Since the compression flange of non-compact steel sections buckle locally under compression before reaching yield stress f_y, the resistance of the composite section consisting of non-compact sections is guided by that of compact sections as above, wherein the effective width of the compression flange is restricted to that of the compact section limiting value.

C.I.3 Moment of Resistance of Composite Section (Negative Moment) [Continuous Girder] I.3

Fig. I.5 below shows stress distribution across a composite beam section subjected to hogging bending moment. The steel bottom flange is in compression. Section classification shall be done as per the relevant table given in IRC:22. For classification of the web, the distance $\bar{y}$ of the plastic neutral axis of the hogging section, must first be found.

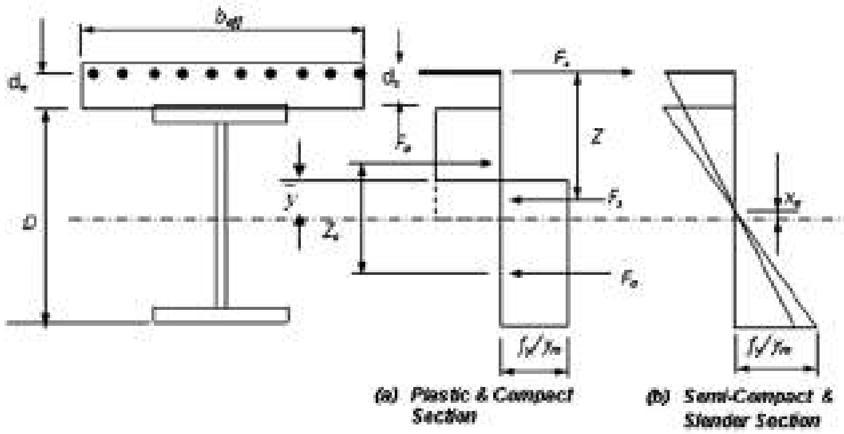


Fig. 1.5 Stress Diagram for Hogging Moment

C.I.3.1 Moment of Resistance for Plastic and Compact Section I.3.1

In the absence of any steel reinforcement in the slab the bending resistance of the section would be given by the steel section alone and is given by,

$$M_p = \frac{Z_p . f_y}{\gamma_m}$$

Where M_p, is the Plastic moment of resistance of the steel section alone, Z_p is the plastic section modulus and γ_m is the material safety factor to be taken as 1.10.

The design tensile force of the reinforcements is,

$$F_s = f_{yk} . A_{st} / \gamma_s$$

Where,

γ_s = partial safety factor for reinforcement = 1.15

f_{yk} is the characteristic yield strength of the reinforcements and A_{st} is the effective area of longitudinal reinforcement within the effective width b_{eff} of the beam. The simplest way for allowing for the reinforcements is to assume that the stress in a depth, $\bar{y}$ of the web changes from tension to compression, where $\bar{y}$ is given by,

$$\bar{y}..t_w \frac{2 f_y}{\gamma_m} = F_s$$

The bending resistance of the composite section is indicated exhaustively in the table given in Table I.2 of IRC:22.

C.I.3.2 Moment of Resistance for Non-compact Section I.3.2

For non-compact section, the stresses in the steel section shall remain within the elastic limit. Here, at the section considered, the loading causes hogging bending moment $M_{e(s)}$ in the steel member alone and $M_{e(c)}$ in the composite member. The effect of modular ratio m (= E_s / E_c) for creep of concrete is considered for elastic analysis.

Neutral axis distance from the centroid of the steel section x_e is given as,

$$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$$

and the second moment of area of the composite section is

$$I_c = I_s + A_s.x_e^{\,2} + A_{st}\left(\frac{D}{2} + d_s - x_e\right)^2$$

Where, I_s is the second moment of area of the steel section alone. The yield moment is mostly governed by the total stress in the steel bottom flange. The locations of the neutral axis and the moment of resistance for non-compact section is given in Table I.3 of IRC:22.

It is obvious, that the bending moment $M_{e(s)}$ causes no stress in the slab reinforcements. In propped construction, the tensile stress in the reinforcement may govern the design. It is given as

$$\sigma_{sr} = \frac{(f_y/\gamma_m - f_s)(D/2 + d_s - x_e)}{(D/2 + x_e)} \leq f_{st}/\gamma_{yk}$$

C.I.4 Flange Stress Reduction Factor I.3.4

In sections, where the grade of steel in flange is higher than that in web (hybrid steel girders), and where the section is non-compact and thus plastic moment capacity cannot be generated, the maximum limit of stress in both compression steel and tensile steel is limited by the reduction factor, to attain acceptable linear stress distribution from flange to web. The maximum stress in flange is given as,

$f_n = R_h \,.\, f_y/\gamma_m$, where R_h is the stress reduction factor for flange.

Obviously, for homogeneous sections, R_h shall be 1.0. As also, the reduction factor should not be applied to compact or plastic sections because the effect of lower strength material in the web is accounted for in calculating the plastic moment as specified in section C.I.1.1 above.

C.I.5 Buckling Resistance Moment (Construction Stage) I.5

During construction stage, before casting of concrete, the steel girder erected takes up all construction loads on its own. Thus, it should be designed to resist all moments and shear under lateral torsional buckling effect. Hence these girders shall be designed as per the stipulations laid down in IRC:24.

C.I.6 Moment of Resistance of Filler Beam Decks I.6

Fig. I.6 gives the stress diagram for filler beams.

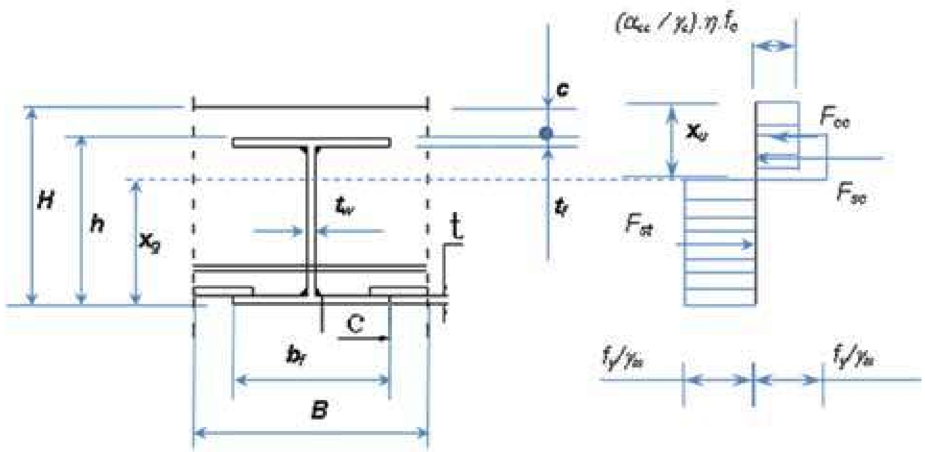


Fig. I.6 Stress Diagram for Filter Beam

Depth of Neutral axis as is evident is

$\mathbf{x_u = H - x_g}$

Where,

$$x_g = \frac{\frac{\alpha_{cc}}{\gamma_c}.\eta.f_{ck}.[B.H - b_f.t_f - t_w.(h - t_f)] + t_w.h.f_y/\gamma_m}{\frac{\alpha_{cc}}{\gamma_c}.\eta.f_{ck}.(B - t_w) + 2t_w.f_y/\gamma_m}$$

Moment of resistance,

$M_p = F_{sc}. X_{sc} + F_{cc}. X_{cc} + F_{st}. X_{st}$

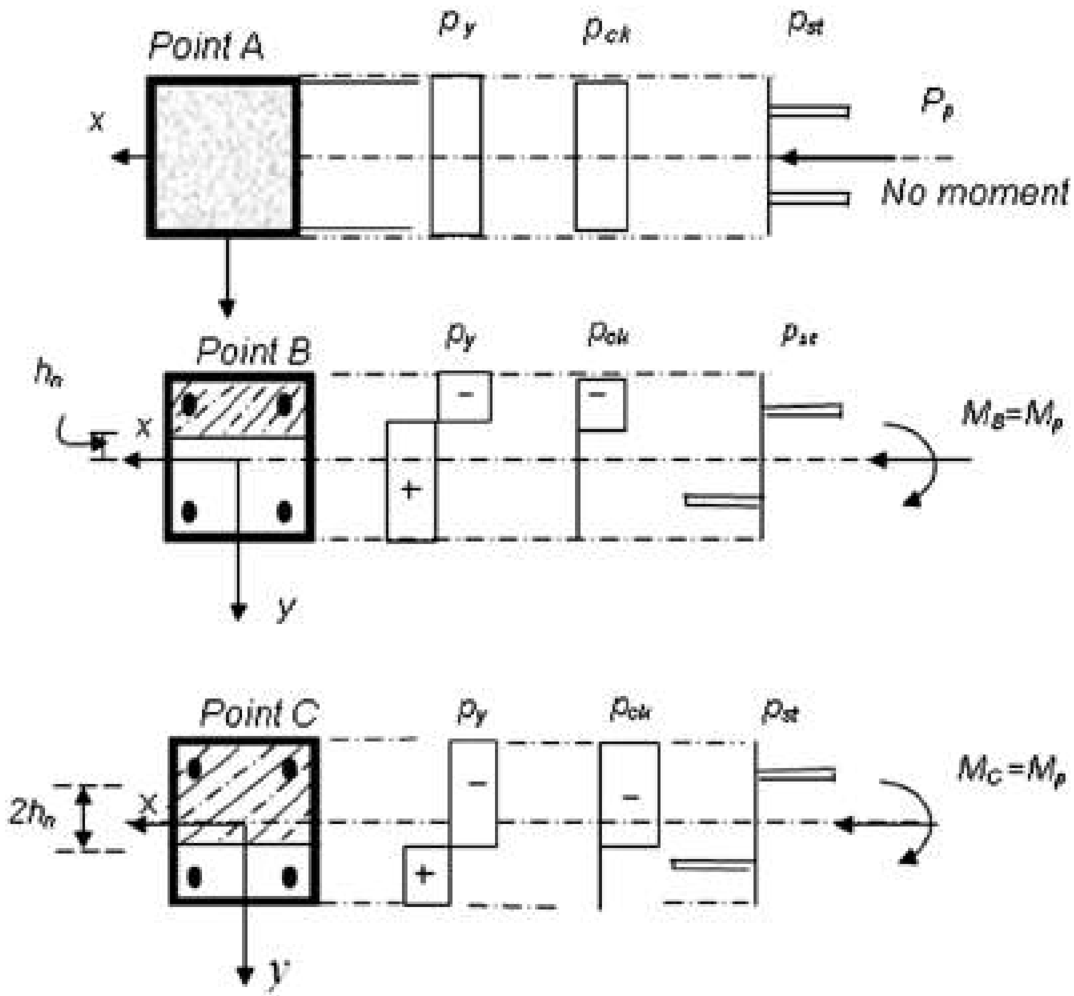
[X_{sc}, X_{cc} and X_{st} are respectively the distance between the neutral axis of the composite girder and the individual centre of gravities of the corresponding forces]

APPENDIX – II

LOCATION OF PLASTIC NEUTRAL AXIS IN COMPOSITE COLUMNS

C.II.1 General Introduction II.1

It is important to note that the positions of the neutral axis for points B and C in the interaction curve is as shown in **Fig. II.1**, h_n, can be determined from the difference in stresses at points B and C.



$$p_y = \frac{f_y}{\gamma_m} \qquad p_{ck} = \frac{0.8.\alpha_c . f_{ck}}{\gamma_c} \qquad p_{st} = \frac{f_{yk}}{\gamma_s}$$

Fig. II.1 Stress Distribution for the Points of the Interaction Curve for Concrete Filled Rectangular Tubular Sections

The resulting axial forces, which are dependent on the position of the neutral axis of the cross-section, h_n, can easily be determined as shown in **Fig. II.2**. The sum of these forces is equal to P_c. This calculation enables the equation defining h_n to be determined, which is different for various types of sections.

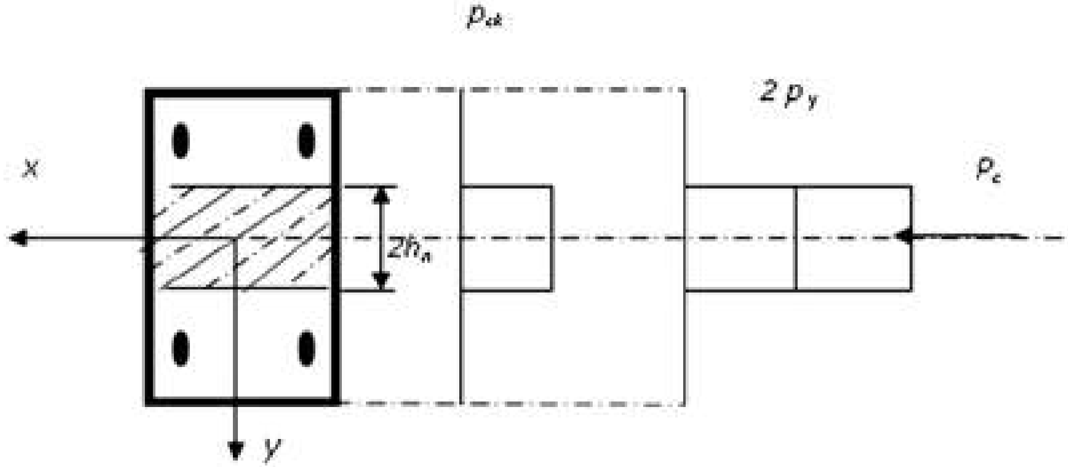


Fig. II.2 Variation in the Neutral Axis Positions

The locations for location of neutral axis for different types of composite columns have been given in eqn. II.1 to II.7 of IRC:22.

APPENDIX – III

MATERIALS AND PROPERTIES

C.III.0 General Introduction III.0

The general specification and properties of materials of construction, such as structural steel, casting & forgings, welding, wire ropes for cables, concrete of various grades, reinforcement steel for concrete, etc. has been elaborated in detail in Annexure III of IRC:22.

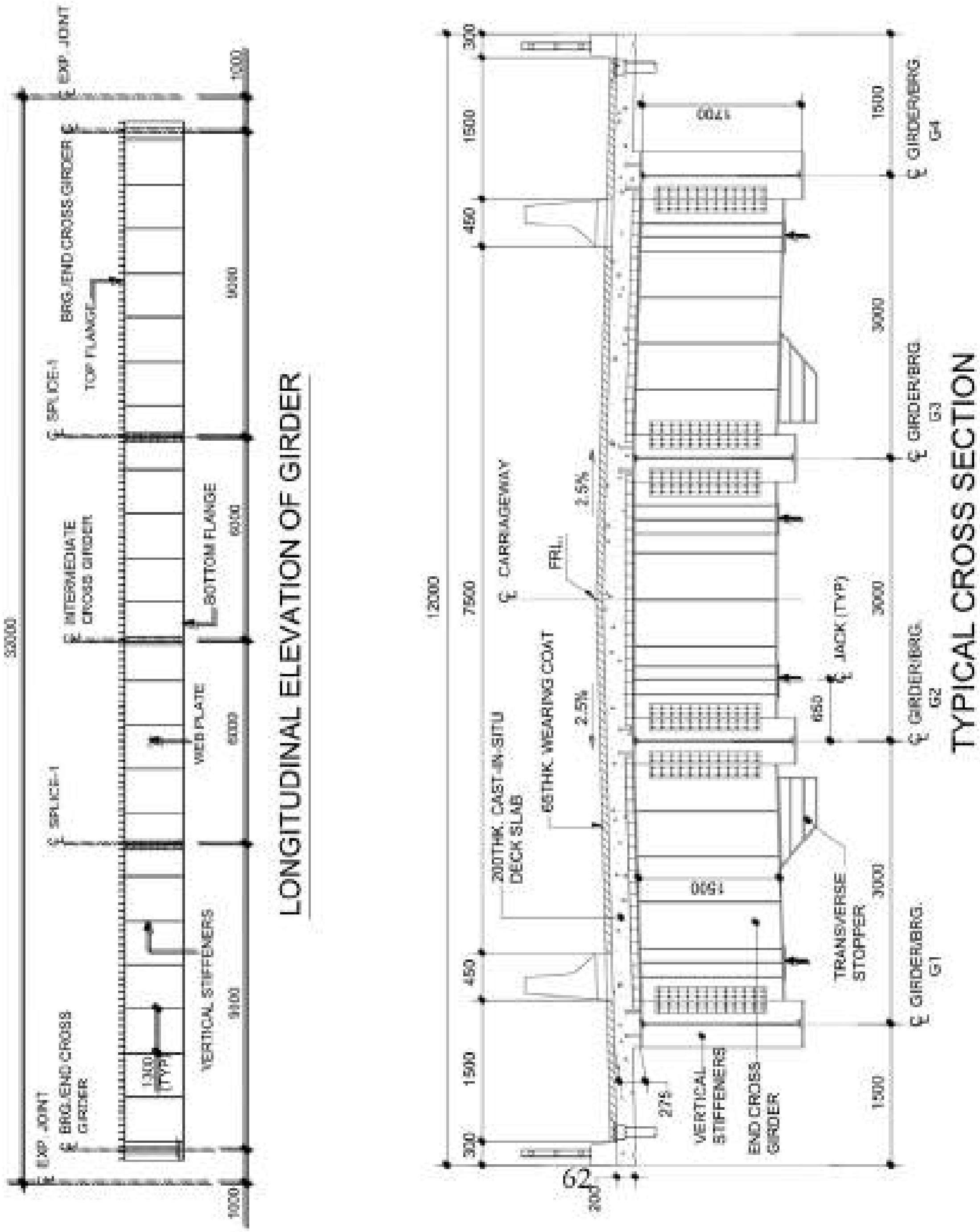
APPENDIX – IV

STANDARD WORKED-OUT EXAMPLE

1 INTRODUCTION

1.1 Description of the Bridge

The proposed 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 over a stream near Delhi. 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 proposed 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. Sketch below shows the proposed structural scheme for Superstructure. Corrugated Profiled Sheet is proposed to be used for the casting of deck slab.



1.2 Scope

A steel concrete composite plate girder superstructure is analyzed and designed in this worked example. The scope of work is kept limited to the design of structural steel components only. The design is presented for a typical girder (outer girder G1) to highlight the steps involved in designing a plate girder.

2 DESIGN BASIS

Reference Codes:
- IRC:6-2017
- IRC:22-2015
- IRC:24-2010
- IRC:112-2011
- IS:2062-2011

The bridge is to be designed in accordance with the provisions of relevant IRC Codes. The basis of design set out in IRC:22-2015 is verification by limit state method of design. Actions, Action combinations and partial load factors are considered as per IRC:6-2017.

STRUCTURAL MATERIAL PROPERTIES

Following structural material grade will be used:

Structural steel : Grade E350 BR : conforming to IS 2062-2011

For structural steel, the value of f_y depends on the product standard.

(Use 320 N/mm^2 for t > 40mm, 330 N/mm^2 for 20mm < t ≤ 40mm and 350 N/mm^2 for t < 20 mm)

Concrete : M40 : conforming to IRC:112-2011
f_{ck} = 40 MPa
Reinforcement : Fe 500 D : conforming to IS:1786-2008
f_{yk} = 500 N/mm^2

The modulus of elasticity of both structural steel and reinforcement steel is taken as 200 GPa (as permitted by IRC:112 for reinforcement and IRC:24 for steel).

For concrete, it is assumed that the average age at first loading is the same and thus the values of the modulus of elasticity of the concrete and modular ratio are:

Component	Short term (Transient Loading)	Long term (Permanent Loading)
Mean E_{cm} Modular ratio, m	33 GPa m = 7.5	16.5 GPa m = 15.0

In this example, the shrinkage effects will be considered at their long-term values where they are unfavourable.

3 ACTIONS ON THE BRIDGE

A) Self Weight of Structural Elements

The 'density' of steel is taken as 7.85 t/m^3 and the density of reinforced concrete is taken as 2.5 t/m^3. The self-weights are based on nominal dimensions. Additionally, to account for concrete in profile sheet, an average thickness of 40 mm concrete is assumed.

B) Weight of Crash Barrier & Railing

The weight of normal containment crash barrier is assumed as 0.75 t/m & that of railing is taken as 0.5 t/m.

C) Self Weight of Surfacing

The total nominal thickness of the surfacing is 65 mm. Assume that the 'density' is 2.2 t/m^3 for the surfacing. The self-weight generally produces adverse effects and for that case the self-weight is based on nominal thickness of 90 mm, to account for future overlay. Thus:

g_k = 0.090 x 2.2 = 0.2 t/m^2

D) Construction Loads

For global analysis, a uniform construction load of Q_{ca} = 150 kg/m^2 is assumed during casting. This is over and above the weight of profile sheet.

E) Traffic Loads

a) Road Traffic

Normal traffic is represented by loads as given in IRC:6-2017. The bridge is designed for Class A (1 Lane/2 Lane) or Class 70R (Wheeled) loadings. Since, the bridge is not located in areas close to any ports, heavy industries and mines, congestion factor as per Clause 204.4 of IRC:6 is not considered applicable for this bridge. Also SV loading as per clause 204.5 is not considered applicable for this bridge.

b) Pedestrian Traffic

Pedestrian traffic is represented by the Basic load intensity of 400 kg/m^2 over the footway area, reduced for global analysis to 240 kg/m^2 as per clause 206.3 of IRC:6-2017.

F) Thermal Actions

a) Range of Effective Bridge Temperature:

(Uniform Temperature Variation)

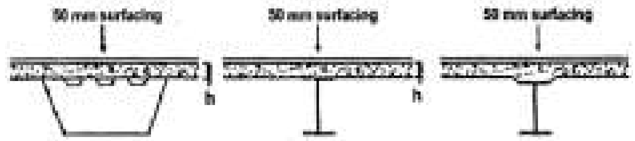
Maximum and minimum shade temperatures, based on a 50-year return period are defined in IRC:6-2017 (Clause 215.2). For this bridge location, the values are:

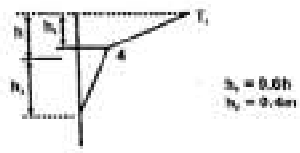
- Maximum 48.4°C
- Minimum -2.2°C
- Mean Temperature (Average) = 1/2x(48.4 – (-2.2)) = 25.30°C Say 26°C
- Bridge temperature to be assumed when the structure is effectively restrained = 26 ± 10 = 36°C and 16°C
- Design Temperature Variation to be considered = 26 + 10 = 36°C.

This does not have any impact on the design of superstructure.

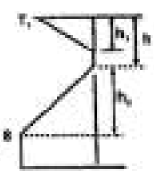
b) **Temperature Difference Loading :**
(Vertical Temperature Gradient)

Clause 215.3 of IRC:6-2017 has been considered for Thermal Gradient Loading. For change of length in composite sections, the coefficient of linear thermal expansion is 12 x 10^{-6} per °C.





H (m)	T_1 (°C)
0.2	18
0.3	20.5



h (m)	T_1 (°C)
0.2	4.4
0.3	6.8

Temperature Differences Across Steel and Composite Section

G) Wind Loads

For wind load assessment, Wind Load as given in Clause 209 of IRC:6-2017 is used. The basic wind speed for Delhi is 47 m/s with average height of exposed surface as 10 m (assumed). For this, wind load at two instances is considered as follows:

a) **Wind with LL:** As per clause 209.3.7, the wind load at deck level (V_z) when live load is considered should be taken as 36 m/s with pressure intensity as 779 N/m^2.

Hence, $F_v = P_z \times A_3 \times G \times C_L = 0.078 \times A_3 \times 2 \times 0.75 = 0.117$ t/m^2

b) **Wind without LL:** As per basic wind speed map as shown in Fig. 10 of IRC:6-2017 with pressure intensity as 940 N/m^2.

Hence, $F_v = P_z \times A_3 \times G \times C_L = 0.094 \times A_3 \times 2 \times 0.75 = 0.141$ t/m^2

H) Fatigue Loads

For fatigue assessment, Fatigue Load as given in Clause 204.6 of IRC:6-2017 is used.

4 GIRDER CROSS SECTIONS

4.1 Section Properties – Main Girders (Outer Girder)

Clause 603.2.1, Pg. 16. IRC:22 - 2015

For determination of stress in the cross section and resistances of the cross section, the effective width of the slab, allowing for shear lag is needed. The following calculations summarize the effective section properties for the sections considered.

The equivalent spans for effective width, Lo = 30 m

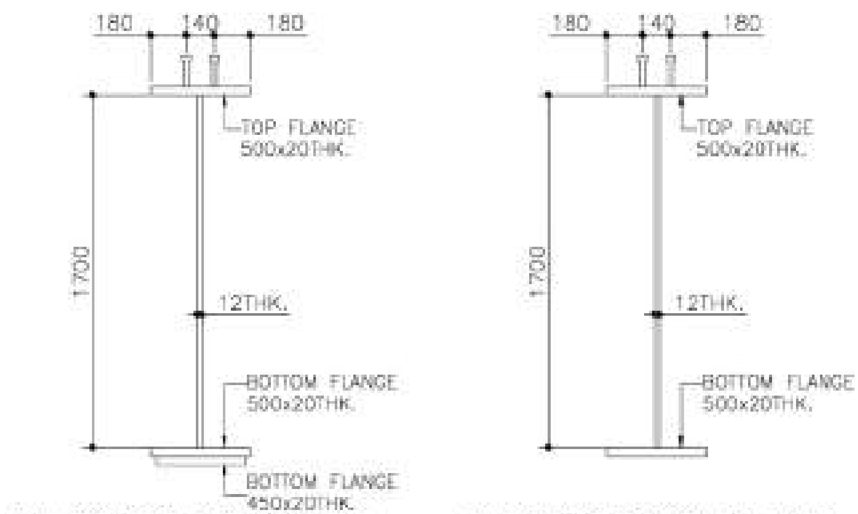
B_1 = 3.0 m

B_o = 1.50 m

X = Lo/8= ≤ Bo = 1.50 m

b_{eff} = Lo/8 + X ≤ (B_1/2 + X) = 3.0 m

Classification of cross sections is determined separately for bending in accordance with Clause 603.1.4 of IRC:22 of the code.



TYP. CROSS SECTION OF LONG. GIRDER AT MID SPAN

TYP. CROSS SECTION OF LONG. GIRDER AT SUPPORT

Section at Midspan

Element	**B**	**t**	**b/t**	**During Construction**	**After Slab is Hardened**
Top flange	500	20	10.3	Semi-compact section	Plastic section
Web	1700	12	141.7	Slender section	Slender section
Bottom flange	500	20	---	---	---
	450	20			
				Slender section	**Slender section**

Section at Support

Element	B	t	b/t	During Construction	After Slab is Hardened
Top flange	500	20	10.3	Semi-compact section	Plastic section
Web	1700	12	141.7	Slender section	Slender section
Bottom flange	500	20	---	---	---
				Slender section	**Slender section**

Section at Splice

Element	B	t	b/t	During Construction	After Slab is Hardened
Top flange*	396	20	7.7	Semi-compact section	Plastic section
Web	1700	12	141.7	Slender section	Slender section
Bottom flange*	396	20	---	---	---
	346	20			
				Slender section	**Slender section**

*Area of 4 holes of 26 mm dia. deducted for 24 mm dia. bolts.

Note :

1) 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.

2) The width of web more than the semi compact limit is deducted as per clause 603.1.3 of IRC:22-2015. Hence, the slender section is now converted to a semi-compact section and checked as a semi-compact section.

Cross section properties :

Bare steel cross sections - Section properties for Outer Girder

Component	Symbol Used	At Mid-span	At Support	At Splice	Unit
Area	A	49400	40400	43200	mm^2
Height of NA from bottom	yb	671	809	682	mm
Second moment of area	$I_{z\text{-}z}$	2.37E+10	1.85E+10	1.95E+10	mm^4
Z, centriod top flange	$W_{bf,y}$	2.18E+07	1.99E+07	1.81E+07	mm^3
Z, centriod bottom flange	$W_{tf,y}$	3.53E+07	2.29E+07	2.85E+07	mm^3
Section classification	---	Semi-Compact	Semi-Compact	Semi-Compact	---
Ultimate bending resistance	M_{pl}	546	485	414	tm

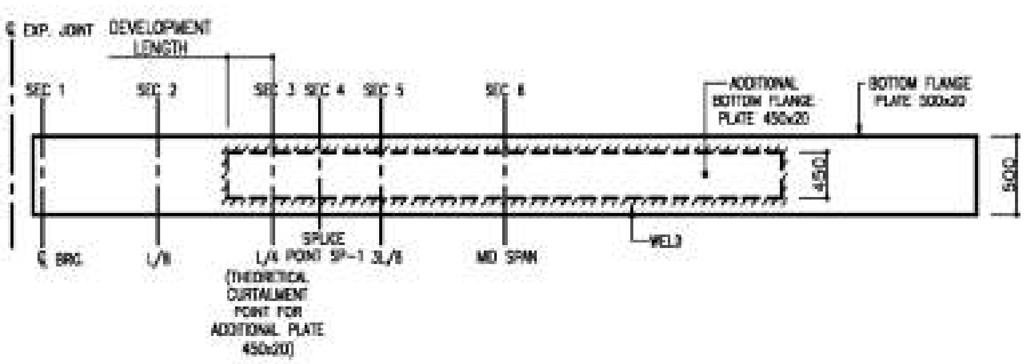
Composite X section –Section Property - short term (m = 7.5) [Sagging]

Component	Symbol Used	At Mid-span	At Support	At Splice	Unit
Area	A	137400	123800	131200	mm^2
Height of NA	y	1466	1549	1508	mm
Second moment of area	$I_{z\text{-}z}$	6.67E+10	4.64E+10	5.76E+10	mm^4
Z, top of slab	W_c	12.97E+07	11.29E+07	12.22E+07	mm^3
Z, top flange	$W_{bf,y}$	22.67E+07	24.31E+07	22.89E+07	mm^3
Z, bottom flange	$W_{tf,y}$	4.55E+07	3.00E+07	3.82E+07	mm^3
Section Classification	---	Semi-Compact	Semi-Compact	Semi-Compact	---
Ultimate bending resistance	M_{pl}	1584	1123	1370	tm

Composite X section –Section Property - Long term (m = 15.0) [Sagging]

Component	Symbol Used	At Mid-span	At Support	At Splice	Unit
Area	A	93400	84400	87200	mm^2
Height of NA	y	1265	1383	1315	mm
Second moment of area	$I_{z\text{-}z}$	5.58E+10	4.00E+10	4.86E+10	mm^4
Z, top of slab	W_c	7.81E+07	6.94E+07	7.32E+07	mm^3
Z, top flange	$W_{tf,z}$	11.28E+07	11.22E+07	10.94E+07	mm^3
Z, bottom flange	$W_{bf,z}$	4.41E+07	2.90E+07	3.70E+07	mm^3
Section Classification	---	Semi-Compact	Semi-Compact	Semi-Compact	---
Ultimate bending resistance	M_{pl}	1584	1123	1370	tm

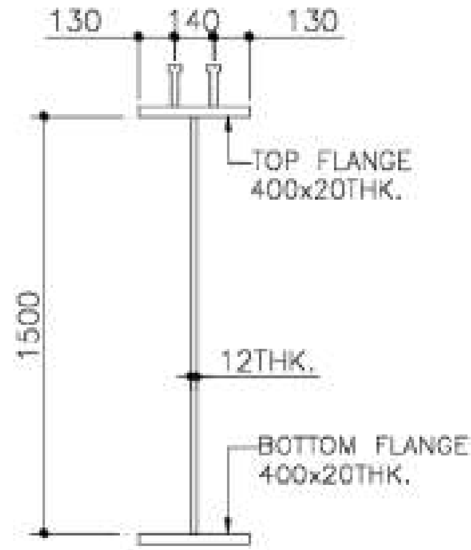
In this example, only bottom flange is curtailed whereas the web and top flange are kept constant throughout. A plan of the bottom flange to show its arrangement is presented below:



PLAN OF BOTTOM FLANGE : GIRDER G1

4.2 Sections Properties – Cross Girders

The gross composite section of the cross girder includes part width of slab towards expansion gap and effective width of slab towards the mid span.



TYP. CROSS SECTION
OF CROSS GIRDER

The section properties of the cross girder are:

Component	Symbol Used	Bare Steel Section	Composite X section Short term	Composite X section Long term	Unit
Area	A	34000	48000	41000	mm^2
Height of NA	y	740	1015	902	mm
Second moment of area	$I_{z\text{-}z}$	1.22E+10	2.01E+10	1.68E+10	mm^4
Z, Top of slab	W_c	---	2.77E+07	2.01E+07	mm^3
Z, centriod top flange	$W_{bf,y}$	1.52E+07	3.83E+07	2.64E+07	mm^3
Z, centriod bottom flange	$W_{tf,y}$	1.64E+07	1.98E+07	1.87E+07	mm^3
Section class	---	Semi-Compact	Semi-Compact	Semi-Compact	---

4.3 Minimum Web Thickness

As per clause 509.6.1 of IRC:24-2010, provisions for minimum web thickness are as follows :

S. No.	Component	Unit	Main Girder	Cross Girder
1	f_{yw}	MPa	350	350
2	ε_w	---	0.845	0.845
3	f_{yf}	MPa	330	330

S. No.	Component	Unit	Main Girder	Cross Girder
4	ε_f	---	0.87	0.87
5	t_w	mm	12	12
6	c	mm	1300	1300
7	d	mm	1700	1500
8	c/d	---	0.76	0.86
A) Serviceability requirement : Since $0.74d \le c < d$, clause 509.6.1.1, b, ii) is applicable i.e., $c/t_w \le 200\ \varepsilon_w$ or 169.				
9	Min. t_w	mm	7.69	7.69
B) Compression flange buckling requirement : Since $c < 1.5d$, clause 509.6.1.2, b, ii) is applicable i.e., $d/t_w \le 345\ \varepsilon_f$ or 300.				
10	Min. t_w	mm	5.66	5.0
Hence, minimum thickness of web required :				
11	Min. t_w (max of A or B)	mm	7.69	7.69

5 PRIMARY EFFECT OF TEMPERATURE DIFFERENCE & SHRINKAGE

A) Temperature Difference

For calculation of primary effects, use the short-term modulus for concrete:

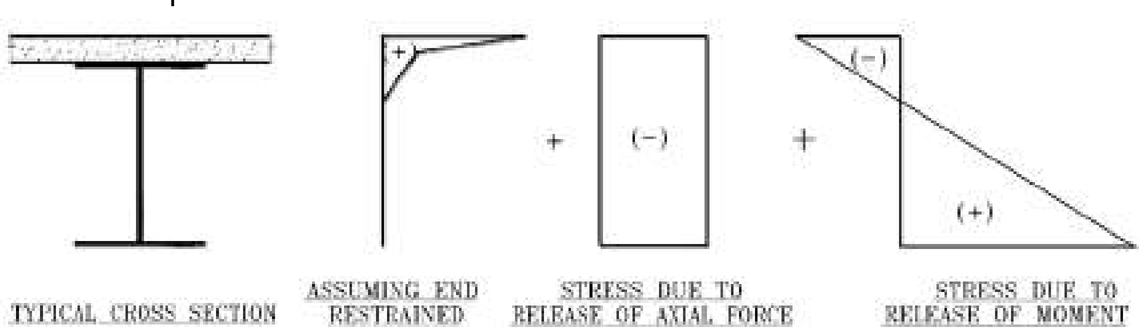
E_{cs} = 33 GPa (For steel, E = 200 GPa),

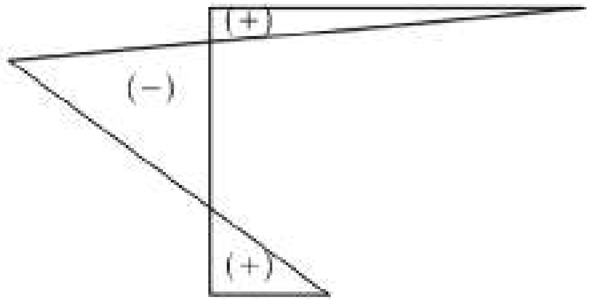
Coefficient of thermal expansion = .000012

Note: For each element of section, calculate stress as strain x modulus of elasticity, then determine force and centre of force for that area.

Note that the full width of slab from the centre line to the edge of the cantilever is used to determine the full effects of restraint. The stress and force are illustrated diagrammatically below:

a) Temperature Rise



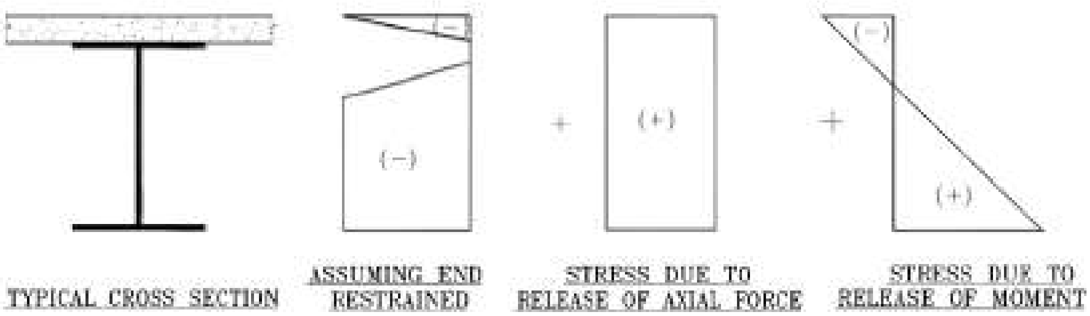


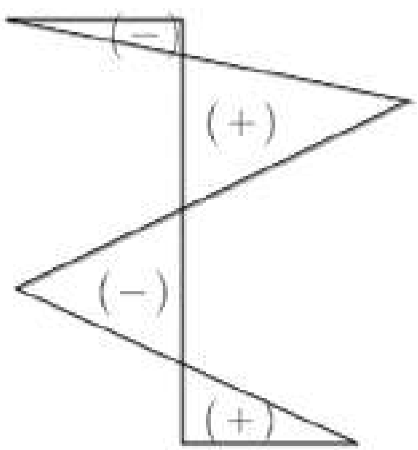
FINAL STRESS

Based on the above the stress at various sections for temperature rise along the cross section are calculated and tabulated below:

Component	**Unit**	**At Mid-span**	**At Support**	**At Splice**
Force	t	191.88	191.88	191.88
Moment	t m	83.16	63.31	74.99
$\sigma_{t\ slab}$	t/m^2	2565.1	2330.2	2521.2
$\sigma_{b\ slab}$ / $\sigma_{t\ girder}$	t/m^2	-1009.2	-1218.3	-1041.2
$\sigma_{b\ girder}$	t/m^2	381.9	353.1	446.1

b) Temperature Fall





FINAL STRESS

Based on the above the stress at various sections for temperature fall along the cross section are calculated and tabulated below:

Component	Unit	At Mid-span	At Support	At Splice
Force	t	-108.68	-108.68	-108.68
Moment	t m	60.67	44.41	65.30
$\sigma_{t\ slab}$	t/m^2	-961.4	-1000.4	-987.8
$\sigma_{b\ slab}$ / $\sigma_{t\ girder}$	t/m^2	550.8	538.6	573.5
$\sigma_{b\ girder}$	t/m^2	192.8	375.0	609.0

B) Shrinkage

Shrinkage effects should be calculated at two instances (1) at the time of opening to traffic and (2) at the end of the service life and the more onerous values used. Shrinkage stresses are considered only for SLS checks. The characteristic value of shrinkage strain is calculated as per IRC:112 and presented below:

S. No.	Component	At the time of opening of traffic (t = 90 days)	At the end of service life (t = ∞)
1	Autogenous shrinkage	30.65×10^{-6}	38.9×10^{-6}
2	Drying shrinkage	144.38×10^{-6}	396.1×10^{-6}
3	Total shrinkage	175.03×10^{-6}	435.0×10^{-6}
4	Creep coefficient	1.838	3.366
5	Creep factor	0.458	0.287
6	Creep factor x Total shrinkage	80.08×10^{-6}	124.76×10^{-6}

Hence, it can be seen that the shrinkage effects are more critical at the end of service life.

Note :

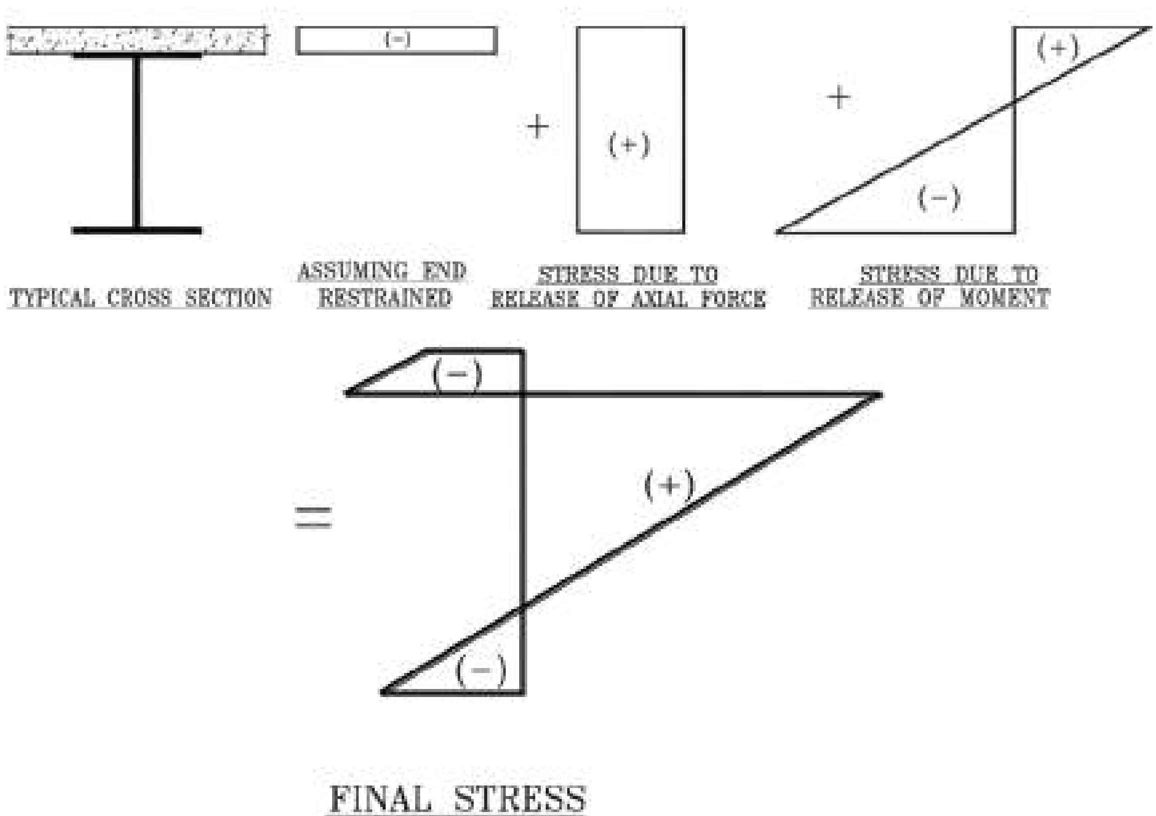
(1) The composite action is considered after the concrete has attained at least 75% of its strength. (t = 3 days).

(2) The Autogenous shrinkage strains are considered from the moment the concrete is poured (t = 3 days).

(3) The Drying shrinkage strains are considered from the end of curing of concrete (t = 14 days).

(4) The effect of creep is considered from the time concrete is hardened (t = 3 days).

(5) Relative humidity is considered as 50%.



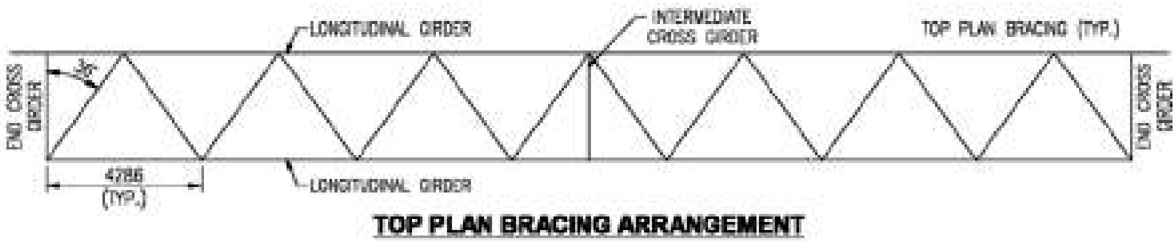
Based on the above the stress at various sections for temperature rise along the cross section are calculated and tabulated below:

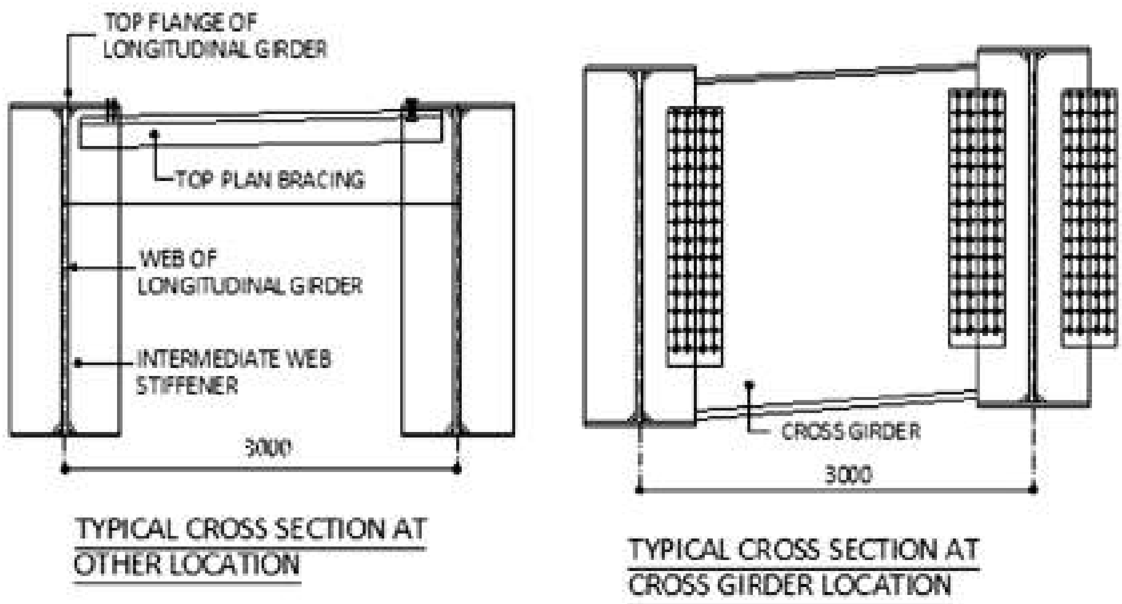
Component	**Unit**	**At Mid-span**	**At Support**	**At Splice**
Force	t	-277.15	-277.15	-277.15
Moment	t m	-167.60	-153.71	-129.38
$\sigma_{t\ slab}$	t/m^2	-1030.2	-841.1	-961.9
$\sigma_{b\ slab}$	t/m^2	-1691.2	-1536.4	-1672.6
$\sigma_{t\ girder}$	t/m^2	4607.7	4762.5	4626.2
$\sigma_{b\ girder}$	t/m^2	-680.1	-800.1	-995.3

6 GLOBAL ANALYSIS

6.1 Construction Stage Analysis

A construction stage is checked to establish whether the sections are feasible during erection of girders. For this it is assumed that two girders braced together will be lifted and placed on temporary/permanent girders. It is important to check this stage as the girder may fail (locally buckle under its own weight) just after it is placed.

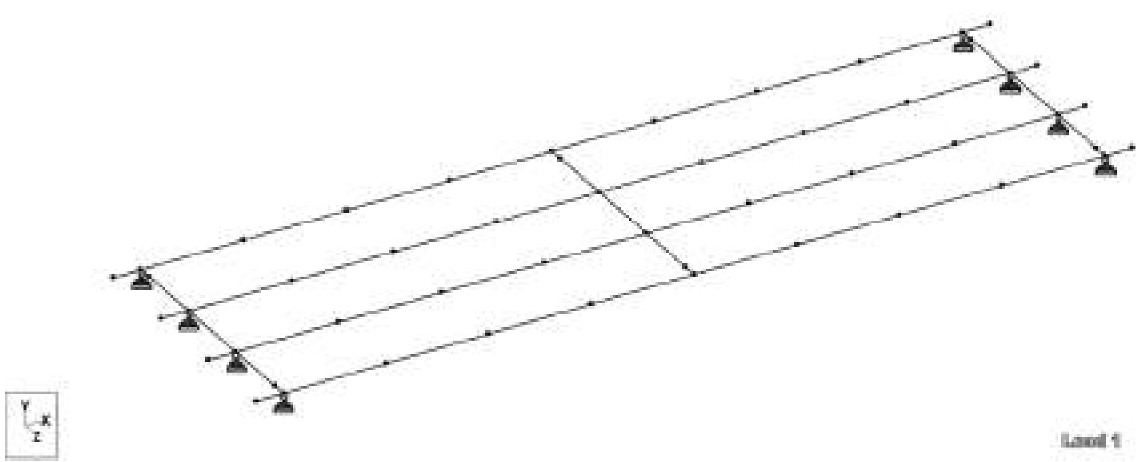




6.2 Grillage Analysis

A 2D Grillage model of the structure was created, as shown below. The model is prepared considering the deck slab as either a wet concrete or hardened concrete depending on age of loading. This has led to two different types of model and three different types of properties:

Model 1 Non-composite steel T girder properties (Bare Steel):
for self-weight of steel sections & deck slab loading

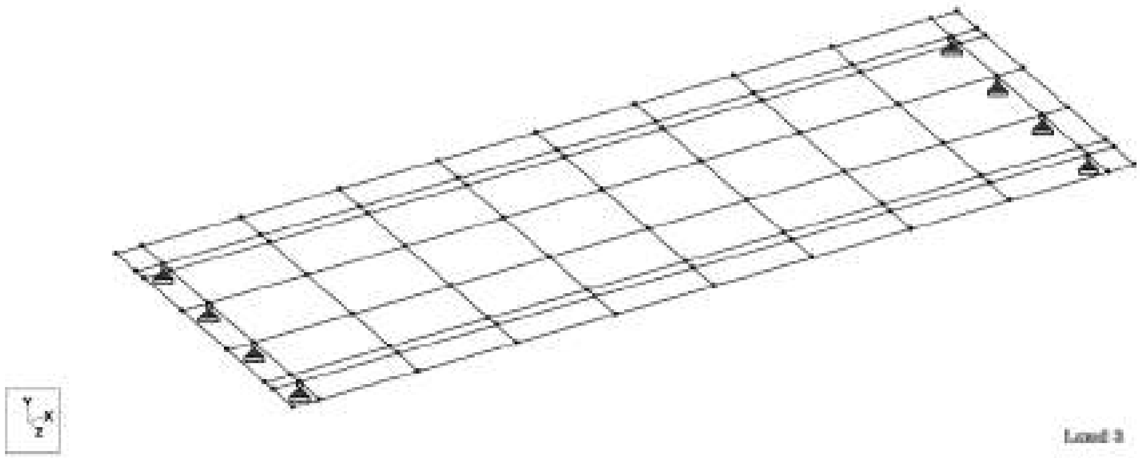


Analysis model, showing Grillage model with Bare Steel properties

Model 2 Composite steel T girder properties:

(a) for permanent imposed loading (with Long term properties)

(b) for live load & wind load (with Short term properties)



Analysis model, showing Grillage model with Composite properties

The global analysis, done on two separate models with appropriate properties effectively gives a comprehensive pattern of forces in all elements in the model. The extracted effects on outer girder section (as defined above) were combinations of moment and shear force.

6.3 Analysis Results

The following table presents the summary of unfactored forces for the primary forces taken from STAAD:

Component			Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
Distance from Left (m)			0.00	3.75	7.50	9.00	11.25	15.00
Staad Member No.			2	3	4	4	5	6
Distance from Start (m)			0.00	0.00	0.00	1.50	0.00	0.00
Self wt.		BM (tm)	0.0	24.1	42.6	48.5	54.6	59.6
		SF (t)	7.3	5.7	4.2	3.4	2.3	0.4
Deck slab		BM (tm)	0.0	95.0	163.6	184.0	204.7	218.4
		SF (t)	29.3	22.0	14.7	11.6	7.4	0.0
SIDL		BM (tm)	0.0	42.7	65.9	67.9	70.6	56.9
		SF (t)	11.2	11.2	6.3	1.3	1.3	3.7
Surfacing		BM (tm)	0.0	29.3	49.9	55.2	62.0	65.5
		SF (t)	8.1	7.5	5.2	3.3	3.0	0.7
CWLL with Impact	(Hog.)	BM (tm)	-3.9	-3.2	-3.2	-3.2	-3.2	-3.8
	(Sag.)	BM (tm)	0.0	81.1	158.6	186.0	224.8	277.5
		SF (t)	21.7	21.7	20.8	19.2	19.2	16.7

Component		Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
FPLL	BM (tm)	0.0	15.4	23.8	24.7	25.2	19.6
	SF (t)	4.5	2.6	0.7	0.5	1.1	1.9
Wind with LL	BM (tm)	0.0	17.2	29.2	32.3	36.3	38.3
	SF (t)	4.8	4.4	3.1	2.0	1.8	0.4
Wind w/o LL	BM (tm)	0.1	20.6	35.1	38.9	43.6	46.1
	SF (t)	5.7	5.3	2.5	2.3	0.9	0.5
Construction load	BM (tm)	-0.2	21.9	37.7	42.5	47.2	50.4
	SF (t)	6.8	5.1	3.4	2.7	1.7	0.0

7 DESIGN FOR LIMIT STATE

7.1 Limit State of Serviceability (SLS)

In this limit state the following conditions are checked:

A) The stresses in structural steel & cast in situ concrete slab are within the prescribed limit (clause 604.3.1, pg.23, IRC:22-2015).

B) The deflection is within the prescribed limit (clause 604.3.2, pg.24, IRC:22-2015).

(A) STRESS LIMITATION

The load combinations adopted for this are as follows:

S. No.	Loads	SLS load combinations				
		Rare combination				
		With LL				W/o LL
		LL leading with wind	Wind leading with LL	Thermal leading with LL	LL leading with thermal	Wind leading
		LC 1	LC 2	LC 3	LC 4	LC 5
1	Self weight	1	1	1	1	1
2	Deck slab	1	1	1	1	1
3	Crash barrier & railing	1	1	1	1	1
4	Surfacing	1.2	1.2	1.2	1.2	1.2
5	CWLL	1	0.75	0.75	1	0
6	Wind with LL	0.6	1	0	0	0
7	Wind w/o LL	0	0	0	0	1

S. No.	Loads	SLS load combinations				
		Rare combination				
		With LL				W/o LL
		LL leading with wind	Wind leading with LL	Thermal leading with LL	LL leading with thermal	Wind leading
		LC 1	LC 2	LC 3	LC 4	LC 5
8	Temperature differences*	0	0	1	0.6	0
9	Differential shrinkage*	1	1	1	1	1

* Effect of Eigen Stress Evaluated Separately

A sample calculation of stress is presented below:

Section 6 at midspan (max tensile stress in girder: LC 1)

Stress @	Unit	Self wt.	Deck slab	SIDL	SURF.	CWLL + FPLL	Wind	Diff. Shrink.
M	t m	59.6	218.4	56.9	78.6	297.1	23.0	-
$\sigma_{t\ slab}$	t/m^2	-	-	49	67	305	24	-69
$\sigma_{b\ slab}$	t/m^2	-	-	34	46	175	14	-113
$\sigma_{t\ girder}$	t/m^2	2737	10028	505	697	1091	101	4608
$\sigma_{b\ girder}$	t/m^2	-1687	-6183	-1291	-1783	-7441	-505	-680

The forces at all sections are calculated in the same manner and are tabulated below:

Max Bending Compressive Stress

Stress @	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at mid-span
$\sigma_{t\ slab}$	t/m^2	274.6	434.8	556.9	574.7	589.9	618.1
$\sigma_{b\ slab}$	t/m^2	---	47.0	112.0	147.9	164.1	186.2
$\sigma_{t\ girder}$	t/m^2	5170.2	12156.0	17240.5	20117.0	19233.2	20215.4
$\sigma_{b\ girder}$	t/m^2	---	---	---	---	---	---
Max Combined Compressive Stress							
$\tau_{b\ girder}$	t/m^2	4245.1	3668.6	2686.3	2017.6	1763.7	1165.7
f_c	t/m^2	8988.5	13716.6	17857.3	20418.3	19474.3	20316.0

Permissible Compressive Stress : (a) Slab $= 0.48f_{ck}$ = 1920 t/m^2

(b) Girder $= f_{yk}$ = 33000 t/m^2

Max Bending Tensile Stress

Stress @	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at mid-span
$\sigma_{t\ slab}$	t/m^2	-199.4	-124.0	-75.5	-68.2	-75.5	-84.1
$\sigma_{b\ slab}$	t/m^2	-247.5	-201.0	-172.5	-160.9	-163.0	-168.9
$\sigma_{t\ girder}$	t/m^2	---	---	---	---	---	---
$\sigma_{b\ girder}$	t/m^2	-334.2	-12462.7	-21037.5	-18595.2	-17280.2	-18655.2
Max Combined Tensile Stress							
$\tau_{b\ girder}$	t/m^2	4245.1	3668.6	2686.3	2017.6	1763.7	1165.7
f_{et}	t/m^2	7360.3	13989.1	21545.8	18920.7	17548.2	18764.1

Permissible Tensile Stress : (a) Slab r/f = 0.60f_y = -30000 t/m^2

(b) Girder = f_{yk} = -33000 t/m^2

(B) LIMIT OF DEFLECTION & CAMBER

As per clause 504.5 of IRC:24-2010

Allowable deflection for total load = span/600 = 30000/600 = 50mm

Allowable deflection for live load and impact = span/800 = 30000/800 = 37.5mm

Deflection Check for Girder G1 : At mid-span

S. No.	Deflection due to	mm
1	Self-weight of girder	11.5
2	Deck slab weight	43.2
3	SIDL	6.4
4	Surfacing	5.6
5	CWLL	16.6
6	FPLL	2.0
Deflection due to total loads		85.3
Deflection due to live load and impact		18.6

Since the deflection due to total load is exceeding the allowable deflection, an in-place pre-camber needs to be provided to compensate for all dead load and super imposed dead load deflections.

Pre-camber calculation:

		SP - 1	Mid span
S. No.	**Deflection due to**	**mm**	**mm**
1	Self-weight of girder	9.4	11.5
2	Deck slab weight	35.6	43.2
3	SIDL	5.4	6.4
4	Surfacing	4.8	5.6
Deflection due to total loads		55.2	66.7

Hence, a pre-camber of 56 mm is provided.

7.2 Limit State of Strength (ULS)

The load combinations adopted for this are as follows:

S. No.	Loads	ULS load combinations										
		Basic										Cons. Stage
		With LL								W/o LL		
		LL leading with wind		Wind leading with LL		Thermal leading with LL		LL leading with thermal		Wind w/o LL		
		1	2	3	4	5	6	7	8	9	10	11
1	Self-weight	1.35	1	1.35	1	1.35	1	1.35	1	1.35	1	1.05
2	Deck slab	1.35	1	1.35	1	1.35	1	1.35	1	1.35	1	1.05
3	Crash barrier & railing	1.35	1	1.35	1	1.35	1	1.35	1	1.35	1	0
4	Surfacing	1.75	1	1.75	1	1.75	1	1.75	1	1.75	1	0
5	CWLL	1.5	1.5	1.15	1.15	1.15	1.15	1.5	1.5	0	0	0
6	Wind with LL	0.9	0.9	1.5	1.5	0	0	0	0	0	0	0
7	Wind w/o LL	0	0	0	0	0	0	0	0	1.5	1.5	0.7x1.5
8	Temperature differences	0	0	0	0	1.5	1.5	0.9	0.9	0	0	0
9	Construction load	0	0	0	0	0	0	0	0	0	0	1.05

Based on the above load combinations, the factored forces are calculated and the forces critical at each sections are tabulated below:

S. No.	Force	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
CONSTRUCTION STAGE								
1	Critical BM	t m	***-0.1***	***169.7***	***293.0***	***329.6***	***367.6***	***393.2***
Load Combination			LC 11	LC 11	LC 11	LC 11	LC 11	LC 11

S. No.	Force	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
				ULTIMATE STAGE				
2	Critical BM	t m	***-5.9***	***429.9***	***754.5***	***847.3***	***961.5***	***1046.9***
Load Combination			LC 1	LC 1	LC 1	LC 1	LC 1	LC 1
3	Critical SF	t	***122.3***	***106.1***	***78.2***	***59.1***	***52.2***	***35.0***
Load Combination			LC 1	LC 1	LC 1	LC 1	LC 1	LC 1

A) Moment Capacity Check : Construction Stage

A sample calculation is presented for calculation of moment capacity at midspan of girder G1.

Annexure – I, Clause I.5, Pg. 75, IRC:22 - 2015

During construction stage, the top flange is either unrestrained or partially restrained till deck slab is hardened. In case of a beam which is symmetrical only about the minor axis, and bending about major axis, the elastic critical moment for lateral torsional buckling is given by the general equation,

$$M_{cr} = C_1 \frac{\pi^2 EI_y}{(L_{LT})^2}\left\{\left[\left(\frac{K}{K_w}\right)^2 \frac{I_w}{I_y} + \frac{GI_t (L_{LT})^2}{\pi^2 EI_y} + \left(c_2 y_g - c_3 y_j\right)^2\right]^{0.5} - \left(c_2 y_g - c_3 y_j\right)\right\}$$

Annexure – C, Clause C 1.2, Pg. 177, IRC:24 - 2010

where,

C_1 = 1.052, C_2 = 0.382, C_3 = 0.753

(for uniformly distributed loading & partial restraint)

K = 0.75 (considering partial fixity)

K_w = 1.0

b_1 = Width of top flange = 500 mm

b_2 = Width of bottom flange = 500 mm

t_1 = Thickness of top flange = 20 mm

t_2 = Average Thickness of bottom flange

= (500 x 20 + 450 x 20)/500 = 38 mm

d' = distance between the centre of both flanges

= 20/2 + 1700 + 38/2

= 1729 mm

$$\alpha = \frac{1}{1 + (b_1/b_2)^3 (t_1/t_2)}$$

$$= \frac{1}{1 + (500/500)^3 (20/38)}$$

= 0.65

y_g = Distance between shear centre and point of application of load

= $y_t – t_1/2 – \alpha.d'$

= 1088.73 – 20/2 – 0.655 x 1729

= -54.0 mm from centroid

β_f = $I_{fc} /(I_{fc} + I_{ft})$

= 0.000208/(0.000208 + 0.000360)

= 0.366

h_y = distance between shear centre of the two flanges of the cross section

= (20/2 + 1700 + 38/2) = 1729 mm

y_j = $1.0\ (2\beta_f – 1)\ h_y/2.0$ (when $\beta_f \le 0.5$)

= 1.0 (2 x 0.366 – 1) x 1729/2.0

= -231 mm

I_t = $\sum b_t(t_i)^3/3$

= [500 x 20^3 + 1316 x 12^3 + 500 x 20^3 + 450 x 20^3)/3]

= 4.62 x 10^{-6} m^4

I_y = .00057 m^4

I_w = $(1-\beta_f)\ \beta_f\ I_y\ h_y^2$

= (1 - 0.366) x 0.366 x 0.00057 x 1.729^2

= 3.95 x 10^{-4} m^6

L_{LT} = 4.3 m x 1.2 = 5.16 m

(distance between top plan bracings providing lateral restraint at intervals)

Hence,

The elastic critical moment for lateral torsional buckling:

$$M_{cr} = 1.052\ x\ \frac{\pi^2\ x\ 200000 \times 102 \times 0.00057}{(5.16)^2}\left\{\left[\left(\frac{0.70}{1.0}\right)^2 \frac{0.000395}{0.00057} + \frac{0.77\ x\ 10^5 x\ 102\ x\ 4.62\ x\ 10^{-6}\ (5.16)^2}{\pi^2 \times 200000\ x\ 102\ x\ 0.00057} + (0.382\ x - 0.054 - 0.753\ x - 0.231)^2\right]^{0.5} - (0.382\ x - 0.054 - 0.753\ x - 0.231)\right\}$$

= 2511.4 tm

Z_e = 0.022 m^4

Z_P = 0.0282 m^4

β_b = Z_e / Z_P = 0.77

The non dimensional slenderness ratio,

Annexure – I, Clause I.5, Pg. 75, IRC:22 - 2015

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y \,/\, M_{cr}}$$

$$\lambda_{LT} = \sqrt{0.77 \times 0.0282 \times 330 \,/\, 2511.4}$$
$$= 0.54$$

$$\emptyset_{LT} = 0.5\left(1 + \alpha_{LT}(\lambda_{LT} - 2.0) + \lambda_{LT}{}^{2}\right)$$
$$= 0.5(1 + 0.49(0.54 - 2.0) + 0.54^2)$$
$$= 0.729$$

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

$$\chi_{LT} = \frac{1}{\{0.729 + [0.729^2 - 0.54^2]^{0.5}\}} = 0.82 \leq 1.0$$

The design buckling resistance moment of laterally unrestrained/partially restrained girder:

$M_{el\,(buck)}$ $= \chi_{LT}\, M_{el}$
$= \chi_{LT}\, \beta_b . Z_P . f_y \,/\, \lambda_{mo}$
$= 0.82 \times 0.77 \times 0.0282 \times 330 \times 102 \,/\, 1.10$
$= 546$ tm

The moment capacity for all sections are calculated in the same manner and are tabulated below :

Component	**Symbol Used**	**At Mid-span**	**At Support**	**At Splice**	**Unit**
Ultimate bending resistance	$M_{el\,(buck)}$	546	485	414	tm

B) Shear Capacity Check

Clause 603.3.3.2, Pg. 18, IRC:22 - 2015

As per clause, 603.3.3.2: The nominal shear strength, V_n may be governed by :

1) Plastic shear resistance, or
2) Strength of web as governed by shear buckling

1) Plastic shear resistance:

Clause 603.3.3.2 (1), Pg. 19, IRC:22 - 2015

The plastic shear resistance is given by:

$V_n = \dfrac{A_v f_{yw}}{\sqrt{3}} = \dfrac{20400 \, x \, 330}{1.732}$

$= 3886722$ N

$= 388.67$ t

where,

f_{yw} = 330 MPa

$A_v = d \, t_w$ = 1700 x 12 = 20400 mm^2

γ_{m0} = 1.1

Now,

$V_d = \dfrac{V_n}{\gamma_{m0}} = \dfrac{388.67}{1.1}$ = 353.3 t

Table 1, Clause 601.4, Pg. 10, IRC:22 - 2015

2) Shear buckling resistance : Simple post-critical method

$\dfrac{d}{t_w} = \dfrac{1700}{12}$ = 141.67

Clause 603.3.3.2 (2), Pg. 19, IRC:22 - 2015

$\dfrac{c}{d} = \dfrac{1300}{1700}$ = 0.76

$K_v = 4 + 5.35 / (c/d)^2$ = 13.15

E = 200000 MPa

μ = 0.3

$\tau_{cr,e} = \dfrac{K_v \pi^2 E}{12(1-\mu^2)(\frac{d}{tw})^2} = \dfrac{13.15 \, x \, 9.87 \, x \, 200000}{12 \, x \, 0.91 \, x \, 20069.4}$ = 118.4 MPa

$\lambda_w = 0.76 \, x \sqrt{f_{yw}/\tau_{cr,e}}$ = 1.189

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

$= [1 - 0.8x(1.189 - 0.8)] \, x \, (330 / 1.732)$ = 118.38 MPa

$A_v = d \, t_w$ = 1700 x 12 = 20400 mm^2

$V_n = V_{cr} = A_v \, \tau_b = 20400 \, x \, 118.38 / 10^4$ = 241.49 t

Table 1, Clause 601.4, Pg. 10, IRC:22 - 2015

γ_{m0} = 1.1

Now,

$V_d = \dfrac{V_n}{\gamma_{m0}} = \dfrac{241.49}{1.1}$ = 219.5 t

Therefore,

S. No.	Method	Shear strength	Unit
1	Plastic shear resistance	353.3	t
2	Shear buckling resistance	219.5	t
3	Governing	219.5	t

Now,

V/V_d = 122.3 / 219.5 = 0.56 < 0.6

Clause 603.3.3.3, Pg. 21, IRC:22 - 2015

Hence, the section is not under high shear force and there is no reduction in the plastic bending resistance of the section because of high shear force.

C) Moment Capacity Check : Ultimate Stage

Annexure – I, Clause I.2, Pg. 71-72, IRC:22 - 2015

A sample calculation is presented for calculation of moment capacity at midspan of girder G1.

y_t = C.G. of steel beam from top of girder
= 1088.7 mm

A_s = Cross sectional area of steel beam
= 500 x 20 + 1316 x 12 + 500 x 20 + 450 x 20
= 44792.3 mm^2

A_f = Area of top flange of steel beam
= 500 x 20
= 10000 mm^2

b_{eff} = 1500 + 3000/2
= 3000 mm

b_f = Width of top flange of steel beam
= 500 mm

d_s = Overall depth of concrete slab
= 220 mm

d_c = Vertical distance between centroids of concrete slab and steel beam
= 1088.7 + 220/2
= 1198.7 mm

t_f = Average thickness of the top flange of the steel section
= 20 mm

t_w = Thickness of the web of the steel section
= 12 mm

α_{cc} = 0.67

γ_c = Material factor of safety for concrete
= 1.50

γ_m = Material factor of safety for structural steel
= 1.10

f_{ck} = 40 MPa

η = 1.00 (for $f_{ck} \leq 60$ MPa)

λ = 0.80 (for $f_{ck} \leq 60$ MPa)

a $= f_y/\gamma_m/\{(\alpha_{cc}/\gamma_c)\ \eta\ \lambda\ (f_{ck})\}$

= 330/1.10/{0.67/1.50) x 1.00 x 0.80 x 40)

= 300/14.293

= 20.99

$b_{eff} d_s$ = 3000 x 220

= 660000 mm^2

$a\ A_s$ = 20.99 x 44792.3

= 940137.4 mm^2

$a\ A_f$ = 20.99 x 10000

= 209888.1 mm^2

Now,

$b_{eff}\ d_s + 2a.A_f$ = 660000 + 2 x 209888.1

$= 1079776.2 > a.A_s > b_{eff}\ d_s$

Hence,

Position of Plastic Neutral axis : **Lies within top flange**

Distance of Plastic Neutral axis (x_u)

$= d_s + \{aA_s - b_{eff}\ d_s)/2b_f\ a$

= 220 + {940137.4 – 660000)/(2 x 500 x 20.99)

= 233.35 mm from top

Moment Capacity M_P

$= f_y\ [A_s.\{d_c + 0.5.d_s\ (1-\lambda)\} - b_f\ (x_u - d_s)\ \{x_u + (1-\lambda)\ d_s\}]/\gamma_m$

= 330 [44792.3 x{1198.7 + 0.5 x 220 (1 – 0.8)} – 500 (233.35 – 220) {233.35 + (1-0.8) x 220)/1.1

= 1584 tm

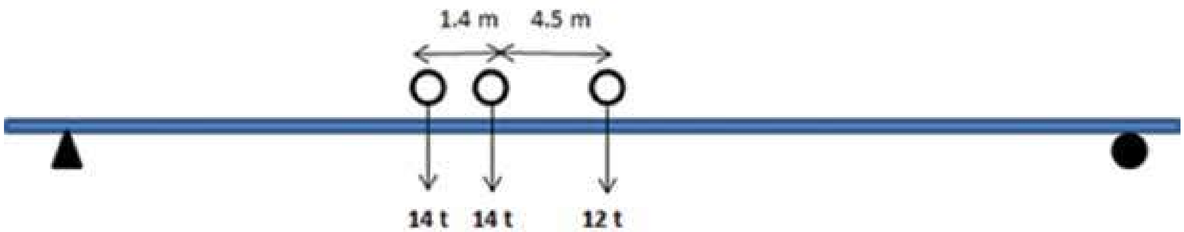
The moment capacity for all sections are calculated in the same manner and are tabulated below:

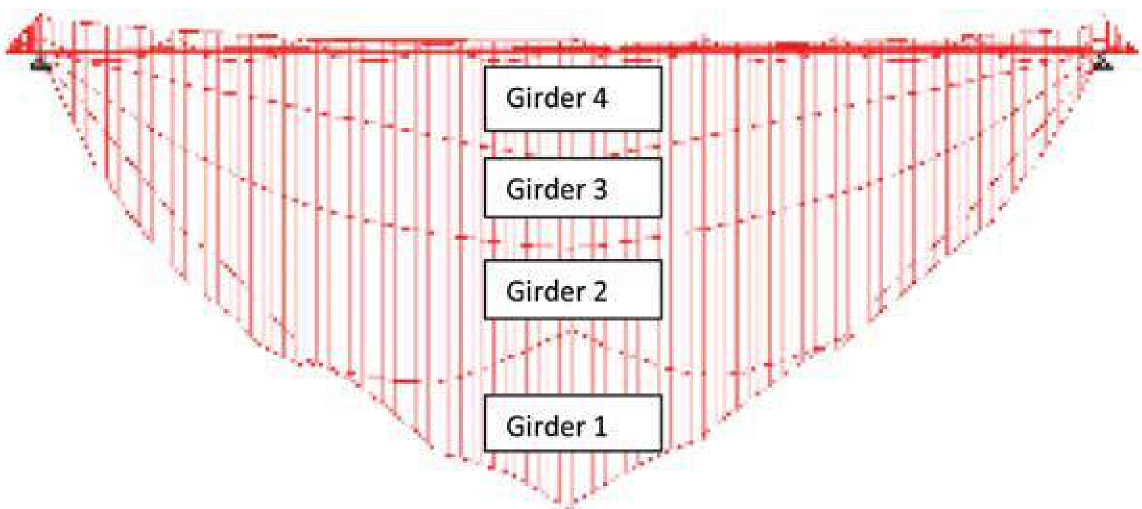
Component	**Symbol Used**	**At Mid-span**	**At Support**	**At Splice**	**Unit**
Ultimate bending resistance	M_{pl}	1584	1123	1370	tm

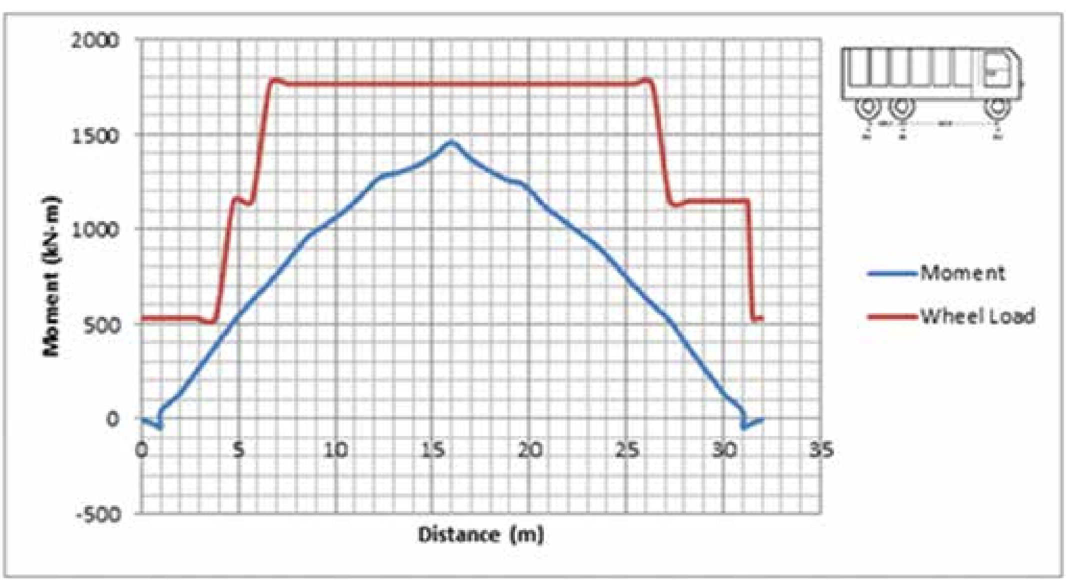
7.3 Limit State of Fatigue

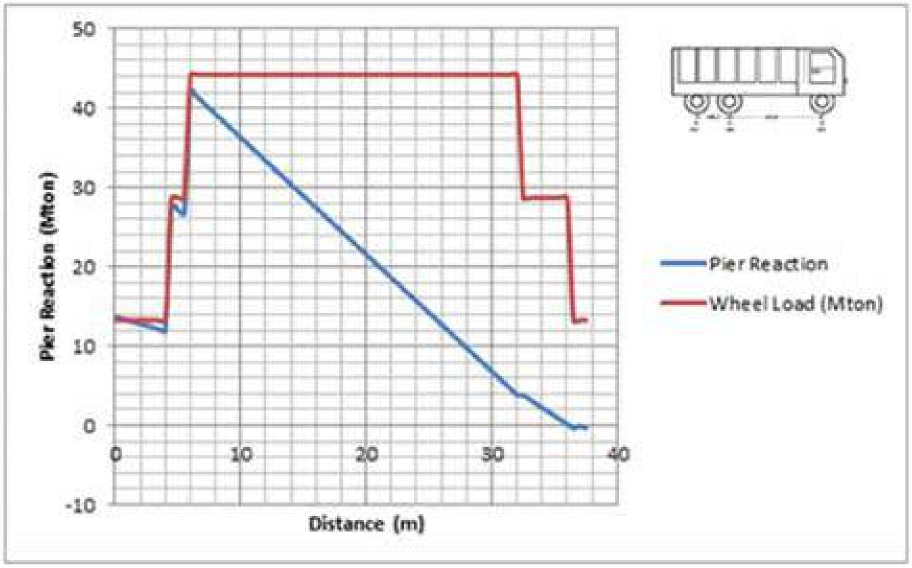
For fatigue assessment, fatigue Load as given in Clause 204.6 of IRC:6-2014 is used. The 40T fatigue vehicle is moved over the girders and the envelope of bending moment for various girders are shown below :


1.35 m
0.525 m
2.47 m
3 m
2.48 m
0.525 m
1.35 m
1 m
3.75 m
3.75 m
3.75 m
3.75 m
3.75 m
3.75 m
3.75 m
3.75 m
1 m


1.4 m
4.5 m
14 t
14 t
12 t


Girder 4
Girder 3
Girder 2
Girder 1


2000
1500
1000
500
0
-500
Moment (kN-m)
0
5
10
15
20
25
30
35
Distance (m)
Moment
Wheel Load



CHECK FOR LOW FATIGUE

Fatigue assessment is not required if following condition is satisfied (IRC:22-2015, Cl 05.2)

i) $f < \dfrac{27}{f\,\gamma_{mft}}$ or

ii) $N_{SC} < 5x10^6 \left[\dfrac{27/\gamma_{mft}}{f\,\gamma_{fft}}\right]^3$

Where,

f = actual fatigue stress range for the detail

N_{SC} = Actual number of stress cycles = $2x10^6$

γ_{fft} = Partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0

γ_{mft} = 1.25 from Table 3 of IRC:22 for Non-Fail-Safe with periodic inspection & maintenance, accessibility to detail is good

Moment at Midspan = $1458x10^6$ N-mm

Section modulus @ bottom of girder = $4.55x10^7$ mm^3

Actual stress range, $f = \dfrac{1458x10^6}{4.55x10^7} = 32.04$ N/mm^2

i) $f > \dfrac{27}{1.25} = 21.6$

ii) $N_{SC} = 5x10^6 \left[\dfrac{27/1.25}{1.0\ x\ 32.04}\right]^3 > 1.53x10^6$

Hence, either condition i) or ii) not satisfied, detailed fatigue assessment is required.

For fatigue loading case,

$$f_{fd} = \frac{\mu_r f_f}{r_{mft}}$$

thickness of flange t_f = 20 mm

The nominal fatigue stress can be worked out as,

$$f_f = f_{fn} \sqrt[3]{5 \times 10^6 / N_{SC}}$$

N_{SC} – 2 x10^6 given in IRC:6-2017 clause 204.6. Assuming continuous longitudinal fillet welds with no stop starts, detail category = 92

$$f_f = 92 \sqrt[3]{\frac{5 \times 10^6}{2x10^6}} = 124.86\, N/mm^2$$

t = 20 mm (provided)

$$\mu_r = \left(25/t_p\right)^{0.25} \leq 1.0$$

$$\mu_r = \left(25/20\right)^{0.25} = 1.05 \leq 1.0$$

γ_{mft} = 1.25

$$f_{fd} = \frac{\mu_r f_f}{\gamma_{mft}} = \frac{1.0\,x\,124.86}{1.25} = 99.89\ N/mm^2$$

Moment capacity of I- Welded plate girder

Ze = 21.8 x 10^6 mm^3

Limit State of fatigue:

M_d = 21.8 x 10^6 x 99.89 = 2178 x 10^6 > 1458 x 10^6 N.mm

Hence, the trial section is adequate.

(i) *For shear stresses (Support section):*

(ii) *Limit state of fatigue:*

Maximum shear force V_{max} = 312 x 10^3 N

∴Maximum shear stress,

$$\tau_{max} = \frac{312x10^3}{1700x12} = 15.29\ N/mm^2$$

From clause 605.2 of IRC:22

$$\mu_r = \left({}^{25}/_{20}\right)^{0.25} = 1.05 \leq 1.0$$

Detail category = 92

$$\tau_f = 92\ \sqrt[5]{\frac{5x10^6}{2x10^6}} = 110.5\ N/mm^2$$

Good maintenance and inspection and accessibility to detail,

Table 3 of IRC:22, r_{mft} = 1.25

$$\tau_{fd} = \frac{1.0\ x\ 110.5}{1.25} = 88.4\ N/mm^2 > 15.29\ N/mm^2$$

Hence, the section provided at support is safe

8 DESIGN OF SHEAR CONNECTOR

Design of Shear Connector: G-1:

Shear connectors are designed for following two considerations:

Clause 606, Pg. 36, IRC:22-2015

1) Design of Shear Connector in Ultimate limit state (Strength criteria)
   a) As per clause 606.4.1, for the actual ultimate shear force due to dead load and live load (including impact)
   b) As per clause 606.4.1.1, for full shear connection
2) Design of Shear Connector in Serviceability limit state (limit state of fatigue):

   As per clause 606.4.2, the shear connectors shall be designed for fatigue due to the effect of Live load and impact

Calculation for spacing of shear connector at support of girder G-1

Criteria 1(a)

Clause 606.4.1, Pg. 40, IRC:22-2015

Design of Shear Connector in Ultimate limit state (Strength criteria) : As per clause 606.4.1 for the actual ultimate shear force due to dead load and live load (including impact)

The calculated Longitudinal shear per unit length at the interface is given by :

$$V_L = \sum \left[\frac{V\ A_{ec}\ Y}{I}\right]_{dl,ll}$$

where,

V_L = Longitudinal shear per unit length

V = The vertical shear forces due to dead load and live load (including impact)

A_{ec} = Area of effective concrete transformed into equivalent steel section on the compression side of the neutral axis.

Y = The distance from the neutral axis of the composite section to the centroid of the transformed area under consideration.

I = The Moment of Inertia of the Composite section using appropriate modular ratio.

Q_u = Ultimate static strength of one shear connector

= 115KN (for 22mm Nominal dia. & M40 concrete)

Table 7, Pg. 38, IRC:22-2015

S. No.	Component	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
1	V_{dl}	t	65.6	65.6	43.1	27.8	20.1	6.8
2	A_{ec}	m^2	0.044	0.044	0.044	0.044	0.044	0.044
3	Y	m	0.43	0.43	0.43	0.52	0.57	0.57
4	I	m^4	0.0400	0.0400	0.0400	0.0486	0.0558	0.0558
5	V_{Ldl}	t	37.1	31.0	20.3	13.0	9.0	3.0
6	V_{ll}	t	39.3	36.5	32.3	29.6	30.5	27.9
7	A_{ec}	m^2	0.088	0.088	0.088	0.088	0.088	0.088
8	Y	m	0.26	0.26	0.26	0.32	0.37	0.37
9	I	m^4	0.0464	0.0464	0.0464	0.0576	0.0667	0.0667
10	V_{Lll}	t	19.6	18.2	16.1	14.6	14.7	13.5
11	V_L	t	56.8	49.2	36.4	27.6	23.7	16.5
12	Q_u	t	11.5	11.5	11.5	11.5	11.5	11.5
13	Nos.	---	2	2	2	2	2	2
14	S_{L1}	mm	405.2	467.8	631.0	832.5	969.4	1391.9

Criteria 1(b)

Design of Shear Connector in Ultimate limit state (Strength criteria) : As per clause 606.4.1.1 for full shear connection

Clause 606.4.1.1, Pg. 40, IRC:22-2015

Longitudinal force due to bending, H = minimum of H_1 & H_2

Where,

A) H_1 = $A_{sl} \cdot f_y \cdot 10^{-3}/\gamma_m$

A_{sl} = Area of tensile steel in longitudinal direction

f_y = 500 MPa

γ_m = 1.25

B) $H_2 = 0.36.f_{ck}.A_{ec}.10^{-3}$

f_{ck} = 40 MPa

A_{ec} = Effective area of concrete

= $b_{eff}.d_s$

Hence, H = minimum of H_1 & H_2

S. No.	Component	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
1	A_{sl}	mm^2	3390	3390	1695	1695	1695	1695
2	f_y	MPa	500	500	500	500	500	500
3	γ_m	---	1.25	1.25	1.25	1.25	1.25	1.25
4	H_1	t	135.6	135.6	67.8	67.8	67.8	67.8
5	f_{ck}	MPa	40	40	40	40	40	40
6	A_{ec}	mm^2	660000	660000	660000	660000	660000	660000
7	H_2	t	950.4	950.4	950.4	950.4	950.4	950.4
8	H	t	135.6	135.6	67.8	67.8	67.8	67.8
9	Q_u	t	11.5	11.5	11.5	11.5	11.5	11.5
10	Nos.	---	2	2	2	2	2	2
11	SL_2	mm	169.6	169.6	339.2	339.2	339.2	339.2

Criteria 2

Clause 606.4.2, Pg. 41, IRC:22-2015

Design of Shear Connector in Serviceability limit state (limit state of fatigue) : The shear connectors shall be designed for fatigue due to the effect of Live load and impact

$$V_r = \sum \left[\frac{V_R\, A_{ec}\, Y}{I}\right]_{ll}$$

where,

V_R = Range of vertical shear i.e. the difference between the maximum and minimum shear envelope due to live load including impact.

A_{ec} = Area of effective concrete transformed into equivalent steel section on the compression side of the neutral axis.

Y = The distance from the neutral axis of the composite section to the centroid of the transformed area under consideration.

I = The Moment of Inertia of the Composite section for short term loading.

Table 8, Pg. 39, IRC:22-2015

Q_r = Nominal fatigue strength of one shear connector.

= 30 KN (for 22mm Nominal dia. & 2 x 106 cycles).

S. No.	Component	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
1	V_{max}	t	21.7	20.8	19.2	19.2	16.7	0.2
2	V_{min}	t	0.3	0.2	0.2	0.2	0.2	16.5
3	V_R	t	22.0	21.0	19.4	19.4	16.9	16.7
4	A_{ec}	m_2	0.088	0.088	0.088	0.088	0.088	0.088
5	Y	m	0.26	0.26	0.26	0.32	0.37	0.37
6	I	m_4	0.0464	0.0464	0.0464	0.0576	0.0667	0.0667
7	V_r	t	10.99	10.49	9.69	9.60	8.18	8.08
8	Q_u	t	3.0	3.0	3.0	3.0	3.0	3.0
9	*Nos.*	---	2	2	2	2	2	2
10	S_R	mm	546.0	572.0	619.2	624.7	733.8	742.5

As per clause 606.4.2 : For full shear connection, the actual spacing to be provided is lowest of {S_{L1}, S_{L2} & S_R}:

S. No.	Component	Unit	Sect. 1 at End support	sect. 2 at L/8	sect. 3 at L/4	sect. 4 at (SP-1)	sect. 5 at 3L/8	sect. 6 at midspan
1	S_{L1}	mm	405.2	467.8	631.0	832.5	969.4	1391.9
2	S_{L2}	mm	169.6	169.6	339.2	339.2	339.2	339.2
3	S_R	mm	546.0	572.0	619.2	624.7	733.8	742.5
4	S	mm	169.6	169.6	339.2	339.2	339.2	339.2

9 DESIGN OF STIFFENERS

The different stiffeners for which design is presented in this example are as follows:

A) Intermediate web stiffener:
Provided at main girders & cross girders to improve the buckling strength.

B) Load carrying stiffeners:
Provided at bearing, cross girder & jack locations.

A) Intermediate Web Stiffener: Longitudinal Girder G1

Clause 509.7.2, Pg. 103, IRC:24 - 2010

c = 1.3 m

d = 1.7 m

ε = $\sqrt{(250/f_y)} = \sqrt{(250/350)}$ = 0.845

K_v = $4.0 + 5.35/(c/d)^2$ = 13.15

tw = 8 mm

c/d = 1.3/1.7 = $0.76 < \sqrt{2}$

Hence,

$$I_s \quad \geq \frac{1.5\ d^3\ t_w{}^3}{c^2} \quad \geq \frac{1.5\ x\ 1700^3\ x\ 8^3}{1300^2} \qquad \geq 22.33 \text{ x } 10^5 \text{ mm}^4$$

Try intermediate stiffener of 75 x 10 mm :

$I_{s\ provided}$ = 2 x {10 x 75^3/12 + 10 x 75 x $(75/2 + 12/2)^2$}
= 35.41 x 105 mm^4

Check for Outstand

Outstand of stiffener = 75 mm
Permissible limit,
$14t_q\varepsilon$ = 14 x 10 x 0.845 = 118.3 mm

Check for Buckling

Stiffeners not subjected to external loads or moments should be checked for a stiffener force: $F_q = V - V_{cr} / \gamma_{mo} \leq F_{qd}$. Since, in this case $V < V_{cr}$, there is no force in stiffener.

B) Load Bearing Stiffener : at Bearing Location
Check for Shear Capacity of End Panel :

Clause 509.5,
Pg. 94,
IRC:24 - 2010

$$H_q \quad = 1.25\ V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$$

where,

V_p = $d.t.f_y/\sqrt{3}$
= 1700 x 12 x $350/\sqrt{3}/10^4$ = 412.2 t

Therefore, Longitudinal shear

$$H_q \quad = 1.25 \text{ x } 412.2 \left(1 - \frac{219.5}{412.2}\right)^{1/2} \qquad = 352.2 \text{ t}$$

R_{tf} = H_q /2 = 261.1/2 = 176.1 t

$$V_n \quad = \frac{d.t.fy}{(\sqrt{3}\ \gamma\text{mo})} \qquad = \frac{1700\ x\ 12\ x\ 350\ x\ 10^{-4}}{(\sqrt{3} \text{ x } 1.10)} \qquad = 374.7 \text{ t}$$

Hence, $V_n > R_{tf}$ and the end panel is safe to carry shear due to anchoring forces.

Check for Moment Capacity of End Panel :

Clause 509.5.3,
Pg. 97,
IRC:24 - 2010

In this example, it is assumed that end post consists of a single stiffener.

M_{tf} = H_q.d/10 = 352.2 x 1.7/10 ≈ 59.9 tm

y = c/2 = 1.3/2 = 0.65 m

I = $t_w\ c^3/12$ = .012 x 1.3^3/12 = .002197 m^4

$$M_q \quad = \frac{I\ .\ fy}{y\ .\ \gamma\text{mo}} \qquad = \frac{.002197\ x\ 350\ x\ 102}{0.65\ x\ 1.10} \qquad \approx 109.7 \text{ tm}$$

Hence, M_q > 2/3 M_{tf} and the end panel is safe to carry bending moment due to anchoring forces.

Now,

Force F_m due to M_{tf} = M_{tf}/c	= 59.9/1.3	= 46.1 t
Reaction @ bearing location		≈ 180 t
Total compressive force	= 46.1 + 180	= 226.1 t

Bearing Check :

Area of stiffener, $A_q > \left(\frac{0.8 \times F_x \times \gamma_{m0}}{f_{yq}}\right) > \left(\frac{0.8 \times 226.1 \times 1.10}{350 \ x \ 102}\right) \text{ x } 10^6 \qquad > 5574 \text{ mm}^2$ — Clause 509.7.5.2, Pg. 107, IRC:24 - 2010

Using 4Nos. 10 x 115 mm & 2Nos. 12 x 140 mm stiffeners,

Area of core, A = 4 x 10 x 115 + 2 x 12 x 140 = 7960 mm^2

Check for Outstand :

A) For 10 x 120 mm stiffeners : 14.t_q.ε = 14 x 10 x 0.845 = 118.3 mm > 115 mm — Clause 509.7.1.2, Pg. 102, IRC:24 - 2010

B) For 12 x 140 mm stiffeners : 14.t_q.ε = 14 x 12 x 0.845 = 141.9mm > 140 mm

Buckling Check : — Clause 509.7.1.5, Pg. 102, IRC:24 - 2010

Effective section : The area of the core of the stiffener + 20 t_w on either side

Area of effective section = 7960 + 20 x 12 x 12 x 2

= 13720 mm^2

MOI of effective section

$$= \frac{2 \ x \ 10 \ x \ 242^3}{12} - \frac{2 \ x \ 10 \ x \ 12^3}{12} + \frac{12 \ x \ 292^3}{12} - \frac{12 \ x \ 12^3}{12} + \frac{2 \ x \ 240 \ x \ 12^3}{12}$$

= 4.86 E+07 mm^4

Radius of gyration, $r_x = \sqrt{\frac{I}{A}} = \sqrt{\frac{4.86E+07}{13720}} = 59.5 \text{ mm}$

Flange is restrained against rotation and lateral deflection.

L_e = 0.7L	= 0.7 x 1700	= 1190 mm

$$\lambda = \frac{L_e}{r_x} = \frac{1190}{59.5} \approx 20$$

For buckling curve *c* & f_y = 350 MPa

f_{cd} = 307 MPa

Buckling resistance of the stiffener:

f_{cd} x A_e = 307 x 13720/10^4 = 421 t > 226.1 t

Check Stiffener as Load Bearing Stiffener

The bearing stiffener is designed for the force in excess of the local capacity of the web, Local capacity of web, $F_w = (b_1 + n_2)t_w \frac{f_{yw}}{\gamma_{m0}}$

Clause 509.7.4, Pg. 106, IRC:24 - 2010

where,

b_1 = 0 (for simplicity)

n_2 = 2.5 x 20 = 50 mm

t_w = 12 mm

f_{yw} = 350 MPa

γ_{mo} = 1.10

Hence,

Local capacity of web, $F_w = (b_1 + n_2)t_w \frac{f_{yw}}{\gamma_{m0}} = (0 + 50)\ x\ 12\ x\ \frac{350}{1.10}$ / 10^4 ≈ 19.1 t

Bearing stiffener is to be designed for the force = 226.1 – 19.1 = 207 t

Bearing capacity of the stiffener = (7960 x 350/1.10)/10^4 = 253 t

ANNEXURE – 1

CALCULATION OF EFFECTIVE SECTION PROPERTIES

Input Data:

Grade of concrete	= M40
Modulus of elasticity of Concrete	= 33000
Modulus of elasticity of Steel	= 200000 MPa
Grade of Steel	= E 350 BR
Design Yield strength	= 330 MPa
Modular Ratio (m)	= 200000/33000 = 6.06 < 7.50
	= 7.50
Creep Factor (Kc)	= 0.5

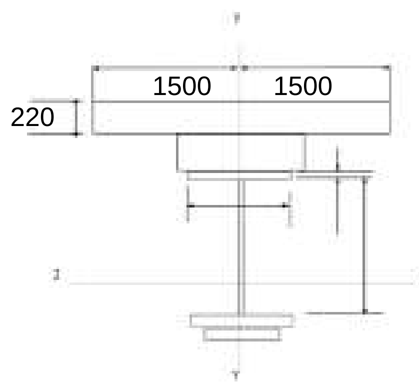
Modular Ratio for permanent loadings i.e., DL & SIDL (mp):

= m/Kc = 7.50/0.5 = 15.00

Modular Ratio for transient loadings i.e., LL (mt): = 7.50

Section at Midspan:

Top flange plate:	**b**	**t**
Plate-1 welded with web	500	20
Actual member b_w/t_w	12.20	
Limiting Class 3 b_w/t_w	11.84	
Hence, class 3.		
Bottom flange plate:	**b**	**t**
Plate-1 welded with web	500	20
Plate-2 below Plate-1	450	20
Web plate:	**b**	**t**
Actual member	12	1700
Actual member b_w/t_w (Ref: **Annexure-2**)	141.67	
Limiting Class 3 b_w/t_w	$126 \times (250/330)^{0.5}$ = 109.67	
Ineffective excess width of web	(141.67 – 109.67) x 12 ≈ 384mm	
Effective width of web	1700 – 384 = 1316 mm	
Start of ineffective web from top of web	(1030.3 – 20 – 384)x0.4 = 250.54 mm	
End of ineffective web from top of web	250.54 + 384 =634.51 mm	
Hence, section is Class - 3: Semi-compact (after deducting the excess width of web).		



Calculation of Iz-z: **Avg. thk. of slab** **0.220** **Ecc of slab from top** **0.110**

SL. No.	No.	A (m^2)	y (m)	Ay (m^3)	Ay^2 (m^4)	Io (m^4)	$I_o + Ay^2$ (m^4)
1. Deck Slab (Transient Loading)	1	0.088000	1.8700	0.1646	0.3077	0.000355	0.308082
2. Deck Slab (Permanent Loading)		0.044000	1.8700	0.0823	0.1539	0.000177	0.154041
3. Top Flange Plate	1	0.010000	1.7500	0.0175	0.0306	0.000000	0.030625
4. Web Plate	1	0.020400	0.8900	0.0182	0.0162	0.004913	0.021072
5. Bottom Flange Plate	1	0.010000	0.0300	0.0003	0.0000	0.000000	0.000009
6. Bottom Flange Plate	1	0.009000	0.0100	0.0001	0.0000	0.000000	0.000001
Composite Section (T-Loading)	Σ	0.137400		0.200606	0.354521	0.005269	0.359790
Composite Section (P-Loading)	Σ	0.093400		0.118326	0.200657	0.005091	0.205749
Girder Only	Σ	0.049400		0.036046	0.046794	0.004914	0.051708

Calculation of Iy-y:

SL. No.	No.	A (m^2)	z (m)	Az (m^3)	Az^2 (m^4)	Io (m^4)	$I_o + Az^2$ (m^4)
1. Deck Slab (Transient Loading)	1	0.088000	0.0000	0.0000	0.0000	0.066000	0.066000
2. Deck Slab (Permanent Loading)		0.044000	0.0000	0.0000	0.0000	0.033000	0.033000
3. Top Flange Plate	1	0.010000	0.0000	0.0000	0.0000	0.000208	0.000208
4. Web Plate	1	0.020400	0.0000	0.0000	0.0000	0.000000	0.000000
5. Bottom Flange Plate	1	0.010000	0.0000	0.0000	0.0000	0.000208	0.000208
6. Bottom Flange Plate	1	0.009000	0.0000	0.0000	0.0000	0.000152	0.000152
Composite Section (T-Loading)	S	0.137400		0.000000	0.000000	0.066569	0.066569
Composite Section (P-Loading)	Σ	0.093400		0.000000	0.000000	0.033569	0.033569
Girder Only	Σ	0.049400		0.000000	0.000000	0.000569	0.000569

Section Properties of Girder

Area	ΣA	=	0.0494	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	0.7297	m
Moment of inertia	$(I_z) = \Sigma(Io+A.y^2)-\Sigma A.Y^2$	=	0.0254	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(I_y) = \Sigma(Io+A.z^2)-\Sigma A.Z^2$	=	0.00057	m^4

Section Properties of Composite Section (T-Loading)

Area	ΣA	=	0.1374	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	1.4600	m
Moment of inertia	$(I_z) = \Sigma(Io+A.y^2)-\Sigma A.Y^2$	=	0.0669	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(I_y) = \Sigma(Io+A.z^2)-\Sigma A.Z^2$	=	0.06657	m^4

Section Properties of Composite Section (P-Loading)

Area	ΣA	=	0.0934	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	1.2669	m
Moment of inertia	$(I_z) = \Sigma(Io+A.y^2)-\Sigma A.Y^2$	=	0.0558	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(I_y) = \Sigma(Io+A.z^2)-\Sigma A.Z^2$	=	0.03357	m^4

Summary of calculated properties:

SECTION	Item	Value		
		Steel only	Short term	Long term
SECTION AT MIDSPAN	Cross sectional area (A) (m^2):	0.0494	0.1374	0.0934
	C.g. distance from bottom (m):	0.7297	1.4600	1.2669
	I_{z-z} (I about z-axis) (m^4):	0.0254	0.0669	0.0558
	I_{y-y} (I about y-axis) (m^4):	0.00057	0.0666	0.0336
	I_{x-x} (tortional constant) (m^4):	0.0001	0.0001	0.0001
	y_t (m):	-	0.5200	0.7131
	y_i (m):	1.0303	0.3000	0.4931
	y_b (m):	0.7297	1.4600	1.2669
	z_t (m^3):	-	0.1287	0.0783
	z_i (m^3):	0.0247	0.2230	0.1132
	z_b (m^3):	0.0348	0.0458	0.0441

CALCULATION OF EFFECTIVE SECTION PROPERTIES:

Calculation of Ix-x:

SL. No.	No.	A (m^2)	y (m)	Ay (m^3)	Ay^2 (m^4)	I_o (m^4)	$I_o + Ay^2$ (m^4)
Deductions for ineffective web :							
1. Web plate	1	0.004608	1.297473	0.0060	0.0078	0.000057	0.007813
Composite Section (T-Loading)	Σ	0.132792		0.194628	0.346764	0.005212	0.351977
Composite Section (P-Loading)	Σ	0.088792		0.112348	0.192901	0.005035	0.197935
Girder Only	Σ	0.044792		0.030068	0.039037	0.004857	0.043894

Calculation of Iy-y:

SL. No.	No.	A (m^2)	z (m)	Az (m^3)	Az^2 (m^4)	I_o (m^4)	$I_o + Az^2$ (m^4)
Deductions for ineffective web:							
1. Web plate	1	0.004608	0.000000	0.0000	0.0000	0.000000	0.000000
Composite Section (T-Loading)	Σ	0.132792		0.000000	0.000000	0.066569	0.066569
Composite Section (P-Loading)	Σ	0.088792		0.000000	0.000000	0.033569	0.033569
Girder Only	Σ	0.044792		0.000000	0.000000	0.000569	0.000569

Section Properties of Girder

Area	ΣA	=	0.0448	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	0.6713	m
Moment of inertia	$(I_z) = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.0237	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(Iy) = \Sigma(Io + A.z^2) - \Sigma A.Z^2$	=	0.00057	m^4

Section Properties of Composite Section (T-Loading)

Area	ΣA	=	0.1328	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	1.4657	m
Moment of inertia	$(I_z) = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.0667	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(Iy) = \Sigma(Io + A.z^2) - \Sigma A.Z^2$	=	0.06657	m^4

Section Properties of Composite Section (P-Loading)

Area	ΣA	=	0.0888	m^2
Distance of cg from bottom fibre (Y)	$Y_b = \Sigma(A.y)/\Sigma A$	=	1.2653	m
Moment of inertia	$(I_z) = \Sigma(I_o + A.y^2) - \Sigma A.Y^2$	=	0.0558	m^4
Distance of cg from Centre of web (Z)	$Z_L = \Sigma(A.z)/\Sigma A$	=	0.0000	m
Moment of inertia	$(Iy) = \Sigma(Io + A.z^2) - \Sigma A.Z^2$	=	0.03357	m^4

Summary of calculated properties:

Section	Item	Reduced Properties		
		Steel only	Short term	Long term
Section at Midspan	Cross sectional area (A) (m^2):	0.0448	0.1328	0.0888
	C.g. distance from bottom (m):	0.6713	1.4657	1.2653
	$I_{z\text{-}z}$ (I about z-axis) (m^4):	0.0237	0.0667	0.0558
	$I_{y\text{-}y}$ (I about y-axis) (m^4):	0.0006	0.0666	0.0336
	$I_{x\text{-}x}$ (tortional constant) (m^4):	0.0001	0.0001	0.0001
	y_t (m):	-	0.5143	0.7147
	y_i (m):	1.0887	0.2943	0.4947
	y_b (m):	0.6713	1.4657	1.2653
	z_t (m^3):	-	0.1297	0.0781
	z_i (m^3):	0.0218	0.2267	0.1128
	z_b (m^3):	0.0353	0.0455	0.0441

ANNEXURE – 2

CALCULATION OF EFFECTIVE WIDTH OF WEB

Calculation of b_{eff} for midspan section with gross cross section properties:

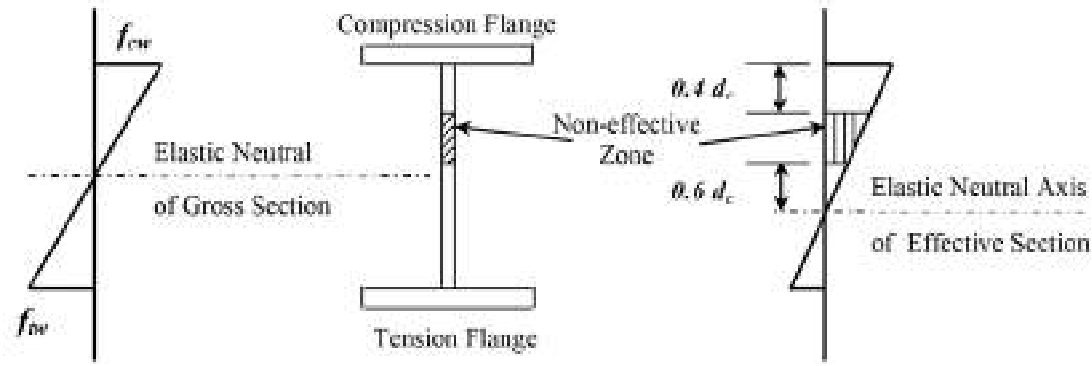


Fig. 4b Effective Width of Slender Web

$$\text{beff} = \frac{120\ \varepsilon\ t}{\left[1 + \dfrac{f_{cw} - f_{tw}}{\rho_{yw}}\right]\left[1 + \dfrac{f_{tw}}{f_{cw}}\right]}$$

SW

$$\frac{120 \times 0.87\ t}{\left[1 + \dfrac{2417 - 1712}{33660}\right]\left[1 + \dfrac{1712}{2417}\right]} = \frac{104.4}{1.7} = 59.9\ t$$

DS

$$\frac{120 \times 0.87\ t}{\left[1 + \dfrac{8857 - 6273}{33660}\right]\left[1 + \dfrac{6273}{8857}\right]} = \frac{104.4}{1.8} = 56.8\ t$$

SIDL

$$\frac{120 \times 0.87\ t}{\left[1 + \dfrac{502 - 1291}{33660}\right]\left[1 + \dfrac{1291}{502}\right]} = \frac{104.4}{3.5} = 29.9\ t$$

SURF.

$$\frac{120 \times 0.87\, t}{\left[1 + \frac{694 - 1783}{33660}\right]\left[1 + \frac{1783}{694}\right]} = \frac{104.4}{3.5} = 30.2\, t$$

CWLL

$$\frac{120 \times 0.87\, t}{\left[1 + \frac{1332 - 6484}{33660}\right]\left[1 + \frac{6484}{1332}\right]} = \frac{104.4}{5.0} = 21.0\, t$$

L/C	b_{eff}	t_w	b_{eff}	Width of web in tension zone	Total effective width of web	Semi-compact limit (126 ε t)	b_{eff} for semi-compact
SW	59.9 t	12	718.6	729.7	1448.3	110 t	1316.0
DS	56.8 t	12	681.4	729.7	1411.1		
SIDL	29.9 t	12	359.3	1266.9	1626.2		
SURF.	30.2 t	12	362.9	1266.9	1629.8		
CWLL	21.0 t	12	252.2	1460.0	1712.2		

The effective width as per semi-compact limit is less than the values as calculated from the detailed stress based calculation. Hence, for the sake of simplicity the semi-compact limit is taken as the limiting width of the web.

Distribution of effective width:

The distribution of the effective width is done as per Fig. 4b of the code. 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 = 1700 – 1316 = 384 mm

Effective width of web in compression zone = dc = yt - Ineffective excess width of web – thk. of top flange

= 1030.3 – 384 – 20 = 626.3 mm

Start of ineffective web from top of web = 0.4 dc = 0.4 x 626.3 = 250.5 mm

End of ineffective web from top of web = 250.5 + 384 = 634.5 mm

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual, etc. from the date specified therein)