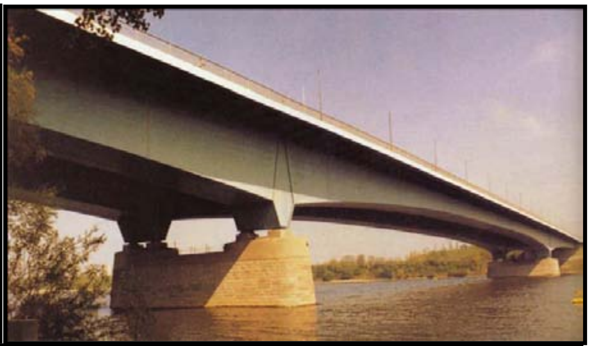
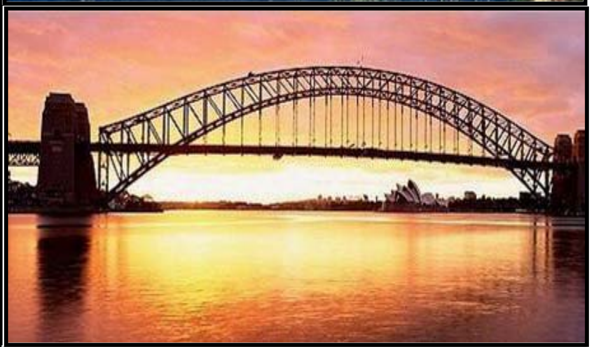
STEEL BRIDGES





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Preface

Bridges have always fascinated people, be it a primitive bridge over a canal or one of the magnificent long span modern bridges. People built bridges to challenge nature where some obstacles like rivers, valleys, or traffic block the way they want to pass through. Our transportation system would not exist without bridges. Their existence allows million of people, cars, and trains to travel every day and everywhere they want to go. It is obvious that both our economy and our society could not function without the technology of bridge engineering.

Bridge building is one of the difficult constructional endeavors that both attracts and challenges structural engineers. The design of such complex structures requires a great deal of knowledge and experience. Depending on the bridge span to be covered, several types of bridge systems exist. Examples of bridge systems are beam bridges for short and moderate spans, arch bridges for moderate spans, and cable stayed bridges and suspension bridges for long spans.

This book covers the design of steel bridges in general with emphasis on bridge systems commonly used to cover short and moderate spans, namely plate girder bridges, box girder bridges, and truss bridges. The book is intended for senior year college students and practicing bridge engineers.

The contents of the book are organized into two parts: the first four chapters cover the design of steel bridges in general while the other four chapters cover the design of specific bridge types. Chapter 1 describes the different structural systems of steel bridges. Chapter 2 presents the design loads on roadway and railway bridges. Chapter 3 presents the design considerations. Chapter 4 covers the design of roadway and railway bridge floor. Chapter 5 covers the design of plate girder bridges. Chapter 6 covers the design of composite plate girders. Chapter 7 covers the design of box girder bridges. Chapter 8 covers the design of truss bridges.

The author hopes that this book will enable structural engineers to design and construct steel bridges with better safety and economy.

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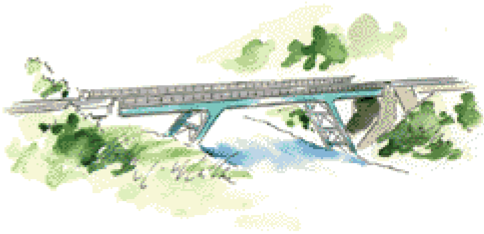
CONTENTS

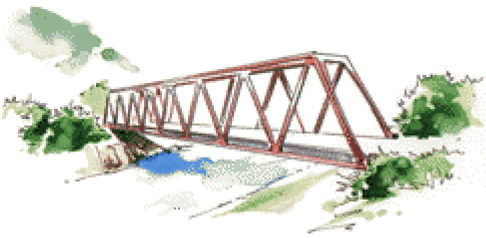
5: PLATE GIRDER BRIDGES

6: COMPOSITE PLATE GIRDER BRIDGES

7: BOX GIRDER BRIDGES

8: TRUSS BRIDGES





CHAPTER 1

INTRODUCTION

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1.1 GENERAL

1.1.1 Historical Background

People have always needed to transport themselves and their goods from one place to another. In early times, waterways were used wherever possible. Navigable waterways, however, do not always go in the direction desired or may not be always available. Therefore, it has been necessary to develop land transportation methods and means of crossing waterways and valleys. Roadway and railway development have therefore become an absolute necessity for economic development. The rapid economic development in Europe, USA, and Japan could not take place until land transportation was developed. Even today, one important factor that has caused many countries to lag behind in economic development is the lack of good land transportation systems.

An important element of land transportation systems is the bridge. A bridge is a structure that carries a service (which may be highway or railway traffic, a footpath, public utilities, etc.) over an obstacle (which may be another road or railway, a river, a valley, etc.), and then transfers the loads from the service to the foundations at ground level.

The history of bridge engineering, which began with stone and wooden structures in the first century BC, can be said to be the history of the evolution of civil engineering. It is not possible to date humanity's conception and creation of the first bridge. Perhaps people derived the first concept in bridge building from nature. The idea of a bridge might have developed from a tree trunk that had fallen across a canal. Early bridges consisted of simple short spans of stone slabs or tree trunks. For longer spans, strands of bamboo or vine were hung between two trees across a stream to make a suspension bridge.

The introduction of new materials – plain, reinforced, and pre-stressed concrete; cast iron; wrought iron; and steel – evolved gradually within the last two centuries. According to known records, the first use of iron in bridges was a chain bridge built in 1734 in Prussia. Concrete was first used in 1840 for a 12-m span bridge in France. Reinforced concrete was not used in bridge construction until the beginning of the twentieth century. Pre-stressed concrete was introduced in 1927. These developments, coupled with advances in structural engineering and construction technology, led to the introduction of different forms of bridges having increasingly longer spans and more load carrying capacities.

1.1.2 Bridge Components

In Figure 1.1 the principal components of a bridge structure are shown. The two basic parts are:

(1) the **substructure**; which includes the piers, the abutments and the foundations.

(2) the **superstructure**; which consists of:

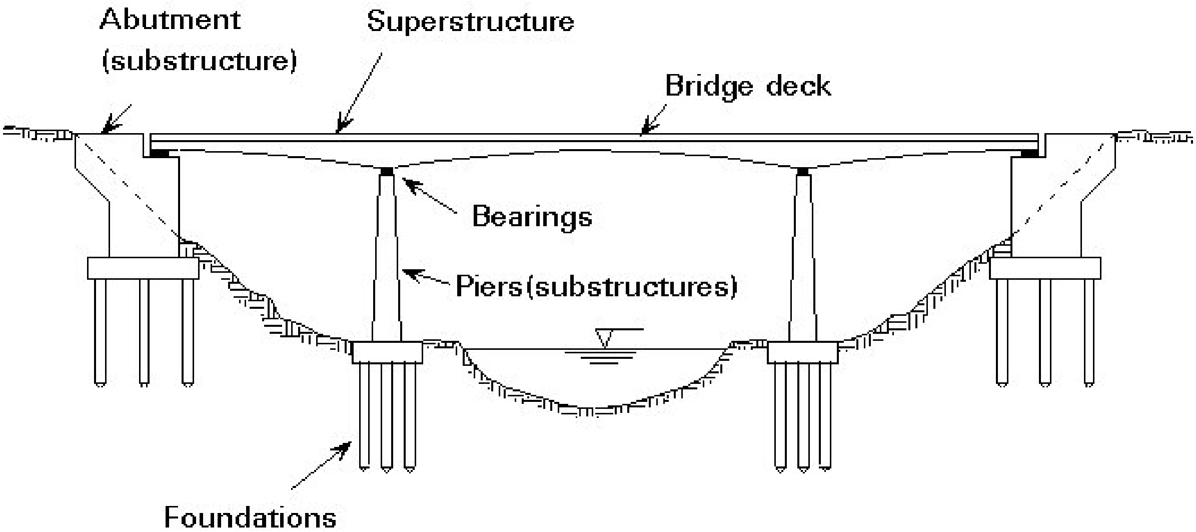
a) the bridge deck, which supports the direct loads due to traffic and all the other permanent loads to which the structure is subjected.
In roadway bridges it includes the deck slab, Fig. 1.1b.
In railway bridges it includes the rails and sleepers, Fig. 1.1c

b) the floor beams, which transmit loads from the bridge deck to the bridge main girders. They consist of longitudinal beams, called stringers, and transversal beams, called cross girders, Fig. 1.1c.

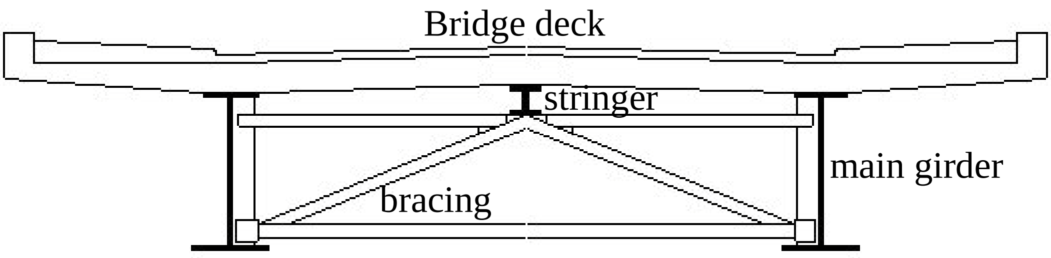
c) the main girders, which transmit the bridge vertical loads to the supports.

d) the bracings, which transmit lateral loads to the supports and also provide lateral stability to compression members in the bridge, Fig. 1.1b.

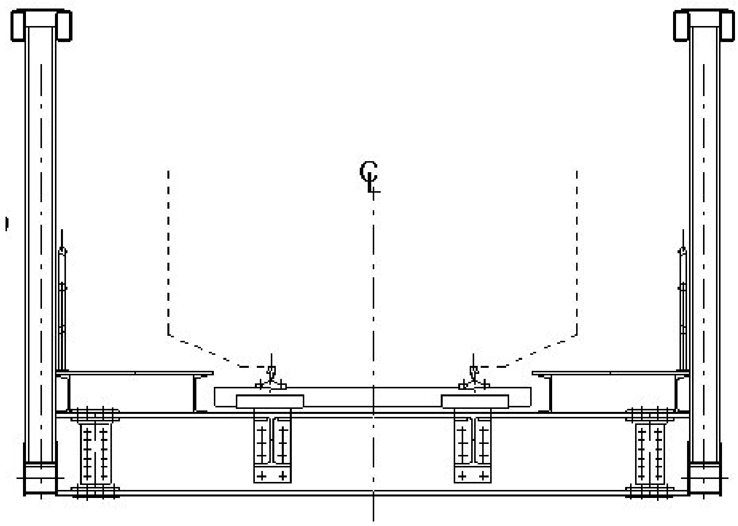
The connection between the substructure and the superstructure is usually made through bearings. However, rigid connections between the piers (and sometimes the abutments) may be adopted, such as in frame bridges, Figs. 1.4a and 1.4b.



a) Bridge Elevation



b) Cross Section of a Roadway Bridge



c) Cross Section of a Railway Bridge

Fig. 1.1 Principal Components of a Bridge Structure

1.2 TYPES OF BRIDGES

Bridges can be classified in several ways depending on the objective of classification. The necessity of classifying bridges in various ways has grown as bridges have evolved from short simple beam bridges to very long suspension bridges. Bridges may be classified in terms of the bridge's superstructure according to any of the following classifications:

1. Materials of Construction
2. Usage
3. Position
4. Structural Forms.
5. Span Lengths

A brief description of these bridge classifications is given next.

1.2.1 Bridge Classification by Materials of Construction

Bridges can be identified by the materials from which their main girders are constructed. The most commonly used materials are ***steel*** and ***concrete***. This classification does not mean that only one kind of material is used exclusively to build these bridges in their entirety. Often, a combination of materials is used in bridge construction. For example, a bridge may have a reinforced concrete deck and steel main girders.

1.2.2 Bridge Classification by Usage

Bridges can be classified according to the traffic they carry as ***roadway, railway,*** Fig. 1.2, and ***footbridges,*** Fig. 1.3. In addition, there are bridges that carry non-vehicular traffic and loads such as ***pipeline bridges*** and ***conveyor bridges***.

Fig. 1.2 Railway Through Bridge

Fig. 1.3 Foot Bridge

1.2.3 Bridge Classification by Position

Most bridges are fixed in place. However, to provide sufficient vertical clearance to facilitate navigation through spanned waterways, bridges are made movable; i.e., the bridge superstructure changes its position relative to the roads that they link. In general, three kinds of movable bridges exist:

1. The ***bascule bridge,*** which has a rotational motion in the vertical plane, Fig. 1.4a.

Fig. 1.4 a) Bascule Bridge

2. The ***lift bridge***, which has a translational motion in the vertical plane, Fig. 1.4b,

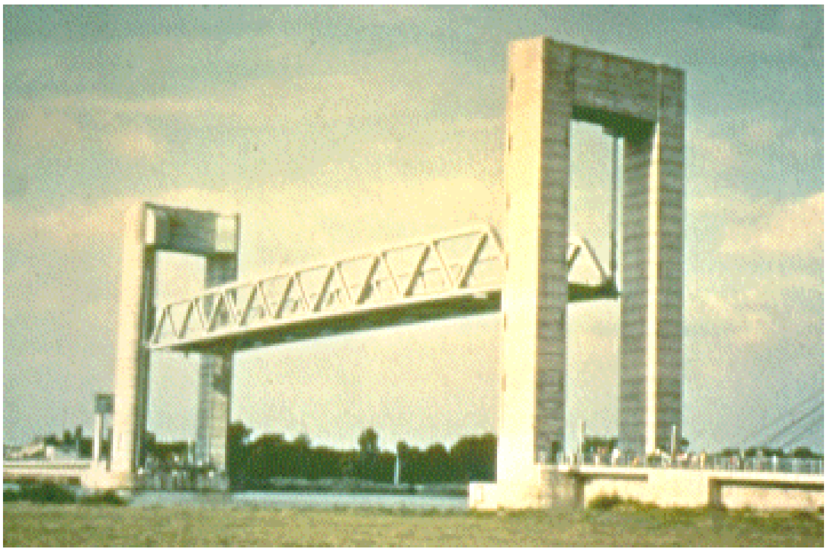


Fig. 1.4 b) Lift Bridge

3. The ***swing bridge***, which has a rotational motion in the horizontal plane, Fig. 1.4c.

Fig. 1.4 c) Swing Bridge

1.2.4 Bridge Classification by Structural Form

From an engineering perspective, bridges are best classified by their structural forms because the methods of analysis used in bridge design depend on the structural system of the bridge. Also, certain types of structural forms are suitable for certain span ranges.

Structural form refers to the load resisting mechanism of a bridge by which it transfers various loads from the bridge deck to the foundation. In different types of bridges, loads follow different paths as they are first applied on the deck and finally resolved in the earth below. From this perspective, several structural systems are used in the elements of the bridge superstructure. It is common in bridge terminology to distinguish between:

a. structural systems in the transversal direction, and
b. structural systems in the longitudinal direction.

The structural systems in the transversal direction are those used for the bridge deck and floor structure to transfer loads to the bridge main girder. Details of different systems used in both roadway and railway bridges are given in Chapter 4.

The structural systems in the longitudinal direction are those used for the bridge main girders to transfer loads to the supporting foundations. It should be understood that bridge structures are basically three-dimensional systems which are only split into these two basic systems for the sake of understanding their behavior and simplifying structural analysis.

The longitudinal structural system of a bridge may be one of the following types:

i) Bridges Carrying Loads Mainly by Bending: a) beam bridges
b) frame bridges

ii) Bridges Carrying Loads Mainly by Axial Forces: a) arch bridges
b) cable stayed bridges
c) suspension bridges.

The cross-section of the main girder incorporated in all these bridge types may be a solid web girder or a truss girder depending on the values of the design straining actions. Solid web girders dimensions are limited by the requirements imposed by fabrication, transportation, and erection. Practical maximum section depths of solid web girders range from 3 to 4 m for economical design. If the required design exceeds this limit, a truss girder has to be used, see Fig. 1.5.

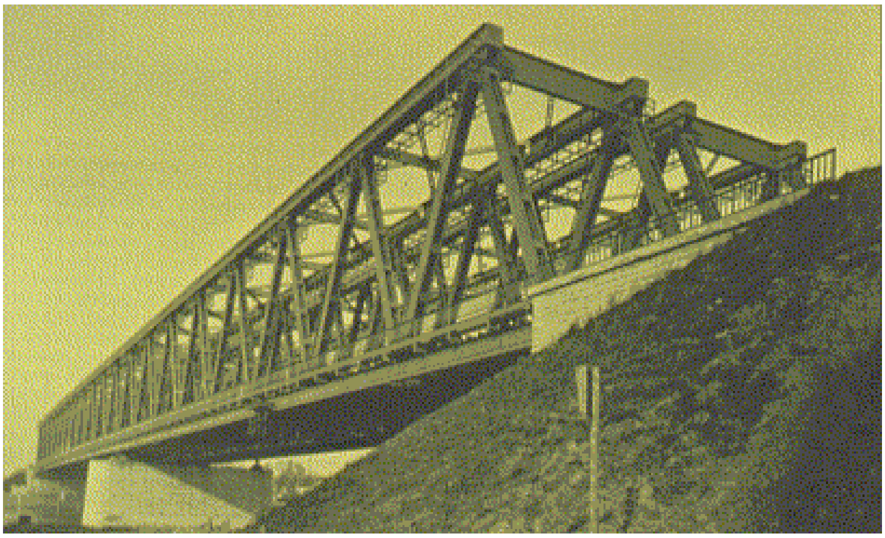


Fig. 1.5 Truss Bridge

A truss used as a girder in flexure carries its bending moments by developing axial loads in its chords, and its shears by developing axial loads in its web members. Truss bridges are not specific bridge forms in themselves – rather, trusses are used to perform the functions of specific members in one of the types above. For example, a girder in flexure or an arch rib in axial compression may be designed as a truss rather than as a solid web plate girder.

1.2.4.1) Bridges Carrying Loads by Bending

By far the majority of bridges are of this type. The loads are transferred to the bearings and piers and hence to the ground by beams acting in bending, i.e. the bridges obtain their load-carrying resistance from the ability of the beams to resist bending moments and shear forces. This type of bridge will thus be referred to generally as a girder bridge.

Beam bridges are the most common and the simplest type of bridges. These may use statically determinate beams (simply supported, Fig. 1.6a, or cantilever beams, Fig. 1.6b) or continuous beams, Fig. 1.6c. Examples of beam bridges are shown in Fig. 1.7:

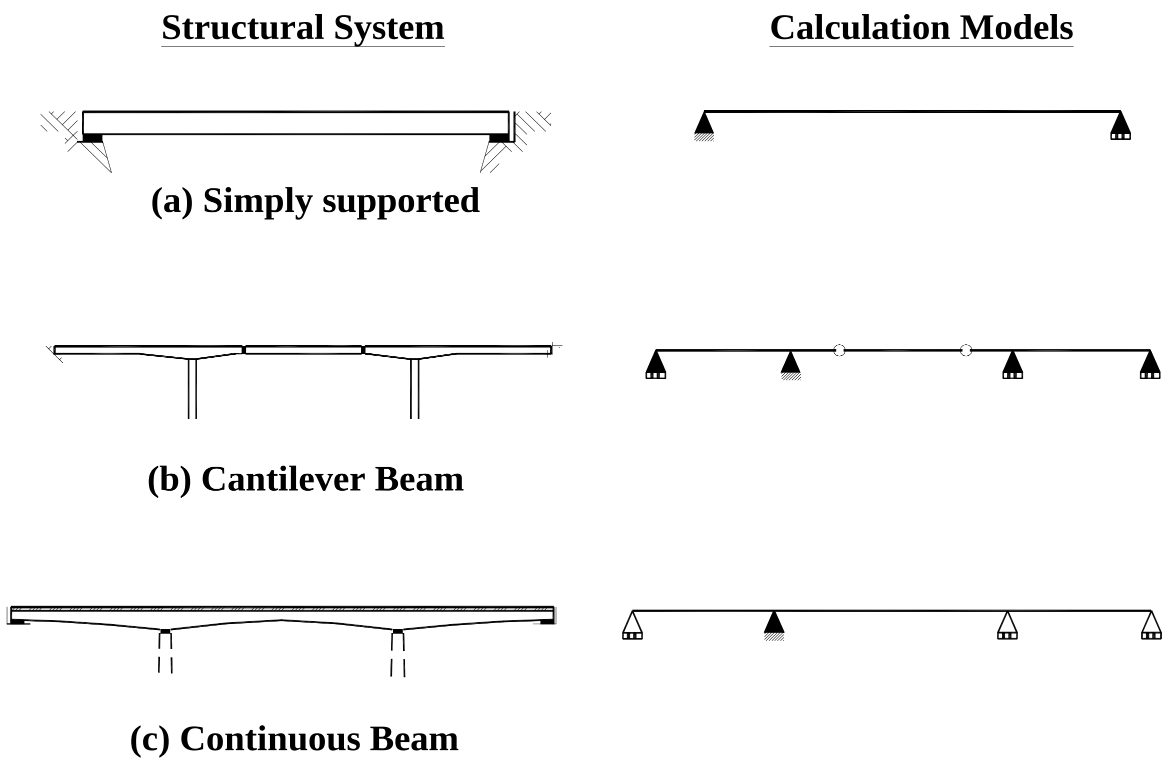
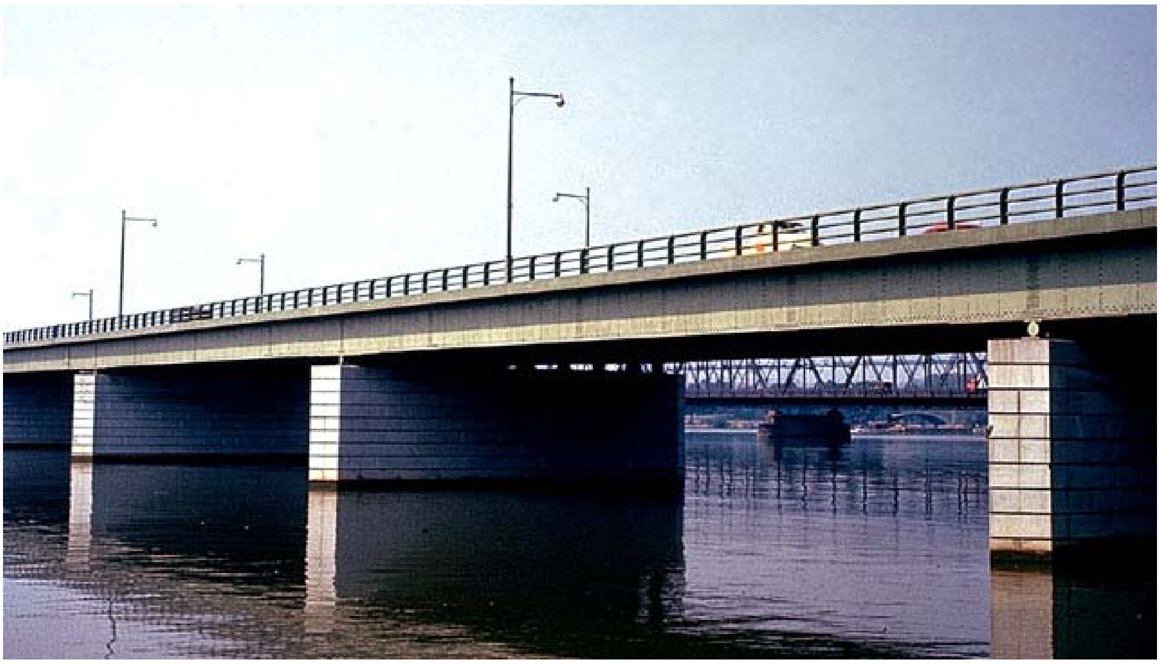
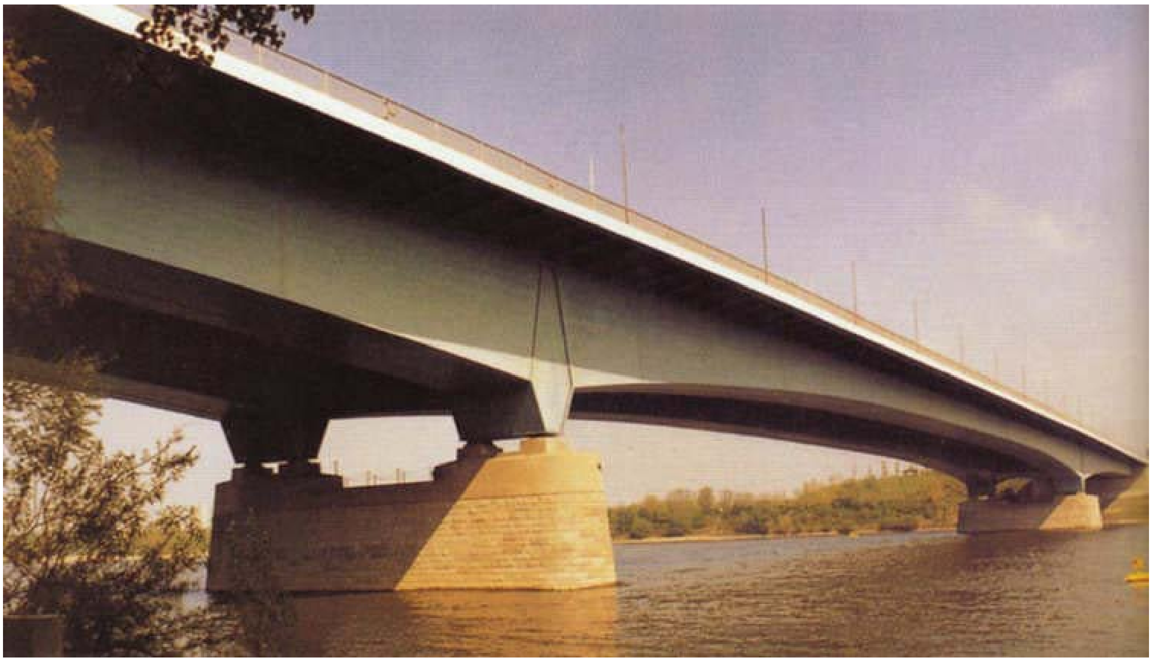


Fig. 1.6 Bridge Systems Carrying Loads by Bending, Beam Bridges



(a) 14th Street Bridge over the Potomac River (USA). Continuous riveted steel girders. Note the absence of internal hinges, and the roller supports at the piers



(b) Continuous steel box girder bridge over the Rhine, Bonn, Germany, 1967. Note the varying depth of the box sections

Fig. 1.7 Examples of Beam Bridges

Simply supported beams are usually adopted only for very small spans (up to 25m). Continuous beams are one of the most common types of bridge. Spans for this system may vary from short (less than 20 m) to medium (20 - 50 m) or long spans (> 100 m). In medium and long spans, continuous beams with variable depth section are very often adopted for reasons of structural behavior, economy and aesthetics. These systems are suitable for bridge spans up to 200 m for solid web girders and up to 300 m for truss girders.

Frame bridges are one of the possible alternatives to continuous beams. Avoiding bearings and providing a good structural system to support horizontal longitudinal loads, e.g. earthquakes, frames have been adopted in modern bridge either with vertical piers or with inclined columns (Fig. 1.8).

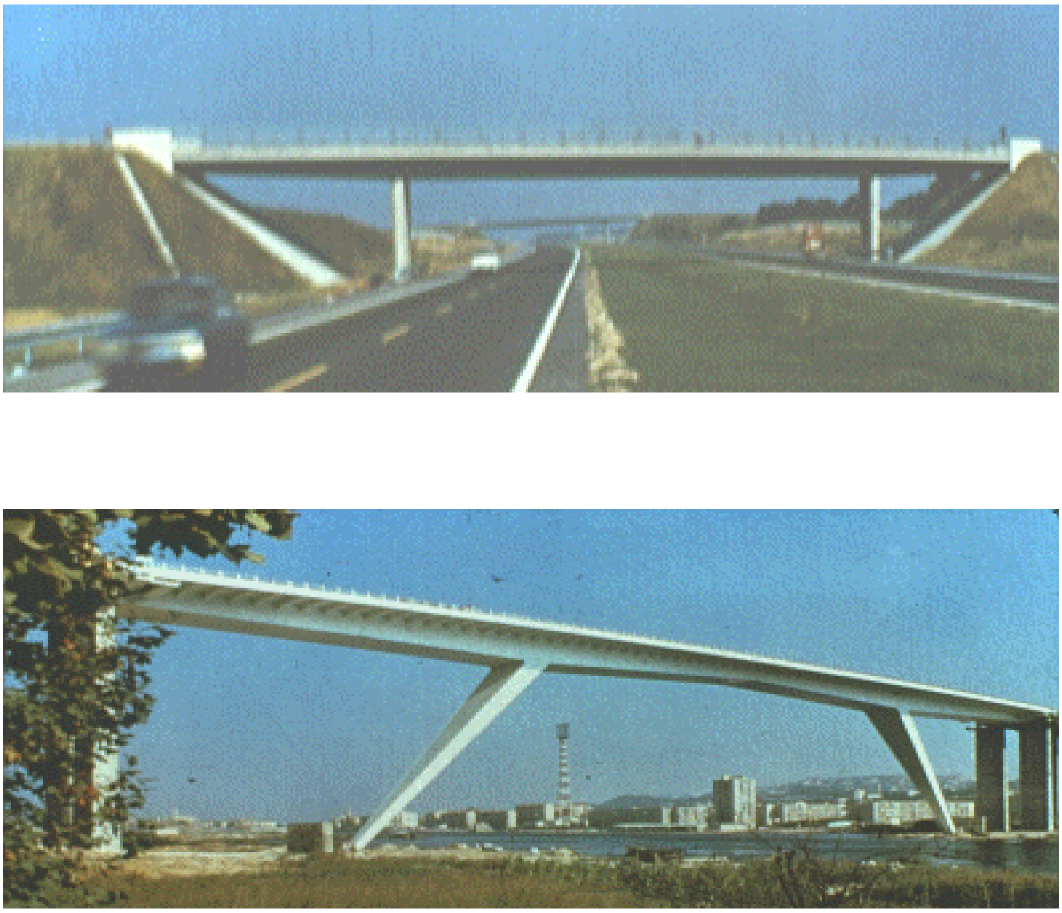
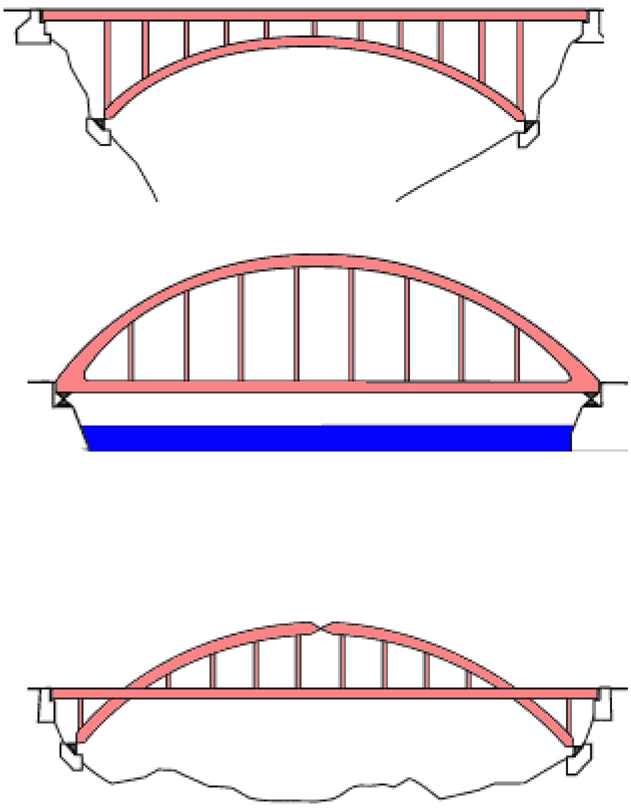


Fig. 1.8 Bridge Systems Carrying Loads by Bending, Rigid Frames with Vertical or Inclined Legs

1.2.4.2) Bridges carrying Loads by Axial Forces

This type can be further subdivided into those bridges in which the primary axial forces are compressive, e.g.; ***arches***, Fig. 1.9, and those in which these forces are tensile, e.g.; ***suspension bridges***, Fig. 1.11, and ***cable-stayed bridges***, Fig. 1.13.

Arches have played an important role in the history of bridges. Several outstanding examples have been built ranging from masonry arches built by the Romans to modern pre-stressed concrete or steel arches with spans reaching the order of 500 m.. The arch may work from below the deck, Fig. 1.9a, from above the deck, Fig. 1.9b, or be intermediate to the deck level, Fig. 1.9c. The most convenient solution is basically dependent on the topography of the bridge site. In rocky sites and good geotechnical conditions for the footings, an arch bridge of the type represented in Fig. 1.9a is usually an appropriate solution both from the structural and aesthetic point of view. Arches work basically as a structure under compressive stress.. The shape is chosen in order to minimize bending moments under permanent loads. The resultant force of the normal stresses at each cross-section must remain within the central core of the cross-section in order to avoid tensile stresses in the arch.



(a) Deck Bridge

(b) Through Bridge (Bow String)

(c) Semi-Deck\Semi Through

Fig. 1.9 Bridge Systems Carrying Loads by Axial Forces; Arch Systems

a) Solid Web Arch Bridge

b) Sydney Harbor Arch Bridge, completed 1932. Almost the longest arch bridge in the world (longest is Bayonne Bridge, New York, completed a few months earlier, 1.5 m longer). Two-hinge arch, span between abutments is 503 m to allow unobstructed passage for ships in Sydney Harbor. Contains 50,300 tons of steel (37,000 in the arch). The widest (49 m) bridge in the world.

Fig. 1.10 Examples of Arch Bridges

The ideal "inverted arch" in its simplest form is a cable. Cables are adopted as principal structural elements in ***suspension bridges*** where the main cable supports permanent and imposed loads on the deck (Fig. 1.11). Good support conditions are required to resist the anchorage forces of the cable. This system is suitable for bridge spans between 300 and 2000 m.

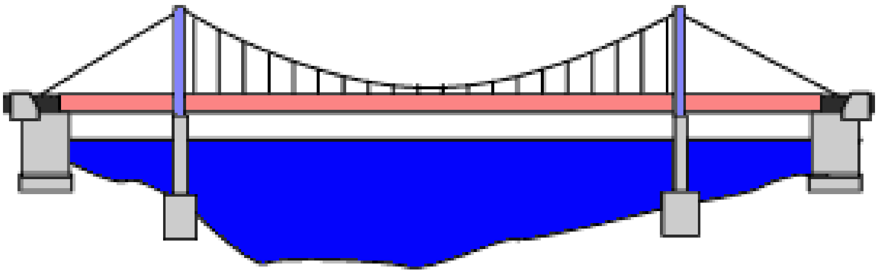


Fig. 1.11a Bridge Systems Carrying Loads by Axial Forces; Suspension Bridges

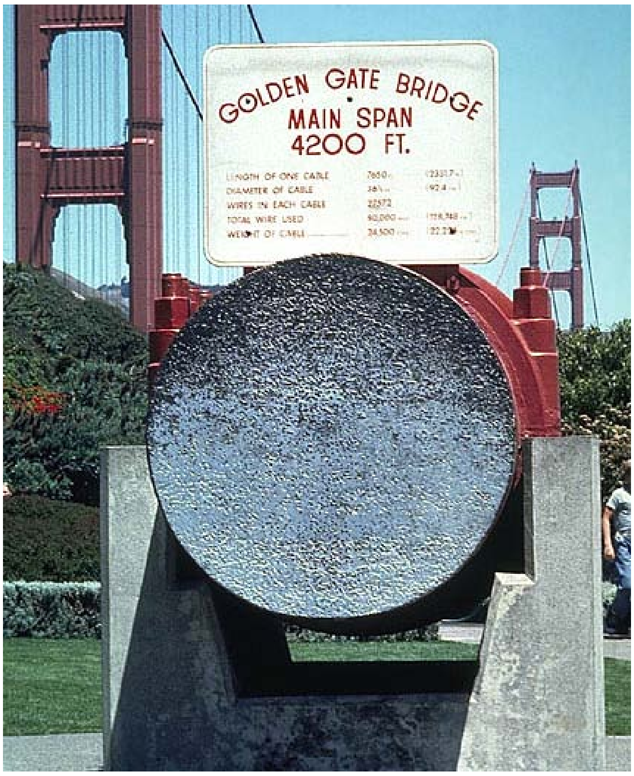
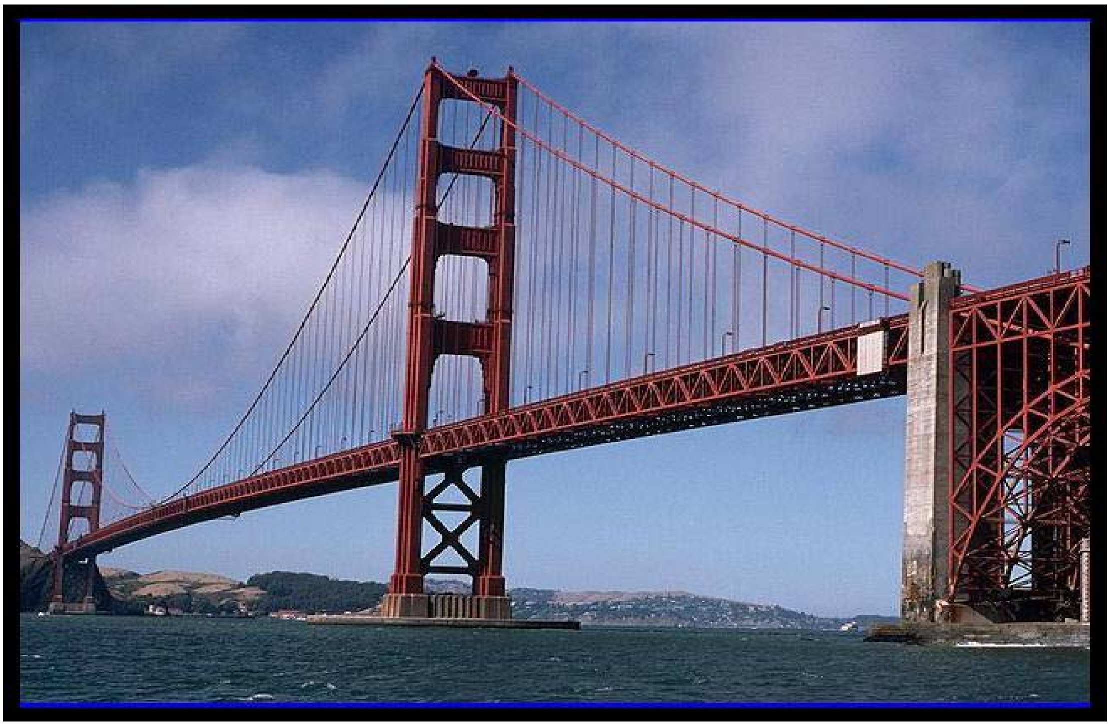


Fig. 1.11b Section of a suspension bridge cable, showing it is made up of a bundle of small cables



a) Golden Gate Bridge, 1937. Main span of 1280 m, was the longest single span at that time and for 29 years afterwards.

b) Akashi-Kaiyko Suspension Bridge, Japan. Links city of Kobe with Awaji Island. World's longest bridge (Main Span 1991 m)

Fig. 1.12 Examples of Suspension Bridges

A simpler form of cable bridges has been used - ***Cable stayed bridges*** (Fig. 1.13). They have been used for a range of spans, generally between 100 m and 500 m, where the suspension bridge is not an economical solution. Cable stayed bridges may be used with a deck made of concrete or in steel.

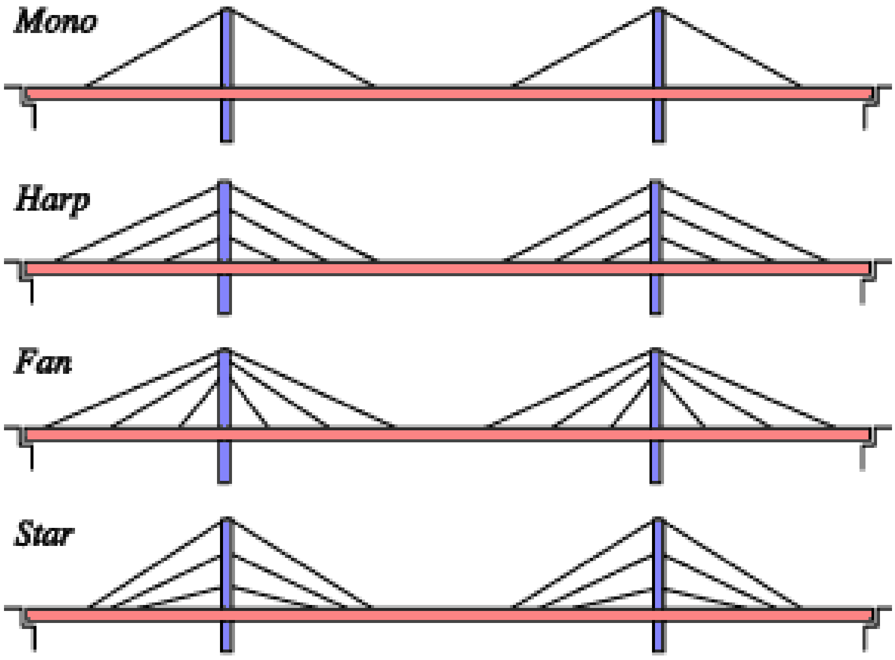


Fig. 1.13 Bridge Systems Carrying Loads by Axial Forces; Cable-Stayed Bridges

Pont du Normandie (River Seine, Le Harve, France). 856 m main span, longest cable stayed bridge in the world up to 1999. Longest now is Tatara Bridge, Japan, 890 m

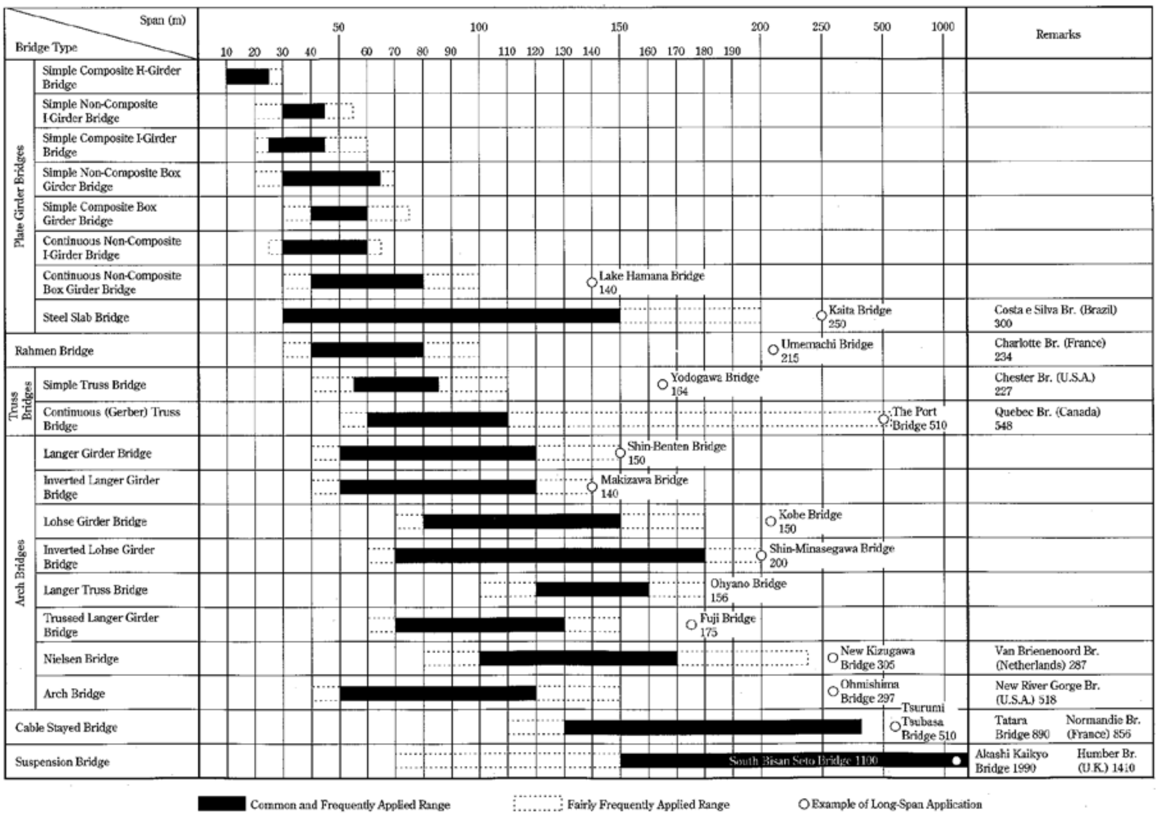
Fig. 1.14 Example of Cable-Stayed Bridges

1.2.5 Bridge Classification by Span Lengths

In bridge engineering, it is customary to identify bridges according to their span lengths as ***short span, medium span, and long span***. Presently there are no established criteria to exactly define the range of spans for these different classifications. A common practice is to classify bridges by span lengths as follows:

Short-span bridges	less than 50 m
Medium-span bridges	50 to 200 m
Long-span bridges	Over 200 m

This classification of bridges is useful only in selecting the structural form most suitable for the bridge span considered, as shown in the following table. Each form of bridge is suited to a particular range of spans. The Table also records the longest span for each type of construction.



1.2.6 Selection of Structural System

Flat girders, i.e. girders of constant depth, are used for all shorter span bridges of both simple spans and continuous construction up to spans of around 30 m. Rolled sections are feasible and usually offer greater economy. Above this span fabricated sections will be required.

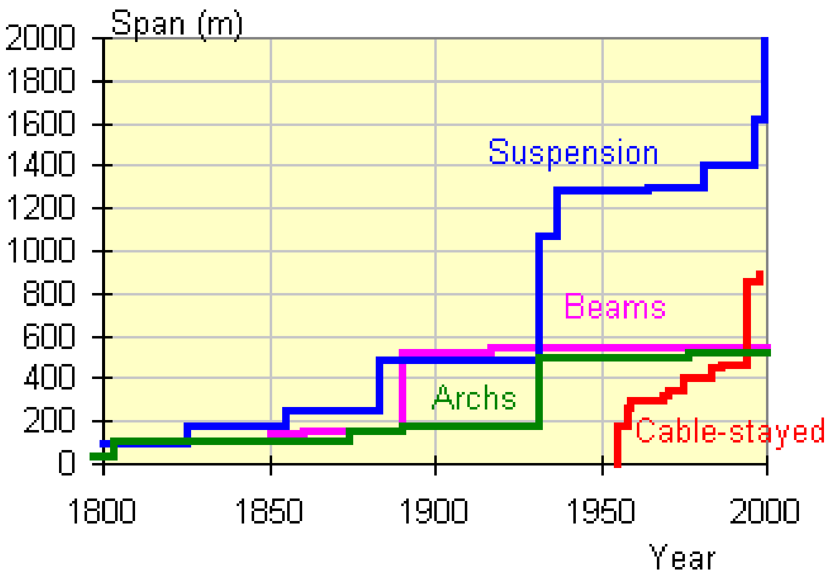
Haunched girders are frequently used for continuous structures where the main span exceeds 50m. They are more attractive in appearance and the greater efficiency of the varying depth of construction usually more than offsets the extra fabrication costs. Both haunched and flat girders can be either plate girders or box girders. Development in the semi-automatic manufacture of plate girders has markedly improved their relative economy. This form of construction is likely to be the preferred solution for spans up to 60 m or so, if depth of construction is not unduly limited. Above 60 m span, and significantly below that figure if either depth of construction is limited or there is plan curvature, the box girder is likely to give greater economy.

Cantilever trusses were used during the early evolution of steel bridges. They are rarely adopted for modern construction.

Arches or ***rigid frames*** may be suitable for special locations. For example, an arch is the logical solution for a medium span across a steep-sided valley. A tied arch is a suitable solution for a single span where construction depth is limited and the presence of curved highway geometry or some other obstruction conflicts with the back stays of a cable stayed bridge. Frame bridges are usually suitable for short or medium spans. In a three span form with sloping legs, they can provide an economic solution by reducing the main span; they also have an attractive appearance. The risk of shipping collision must be considered if sloping legs are used over navigable rivers.

Cable stayed bridges, being self anchored, are less dependent on good ground conditions. However, the deck must be designed for the significant axial forces from the horizontal component of the cable force. The construction process is quicker than for a suspension bridge because the cables and the deck are erected at the same time. ***Suspension or cable stayed bridges*** are the only forms capable of achieving the longest spans. They are clearly less suitable for road or rail bridges of short or medium spans.

The following Figure shows the development of different bridge systems with the span over the years.

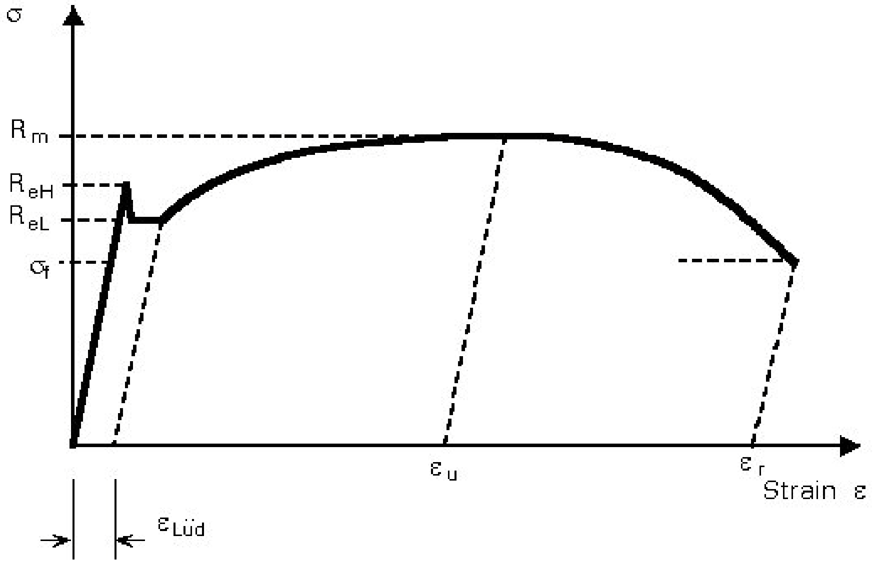


1.3 MATERIALS FOR BRIDGE CONSTRUCTION

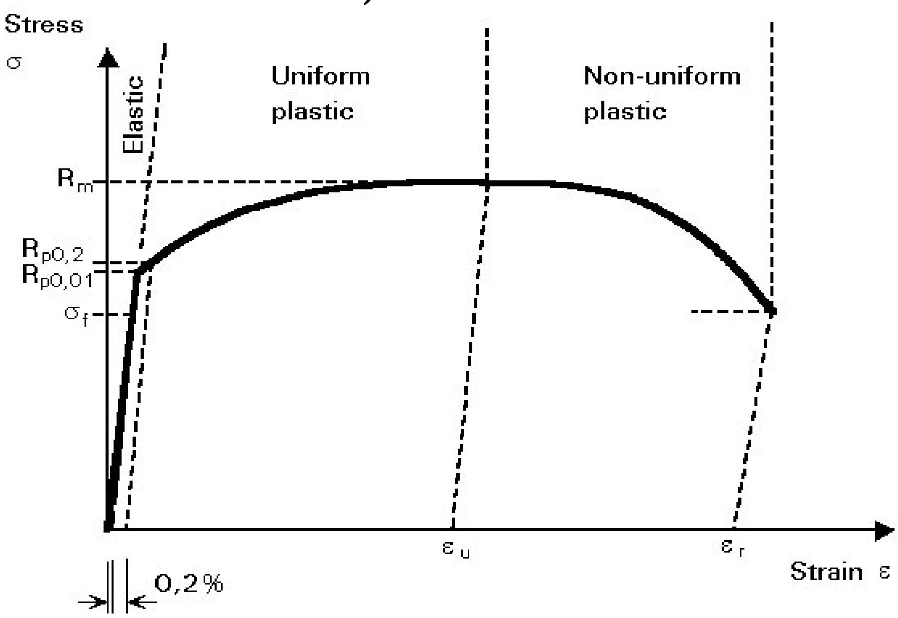
Steel and concrete are the two major materials used in bridge construction. For bridge decks, concrete is predominant. However, for long span bridges, there can be a saving in using steel orthotropic plate decks with an asphalt wearing surface. Concrete is also the predominant material for curbs, sidewalks, parapets, and substructure.

1.3.1 Structural Steels

Structural steel used in bridge construction can be categorized into three main types: (1) Carbon steel, (2) High-strength low-alloy steel, and (3) heat-treated alloy steel. Fig. 1.15 shows typical stress strain curves.



a) Carbon Steel



b) High Strength Steel

Fig. 1.15 Stress Strain Curves for Structural Steels

1. **Carbon steel:** This is the cheapest steel available for structural use. This type of steel is characterized by the following chemical analysis contents:

   Carbon : 0.15 - 0.29 %
   Copper : 0.60 %
   Manganese: 1.65 %

   Examples of these steels are St. 37 which has a minimum yield stress of 24 kg/mm^2.

2. **High-strength low-alloy steel**: Structural steels included in this category have a minimum yield stress of 28 kg/mm^2. The improvement in the mechanical properties is achieved by adding small amounts of alloy elements such as chromium, columbium, molybdenum, nickel, or vanadium. The total of alloying elements does not exceed 5 % of the total composition of steel, hence the term 'low-alloy'. Examples of these steels are St. 44 and St. 52.

3. **Heat-treated alloy steel:** These steels are obtained by heat-treating the low-alloy steels to obtain higher yield strength, 60 to 90 kg/mm^2. The process of heat treating involves quenching or rapid cooling with water or oil from 900 ^oC to about 150 - 200 ^oC, then tempering by reheating to at least 600 ^oC, and then controlled cooling. These steels do not exhibit a well-defined yield point like the carbon and low-alloy steel. Consequently, their yield strengths are determined by the 0.2 percent offset method.

1.3.1.1 Physical Properties of Steel:

Mass Density	**$\rho = 7.85$**	**t/m^3**
Modulus of Elasticity	**$E = 2100$**	**t/cm^2**
Shear Modulus	**$G = 810$**	**t/cm^2**
Poisson's Ratio	**$\upsilon = 0.3$**	
Coefficient of Thermal Expansion	**$\alpha = 1.2 \times 10^{-5}$**	**$/ ^oC$**

1.3.1.2 Mechanical Properties of Steel

Egyptian Standard Specification No.260/71

Grade of Steel	Nominal Values of Yield Stress F_y and Ultimate Strength F_u			
	Thickness t			
	$t \leqslant 40$ mm		40 mm < $t \leqslant 100$ mm	
	F_y (t/cm^2)	F_u (t/cm^2)	F_y (t/cm^2)	F_u (t/cm^2)
St 37	2.40	3.70	2.15	3.4
St 44	2.80	4.40	2.55	4.1
St 52	3.60	5.20	3.35	4.9

1.3.2 Welding Materials

Welding has become the predominant method for connecting parts of steel bridges, especially with respect to shop fabrication. The development of automatic welding has been a major factor in the fabrication of welded bridges.

Structural steels may be welded by one of the following welding processes:

- Shielded Metal Arc Welding (S.M.A.W.): used for manual welding.
- Submerged Arc Welding (S.A.W.): used for automatic welding.
- Gas Metal Arc Welding (G.M.A.W.): used for semi-automatic welding.

The appropriate electrode types used in the weld process as well as their yield and tensile strengths are given in Table 1 according to ECP 2001.

Table (1) Electrodes Used for Welding (ECP 2001)

Process	Electrode Strength *		Chemical Composition	Weld Position	Remarks
	Min. Yield Stress (t/cm^2)	Min. Tensile Strength (t/cm^2)			
Shield Metal Arc WELDING (S.M.A.W.)	3.45 – 6.75	4.25 – 7.6	Electrode: Low Carbon Coating: Aluminium, Silicon, other deoxidizers	All weld positions	Storage of electrodes in drying ovens near the points is a must.
Submerged Arc WELDING (S.A.W.)	3.45 – 6.75	4.25 – 8.95	Electrode: Medium Mn (1.0%) Nominal Carbon (0.12%) Flux: Finely powdered constituents glued together with silitales.	Flat or horizontal weld position	-Fluxes must be kept in storage. -usually used in shop.
Gas Metal Arc WELDING (G.M.A.W.)	4.15 – 6.75	4.95 – 7.6	Electrode: Uncoated mild steel, dioxidized carbon manganese steel Shielding Gas: 75% Argon + 25% CO_2 or 10% CO_2	Flat or horizontal weld position	Co_2 is the least shielding used in buildings and bridges.
Flux Cored Arc WELDING (F.C.A.W.)	3.45 – 6.75	4.25 – 8.6	Electrode: Low Carbon (0.05% Max.) Flux: Filled inside the electrode core (Self Shielded)	All weld positions	Useful for field welding in severe cold weather conditions.

(*) The minimum value depends on the electrode type.

1.3.3 Bolts

Bolts used in bridge construction come in two general categories:

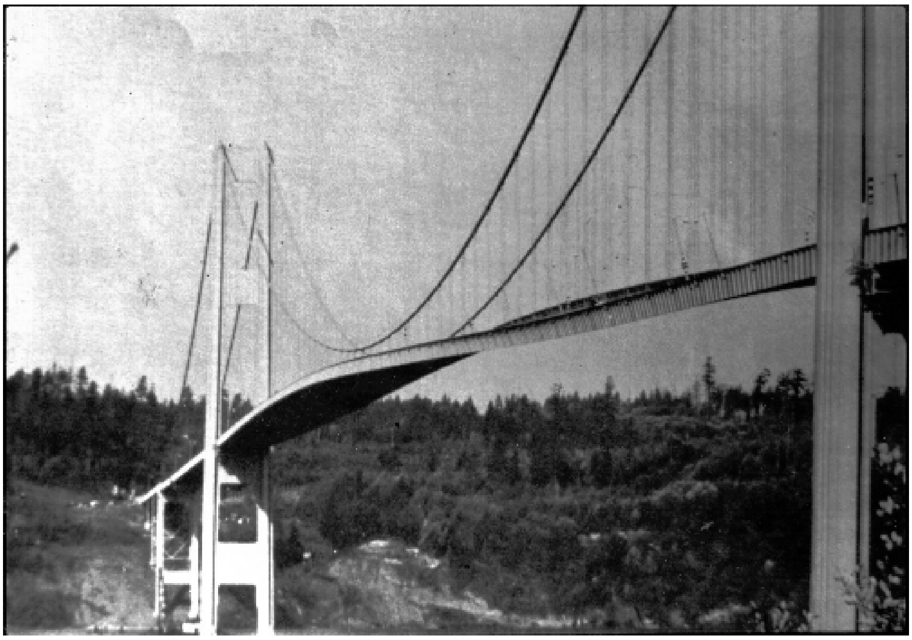
1. **Ordinary Bolts**: which are made from low-carbon steel. Example of this type of bolts are grade 4.6 bolts. Because of their low strength, they are not generally used in joints of main members. They should not be used in joints subjected to fatigue.

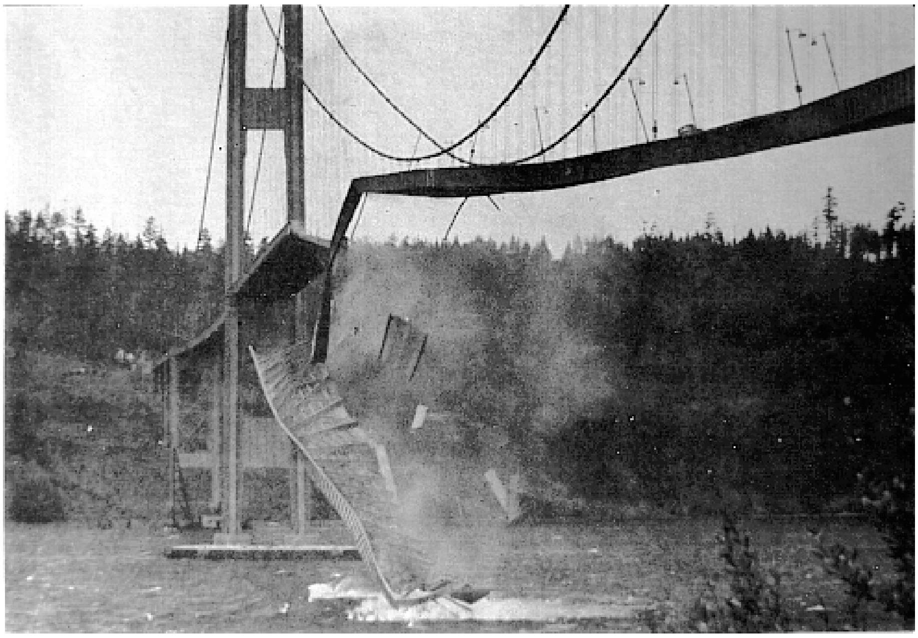
2. **High Strength Bolts**: which are made from high strength alloy steels. Examples of these bolts are grade 8.8 and 10.9 bolts. All high-strength bolts carry markings on their heads to indicate the bolt grade; i.e., 8.8 or 10.9.

The usual bolt diameters used in bridge construction are 20, 22, 24, and 27 mm. The nominal values of the yield stress F_{yb} and the ultimate tensile strength F_{ub} are as given in Table 2 according to ECP 2001. These bolt grades are used in conjunction with structural components in steel up to St 52.

Table (2) (ECP 2001)
Nominal Values of Yield Stress F_{yb} and Ultimate Tensile Strength F_{ub} for Bolts

Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
F_{yb} (t/cm2)	2.4	3.2	3.0	4.0	4.8	6.4	9.0
F_{ub} (t/cm2)	4.0	4.0	5.0	5.0	6.0	8.0	10.0





CHAPTER 2

DESIGN LOADS ON BRIDGES

CHAPTER 2

DESIGN LOADS ON BRIDGES

2.1 INTRODUCTION

Bridge structures must be designed to resist various kinds of loads: vertical as well as lateral. Generally, the major components of loads acting on bridges are dead and live loads, environmental loads (temperature, wind, and earthquake), and other loads, such as those arising from braking of vehicles and collision. Vertical loads are caused by the deadweight of the bridge itself and the live load, whereas the lateral loads are caused by environmental phenomena such as wind and earthquakes.

Bridge structures serve a unique purpose of carrying traffic over a given span. Therefore, they are subjected to loads that are not stationary; i.e., moving loads. Also, as a consequence, they are subjected to loads caused by the dynamics of moving loads; such as longitudinal force and impact and centrifugal forces.

Various kinds of bridge loads are shown in Fig. 2.1 and are described in the following sections.

2.2 ROADWAY DESIGN LOADINGS

a) Dead Load

Dead load on bridges consists of the self-weight of the superstructure plus the weight of other items carried by the bridge such as utility pipes which may be carried on the sides or underneath the deck. The self-weight of the superstructure consists of the deck, including the wearing surface, sidewalks, curbs, parapets, railings, stringers, cross girders, and main girders. Depending on the bridge type, the self-weight of the superstucture may be significant, as in the case of long span bridges, or it may be a small fraction of the total weight, as in the case of short span bridges. In any case, the dead load can be easily calculated from the known or the assumed sizes of the superstructure components.

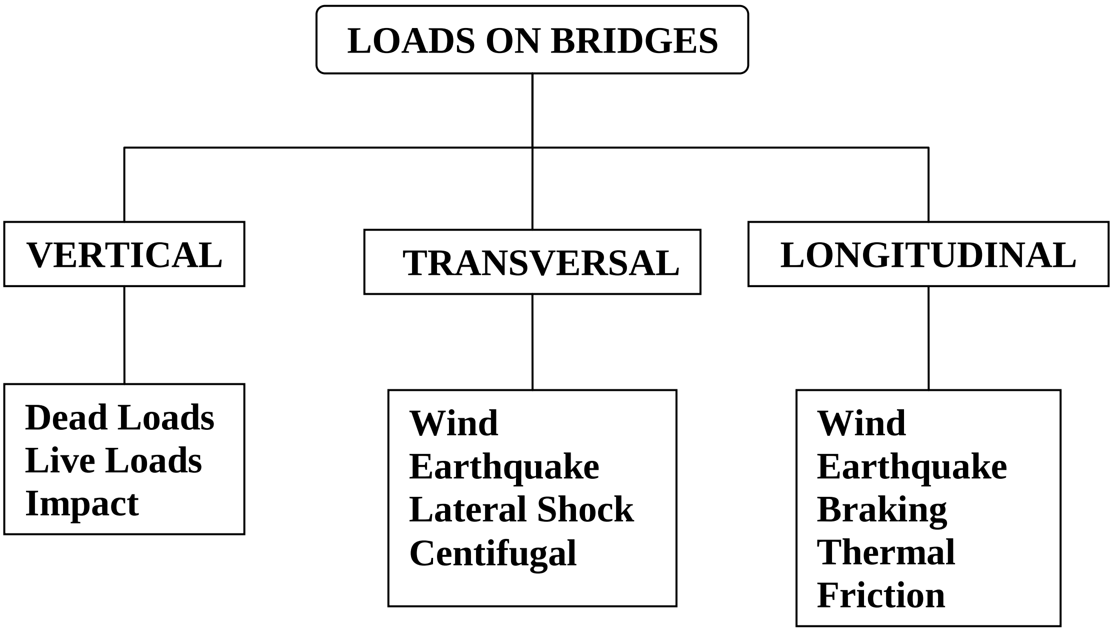


Fig. 2.1 Design Loads on Bridges

In the case of bridge decks consisting of reinforced concrete slabs, it is a common practice to apply the wearing surface and pour curbs, parapets, and sidewalks after the slab has hardened. The weight of these additional components is usually referred to as the superimposed dead load.

An important consideration in dead-load computation is to include, in addition to the a.m. components, weights of anticipated future wearing surface and extra utilities the bridge has to carry.

b) Live Loads

Live loads on bridges are caused by the traffic crossing the bridge. Design live loads are usually specified by relevant design codes in the form of equivalent traffic loads. Some traffic loads represent the weight of real vehicles that can travel over the bridge; other values and distributions are chosen in such a way that they produce maximum internal forces in bridge structures similar to those produced by real vehicles.

According to the Egyptian Code for design loads on roadway bridges, the roadway is divided into traffic lanes of 3 m width; the most critical lane for the design of a structural member is called the main lane. Two types of loads are specified in the Code for design:

i) Truck loads:

This load is intended to represent the extreme effects of heavy vehicles. It consists of a 60-ton truck in the main lane and a 30-ton truck in a secondary lane, which is taken next to the main lane. The arrangement of wheel loads is shown in Fig. 2.2a. The locations of the main and secondary lanes are chosen so as to produce maximum effect on the member considered.

For main girders with spans longer than 30 meters, an equivalent uniform load of 3.33 t/m^2 and 1.67 t/m^2 may be used instead of the 60-ton and 30-ton trucks for the design of main girders only.

ii) Uniform distributed load:

This load simulates the effects of normal permitted vehicles. It is applied on the traffic lanes and over the lengths that give the extreme values of the stress (or internal force) being considered. It may be continuous or discontinuous. It consists of a 500 kg/m^2 uniform load in the main lane in front and back of the main truck and 300 kg/m^2 in the remaining bridge floor areas, as shown in Fig. 2.2 b.

The interaction of moving loads and the bridge superstructure results in dynamic amplification of the moving loads, resulting in vibrations and increased stresses. This amplification was found to depend mainly on the natural frequency of the structure which is a function of its length. Consequently, the ***dynamic effect of moving loads*** is considered in the design by increasing the static values of the main lane loading by the ***impact factor*** **I** computed as:

$$\mathbf{I = 0.40 - 0.008\,L > 0} \qquad \textbf{(2.1)}$$

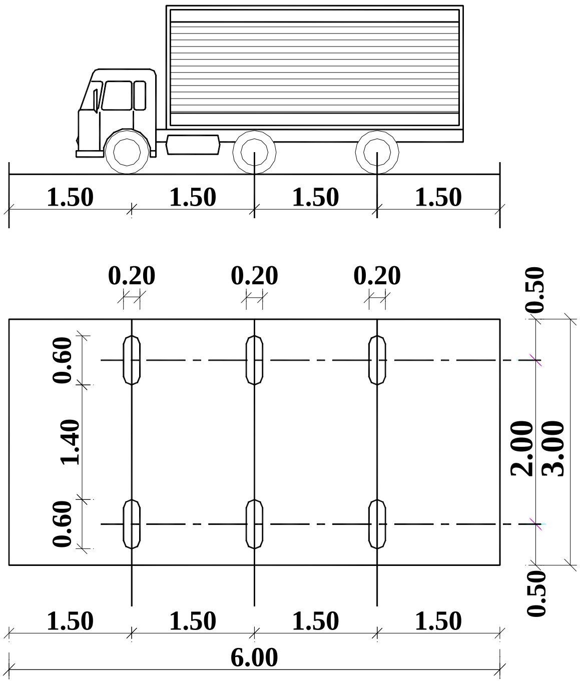
where L = loaded length of main traffic lane giving maximum effect and is evaluated as follows:

a) For directly loaded structural members, L is taken equal to the span length of loaded span or the cantilever length of loaded cantilevers.

b) For indirectly loaded structural members, L is taken equal to the span length of the directly loaded member transmitting the load or the span length of the indirectly loaded member, whichever is greater.

c) For two-way slabs, L is taken equal to the short span length.

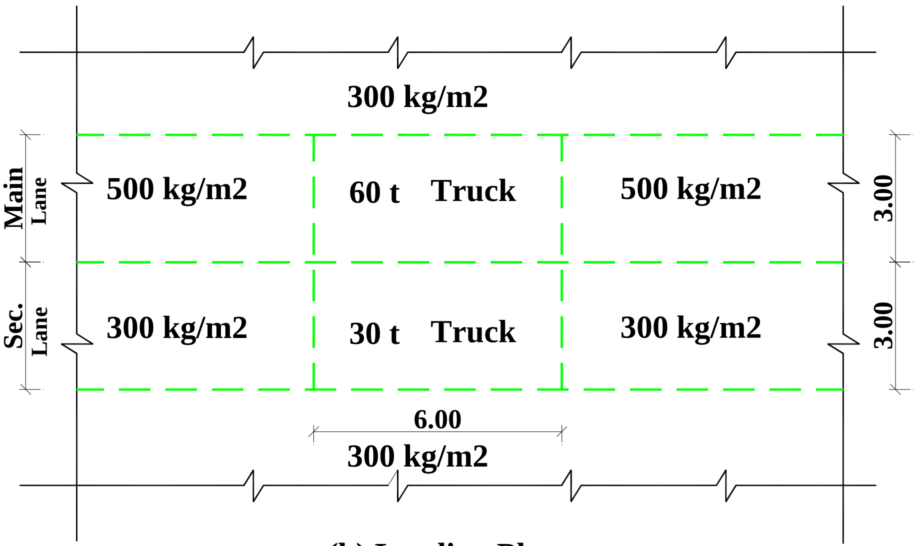
For the assessment of the bridge fatigue strength, the prescribed live load and impact values on roadway bridges shall be reduced by 50 %.



(a) Wheel Arrangement

60 T TRUCK = 6 x 10 T

30 T TRUCK = 6 x 5 T



(b) Loading Plan

Fig. 2.2 Live Loads on Roadway Bridges

c) Longitudinal Tractive Forces

The term longitudinal forces refer to forces that act in the direction of the longitudinal axis of the bridge; i.e., in the direction of traffic. These forces develop as a result of the braking effort (sudden stopping) , or the tractive effort (sudden acceleration). In both cases, the vehicle's inertia force is transferred to the bridge deck through friction between the deck and the wheels.

These forces are applied to the road surface parallel to the traffic lanes as shown in Figure 2.3. According to the Egyptian Code, they are taken equal to 25 % of the main lane loading without impact, with a maximum value of 90 tons.

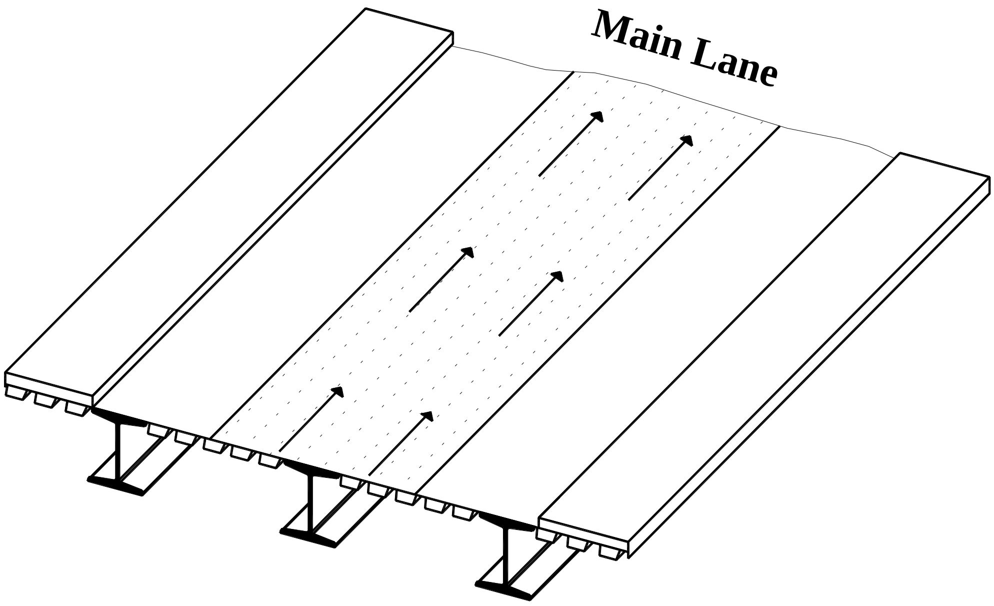


Fig. 2.3 Braking Forces on Roadway Bridges

d) Centrifugal Forces

When a body moves along a curved path with a constant speed, the body is subjected to a horizontal transversal force due to centrifugal acceleration and acts perpendicular to the tangent to the path. Curved bridges are therefore subjected to centrifugal forces applied by the vehicles that travel on them. According to the Egyptian Code, these forces are taken as two concentrated forces applied horizontally spaced at 50 m at the roadway surface level at the bridge centerline as shown in Fig. 2.4. The value of each force is computed from the equation:

$$\mathbf{C = 3000 / (R + 150)} \qquad \textbf{(2.2)}$$

Where C = centrifugal force, ton
R = radius of curvature, m

A vertical load of 30 tons distributed on a roadway area of 6 m long and 3 m wide is assumed to act with each force.

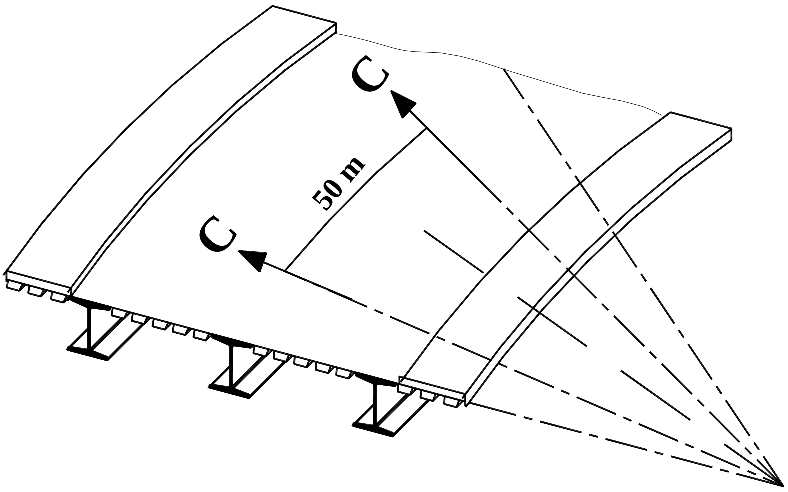


Fig. 2.4 Centrifugal Forces on Curved Roadway Bridges

e) Sidewalks

Many highway bridges, in urban and non-urban areas, have sidewalks (footpaths) for pedestrian traffic. On these areas a uniform distributed load of 300 kg/m^2 shall be considered in addition to the main bridge loads. Alternatively, a uniform load of 500 kg/m^2 acting alone shall be considered. Sidewalks not protected from vehicles cross over (parapet height less than 35 cm) shall be designed for a single wheel load of 5 tons acting on a distribution area 30*40 cm.

Handrails for sidewalks that are protected from highway traffic by an effective barrier are designed to resist a horizontal distributed force of 150 kg/m applied at a height of 1m above the footway. When sidewalks are not separated from the highway traffic by an effective barrier (parapet height less than 35 cm), The elements of the sidewalk shall also be checked for the effect of a vertical or horizontal concentrated load of 4 tons acting alone in the position producing maximum effect. The working stresses for this case are increased by 25 %.

2.3 RAILWAY DESIGN LOADINGS

a) Dead Load

Superimposed dead loads on railway bridges usually include the rails, the sleepers, the ballast (or any other mean for transmission of train loads to the structural elements), and the drainage system.

b) Train Loads

Train loads for railway bridges correspond to Train-type D of the Egyptian Railways as shown in Figure 2.5. Two 100 ton locomotives with 80 ton tenders are to be assumed, followed on one side only by an unlimited number of 80 ton loaded wagons. Different live load positions shall be tried to arrive at the specific position giving maximum effect. If two tracks are loaded at the same time, only 90 % of the specified loads for one track are used for both tracks. In case of three tracks, only 80 % of the specified loads are used. In case of four tracks or more, 75 % of the specified loads are used.

Train loads specified in the code are equivalent static loading and should be multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities. Values of dynamic factors depend on the type of deck (with ballast or open-deck) and on the vertical stiffness of the member being analyzed. For open-deck bridges values of dynamic factors are higher than for those with ballasted decks. Consideration of the vertical stiffness is made by adopting formulae in which the dynamic factor is a function of the length, L, of the influence line for deflection of the element under consideration. According to the Egyptian Code of Practice, impact effects of railway loads are taken into consideration by increasing the static values by the impact factor I computed as:

$$\mathbf{I = 24 / (24 + L)} \qquad \mathbf{(2.3)}$$

Where L (in meters) = Loaded length of one track, or the sum of loaded lengths of double tracks. For stringers L is taken equal to the stringer span. For cross girders L is taken equal to the sum of loaded tracks. For the main girders L is taken equal to the loaded length of one track for single track bridges or the sum of loaded lengthes of two tracks only in multiple track bridges.

The value of I in this formula has a minimum value of 25 % and a maximum value of 75 %. For ballasted floors with a minimum ballast thickness of 20 cm, the value of I computed from the given formula shall be reduced by 20 %. For bridges having multiple number of tracks, the dynamic effect shall be considered for the two critical tracks only.

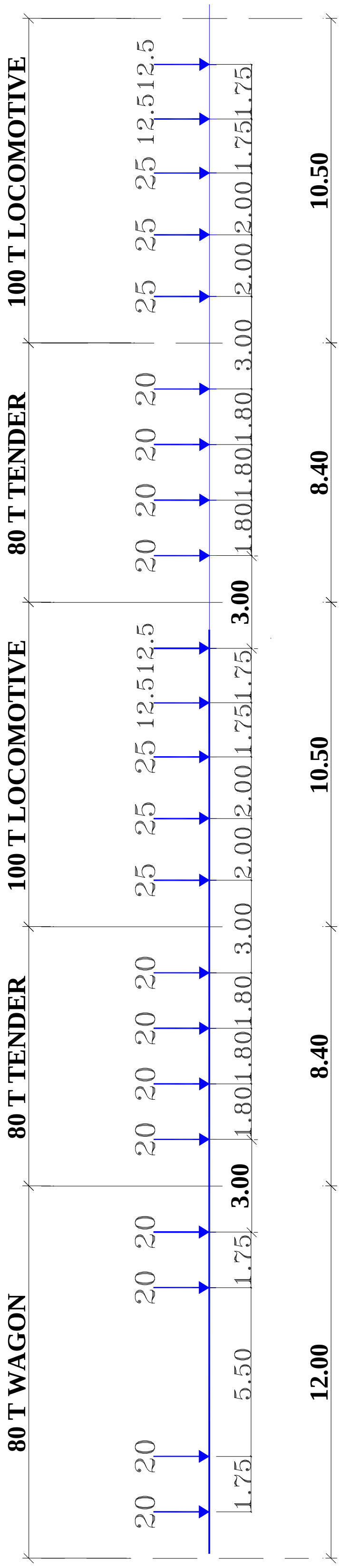


Fig. 2.5 Live Loads on Railway Bridges (Train Type D)

c) Longitudinal Braking and Tractive Forces

These forces, which equals 1/7 of the maximum live loads (without impact) supported by one track only, are considered as acting at rail level in a direction parallel to the tracks, Figure 2.6. For double track bridges, the braking or tractive force on the second track is taken as one half the above value. For bridges with more than two tracks, these forces are considered for two tracks only.

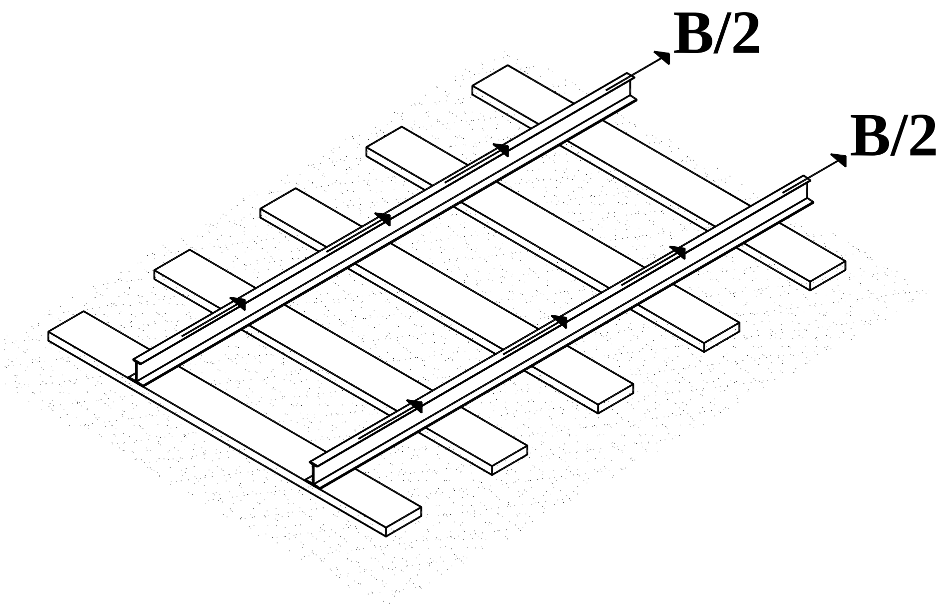


Fig. 2.6 Braking Forces on Railway Bridges

d) Centrifugal Forces

When the railway track is curved, the bridge elements shall be designed for a centrifugal force "C" per track acting radially at a height of 2 m above rail level. Its value is obtained as:

$$\mathbf{C = (V^2 / 127\ R)\ W} \qquad \mathbf{(2.4)}$$

Where C = centrifugal force in tons
V = maximum speed expected on the curve in Km/hr
R = radius of curvature in meters
W = maximum axle load in tons.

e) Lateral Forces From Train Wheels

To account for the lateral effect of the train wheels, the bridge elements are designed for a single load of 6 ton (without impact) acting horizontally in either direction at right angles to the track at the top of the rail, Figure 2.7. This force should be applied at a point in the span to produce the maximum effect in the element under consideration.

For elements supporting more than one track, only one lateral load is considered. For bridges on curves, design shall be based on the greater effect due to the centrifugal forces or the lateral shock.

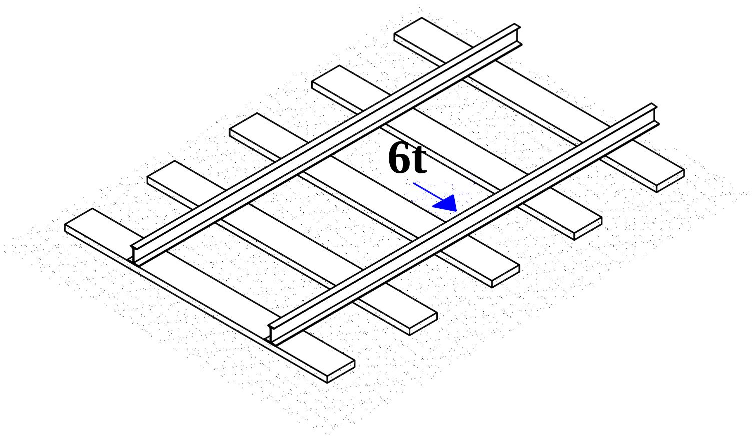


Fig. 2.7 Lateral Shock Forces on Railway Bridges

2.4 OTHER LOADS ON BRIDGES

a) Wind Loads

The wind actions on a bridge depend on the site conditions and the geometrical characteristics of the bridge. The maximum pressures are due to gusts that cause local and transient fluctuations about the mean wind pressure.

Because steel bridges have a low span-to-weight ratios, wind effects on bridges is very important and, if not properly considered, can lead to failure, see Fig 2.8.

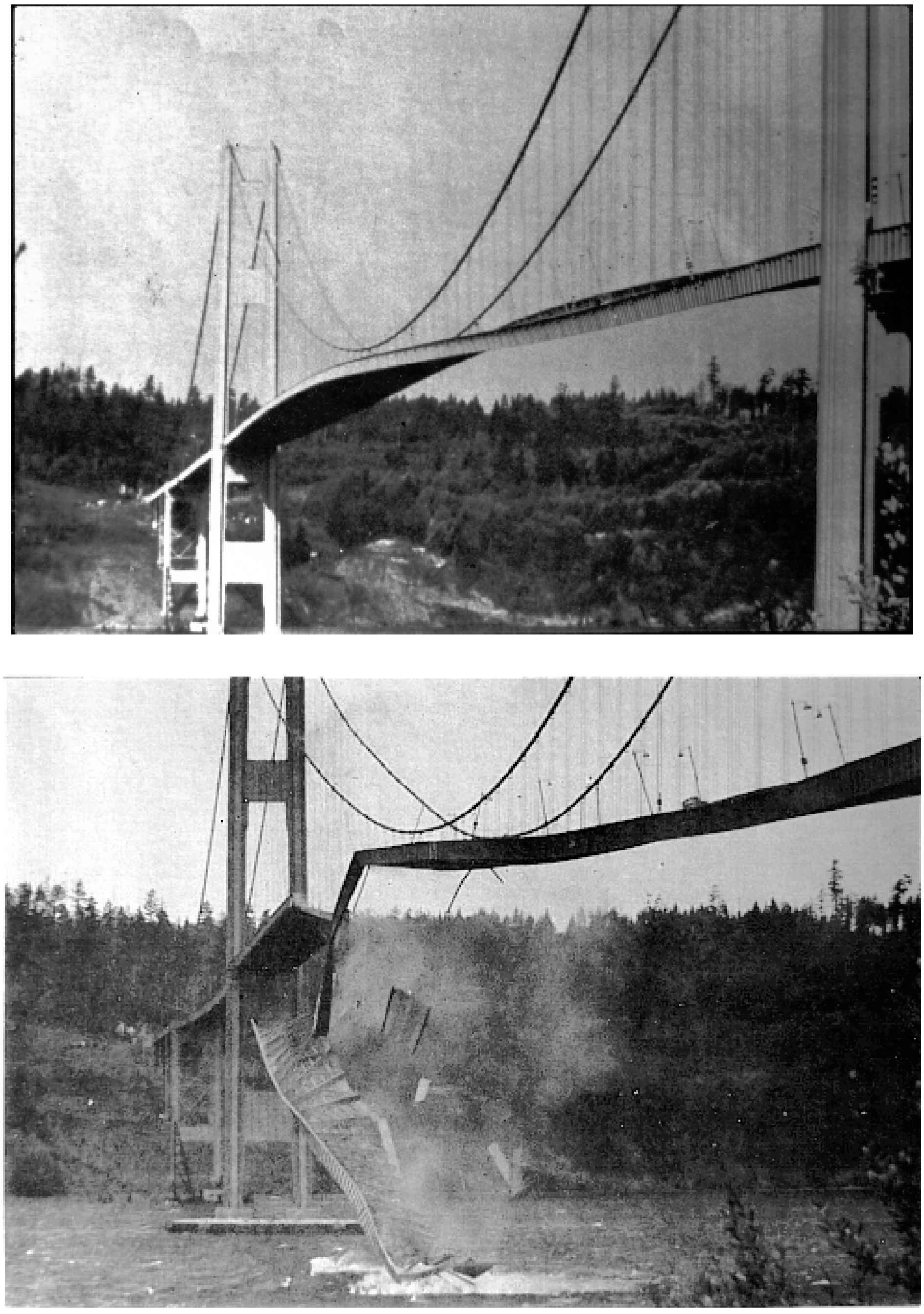


Fig. 2.8 Failure of a Suspension Bridge due to Wind loads

Design wind pressures are derived from the design wind speed defined for a specified return period. The wind load shall be assumed to act horizontally at the following values:

1) ***When the bridge is not loaded by traffic***: the wind pressure, on the exposed area of the bridge, is equal to 200 kg/m^2

2) ***When the bridge is loaded by traffic***: the wind pressure, on the exposed area of the bridge and the moving traffic, is equal to 100 kg/m^2.

Exposed area of traffic on bridges has the length corresponding to the maximum effects and in general a height of 3.00 m above the roadway level in highway bridges and 3.50 m above rail level in railway bridges, Figure 2.9. The exposed area of the bridge before the top deck slab is executed is taken equal to the area of two longitudinal girders. Wind pressure during construction can be reduced to 70 % of the specified values.

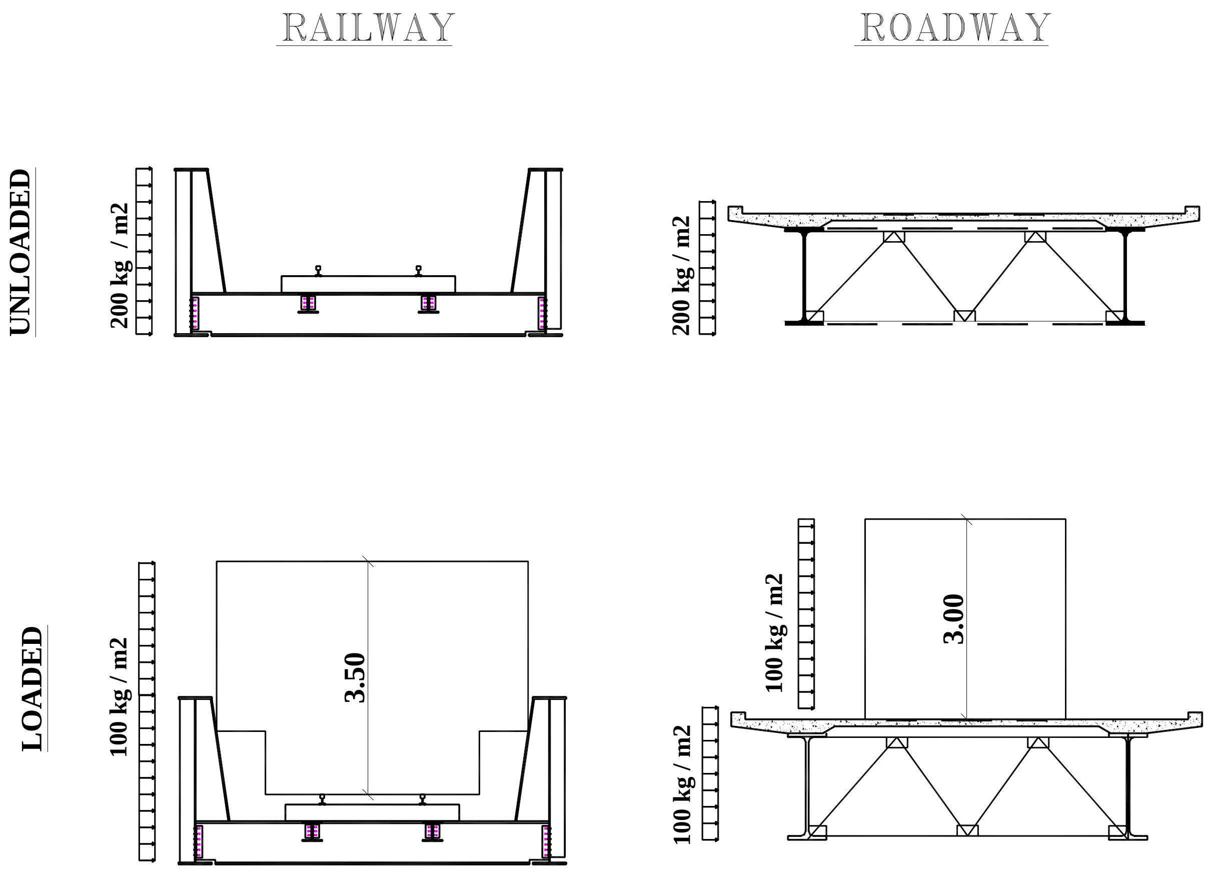


Fig. 2.9 Design Wind loads on Bridges

b) Thermal Effects on Bridge Structures

Daily and seasonal fluctuations in air temperature cause two types of thermal actions on bridge structures:

a) Changes in the overall temperature of the bridge (uniform thermal actions),

b) Differences in temperature (differential thermal actions) through the depth of the superstructure.

The coefficient of thermal expansion for steel may be taken as 1.2×10^{-5}° C. According to the Egyptian Code; bridge elements shall be designed for:

a) a + 30° C uniform change of temperature, Fig. 2.10 a, and

b) a + 15° C difference in temperature through the superstructure depth, Fig. 2.10b.

The mean temperature of the bridge shall be assumed at 20° C.

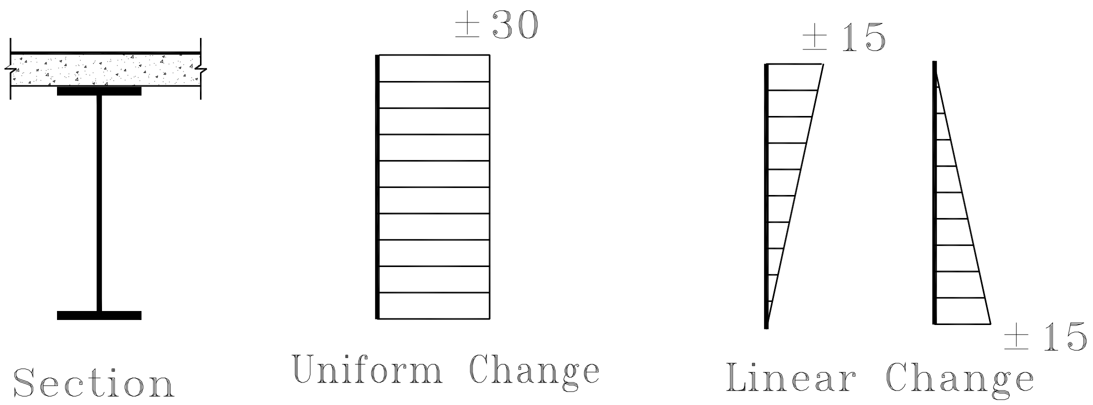


Fig. 2.10 Thermal Loads on Bridges

If the free expansion or contraction of the bridge due to changes in temperature is restrained, then stresses are set up inside the structure. Furthermore, differences in temperature through the depth of the superstructure cause internal stresses if the structure is not free to deform. A differential temperature pattern in the depth of the structure represented by a single continous line from the top to the bottom surface does not cause stresses in statically determinate bridges, e.g. simply supported beams, but will cause stresses in statically indeterminate structures due to reatraints at supports. If differential temperature is not represented by a single continous line from the top to the bottom surface, then thermal stresses are caused even in simple spans.

c) Shrinkage of Concrete

In principle, shrinkage gives a stress independent of the strain in the concrete. It is therefore equivalent to the effect of a differential temperature between concrete and steel. The effect of shrinkage can thus be estimated as equivalent to a uniform decrease of temperature of 20° C.

In composite girders the effect of concrete shrinkage is considered by using a modified value of the modular ratio that is equal to three times of the normal value. Generally, shrinkage effects are only taken into account when the effect is additive to the other action effects.

d) Settlement of Foundations

The settlements of foundations determined by geotechnical calculations should be taken into account during design of the superstructure. For continuous beams the decisive settlements are differential vertical settlements and rotations about an axis parallel to the bridge axis. For earth anchored bridges (arch bridges, frame bridges and suspension bridges) horizontal settlements have to be considered.

Where larger settlements are to be expected it may be necessary to design the bearings so that adjustments can be made, e.g. by lifting the bridge superstructure on jacks and inserting shims. In such a case the calculations should indicate when adjustments have to be made.

e) Friction of Bearings

It should be checked whether the unavoidable friction of bearings can induce forces or moments that have to be considered in the design of the structural elements.

According to the Egyptian Code, the force due to friction on the expansion bearings under dead load only shall be taken into account and the following coefficients of friction shall be used:

a. Roller Bearings:	One or two rollers	0.03
	Three or more rollers	0.05
b. Sliding Bearings:	Steel on Cast iron or steel	0.25

In a continuous beam with a hinged bearing at the center and longitudinally movable bearings on both sides, expansion (or contraction) of the beam induces symmetrical frictional forces. These forces are in horizontal equilibrium if a constant coefficient of friction is assumed, and they normally result in moderate axial forces in the main girders. However, to take into account the uncertainty in the magnitude of frictional forces it may be reasonable to assume full friction in the bearings on one side of the fixed bearing and half friction on the other side.

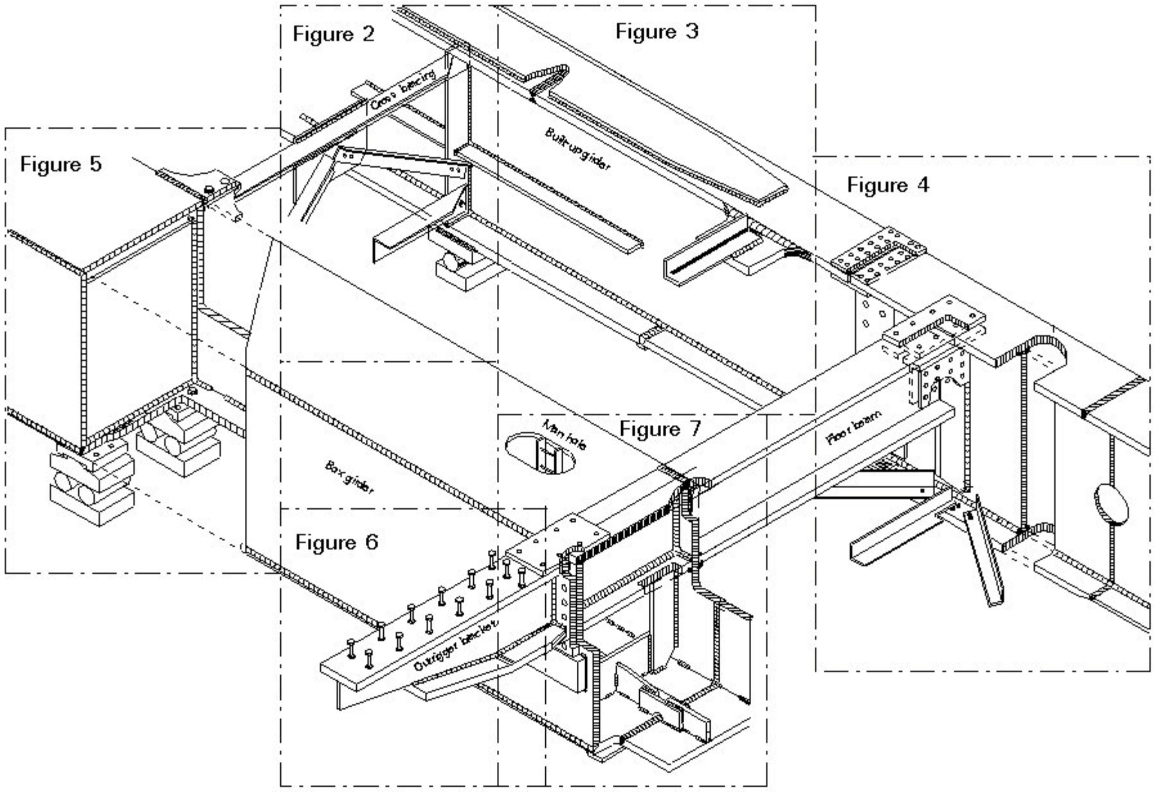


Figure 1 General overview of case studies

CHAPTER 3

DESIGN CONSIDERATIONS

CHAPTER 3

DESIGN CONSIDERATIONS

3.1 DESIGN PHILOSOPHIES

The aim of design is that the bridge should sustain all loads and deformations liable to occur during its construction and use. A bridge design should satisfactorily accomplish the objectives of constructability, safety, and serviceability. Simply stated, a bridge design should permit safe structural erection as planned and be able to safely perform its intended function during its design life.

The basis for structural design philosophies is the known stress-strain relationship of the material. It is usually assumed that the material is (a) homogeneous, i.e., has the same physical properties at all points, (b) obeys Hook's low, i.e., the material is linearly elastic, and (c) isotropic, i.e., has the same elastic properties in all directions.

Two philosophies of design are in current use. The ***working stress design*** philosophy has been the principal one used during the past 100 years. According to this philosophy, a structural element is designed so that stresses computed under the action of working, or service, loads do not exceed predesignated allowable values. These allowable stresses are predescribed by a building code or specification to provide a factor of safety against attainment of some limiting stresses, such as the minimum specified yield stress or the stress at which buckling occurs. The computed stresses are well within the elastic range; i.e., stresses are proportional to strains.

The other design philosophy is generally referred to as ***limit states design.*** This relatively recent term includes the methods commonly referred to as "ultimate strength design", "strength design", "plastic design", "load factor design", "limit design", and more recently, "load and resistance factor design (LRFD)". Limit states is a general term meaning "those conditions of a structure in which the structure ceases to fulfill the function for which it was designed". Those states can be divided into the categories of ***strength*** and ***serviceability***. Strength (i.e., safety) limit states are plastic strength, buckling, fatigue, fracture, overturning and sliding. Serviceability limit states are those concerned with the use of the structure, such as deflection, vibration, permanent deformation and cracking. In limit states design, the strength limit states are dealt with by applying factors to the loadings, focusing attention on the failure modes (limit states) by making comparisons for safety at the limit state condition, rather than in the service load range as is done for working stress design.

The design philosophy followed throughout this book is based on the latest edition (2001) of the **Egyptian Code of Practice for Steel Constructions and Bridges (ECP).** This code follows the allowable stress design method in which the bridge elements (members and connections) are proportioned on the basis of design loads and allowable stresses for the materials under service conditions. Values of the basic allowable stresses for different cases are given in Egyptian Building Code for the Design of Steel Structures and Bridges (ECP 2001) chapter 2 for members, chapter 3 for fatigue, and chapters 5, 6 for welded and bolted connections. The main sections of the code are summarized in this Chapter.

3.2 ALLOWABLE STRESSES FOR STRUCTURAL STEEL

3.2.1 GENERAL APPLICATION

The following prescriptions, together with any other provisions stipulated in the special specifications, are intended to apply to the design and construction of steel bridges and buildings.

The structural safety shall be established by computing the stresses produced in all parts and ascertaining that they do not exceed the allowable (working) stresses specified herein, when these parts are subjected to the most unfavourable conditions or combinations of the loads and forces according to the current Egyptian Code of Practice for Loads and Forces for Structural Elements. In applying the said prescriptions, approved scientific methods of design shall be used. Deflections shall be computed and they shall in no case exceed the limits herein after specified.

3.2.2 PRIMARY AND ADDITIONAL STRESSES

3.2.2.1 For the purpose of computing the maximum stress in a structure, the straining actions shall be calculated for two cases:

Case I: Primary Stresses due to:
Dead Loads + Live Loads or Superimposed Loads + Dynamic Effects + Centrifugal Forces.

Case II: Primary and Additional Stresses due to:
Case **I** + [(Wind Loads or Earthquake Loads), Braking Forces, Lateral Shock Effect, Change of Temperature, Frictional Resistance of Bearings, Settlement of Supports in addition to the Effect of Shrinkage and Creep of Concrete]

3.2.2.2 Stresses due to Wind Loads shall be considered as primary for such structures as towers, transmission poles, wind bracing systems, etc...

3.2.2.3 In designing a structure, members shall, in the first instance, be so designed that in no case the stresses due to case **I** exceed the allowable stresses specified in the present code.

The design should then be checked for case **II** (primary + additional stresses), and the stresses shall in no case exceed the aforesaid allowable stresses by more than **20** %.

3.2.3 SECONDARY STRESSES

Structures should be so designed, fabricated and erected as to minimize, as far as possible, secondary stresses and eccentricities.

Secondary stresses are usually defined as bending stresses upon which the stability of the structure does not depend and which are induced by rigidity in the connections of the structure already calculated on the assumption of frictionless or pin-jointed connections.

In ordinary welded, bolted or riveted trusses without sub-panelling, no account usually needs to be taken of secondary stresses in any member whose depth (measured in the plane of the truss) is less than **1/10** of its length for upper and lower chord members, and **1/15** for web members. Where this ratio is exceeded or where sub-panelling is used, secondary stresses due to truss distortion shall be computed, or a decrease of **15%** in the allowable stresses prescribed in this code shall be considered.

Bending stresses in the verticals of trusses due to eccentric connections of cross-girders shall be considered as secondary.

The induced stresses in the floor members and in the wind bracing of a structure resulting from changes of length due to the stresses in the adjacent chords shall be taken into consideration and shall be considered as secondary.

Stresses which are the result of eccentricity of connections and which are caused by direct loading shall be considered as primary stresses.

For bracing members in bridges, the maximum allowable stresses shall not exceed **0.85** of the allowable stresses specified in this code if the bridge has not been considered as a space structure.

3.2.4 STRESSES DUE TO REPEATED LOADS

Members and connections subject to repeated stresses (whether axial, bending or shearing) during the passage of the moving load shall be proportioned according to Chapter 3 of ECP 2001 which is summarized in section 3.3 of this Chapter.

3.2.5 ERECTION STRESSES

Where erection stresses, including those produced by the weight of cranes, together with the wind pressure, would produce a stress in any part of structure in excess of **25** % above the allowable stresses specified in this code, such additional material shall be added to the section or other provision made, as is necessary, to bring the erection stresses within that limit.

3.2.6 ALLOWABLE STRESSES FOR STRUCTURAL STEEL

3.2.6.1 General

Allowable stresses for structural steel shall be determined according to the grade of steel used. Structural sections shall be classified (depending on the maximum width-thickness ratios of their elements subject to compression) as follows:

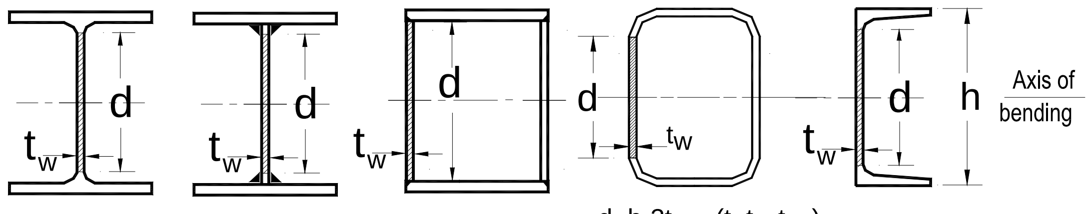
1- **Class 1. (compact sections):**
Are those which can achieve the plastic moment capacity without local buckling of any of its compression elements.

2- **Class 2. (non- compact sections):**
Are those which can achieve the yield moment capacity without local buckling of any of its compression elements.

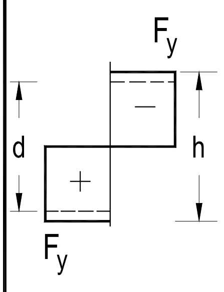
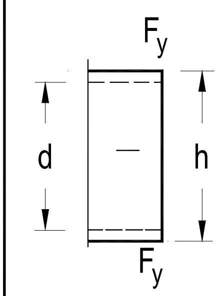
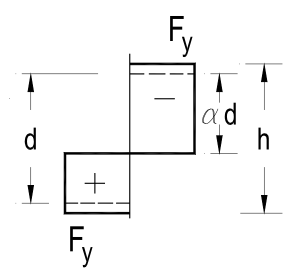
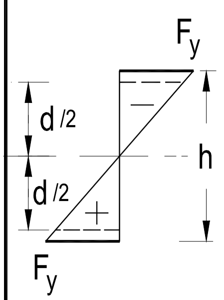
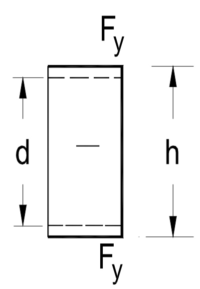
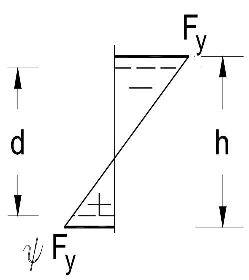
The limiting width to thickness ratios of class 1 and 2 compression elements are given in Table 3.1.

Table (3.1a) Maximum Width to Thickness Ratios for Stiffened Compression Elements

(a) Webs: (Internal elements perpendicular to axis of bending)



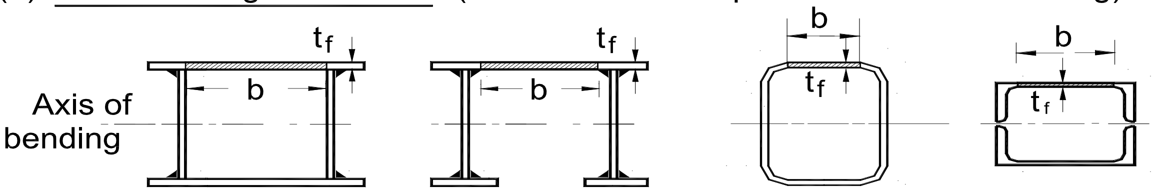
$d = h - 3t$, $(t = t_f = t_w)$

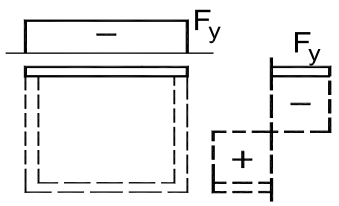
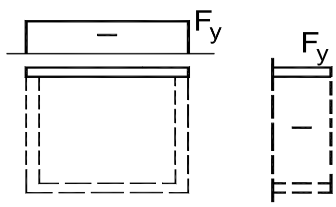
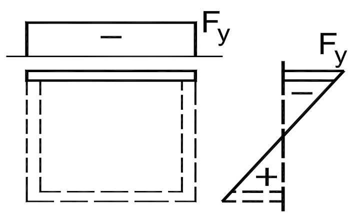
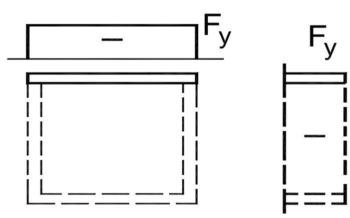
Class / Type	Web Subject to Bending	Web Subject to Compression	Web Subject to Bending and Compression	
1. Compact Stress distribution in element.				
	$\alpha = 0.5$	$\alpha = 1.0$	$\alpha > 0.5$	$\alpha \leq 0.5$
	$\frac{d}{t_w} \leq \frac{127}{\sqrt{F_y}}$	$\frac{d}{t_w} \leq \frac{58}{\sqrt{F_y}}$	$\frac{d}{t_w} \leq \frac{699/\sqrt{F_y}}{13\alpha - 1}$	$\frac{d}{t_w} \leq \frac{63.6/\alpha}{\sqrt{F_y}}$
2.Non-Compact Stress distribution in element.				
	$\psi = -1$	$\psi = 1$	$\psi > -1$	$\psi \leq -1$
	$\frac{d}{t_w} \leq \frac{190}{\sqrt{F_y}}$	$\frac{d}{t_w} \leq \frac{64}{\sqrt{F_y}}$	$\frac{d}{t_w} \leq \frac{190/\sqrt{F_y}}{2+\psi}$	$\frac{d}{t_w} \leq \frac{95(1-\psi)\sqrt{-\psi}}{\sqrt{F_y}}$

F_y in t/cm^2

Table (3.1b) Maximum Width to Thickness Ratios for Stiffened Compression Elements

(b) Internal Flange Elements: (Internal elements parallel to axis of bending)

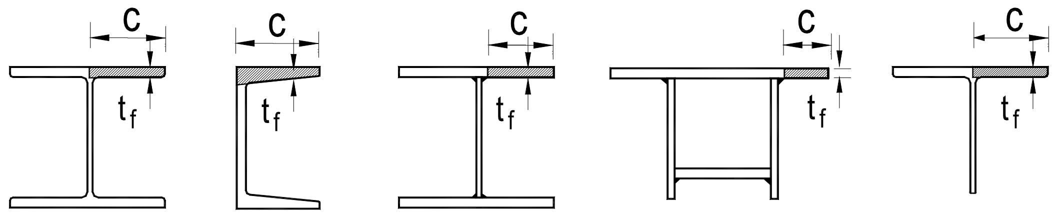


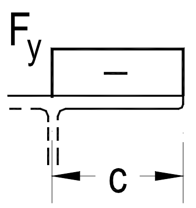
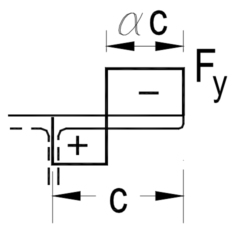
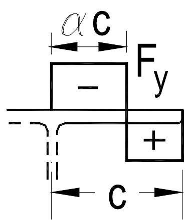
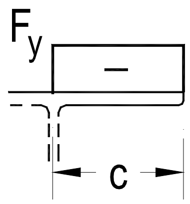
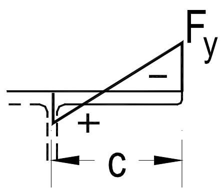
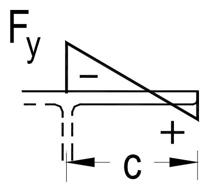
Class / Type	Section in Bending	Section in Compression
1. Compact Stess distribution in element and across section		
	$\frac{b}{t_f} \leqslant \frac{58}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$
2. Non-Compact Stess distribution in element and across section		
	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$

F_y in t/cm^2

Table (3.1c) Maximum Width to Thickness Ratios for Unstiffened Compression Elements

(c) Outstand Flanges

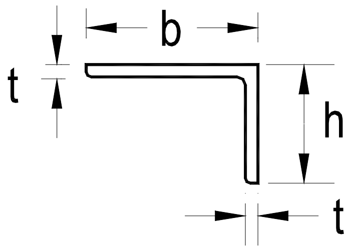
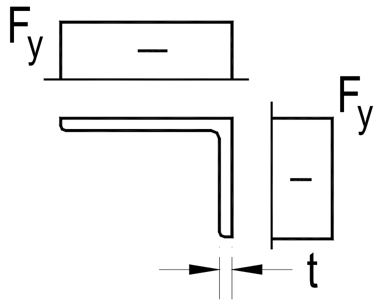
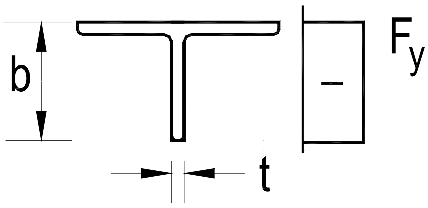
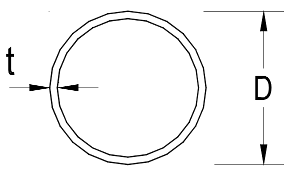


Class / Type	Flange Subject to Compression Due to M_x	Flange Subject to Compression and Bending	
		Tip in Comprssion	Tip in Tension
1. Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{\alpha F_y}$
Welded	$\frac{c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{\alpha F_y}$
2. Non-Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 23/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 35\sqrt{K_\sigma/F_y}$	
Welded	$\frac{c}{t_f} \leqslant 21/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 32\sqrt{K_\sigma/F_y}$	

F_y in t/cm^2

For K_σ see Tables 2.3 & 2.4

Table (3.1d) Maximum Width to Thickness Ratios for Compression Elements

(d) Angles: Refer also to (Table 2.1c) "Outstand flanges"	 (Does not apply to angles in continuous contact with other components)
Class	Section In Compression
Stress distribution across section	
Non-compact	$b/t \leq 23/\sqrt{F_y}$; $(b+h)/2t \leq 17/\sqrt{F_y}$ (*)
Class	Section In Compression
(e) T-section:	
Non-compact	$b/t \leq 30/\sqrt{F_y}$
(f) Tubular section:	
Class	Section In Bending and/or Compression
1. Compact	$D/t \leq 165/F_y$
2. Non-Compact	$D/t \leq 211/F_y$

F_y in t/cm^2 (*) For unequal angles

3- Class 3. (slender sections):

Are those which cannot achieve yield moment capacity without local buckling of any of its compression elements.

When any of the compression elements of a cross-section is classified as class-3, the whole cross section shall be designed as class-3 cross section.

3.2.6.2 Allowable Stress in Axial Tension F_t

On effective net area:

$$\mathbf{F_t = 0.58\ F_y} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.1}$$

Grade of Steel	F_t (t/cm^2)	
	t ≤ 40	**40** mm < **t ≤100** mm
St 37	**1.4**	**1.3**
St 44	**1.6**	**1.5**
St 52	**2.1**	**2.0**

3.2.6.3 Allowable Stress in Shear q_{all}

3.2.6.3.1 The allowable shear stress on the gross effective area in resisting shear is:

$$\mathbf{q_{all} = 0.35\ F_y} \quad \ldots\ldots\ldots \quad \mathbf{3.2}$$

Grade of Steel	q_{all} (t/cm^2)	
	t ≤ 40 mm	**40** mm < **t ≤100**
St 37	**0.84**	**0.75**
St 44	**0.98**	**0.89**
St 52	**1.26**	**1.17**

The effective area in resisting shear of rolled shapes shall be taken as the full height of the section times the web thickness while for fabricated shapes it shall be taken as the **web** height times the web thickness.

In addition, the shear buckling resistance shall also be checked as specified in Clause **3.2.6.3.2** when:

-For unstiffened webs:

$$\frac{d}{t_w} > \frac{105}{\sqrt{F_y}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{3.3}$$

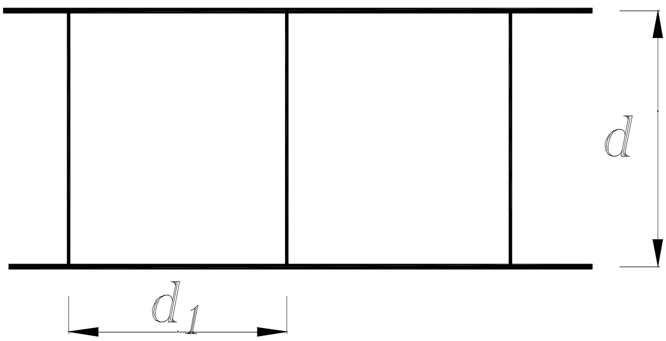
- For stiffened webs:

$$\frac{d}{t_w} > 45\sqrt{\frac{K_q}{F_y}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{3.4}$$

Where K_q **=buckling factor for shear**

$$k_q = 4.00 + 5.34 / \alpha^2 \qquad \alpha < 1 \ \ldots\ldots \ \textbf{3.5}$$
$$k_q = 5.34 + 4.00 / \alpha^2 \qquad \alpha > 1 \ \ldots\ldots \ \textbf{3.6}$$

Where $\alpha = d_1 / d$ & d_1 = spacing of transversal stiffeners



3.2.6.3.2 Allowable Buckling Stress in Shear q_b

Depending on the web slenderness parameter :

$$\lambda_q = \frac{d/t_w}{57}\sqrt{\frac{F_y}{K_q}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{3.7}$$

The buckling shear stress is :

For $\lambda_q \le 0.8$ $\quad q_b = 0.35\, F_y$ …………………………… **3.8**

$0.8 < \lambda_q < 1.2$ $\quad q_b = (1.5 - 0.625\,\lambda_q)\,(0.35\, F_y)$ ……. **3.9**

$\lambda_q \ge 1.2$ $\quad q_b = \dfrac{0.9}{\lambda_q}\,(0.35\, F_y)$ …………………. **3.10**

3.2.6.4 Allowable Stress in Axial Compression F_c

On gross section of axially loaded symmetric (having compact, non-compact or slender section) compression members in which the shear center coincides with the center of gravity of the section and meeting all the width-thickness ratio requirements of Clause 3.2.6.1:

For λ = slenderness ratio = k l/ r < 100 :

$$F_c = 0.58F_y - \frac{(0.58F_y - 0.75)}{10^4}\lambda^2 \quad \ldots\ldots \quad 3.11$$

Grade of Steel	F_c (t/cm²)		
	t ≤ 40 mm	**40 mm < t≤100 mm**	
St 37	$F_c = (1.4 - 0.000065\lambda^2)$	$F_c = (1.3 - 0.000055\lambda^2)$	……**3.12**
St 44	$F_c = (1.6 - 0.000085\lambda^2)$	$F_c = (1.5 - 0.000075\lambda^2)$	……**3.13**
St 52	$F_c = (2.1 - 0.000135\lambda^2)$	$F_c = (2.0 - 0.000125\lambda^2)$	……**3.14**

For all grades of steel

For $\lambda = kl/r \geq 100$: $F_c = 7500/\lambda^2$ …………………….. **3.15**

For compact and non-compact sections, the full area of the section shall be used, while for slender sections, the effective area shall be used.

In case of sections eccentrically connected to gusset plates (e.g. one angle), unless a more accurate analysis is used, the allowable compressive stresses shall be reduced by 40% from Fc in case the additional bending stresses due to eccentricity are not calculated.

3.2.6.5 Allowable Stress in Bending F_b

2.6.5.1 Tension and compression due to bending on extreme fibers of "compact" sections symmetric about the plane of their minor axis:

$$F_b = 0.64\, F_y \quad \ldots\ldots \quad 3.16$$

Grade of Steel	F_b (t/cm²)	
	$t \leq 40$ mm	40 mm < $t \leq 100$ mm
St 37	**1.54**	**1.38**
St 44	**1.76**	**1.63**
St 52	**2.30**	**2.14**

In order to qualify under this section:

1- The member must meet the compact section requirements of Table 3.1.

2- The laterally unsupported length (L_u) of the compression flange is limited by

i- For box sections:

$$\left.\begin{aligned} &L_u < \frac{84}{F_y} b_f \\ &L_u \leq \left(137 + 84\frac{M_1}{M_2}\right) b_f / F_y \end{aligned}\right\} \quad \mathbf{3.17}$$

ii- For other sections

$$\left.\begin{aligned} &L_u \leq \frac{20 b_f}{\sqrt{F_y}} \\ &L_u \leq \frac{1380 A_f}{d F_y} C_b \end{aligned}\right\} \quad \mathbf{2.18}$$

Where $\mathbf{b_f}$ is the compression flange width, $\mathbf{M_1/M_2}$ is the algebraic ratio of the smaller to the larger end moments taken as positive for reverse curvature bending, $\mathbf{d}$ is the web depth and $\mathbf{C_b}$ is given in Equation 3.27.

3.2.6.5.2 Tension and compression due to bending on extreme fibers of doubly symmetrical I-shape members meeting the compact section requirements of

Table 2.1(c), and bent about their minor axis; solid round and square bars; solid rectangular sections bent about their minor axis:

$$F_b = 0.72\ F_y \qquad \ldots\ldots\ldots\ldots\ldots\ldots \qquad 3.19$$

3.2.6.5.3 Tension and compression on extreme fibers of rectangular tubular sections meeting the compact section requirements of Table 3.1(b), and bent about their minor axis:

$$F_b = 0.64\ F_y \qquad \ldots\ldots\ldots\ldots\ldots\ldots \qquad 3.20$$

3.2.6.5.4 Tension and compression on extreme fibers of box-type flexural members meeting the "non-compact" section requirements of Table 3.1(b):

$$F_b = 0.58\ F_y \qquad \ldots\ldots\ldots\ldots\ldots\ldots \qquad 3.21$$

3.2.6.5.5 On extreme fibers of flexural members not covered by Clauses 3.2.6.5.1 – 3.2.6.5.4 :

1- Tension F_{bt}

$$F_{bt} = 0.58\ F_y \qquad \ldots\ldots\ldots\ldots\ldots\ldots \qquad 3.22$$

Hence, F_{bt} is taken as follows:

Grade of Steel	F_{bt} (t/cm^2)	
	$t \leqslant 40$ mm	**40 mm < t**
St 37	**1.4**	**1.3**
St 44	**1.6**	**1.5**
St 52	**2.1**	**2.0**

2- Compression F_{bc}

I. When the compression flange is braced laterally at intervals exceeding L_u as defined by Equations 3.17 or 3.18, the allowable bending stress in compression F_{bc} will be taken as the larger value from Equations 3.23 and 3.24 or 3.25) with a maximum value of 0.58 F_y :

i- For shallow thick flanged sections, for any value of L/r_T, the lateral torsional buckling stress is governed by the torsional strength given by:

$$F_{ltb1} = \frac{800}{L_u . d / A_f} C_b \ \leq 0.58 \ F_y \ \ldots\ldots\ldots\ldots\ldots\ldots 3.23$$

ii- For deep thin flanged sections, the lateral torsional buckling stress is governed by the buckling strength given by:

a- When $84\sqrt{\frac{C_b}{F_y}} \leq L_u / r_T \leq 188\sqrt{\frac{C_b}{F_y}}$,then :

$$F_{ltb2} = (0.64 - \frac{(L_u / r_T)^2 F_y}{1.176x10^5 C_b})F_y \ \leq 0.58 \ F_y \ \ldots\ldots\ldots\ldots\ldots 3.24$$

b- When $L_u / r_T > 188\sqrt{\frac{C_b}{F_y}}$, then:

$$F_{ltb2} = \frac{12000}{(L_u / r_T)^2} C_b \ \leq 0.58 \ F_y \ \ldots\ldots\ldots\ldots\ldots\ldots 3.25$$

Alternatively, the lateral torsional buckling stress can be computed more accurately as the resultant of the above mentioned two components as:

$$F_{ltb} = \sqrt{F_{ltb_1}^2 + F_{ltb_2}^2} \ \leq 0.58\, F_y \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.26$$

In the above Equations:

L_u = Effective laterally unsupported length of compression flange

= **K**. (distance between cross sections braced against twist, or lateral displacement of the compression flange in cm).

K = Effective length factor (as given in Chapter 4 of Code)

r_T = Radius of gyration about minor axis of a section comprising the compression flange plus one third of compression web area (in cms)

A_f = ($b_f * t_f$) Area of compression flange (in cm^2)

D = Depth of web (in cm)
$\mathbf{F_y}$ = Yield stress (t/cm^2)
$\mathbf{t_f}$ = Compression flange thickness (in cm)

$\mathbf{C_b}$ = Coefficient depending on the type of load and support conditions as given in Table 3.2. For cases of unequal end moments without transverse loads, ($\mathbf{C_b}$) can be computed from the expression :

$$\mathbf{C_b = 1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \le 2.3} \qquad \mathbf{3.27}$$

…………………...

Where: $\mathbf{(M_1/M_2)}$ is the algebraic ratio of the smaller to the larger end moments taken as positive for reverse curvature bending. When the bending moment at any point within the un-braced length is larger than the values at both ends of this length, the value of ($\mathbf{C_b}$) shall be taken as unity.

II- Compression on extreme fibres of channels bent about their major axis and meeting the requirements of Table 3.1.

$$\mathbf{F_{ltb} = \frac{800}{(L_u.d/A_f)} C_b \le 0.58\,F_y} \quad \ldots. \quad \mathbf{3.28}$$

III. Slender sections which do not meet the non-compact section requirements of Table 3.1 shall be designed using the same allowable stresses used for non-compact sections except that the section properties used in the design shall be based on the effective widths b_e of compression elements as specified in Table 3.3 for stiffened elements and Table 3.4 for unstiffened elements. The effective width is calculated using a reduction factor $\boldsymbol{\rho}$ as $\mathbf{b_e = \rho\ \bar{b}}$

Where:

$$\boldsymbol{\rho = (\bar{\lambda}_p - 0.15 - 0.05\ \psi) / \bar{\lambda}_p^{\,2} \le 1} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\mathbf{3.29}$$

and

$\bar{\lambda}_p$ = normalized plate slenderness given by

$$\boldsymbol{\bar{\lambda}_p = \frac{\bar{b}/t}{44}\sqrt{\frac{F_y}{K_\sigma}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad \mathbf{3.30}$$

K_σ	=	Plate buckling factor which depends on the stress ratio ψ as shown in Tables 3.3 and 3.4.
$\overline{b}$	=	Appropriate width, (see Table 3.1) as follows :
$\overline{b}$	=	**d** for webs
$\overline{b}$	=	**b** for internal flange elements (except rectangle hollow sections)
$\overline{b}$	=	**b-3t** for flanges of rectangle hollow sections.
$\overline{b}$	=	**c** for outstand flanges
$\overline{b}$	=	**b** for equal leg angles
$\overline{b}$	=	**b** or **(b+h)/2** for unequal leg angles
t	=	relevant thickness.

Table (3.2) Values of Coefficients K and C_b

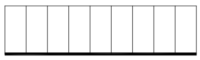
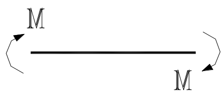
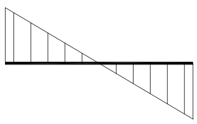
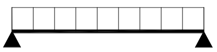
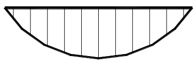
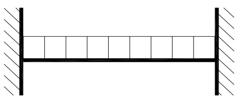
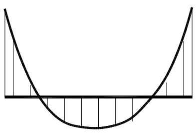
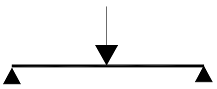
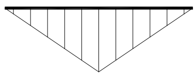
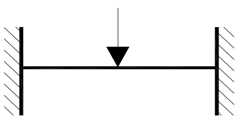
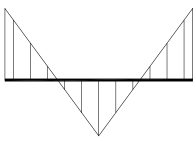
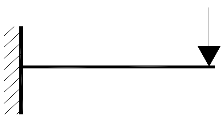
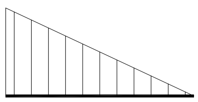
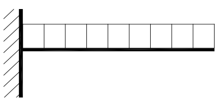
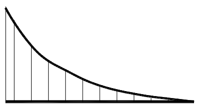
Loading	Bending Moment Daigram	End Restraint About Y-axis	Effective Length Factor K	C_b
		Simple Fixed	1.0 0.5	1.00 1.00
		Simple Fixed	1.0 0.5	2.30 2.30
		Simple Fixed	1.0 0.5	1.13 1.00
		Simple Fixed	1.0 0.5	1.30 0.90
		Simple Fixed	1.0 0.5	1.35 1.07
		Simple Fixed	1.0 0.5	1.70 1.04
		Warping Restrained	1.0	1.50
		Restrained	1.0	2.10

Table (3.3) Effective Width and Buckling Factor for Stiffened Compression Elements

For $1 > \psi \geq -1$: $k_\sigma = \dfrac{16}{[(1+\psi)^2 + 0.112(1-\psi)^2]^{0.5} + (1+\psi)}$						$-1 > \psi > -2$
$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	
Buckling Factor k_σ	4.0	$\dfrac{8.2}{1.05+\psi}$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98(1-\psi)^2$

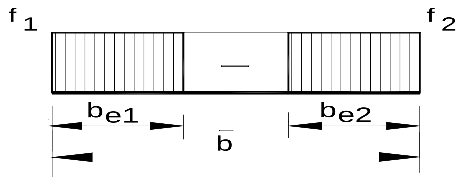
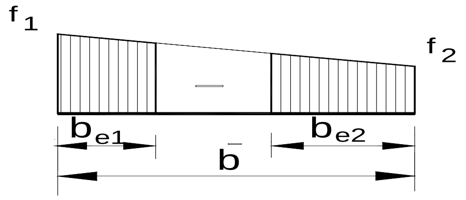
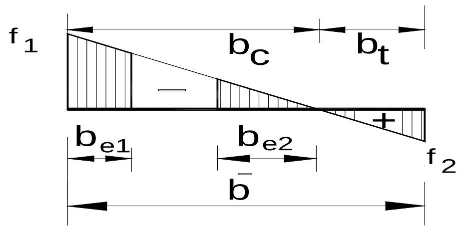
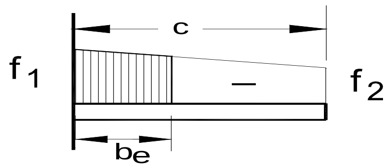
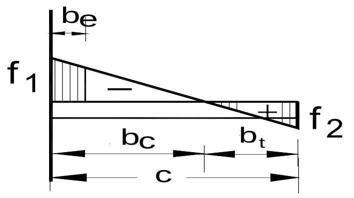
<table>
<tr><th>Stress Distribution</th><th>Effective Width b_e for
$\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi) / \bar{\lambda}_p^2 \leq 1$</th></tr>
<tr><td>
</td><td>$\psi = 1$:
$b_e = \rho \bar{b}$
$b_{e1} = 0.5\, b_e$
$b_{e2} = 0.5\, b_e$</td></tr>
<tr><td>
</td><td>$1 > \psi \geq 0$:
$b_e = \rho \bar{b}$
$b_{e1} = 2\, b_e / (5 - \psi)$
$b_{e2} = b_e - b_{e1}$</td></tr>
<tr><td>
</td><td>$\psi < 0$:
$b_e = \rho b_c = \rho \bar{b} / (1 - \psi)$
$b_{e1} = 0.4\, b_e$
$b_{e2} = 0.6\, b_e$</td></tr>
</table>

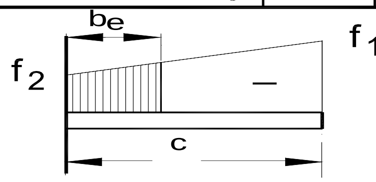
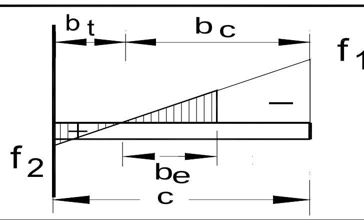
Table (3.4) Effective Width and Buckling Factor For Unstiffened Compression Elements

Stress Distribution	Effective Width b_e for $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi)/\bar{\lambda}_p^2 \leq 1$

$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0.43	$\frac{0.578}{\psi + 0.34}$	1.70	$1.7 - 5\psi + 17.1\psi^2$	23.8

Stress Distribution	Effective Width
	$\underline{1 > \psi > 0:}$ $b_e = \rho c$
	$\underline{\psi < 0}$ $b_e = \rho b_c = \rho c / (1-\psi)$

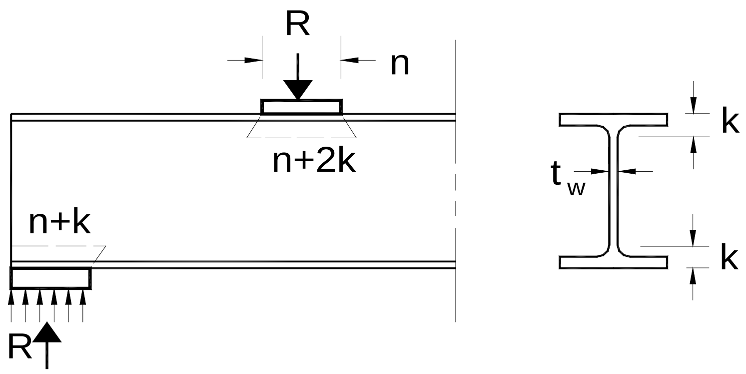
$\psi = f_2 / f_1$	1	0	-1	$1 > \psi > -1$
Buckling factor k_σ	0.43	0.57	0.85	$0.57 - 0.21\psi + 0.07\psi^2$

Stress Distribution	Effective Width
	$\underline{1 > \psi > 0:}$ $b_e = \rho c$
	$\underline{\psi < 0:}$ $b_e = \rho b_c = \rho c / (1-\psi)$

3.2.6.6 Allowable Crippling Stress in Web F_{crp}

Web crippling is a localised yielding that arises from high compressive stresses occurring in the vicinity of heavy concentrated loads.

On the web of rolled shapes or built-up I-sections, at the toe of the fillet, the allowable crippling stress shall not exceed:



$$F_{crp} = 0.75\ F_y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.31$$

Grade of Steel	F_{crp} (t/cm²)	
	$t \leq 40$ mm	40 mm $< t$
St 37	1.8	1.6
St 44	2.1	1.9
St 52	2.7	2.5

The crippling stress (f_{crp}) at the web toes of the fillets resulting from concentrated loads (**R**) not supported by stiffeners shall be calculated from the following Equations:

for interior loads $$f_{crp} = \frac{R}{t_w\,(n + 2k)} \quad \ldots\ldots\ldots\ldots \quad 3.32$$

for edge loads $$f_{crp} = \frac{R}{t_w\,(n + k)} \quad \ldots\ldots\ldots\ldots \quad 3.33$$

3.2.6.7 Combined Stresses

3.2.6.7.1 Axial Compression and Bending

Members subjected to combined axial compression (**N**) and simple bending moment (**M**) about the major axis shall be proportioned to satisfy the following interaction Equation:

$$\frac{f_{ca}}{F_c} + \frac{f_{bx}}{F_{bcx}} A_1 + \frac{f_{by}}{F_{bcy}} A_2 \leq 1.0 \quad \ldots\ldots\ldots \quad 3.34$$

For cases when $\mathbf{f_{ca}/ F_c < 0.15}$, $A_1 = A_2 = \mathbf{1.0}$.
otherwise:

$$A_1 = \frac{C_{mx}}{(1 - \frac{f_{ca}}{F_{Ex}})}, \qquad A_2 = \frac{C_{my}}{(1 - \frac{f_{ca}}{F_{Ey}})}$$

$\mathbf{f_{ca}}$	=	Actual compressive stress due to axial compression.
$\mathbf{F_c}$	=	The allowable compressive stress, as-appropriate, prescribed in Clause **3.2.6.4.**
$\mathbf{f_{bx}, f_{by}}$	=	The actual bending stresses based on moments about the x and y axes respectively.
$\mathbf{F_{bcx}, F_{bcy}}$	=	The allowable compressive bending stresses for the x and y axes respectively, considering the member loaded in bending only as prescribed in Clause 3.2.6.5.
$\mathbf{F_{Ex}, F_{Ey}}$	=	The Euler stress divided by the factor of safety for buckling in the x and y directions respectively (t/cm^2).

$$F_{Ex} = \frac{7500}{\lambda_x^2}, \qquad F_{Ey} = \frac{7500}{\lambda_y^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \mathbf{3.35}$$

$\mathbf{C_m}$ = Moment modification factor, and to be taken according to the following:

a- For frames prevented from sway without transverse loading between supports $\mathbf{C_m = 0.6 - 0.4\ (M_1/ M_2)} > \mathbf{0.4}$ where the end moments $\mathbf{M_1}$ and $\mathbf{M_2}$ carry a sign in accordance with end rotational direction; i.e. positive moment ratio for reverse curvature and negative moment ratio for single curvature ($\mathbf{M_2} > \mathbf{M_1}$).

b- For frames, prevented from sway, with transverse lateral loading between supports, $\mathbf{C_m}$ may be taken:

i- For members with moment restraint at the ends C_m = **0.85.**

ii- For members with simply supported end C_m = **1.0**.

c- For frames permitted to sway, C_m= **0.85.**

d- In addition, sections at critical locations, e.g. at member ends, shall satisfy the following Equation:

$$\frac{f_{ca}}{F_c} + \frac{f_{bx}}{F_{bcx}} + \frac{f_{by}}{F_{bcy}} \leq 1.0 \quad \text{.......................................} \quad \mathbf{3.36}$$

3.2.6.7.2 Axial Tension and Bending

Members subjected to combined axial tension "**N**" and bending moment "**M**" shall be proportioned to satisfy the following conditions:

$$f_N + f_M \leq 0.58\, F_y \quad \text{..} \quad \mathbf{3.37}$$

Where:

f_N = the tensile stress due to the axial tensile force (**N**)=N/A_{net}

f_M = the maximum tensile stress due to the bending moment (**M**).

In addition, the compressive bending stress alone shall be checked against the lateral torsional buckling stress.

3.2.6.8 Equivalent Stress f_e

Whenever the material is subjected to axial and shear stresses, the equivalent stress (f_e) must not exceed the permitted stresses given in this code plus **10%**, and the equivalent stress shall be calculated as follows:

$$f_e = \sqrt{f^2 + 3q^2} \leq 1.1\, F_{all} \quad \text{...................................} \quad \mathbf{3.38}$$

3.2.7 ALLOWABLE STRESSES IN BEARINGS AND HINGES

3.2.7.1 Table 3.5 gives the allowable stresses in (t/cm^2) in the parts of bearings and hinges made of cast iron, cast steel, and forged steel subject to bending or compression.

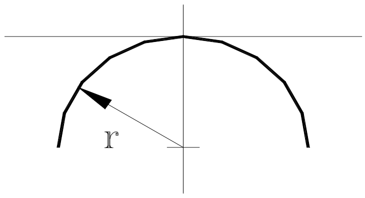
These allowable stresses may be exceeded by **20%** when the maximum combination of primary and additional stresses is taken into account.

Table (3.5) Allowable Stresses in Parts of Bearings and Hinges

Material	Primary Stresses (t/cm^2)	
	Bending	**Compression**
Cast steel **CST 55**	1.80	1.80
Forged steel **FST 56**	2.00	2.00
Cast Iron **CI 14**:		
Tension	0.30	0.90
Compression	0.60	0.90

3.2.7.2 According to Hertz formula, the bearing pressure between a cylinder and a plane surface is calculated as follows:

$$f_{max} = 0.423 \sqrt{\frac{E\,V}{\ell}} \quad \ldots\ldots\ldots\ldots \quad 3.39$$



Where:

f_{max}	=	Maximum actual bearing pressure at the surface of contact (t/cm^2).
r	=	Radius of cylinder or sphere (cm).
E	=	Young's modulus (t/cm^2).
V	=	Maximum load on bearing (ton).
ℓ	=	Bearing length (cm).

For fixed, sliding, and movable bearings with one or two rollers, the allowable bearing stresses (t/cm^2) shall be as given below, when the surface of contact between the different parts of a bearing are lines or points and when their design is carried out according to Hertz formula, assuming these bearings are subjected only to the primary stresses designated in Clause 3.2.2.1.

Material		Allowable Bearing Stress (t/cm^2)
For Cast Iron	**Cl 14**	**5.00**
For Rolled Steel	**St 44**	**6.50**
For Cast Steel	**CST 55**	**8.50**
For Forged Steel	**FST 56**	**9.50**

3.2.7.3 The allowable load **V** (ton) on a cylindrical expansion roller shall not exceed the following values:

Material	**Allowable Reaction (ton)**
Rolled steel **St 37**	**0.040 d.ℓ**
Rolled steel **St 44**	**0.055 d.ℓ**
Cast steel **CST 55**	**0.095 d.ℓ**
Forged steel **FST 56**	**0.117 d.ℓ**

Where:

d = Diameter of roller (cm).
ℓ = Length of roller (cm).

In the case of movable bearings with more than two rollers, where the compressive force affecting the said rollers cannot be equally shared by all their parts, the aforesaid allowable reactions shall be increased by **20%**.

3.2.7.4 When bearings are provided with cylindrical cast steel knuckle pins, the diameter (**d**) of the pins shall be given by the formula:

$$\mathbf{d = \frac{4}{3} . \frac{V}{\ell}} \qquad \mathbf{3.40}$$

Where:

d = Diameter of pin (cm).
V = Vertical load (ton).
ℓ = Length of pin (cm).

The bearing pressure between pins made of cast or forged steel and the gusset plates shall not exceed 2.40 t/cm^2.

3.3 FATIGUE

3.3.1 General

A bridge member may respond to applied loads in one of the following three ways, see Fig 3.1: (a) deform elastically, or (b) deform plastically, or (c) break. Since steel is a ductile material, failure of steel members is normally preceded by a considerable amount of elastic or plastic deformations. This amount depends on the magnitude of applied loads (below or above the yield level) and on the repetitive and cyclic nature of the load.

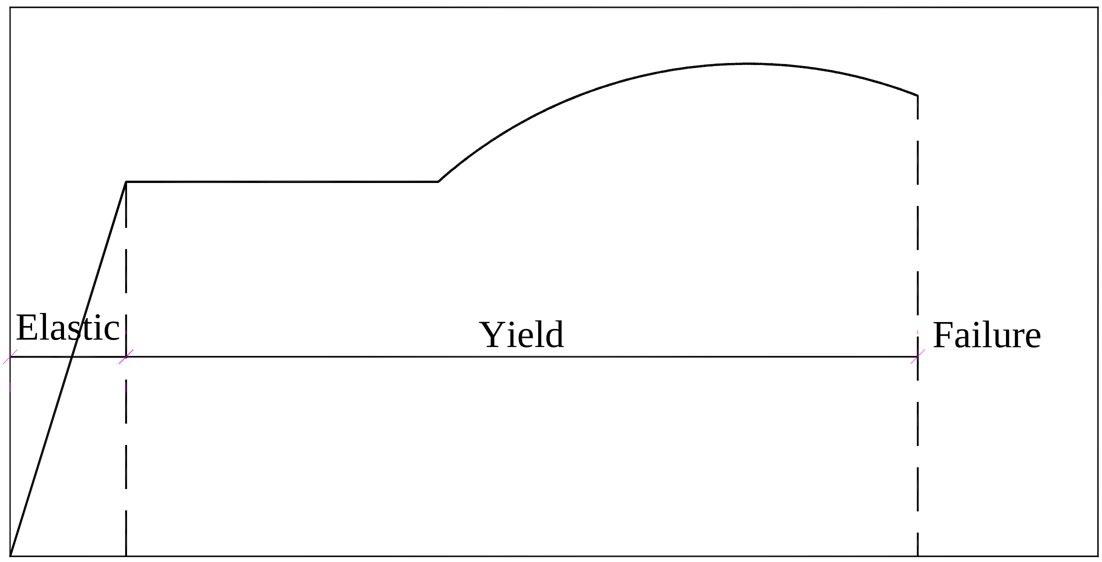
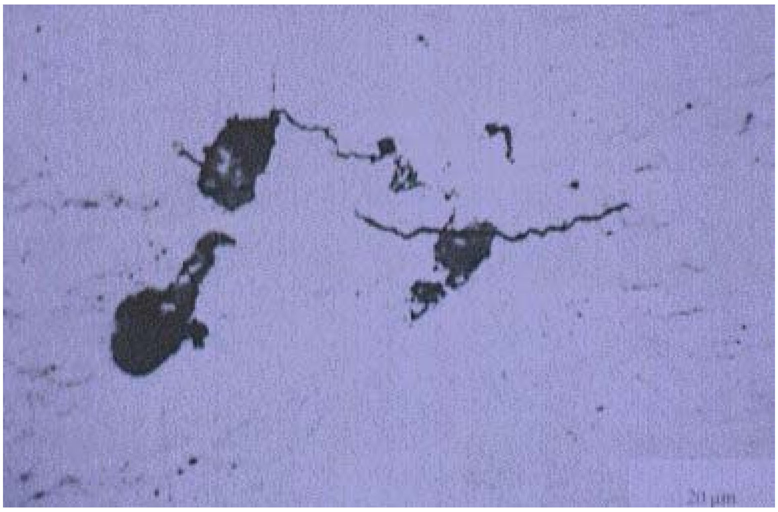


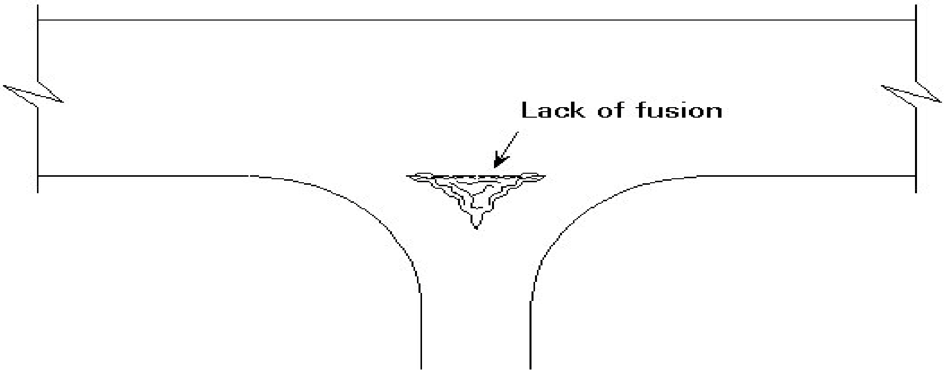
Fig. 3.1 Behaviour Stages: (a) Elastic, (b) Plastic, (c) Failure

Sometimes, however, certain types of steel members may fail suddenly in the form of brittle fracture. It was found that this failure mode starts from the presence of very small defect and cracks in the member during fabrication due to rolling, cutting, drilling, and or welding, see Fig. 3.2. The presence of these defects causes stress concentration around them as shown in Fig. 3.3a for a plate with a hole and in Fig. 3.3b at a fillet weld toe. The stress concentration around the defects causes them, although initially undetected, to increase in size and eventually propagate to failure when the member is subjected to a large number of stress cycles, see Fig 3.4.

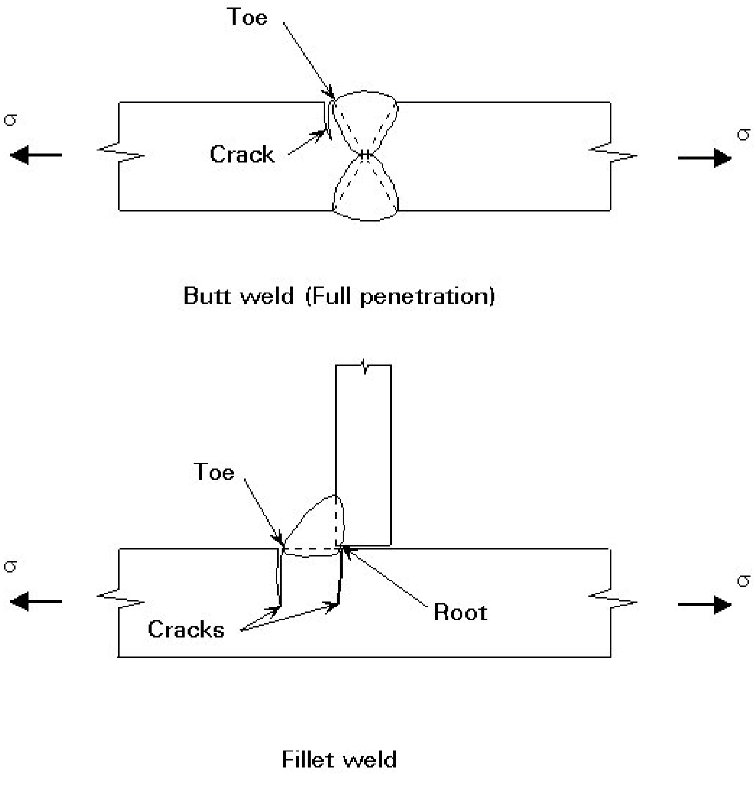
In order to determine the design parameters that can prevent the occurrence of this brittle failure, ***Fracture Mechanics*** concepts can be applied to arrive at the fatigue strength of different bridge components; e.g., members and connections. Control of fatigue failure is then achieved through efficient design and detailing.



a) Porosity / Slag Inclusion



b) Lack of Fusion



c) Weld Cracks

Fig 3.2 Possible Defects Causing Fatigue Failure

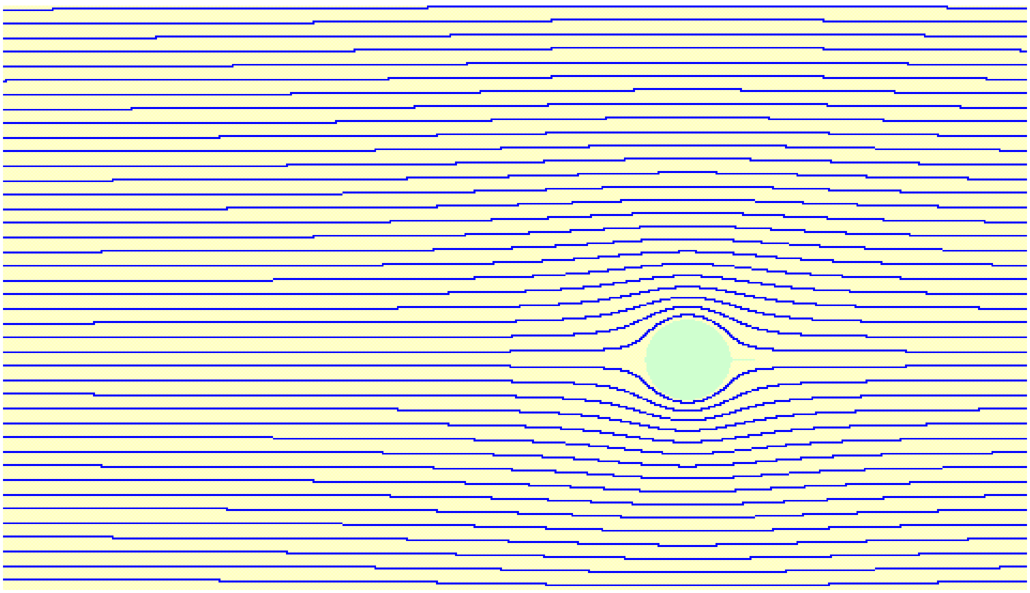


Fig 3.3a Stress Concentration in a Plate with Hole

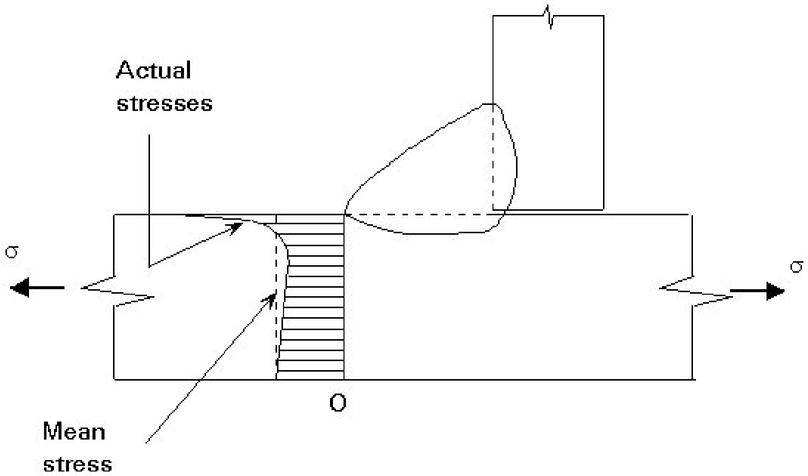


Fig 3.3b Stress Concentration at Fillet Weld Toe

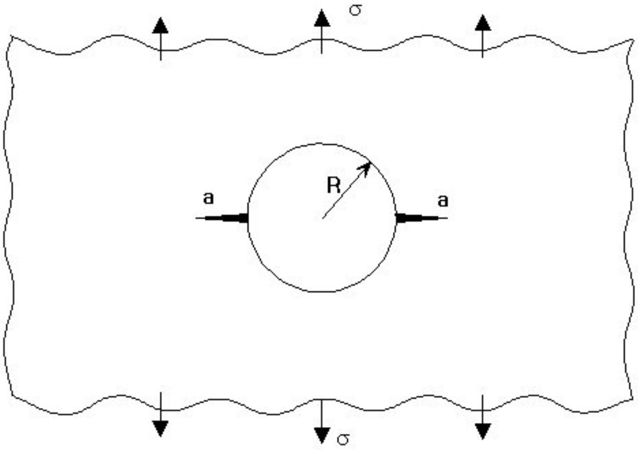


Fig 3.3c Crack Initiation and Propagation

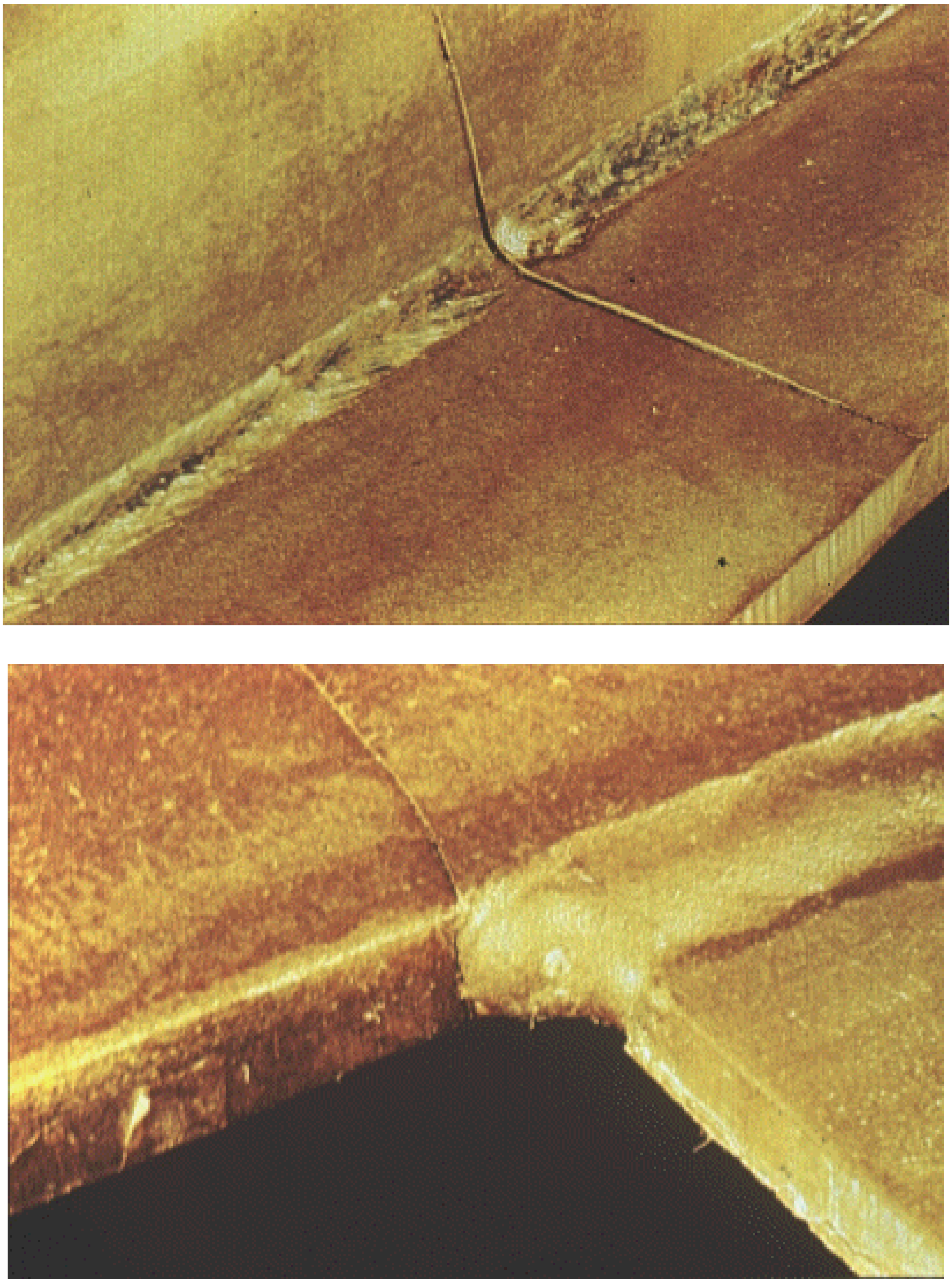


Fig 3.4 Cracks Causing Fatigue Failure due to Weld Defects

This section presents a general method for the fatigue assessment of structures and structural elements that are subjected to repeated fluctuations of stresses. Members subjected to stresses resulting from fatigue loads shall be designed so that the maximum stresses do not exceed the basic allowable stresses under static load conditions and that the ***stress range***, see definition below, does not exceed the allowable fatigue stress range given in this section.

3.3.2 DEFINITIONS

Fatigue: Damage in a structural member through gradual crack propagation caused by repeated stress fluctuations.

Design Life: The period in which a structure is required to perform safely with an acceptable probability that it will not fail or require repair.

Stress Range: The algebraic difference between two extreme values or nominal stresses due to fatigue loads, see Fig. 3.5. This may be determined through standard elastic analysis.

Fatigue Strength: The stress range determined form test data for a given number of stress cycles.

Fatigue Limit: The maximum stress range for constant amplitude cycles that will not form fatigue cracks.

Detail Category: The designation given to a particular joint or welded detail to indicate its fatigue strength. The category takes into consideration the local stress concentration at the detail, the size and shape of the maximum acceptable discontinuity, the loading condition, metallurgical effects, residual stresses, fatigue crack shapes, the welding procedure, and any post-welding improvement.

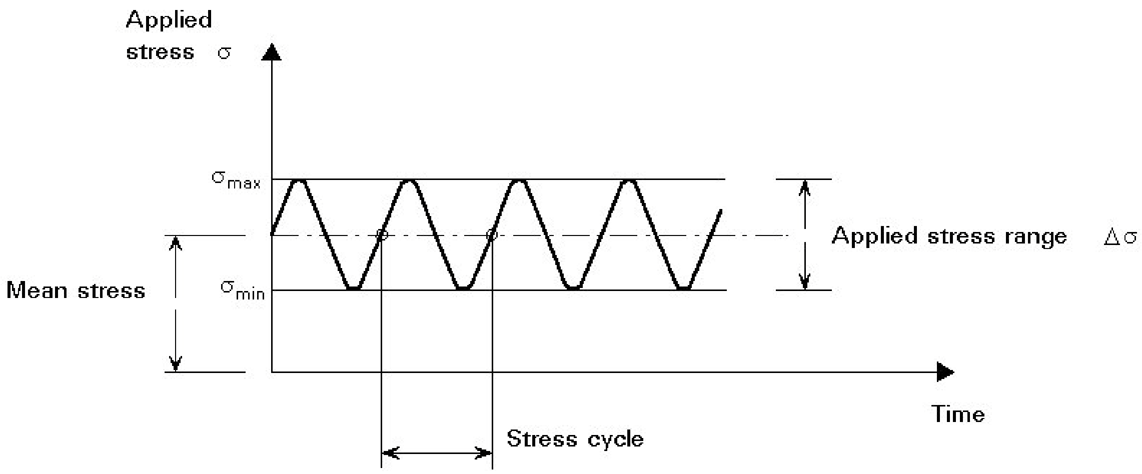


Fig. 3.5a) Stress Definitions Related to Fatigue, Constant Stress Cycles

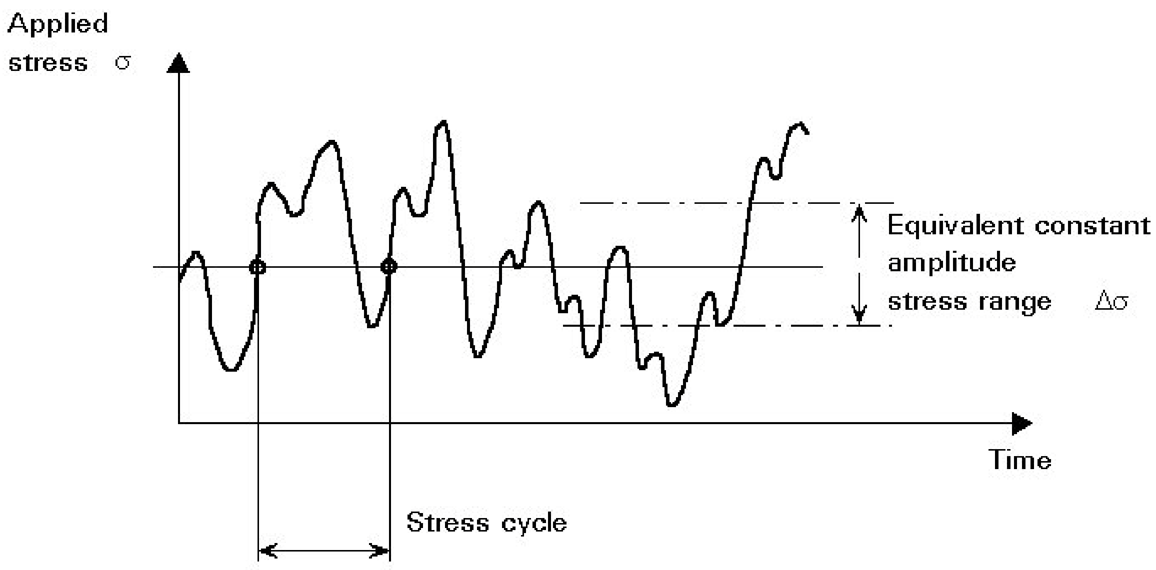


Fig. 3.5b) Variable Amplitude Stress History

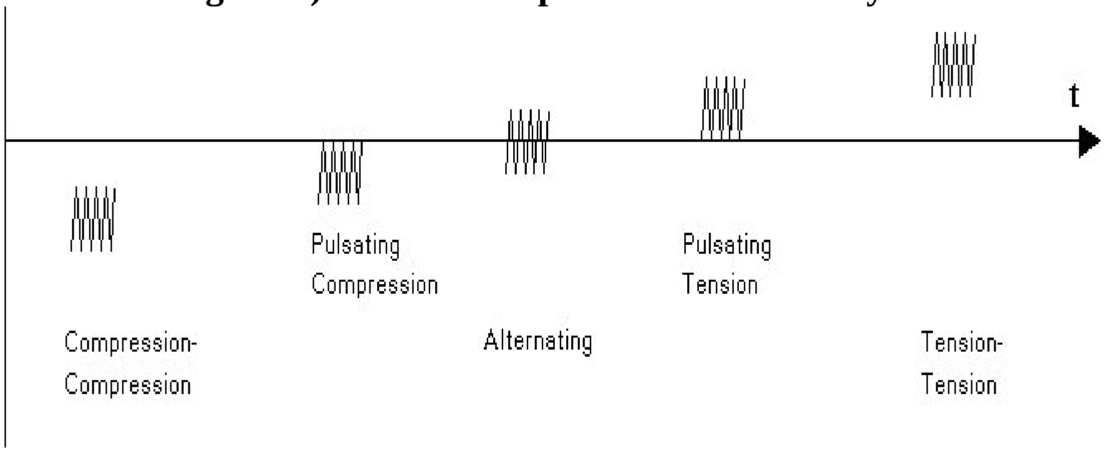


Fig. 3.5c) Various Patterns of Stress Variation

3.3.3 BASIC PRINCIPLES RELATED TO FATIGUE

1- The differences in fatigue strength between grades of steel are small and may be neglected.

2- The differences in fatigue damage between stress cycles having different values of mean stress but the same value of stress range may be neglected.

3- Cracks generally occur at welds or at stress concentration due to sudden changes of cross-sections. Very significant improvements in fatigue strength can be achieved by reducing the severity of stress concentrations at such points.

4- Members subjected to stresses resulting from wind forces only, shall be designed so that the maximum unit stress does not exceed the basic allowable unit stress given in the Code.

5- Cracks that may form in fluctuating compression regions are self-arresting. Therefore, these compression regions are not subjected to fatigue failure.

6- When fatigue influences the design of a structure, details should be precisely defined by the designer and should not be amended in any way without the designer's prior approval. Similarly, no attachments or cutouts should be added to any part of the structure without notifying the designer.

7- Structures in which the failure of a single element could result in a collapse or catastrophic failure should receive special attention when fatigue cracks are a possibility. In such cases, the allowable stress ranges shall be limited to 0.8 times the values given in Table 3.2 or in Figure 3.1.

8- Slotted holes shall not be used in bolted connections for members subjected to fatigue.

3.3.4 Factors Affecting Fatigue Strength

The fatigue strength of the structural elements depends upon:

1- The applied stress range resulting from the applied fatigue loads.
2- The number of stress cycles.
3- The detail category of the particular structural component or joint design.

3.3.5 Fatigue Loads

1- Roadway Bridges: The fatigue loads used to calculate the stress range are 50% of the standard design live loads including the corresponding dynamic effect.

2- Railway Bridges: The fatigue loads used to calculate the stress range are the full standard design live loads.

For bridges carrying both trucks and trains, the fatigue load is the combined effect of the full railway live load and 60% of the traffic live loads.

3.3.6 Fatigue Assessment Procedure

1- The fatigue assessment procedure should verify that the effect of the applied stress cycles expected in the design life of the structure is less than its fatigue strength.

2- The effect of applied stress cycles is characterized by the maximum stress range (f_{sra}). The maximum stress range can be computed from the applied fatigue loads using an elastic method of analysis. The fatigue loads should be positioned to give the maximum straining actions at the studied detail. In some structures such as bridges and cranes, consideration should be given to possible changes in usage such as the growth of traffic, changes in the most severe loading, etc.

3- In non-welded details or stress relieved welded details subjected to stress reversals, the effective stress range to be used in the fatigue assessment shall be determined by adding the tensile portion of the stress range and 60% of the compressive portion of the stress range. In welded details subjected to stress reversals, the stress range to be used in the fatigue assessment is the greatest algebraic difference between maximum stresses.

4- The fatigue strength of a structural part is characterized by the allowable stress range (F_{sr}) which is obtained from Table 3.2 for the specified number of constant cycles and the particular detail category.

5- The number of constant stress cycles to be endured by the structure during its design life is given in Table 3.6a for roadway bridges and Table 3.6b for railway bridges. The number of cycles given in Tables 3.6a and 3.6b is subject to modifications according to the competent authority requirements.

Table (3.6a) Number of Loading Cycles – Roadway Bridges

Type of Road	ADTT *	Number of Constant Stress Cycles (N)	
		Longitudinal Members	Transverse Members
Major Highways and Heavily Traveled Main Roads	≥ 2500	2,000,000	Over 2,000,000
	< 2500	**500,000**	2,000,000
Local Roads and Streets		100,000	500,000

* ADTT = Average Daily Truck Traffic for 50 years design life

Table (3.6b) Number of Loading Cycles – Railway Bridges

Member Description	Span Length (L) (m)	Number of Constant Stress Cycles (N)
Class I Longitudinal flexural members and their connections, or truss chord members including end posts and their connections.	L > 30	500,000
	30 ≥ L ≥ 10	2,000,000
	L < 10	Over 2,000,000
Class II Truss web members and their connections except as listed in class III	Two tracks loaded	200,000
	One track loaded	500,000
Class III Transverse floor beams and their connections or truss verticals and sub-diagonals which carry floor beam reactions only and their connections	Two tracks loaded	500,000
	One track loaded	over 2,000,000

6- In detailing highway bridges for design lives greater than 50 years, the fatigue loads should be increased by a magnification factor, M, given by the following Table:

No. of Years	50	80	100	120
Magnification Factor, M	1.00	1.10	1.15	1.20

7- Each structural element has a particular detail category as shown in Table 7.3 The classification is divided into four parts which correspond to the following four basic groups:

Group 1: non-welded details, plain materials, and bolted plates.
Group 2: welded structural elements, with or without attachments.
Group 3: fasteners (welds and bolts).
Group 4: Orthotropic Deck Bridges.

8- When subjected to tensile fatigue loading, the allowable stress range for High Strength Bolts friction type shall not exceed the following values:

Number of Cycles	Allowable Stress Range F_{sr} (t/cm^2)	
	Bolts Grade 8.8	Bolts Grade 10.9
$N \leq 20,000$	2.9	3.6
$20,000 < N \leq 500,000$	2.6	3.2
$500,000 < N$	2.0	2.5

Table (3.7) Allowable Stress Range (F_{sr})
for Number of Constant Stress Cycles (N)

F_{sr} (t/cm^2)				
	100,000	500,000	2,000,000	Over 2,000,000
A	4.30	2.52	1.68	1.68
B	3.42	2.00	1.26	1.12
B’	2.77	1.52	1.02	0.85
C	2.48	1.45	0.91	0.70
D	1.92	1.12	0.71	0.49
E	1.53	0.89	0.56	0.32
E’	1.11	0.65	0.41	0.18
F	0.72	0.52	0.40	0.36

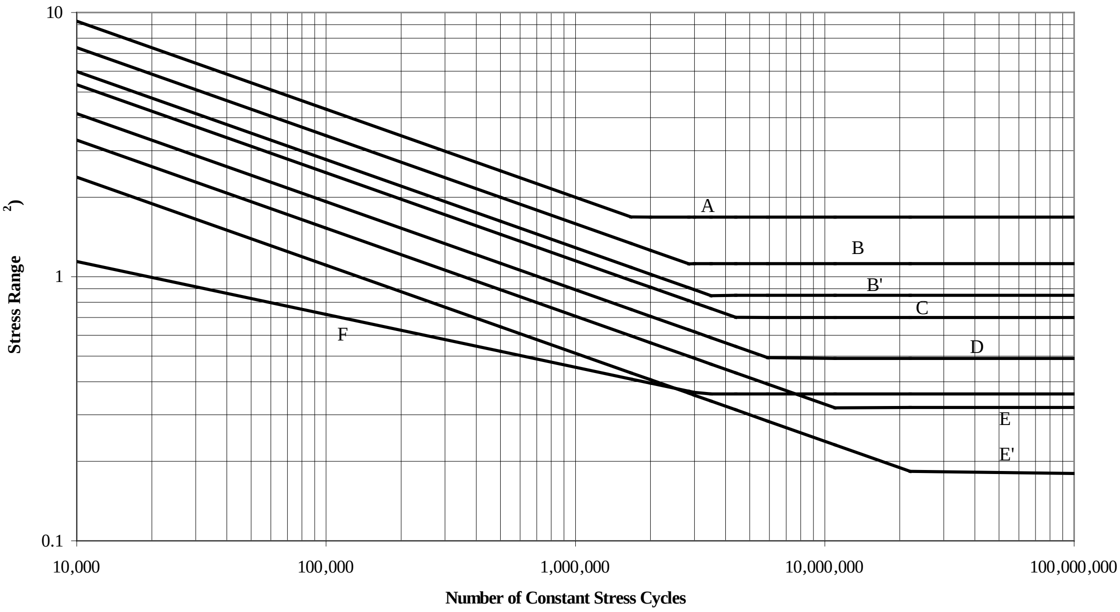
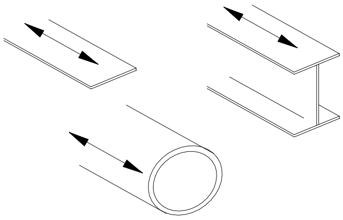
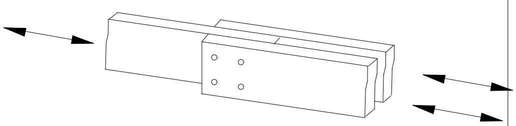
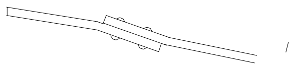
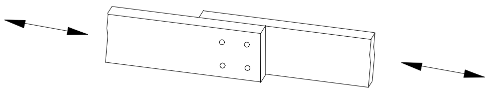
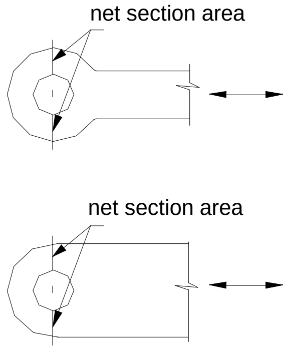


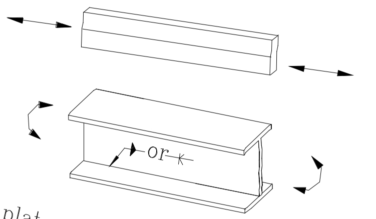
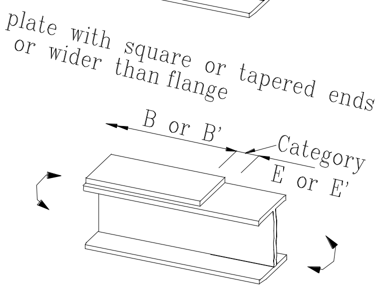
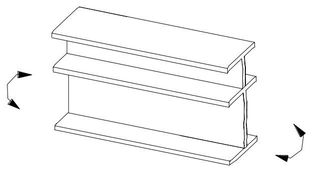
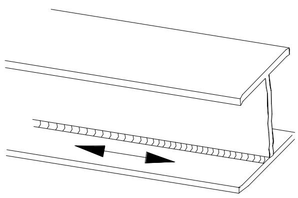
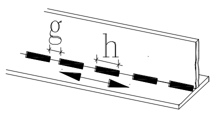
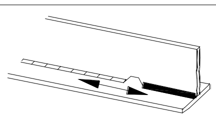
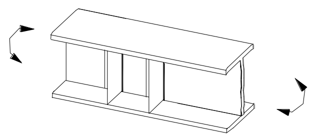
Fig. 3.1. Stress Range Versus Number of Constant Stress Cycles

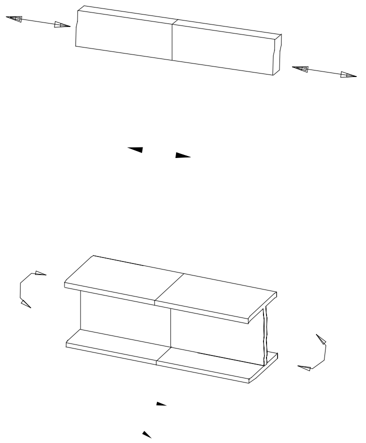
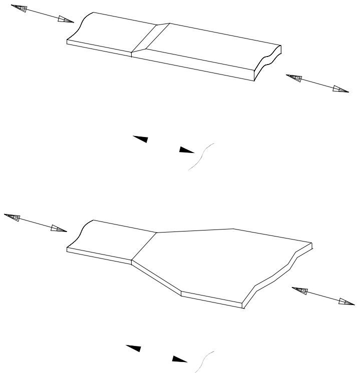
Table (3.8) Classification of Details

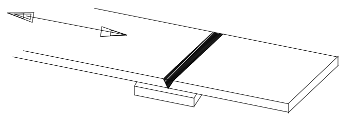
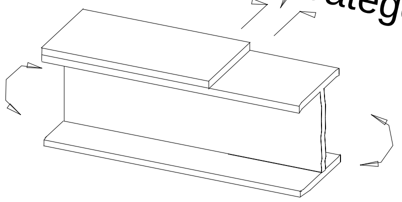
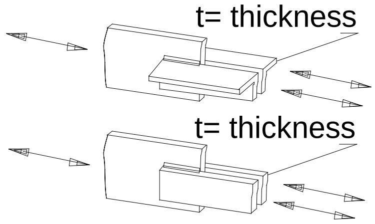
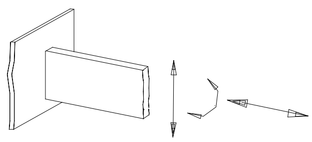
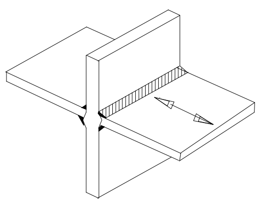
Group 1: Non-Welded Details

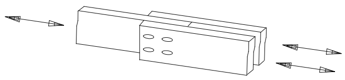
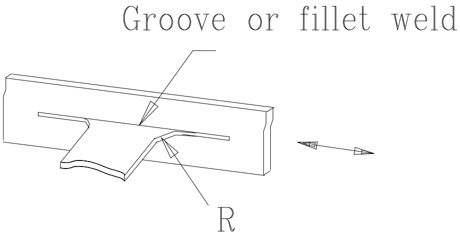
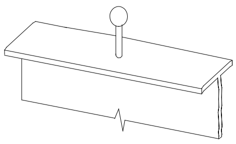
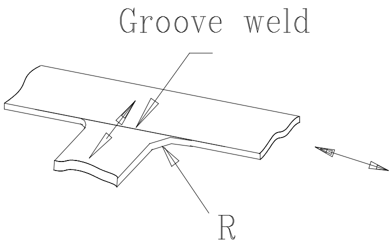
Description	Illustration	Class
1.1. Base metal with rolled or cleaned surfaces; flame cut edges with a surface roughness less than 25 μm		A
1.2. Base metal with sheared or flame cut edges with a surface roughness less than 50 μm		B
2.1. Base metal at gross section of high strength bolted slip resistant (friction) connections, except axially loaded joints which induce out of plane bending in connected material.		B
2.2. Base metal at net section of fully tensioned high strength bolted bearing type connections		B'
2.3. Base metal at net section of other mechanically fastened joints (ordinary bolts & rivets).		D
3. Base metal at net section of eye-bar head or pin plate.		E

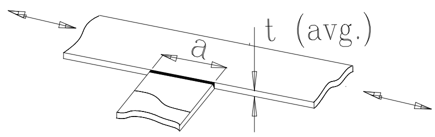
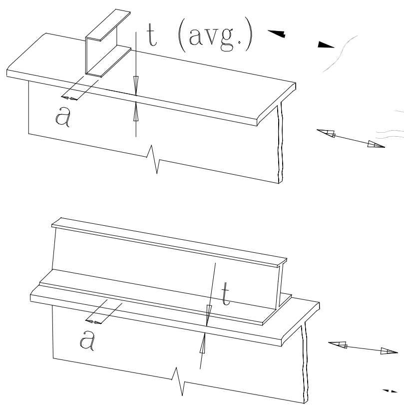
Group 2: Welded Structural Elements

Description	Illustration	Class
4.1. Base metal in members without attachments, built up plates or shapes connected by continuous full penetration groove welds or by continuous fillet welds carried out from both sides without start stop positions parallel to the direction of applied stress.		B
4.2. Same as (4.1.) with welds having stop - start positions.		B'
4.3. Base metal in members without attachments, built-up plates or shapes connected by continuous full penetration groove welds with backing bars not removed, or by partial penetration groove welds parellel to the direction of applied stress.		B'
5. Base metal at continuous manual longitudinal fillet or full penetration groove welds carried out from one side only. A good fit between flange and web plates is essential and a weld preparation at the web edge such that the root face is adequate for the achievement of regular root penetration.		C
6. Base metal at zones of intermittent longitudinal welds with gap ratio g/h < 2.5		D
7. Base metal at zones containing copes in longitudinally welded T-joints.		D
8. Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners.		C

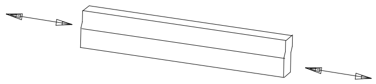
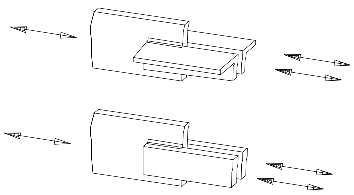
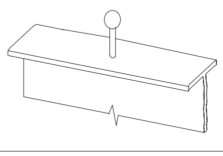
Description	Illustration	Class
9.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) of parts of similar cross sections ground flush, with grinding in the direction of applied stress and weld soundness established by radiographic or ultrasonic inspection.		B
9.2. Same as (9.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
9.3. Same as (9.2.) with reinforcement more than 0.10 of weld width.		D
10.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2.5 with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection.		B'
10.2. Same as (10.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
10.3. Same as (10.2.) with slopes more than 1 to 2.5		D
10.4. Same as (10.1.) to (10.3.) but with welds made from one side only.		E'

Description	Illustration	Class
11.1. Base metal and weld metal at transverse full penetration groove welded splices on a backing bar. The end of the fillet weld of the backing strip is more than 10 mm from the edges of the stressed plate		D
11.2. Same as (11.1) with the fillet weld less than 10 mm from the edges of the stressed plate.		E
12.1. Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than the flange with welds at the ends. Flange thickness ≤ 20 mm		E
Flange thickness > 20 mm		E'
12.2 Base metal at ends of partial length welded cover plates wider than the flange without end welds.		E'
13. Base metal at axially loaded members with fillet welded connections. t ≤ 25 mm		E
t > 25 mm		E'
14. Base metal at members connected with transverse fillet welds.		C
15.1. Base metal at full penetration weld in cruciform joints made of a special quality weld.		D
15.2. Same as (15.1) with partial penetration or fillet welds of normal quality.		E'

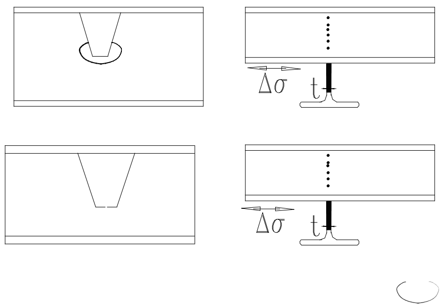
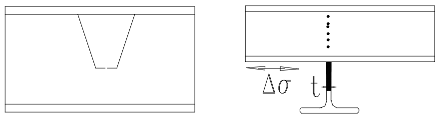
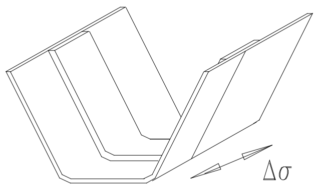
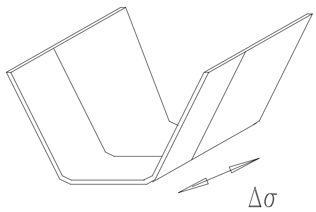
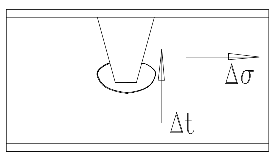
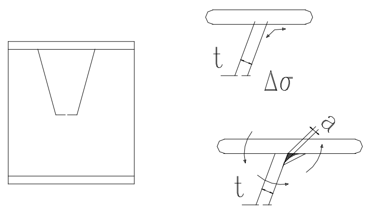
Description	Illustration	Class
16. Base metal at plug or slot welds.		
17. Base metal and attachment at fillet welds or partial penetration groove welds with main material subjected to longitudinal loading and weld termination ground smooth R > 50 mm		D
R < 50 mm		E
18. Base metal at stud- type shear connector attached by fillet weld or automatic end weld.		C
19.1. Base metal at details attached by full penetration groove welds subject to longitudinal loading with weld termination ground smooth. Weld soundness established by radiographic or ultrasonic inspection R > 610 mm		B
610 mm > R > 150 mm		C
150 mm > R > 50 mm		D
R < 50 mm		E
19.2. Same as (19.1.) with transverse loading, equal thickness, and reinforcement removed. R > 610 mm		B
610 mm > R > 150 mm		C
150 mm > R > 50 mm		D
R < 50 mm		E

Description	Illustration	Class
19.3. Same as (19.2.) but reinforcement not removed R > 610 mm		C
610 mm > R > 50 mm		C
150 mm > R > 50 mm		D
R > 50 mm		E
19.4. Same as (19.2.) but with unequal thickness R > 50 mm		D
R < 50 mm		E
19.5. Same as (19.4.) but with reinforcement not removed and for all R		E
20. Base metal at detail attached by full penetration groove welds subject to longitudinal loading 50-mm< a <12t or 100 mm		D
a >12t or 100 mm (t<25 mm)		E
a >12t or 100 mm (t>25 mm)		E'
21. Base metal at detail attached by fillet welds or partial penetration groove welds subject to longitudinal loading a < 50 mm		C
50 mm< a <12t or 100 mm		D
a >12t or 100 mm (t<25 mm)		E
a >12t or 100 mm (t>25 mm)		E'

Group 3: Fasteners (Welds and Bolts)

Description	Illustration	Class
22.1. Weld metal of full penetration groove welds parallel to the direction of applied stress (weld from both sides)		B
22.2. Same as (22.1.) but with weld from one side only.		C
22.3. Weld metal of partial penetration transverse groove weld based on the effective throat area of the weld.		F
23.1 Weld metal of continuous manual or automatic longitudinal fillet welds transmitting continuous shear flow.		D
23.2 Weld metal of intermittent longitudinal fillet welds transmitting a continuous shear flow.		F
23.3 Weld metal at fillet welded lap joints.		E'
24. Transversally loaded fillet welds.		E'
25. Shear on plug or slot welds.		F
26. Shear stress on nominal area of stud-type shear connectors.(Failure in the weld or heat affected zone.)		F
27.1. High strength bolts in single or double shear (fitted bolt of bearing type).		C
27.2. Rivets and ordinary bolts in shear.		D
28. Bolts and threaded rods in tension (on net area)		F

Group 4: Orthotropic Deck Bridges

Description	Illustration	Class
29.1. Base metal at continuous longitudinal rib with or without additional cutout in cross girder. (Bending stress range in the rib) t ≤ 12mm		C
29.2. Same as (29.1.) t > 12mm		D
30. Base metal at separate longitudinal ribs on each side of the cross girder. (Bending stress range in the rib)		E'
31. Base metal at rib joints made of full penetration weld with backing plate.(Bending stress range in the rib)		D
32.1. Base metal at rib joints made of full penetration weld without backing plate. All welds ground flush to plate surface in the direction of stress. Slope of thickness transition < 1:4. (Bending stress range in the rib)		B'
32.2. Same as (32.1.) with weld reinforcement ≤ 0.2		C
33. Base metal at connection of continuous longitudinal rib to cross girder. (Equivalent stress range in the cross girder web).		E'
34.1. Weld metal at full penetration weld connecting deck plate to rib section.		D
34.2. Weld metal at fillet weld connecting deck plate to rib section.		E'

3.3.7 Examples of weld detail classifications

In order to assist a designer in selecting the correct detail category, this section presents a case study of a particular civil engineering structure, and shows how the details can be classified. The case study comprises an imaginary steel bridge, as shown in an exploded isometric view in Figure 1. In order to illustrate as many different details as possible, two forms of construction are shown with a box girder on the left hand side and a braced plate girder on the right hand side. Furthermore, there are differences between the two sides in bearing arrangement, connection of cross girders, etc. It is not suggested that these arrangements, nor even some of the details, are necessarily representative of good design practice; they are presented for the purpose of illustrating a point of discussion.

This imaginary structure is then subdivided as shown in Figure 1 into several close-up details in Figures 2 - 7, which in some cases are further subdivided where necessary for clarity. The detailed figures indicate the direction of principal stress, and the potential crack location and direction; the category into which the detail should be classified according to European Code is shown as a number in a circle beside the detail.

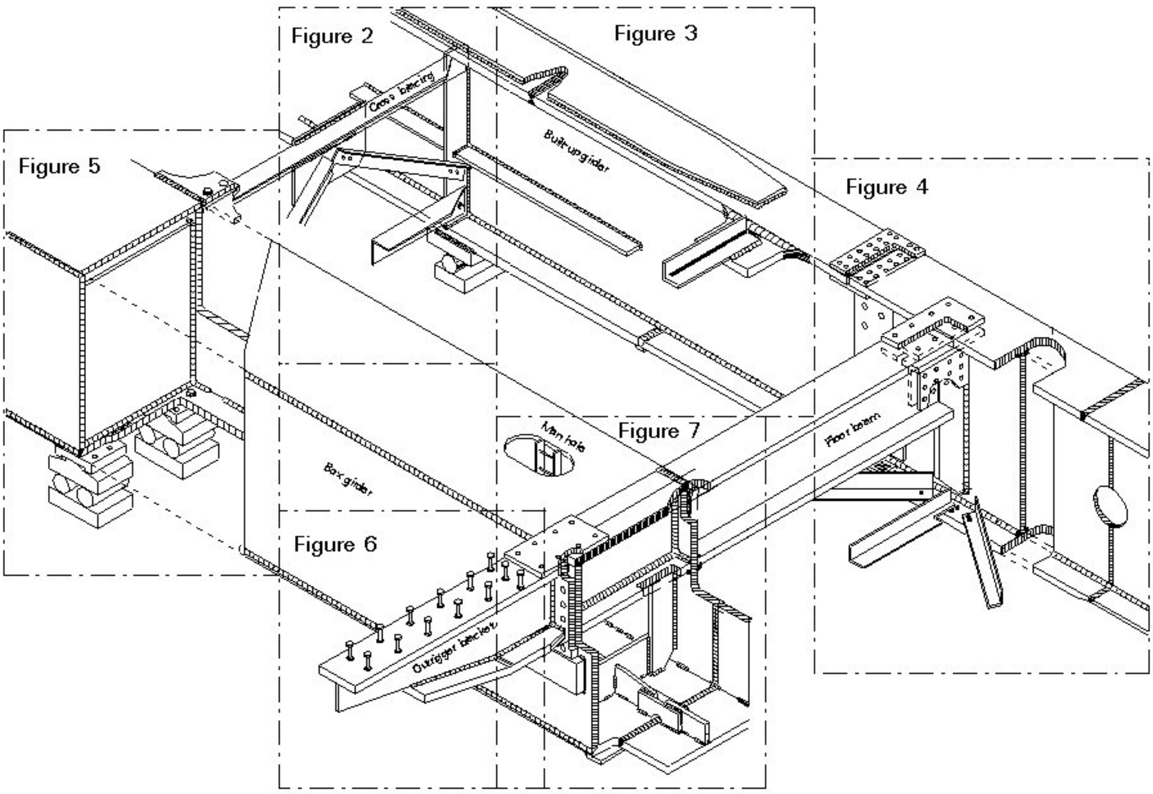


Figure 1 General overview of case studies

Examples of Weld Detail Group Classifications

Notes on Fig. 2:

a. This detail should generally be avoided (it is usually better, and probably easier, to detail the longitudinal stiffeners passing through "mouseholes" in the transverse stiffeners).
b. The category of 112 shown is the "standard" one for automatic fillet welding carried out from both sides, but containing stop-start positions. If it contained no stop-start positions it could be upgraded to category 125, or even 140 if a specialist inspection shows that the welds are free from significant flaws; conversely, if the fillet were placed manually, it would be downgraded to category 100.
c. Stresses should be calculated using the gross section for slip resistant connections, or the net section for all other connections. The effects of eccentricity in the connection should be taken into account when calculating the stresses in a single-sided connection.
d. This detail (at the termination of a longitudinal stiffener) may be treated for cracking in the main plate as a long (>100m) longitudinal attachment within the width of a plate with a non-load carrying weld. Note that the weld may also require checking in shear, with the stress range calculated from the weld throat area.
e. The gusset plate attached as shown to the leg of the angle may be treated as a cover plate wider than the flange (with the leg of the angle representing the flange). Provided all plate thicknesses are 20mm or less, this is category 50* for cracking in the angle; this reduces to 36* if thicknesses exceed 20mm. The weld should be continued down the leg of the angle, and ground to remove undercut if necessary

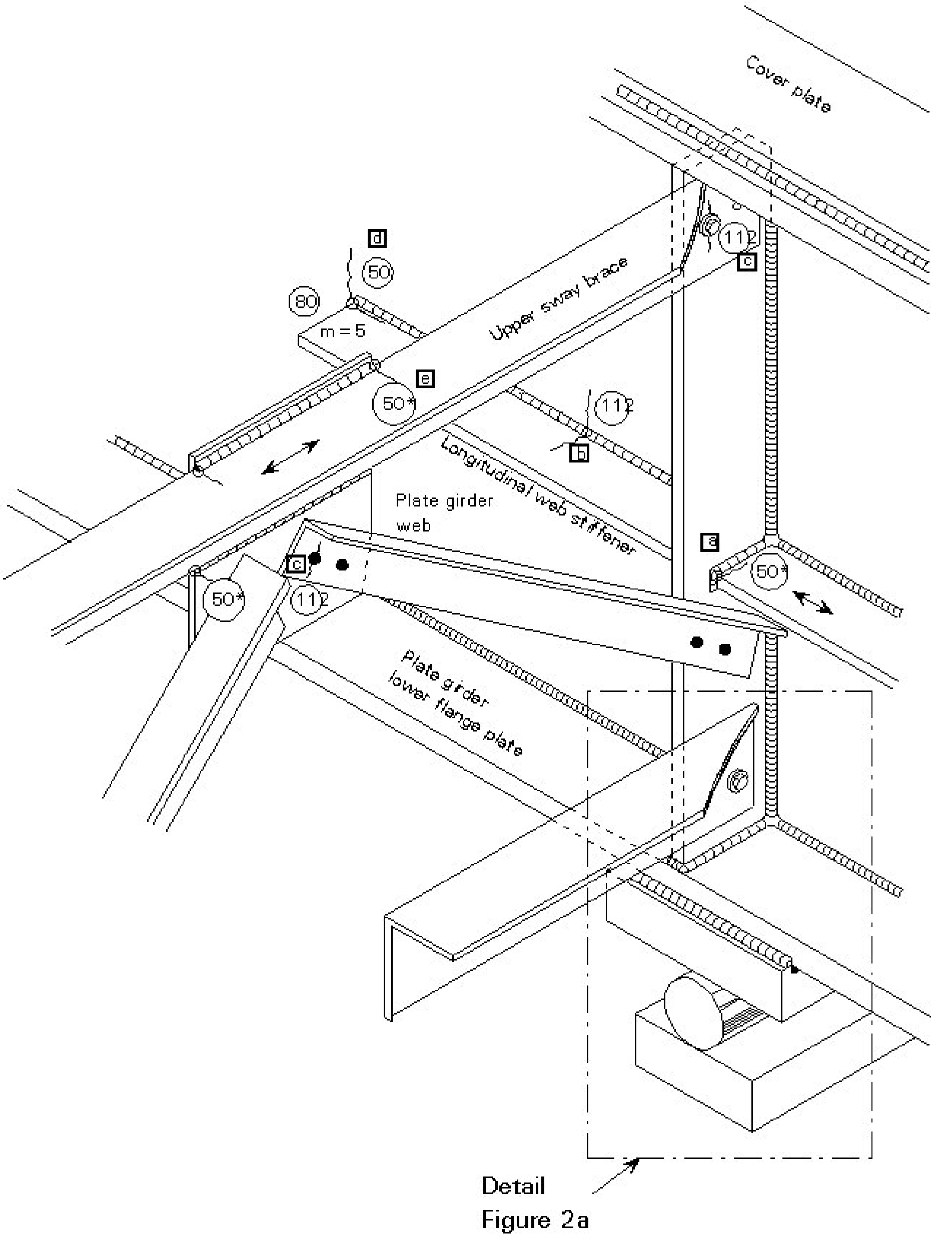

Cover plate
Upper sway brace
m = 5
Longitudinal web stiffener
Plate girder
web
Plate girder
lower flange plate
Detail
Figure 2a

Figure 2

Notes on Fig. 2a:

a & b These details show how cracks may grow in different directions in an area of complex geometry and stress distribution. Considerations are similar to Figure 2, note e, but the bearing plate > 20mm thick and so the category for plate cracking is reduced to 36*.

c. See Figure 2, note b; as the weld to the bearing plate will almost certainly be placed manually, the lower category of 100 is used.

d. The category of the plate edge depends on the method of production; if it is a rolled flat the category could be increased to 160, or if machine flame cut with subsequent machining to 140. The indicated category of 125 is for a machine flame cut edge without subsequent machining. It should contain no repairs by weld infill.

e. As for Figure 2, note b.

f. As for Figure 2, note c.

g. The category of this weld has been reduced from the 71 or 80 shown for web stiffeners since the stiffener is shown flush with the edge of the plate

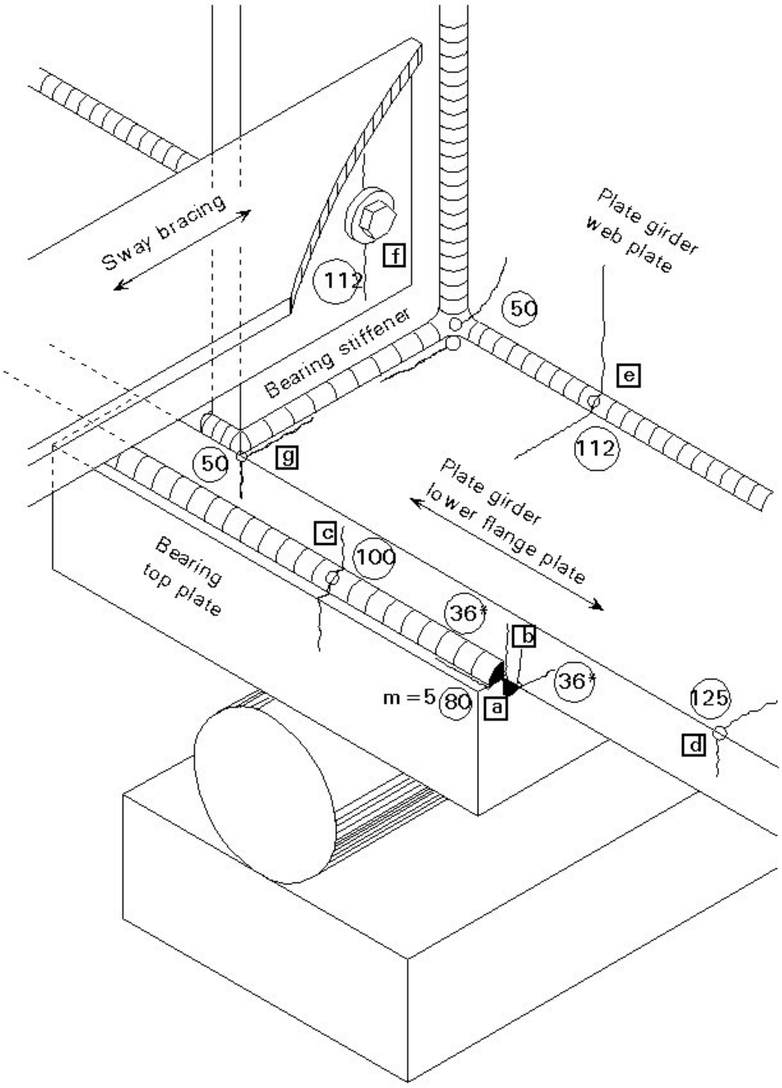

Sway bracing
Plate girder
web plate
112
f
50
Bearing stiffener
e
112
50
g
Plate girder
lower flange plate
Bearing
top plate
c
100
36*
b
36*
m = 5
80
a
125
d

Figure 2a

Notes on Fig. 3:

a. This is a rather poor detail, since because of the taper in the flange a good fit cannot be guaranteed above the backing flat; hence the low category of 50.
b. At the top of the butt weld, provided the "reinforcement" does not exceed 0.1 times the width of the weld bead, the category is 90; up to 0.2 it would be 80. Run off pieces should also be used. (If the weld is ground flush the category could be 125 or higher). Normally there would be little point in making the category of the top surface much higher than that of the bottom, unless the eccentricity arising from the change of plate thickness results in a higher stress range at the top.
c. The comparatively high category of this weld is only true for a gusset plate with a generous radius as drawn (> 150mm, and also > (width of main plate)/3). The radius has to be formed by initial machining or gas cutting, with subsequent grinding of the weld area parallel to the direction of stress. If the radius < (width of main plate)/6 the category falls to 45*, and between the two limits above to 71.
d. It should be noted that the weld should be held back 10mm from the end of the gusset. As it is a single sided connection, the effects of eccentricity should be considered.
e. The calculation of the stress in the main plate requires care, and in a single sided application as shown may have to allow for eccentricity.
f. This is a standard "bad" detail for increasing the area of a plate. The plates in the example are not thicker than 20mm so the category is 50*; above this thickness the category is reduced to 36*. Contrary to what may be thought, tapering the cover plate as shown, or rounding its end, does not, in itself, improve the detail; however, a special detail with tapering welds and chamfered cover plates, being developed by German Railways, may raise the category to 80.
g. This is a two sided butt weld, with the surface ground flush. Significant quality control and inspection is required to permit the use of this high category.
h. As for Figure 2, note d.

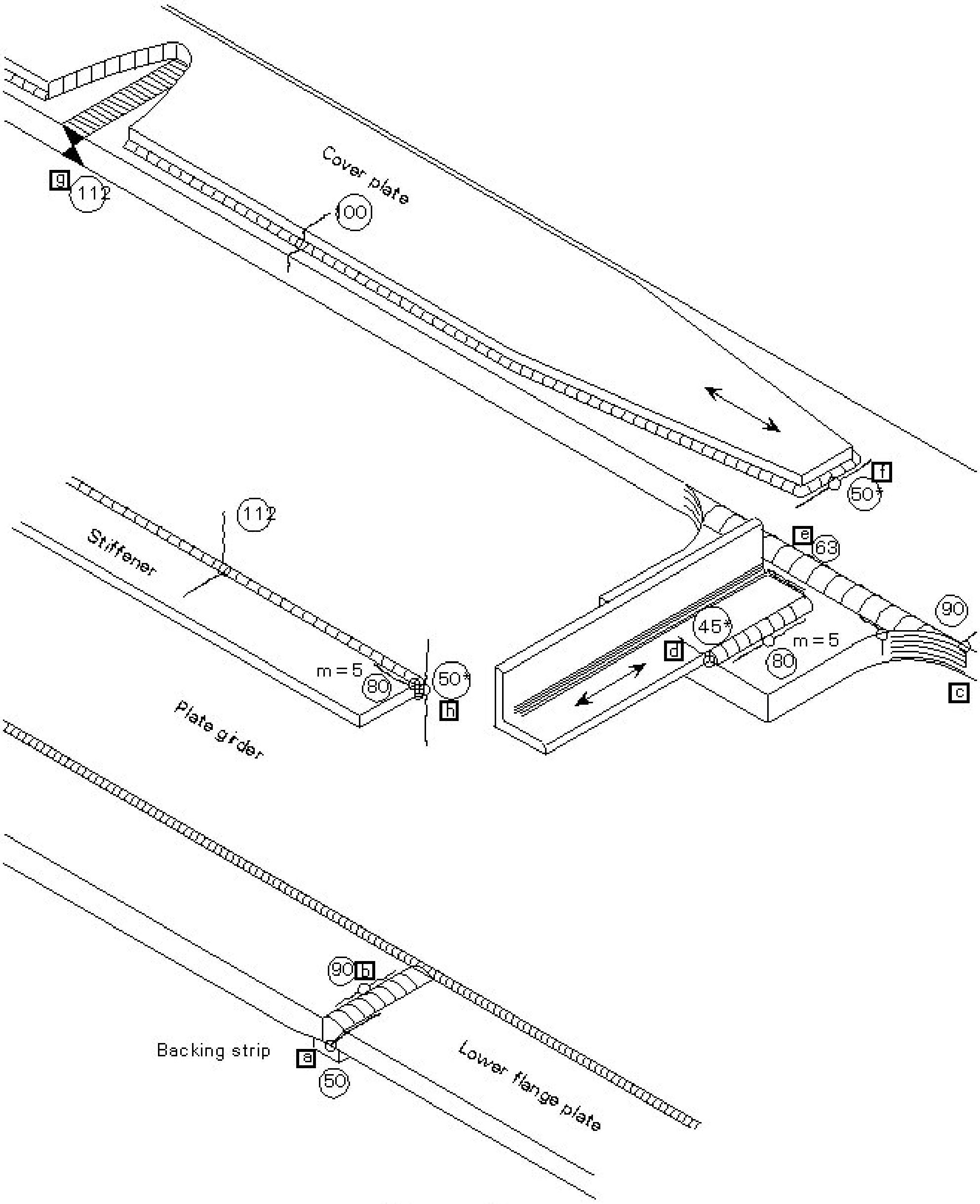

Cover plate
100
112
g
50*
f
e
63
90
Stiffener
112
m = 5
80
50*
h
45*
d
m = 5
80
c
Plate girder
90
b
Backing strip
a
50
Lower flange plate

Figure 3

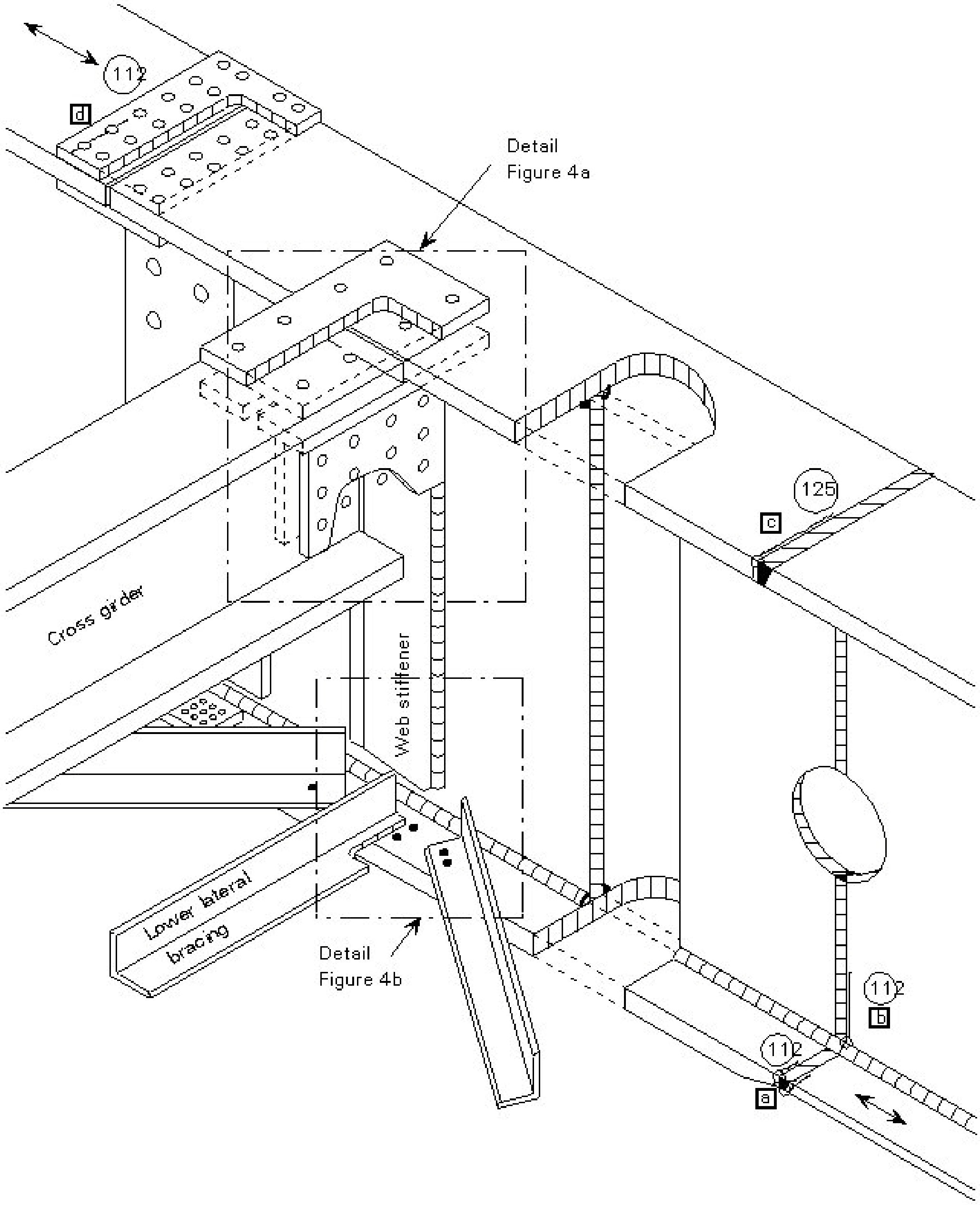


Figure 4

Notes on Fig. 4:

a. This is a butt weld, without backing flat and ground flush, between plates of different thickness. Provided the difference in thickness is taken up by tapering the thicker member with a slope of not greater than 1:4, this still qualifies as a category 112 weld.

b. As Figure 3, note g.

c. As Figure 3, note g, but it is a single sided weld without backing flat and because very high quality of execution and inspection is specified, the category can be raised to 125.

d. As for Figure 2, note c, but as the connection is double sided no eccentricity occurs.

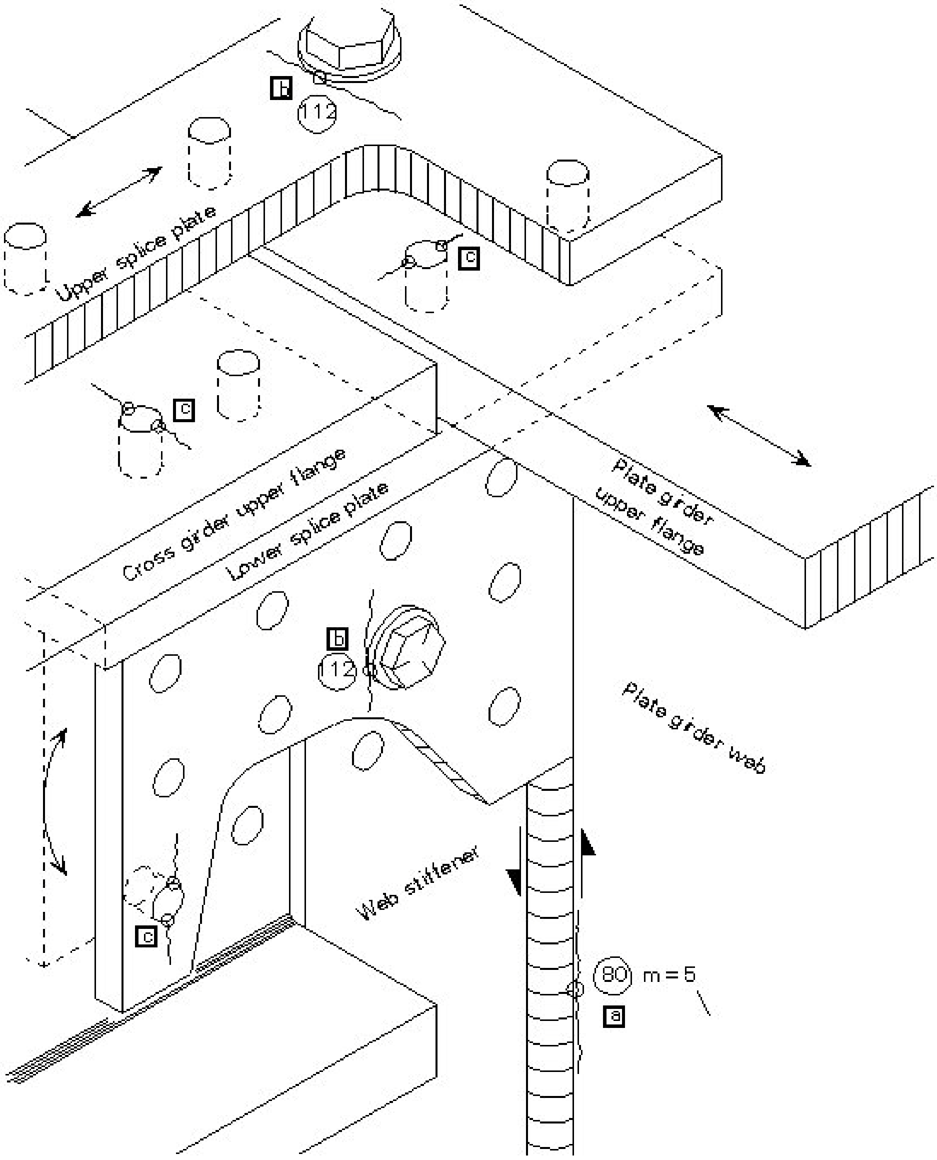


Figure 4a

Notes on Fig. 4a:

a. This is the standard detail for fillet welds in shear. The stress range should be calculated from the weld throat area.

b. As for Figure 2, note c, but as the connection is double sided no eccentricity occurs. Note that crack begins from edge of washer.

c. As for Figure 2, note c, but as the connection is double sided no eccentricity occurs. Note carefully that direction of crack is related to direction of stress.

Notes on Fig. 4b:

a & b Are both as for Figure 2, note c, but note that the direction of stress, and hence the direction of cracking, may differ from hole to hole.

c. As for Figure 4a, note a.

d. Note that the stiffener should terminate at least 10mm above the flange, and the weld should be returned round the bottom of the stiffener. Some recent evidence suggests that out-of-plane flexure of the web plate at the termination of the stiffener could degrade this detail, but research continues on it.

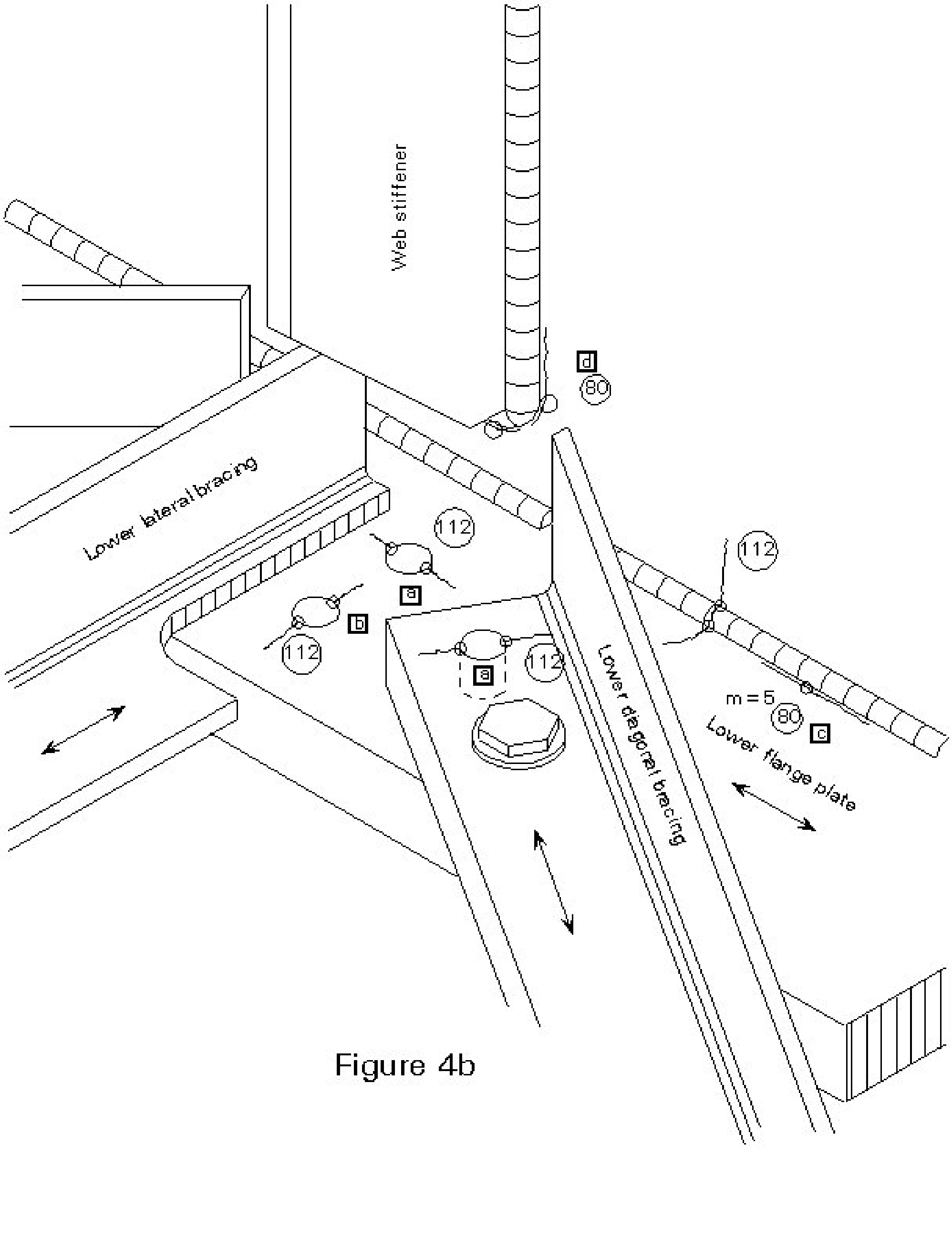

Web stiffener
d
80
Lower lateral bracing
112
a
b
112
a
112
112
m = 5
80
c
Lower diagonal bracing
Lower flange plate

Figure 4b

Notes on Fig. 5:

a. As for Figure 4a, note a. Note crack propagating across direction of principal tensile stress.

b. This is the standard detail for the welding of diaphragms in box girders to the webs and flanges, where the diaphragm thickness is not greater than 12 mm. If the thickness were greater the category would be reduced to 71.

c. This is the standard detail for corner welds of box girders. Note that a good fit between flange and web is essential, so that a one sided weld can be placed without blow through. In certain forms of construction and loading this weld is also prone to bending about its longitudinal axis due either to local traffic loading or distortional effects in the box girder. It is virtually impossible to give a category for such effects, and considerable experience is necessary.

d. As for Figure 2a, note d.

e. See Figure 2, note b. As the weld will be placed manually, it is category 100.

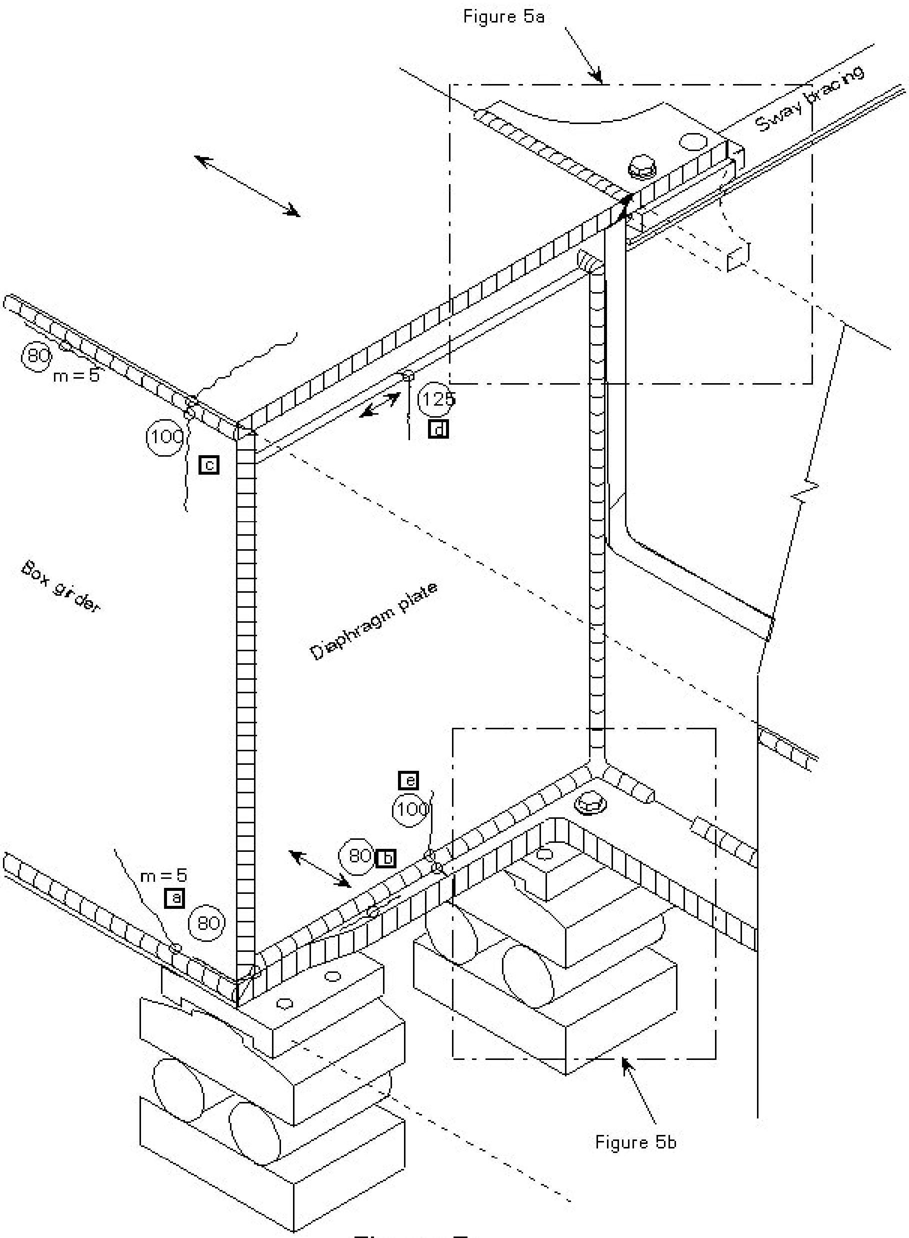


Figure 5a
Sway bracing
80
m = 5
100
c
125
d
Box girder
Diaphragm plate
e
100
80
b
m = 5
a
80
Figure 5b

Figure 5

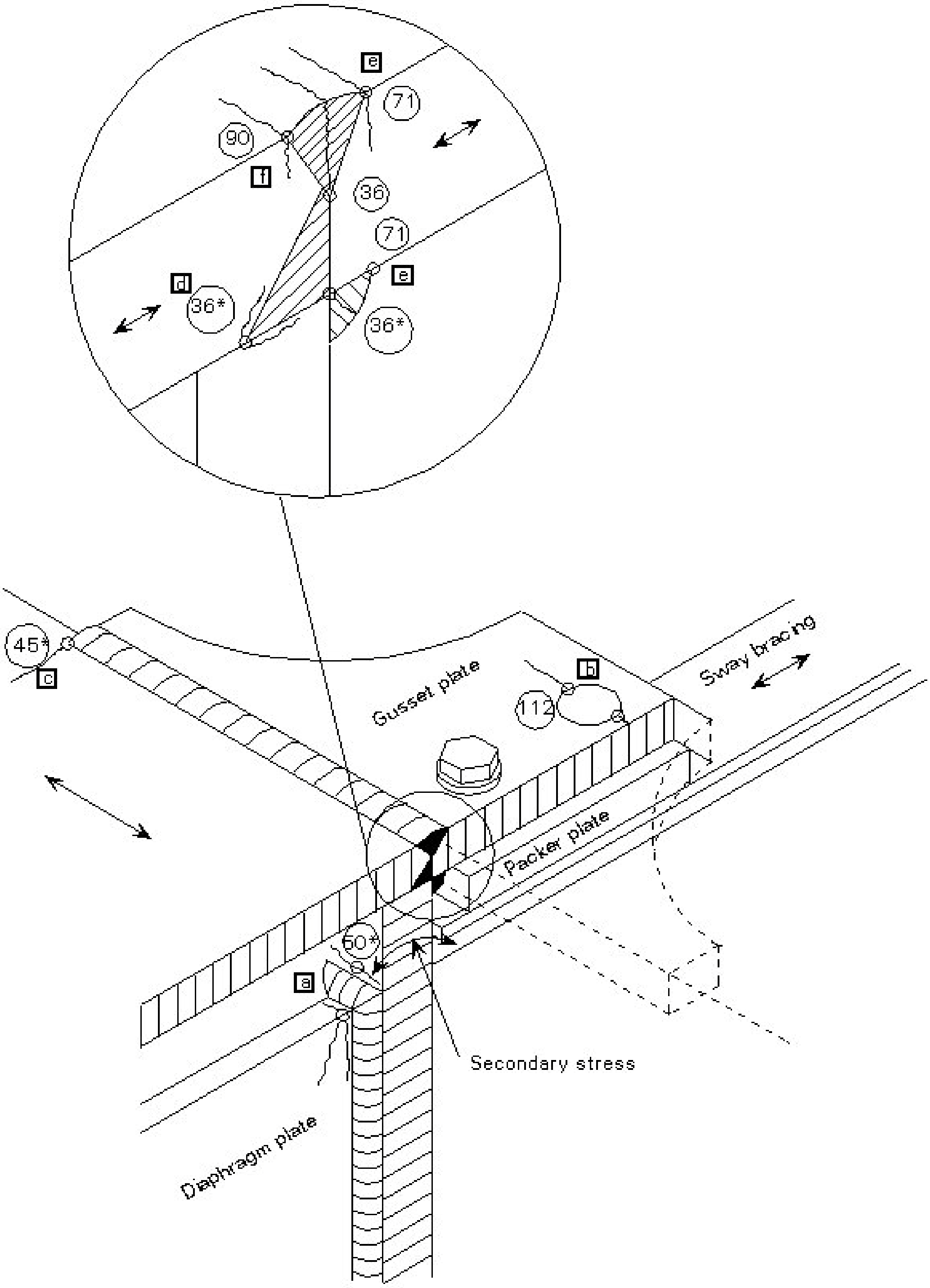
Notes on Fig. 5a:

a. This weld is being stressed by flexure of the web plate and is not readily classifiable from the details in Eurocode 3 Part 1. It is similar, however, to the long attachment, and it is probably safe to use that category (50*).

b. As for Figure 2, note c.

c. See Figure 3, note c. Because the main plate (the flange of the box girder) is wide, the radius of the gusset plate is more severe than it appears and hence the weld falls into the lowest category, for this detail, of 45*.

d, e & f These welds are very difficult to categorize and are not covered explicitly in Eurocode 3 Part 1. Furthermore, although the direction of stress is shown by arrows on the detail, the welds may also be subjected to flexural effects in the web and flange. Considerable caution should therefore be used in attempting to classify them.
Detail d can be thought of as an incomplete penetration butt weld placed from one side only, and hence classified as category 36*.
Details e and f are analogous to the cruciform detail, and so are category 36* as far as cracking from the root is concerned. Cracking in the parent plate from the toe of the weld may be checked at the higher category of 71 in the gusset plate or 90 in the flange, provided the special requirements in the table are met).



e
71
90
f
36
71
e
d
36*
36*
45*
c
Gusset plate
b
112
Sway bracing
Packer plate
50*
a
Secondary stress
Diaphragm plate

Figure 5a

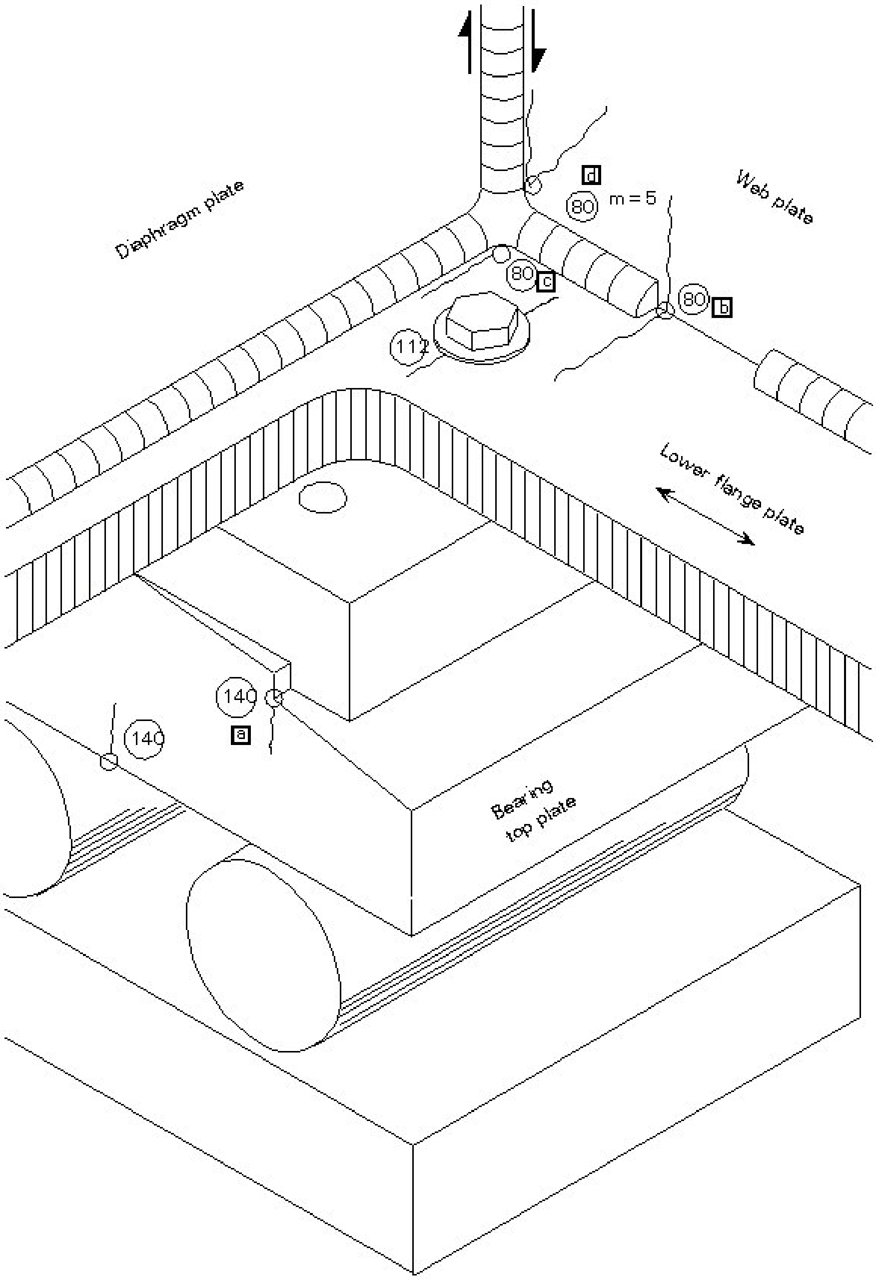


Figure 5b

Notes on Fig. 5b:

a. As this will be a machined plate, the high category of 140 may be used. However, as there is a re-entrant corner, stress concentrations will occur and the magnified stresses should be used in making the check.
b. This is similar to the category at the end of lengths of intermittent fillet weld where the gap is less than 2.5 times the weld length. Hence the category may be taken as 80.
c. As for Figure 5, note b.
d. As for Figure 4a, note a. Note crack propagating across direction of principal tensile stress.

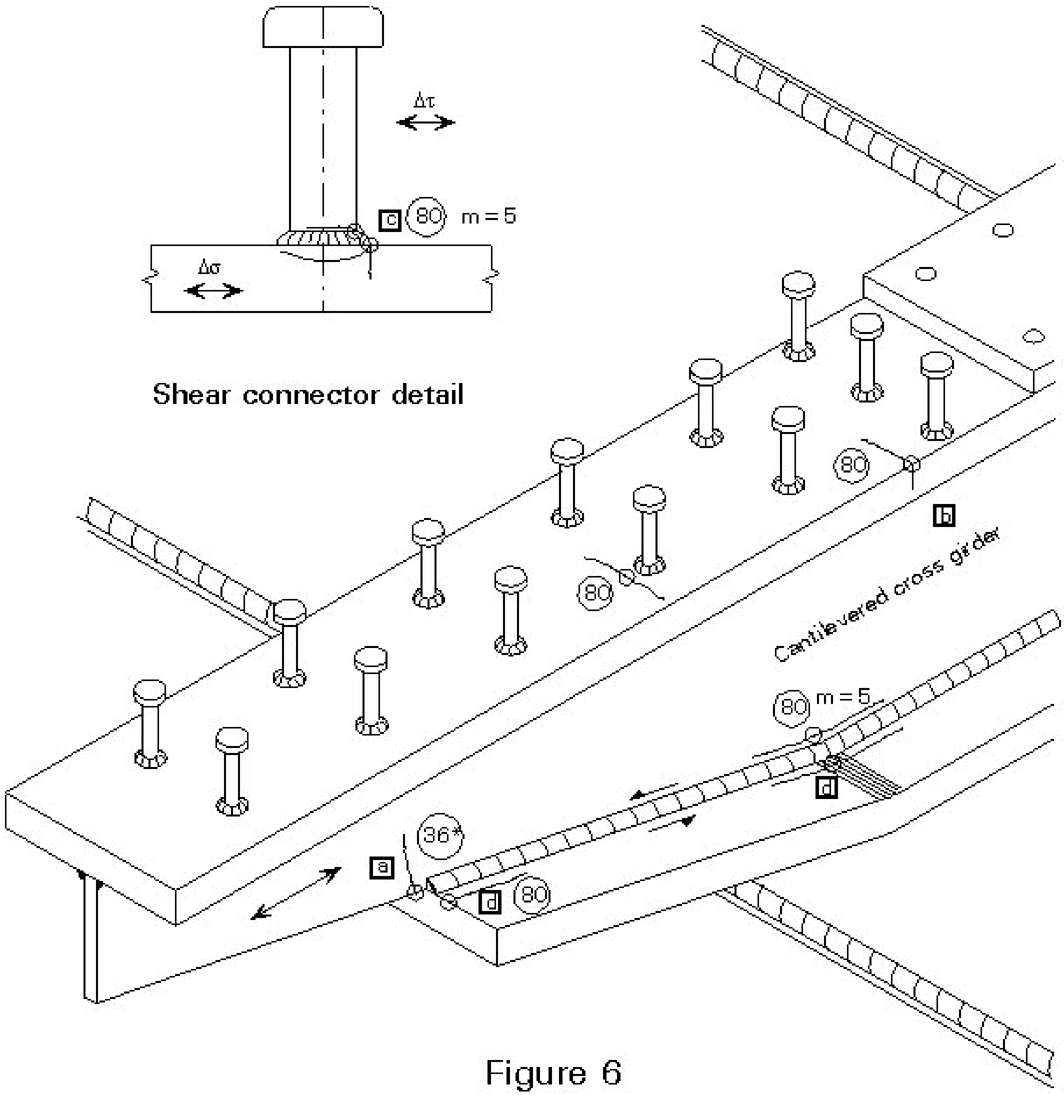


Figure 6

Notes on Fig. 6:

a. This detail is clearly of a very low category and should not be used if the stress range is significant. It would appear appropriate to classify it as the lowest category available, 36*.
b. The effect of the shear connectors on the base plate is to cause a category 80 detail.
c. The weld connecting the shear studs is classified in Code with the shear stress calculated on the nominal cross section of the stud.
d. As for Figure 4a, note a.

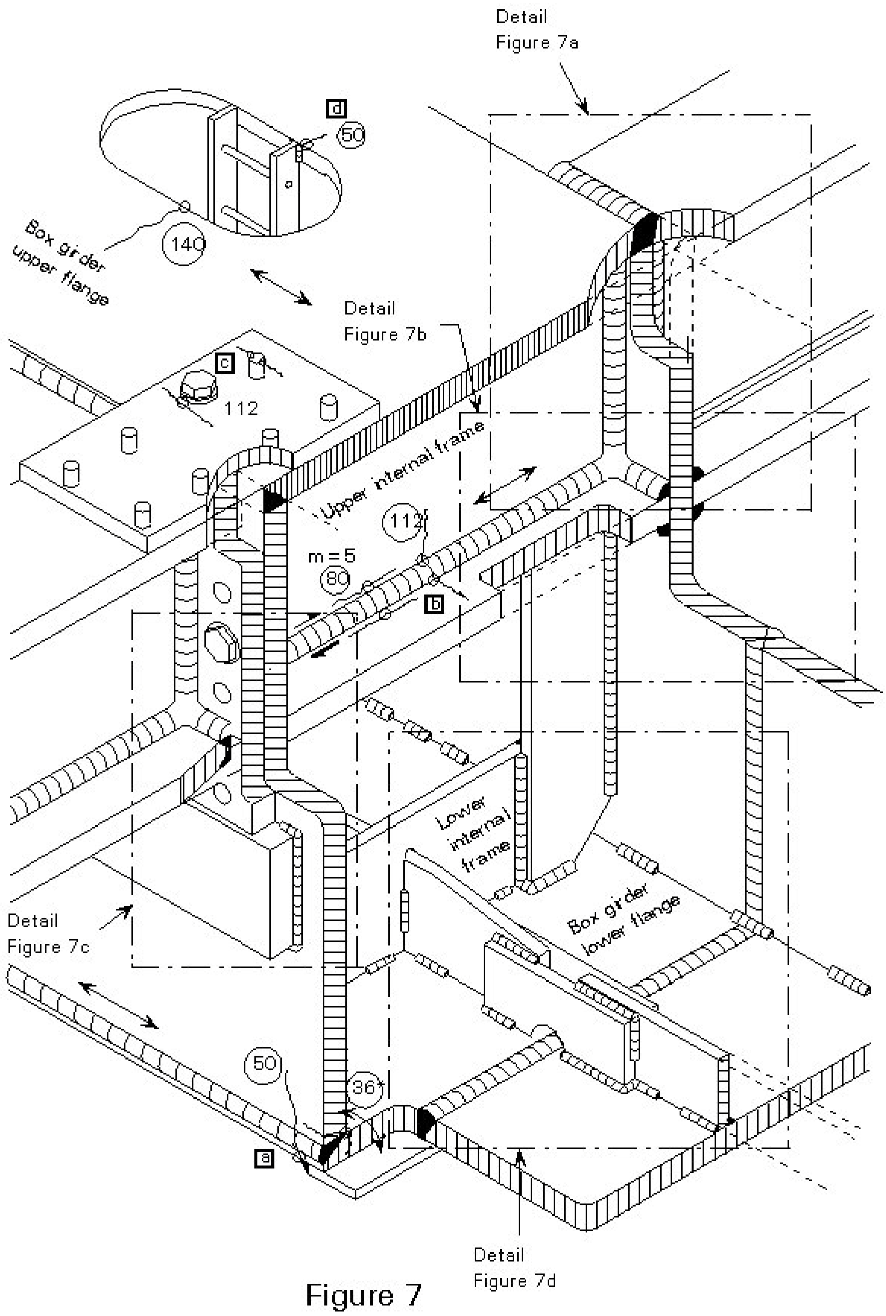


Figure 7

Notes on Fig. 7:

a. This detail represents a butt weld on a permanent backing flat, where the backing flat fillet weld terminates closer than 10mm from the plate edge.
b. As for Figure 2, note b.
c. As for Figure 2, note c.
d. This connection is effectively a welded transverse attachment with a non-load carrying weld. However, the weld terminates at the plate edge, and so the detail is a worse category than in the table. Category 50 appears appropriate. It should be pointed out from this how an apparently minor, non-structural, detail can seriously degrade the fatigue capacity of the structure. If it has to be used, it should be positioned in an area of low stress fluctuation.

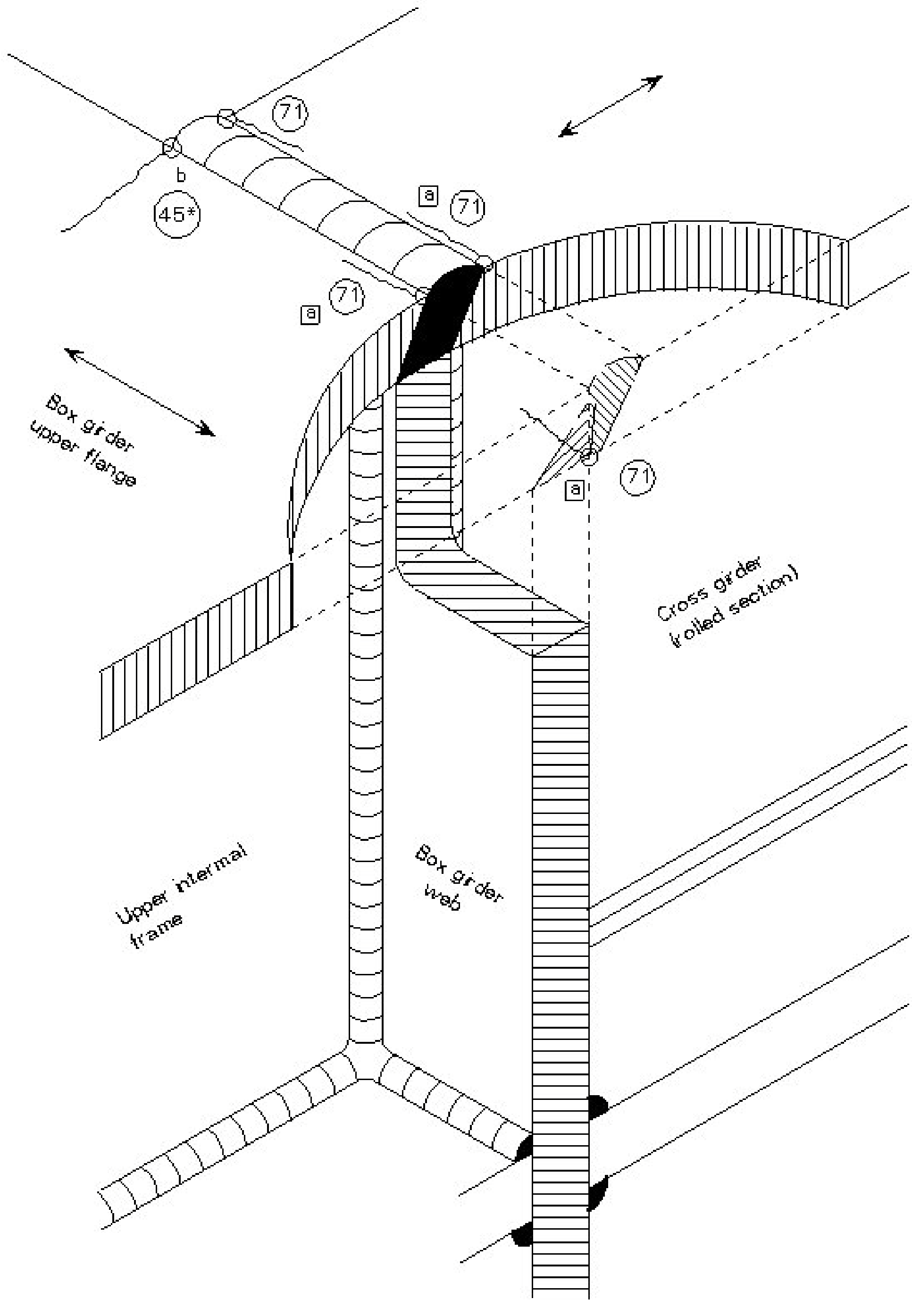


Figure 7a

Notes on Fig. 7a

a. These welds are similar to those associated with cracking in the parent plate from the toe of the weld in cruciform joints.
b. This detail is effectively a gusset with zero plan radius and so falls into category 45*.

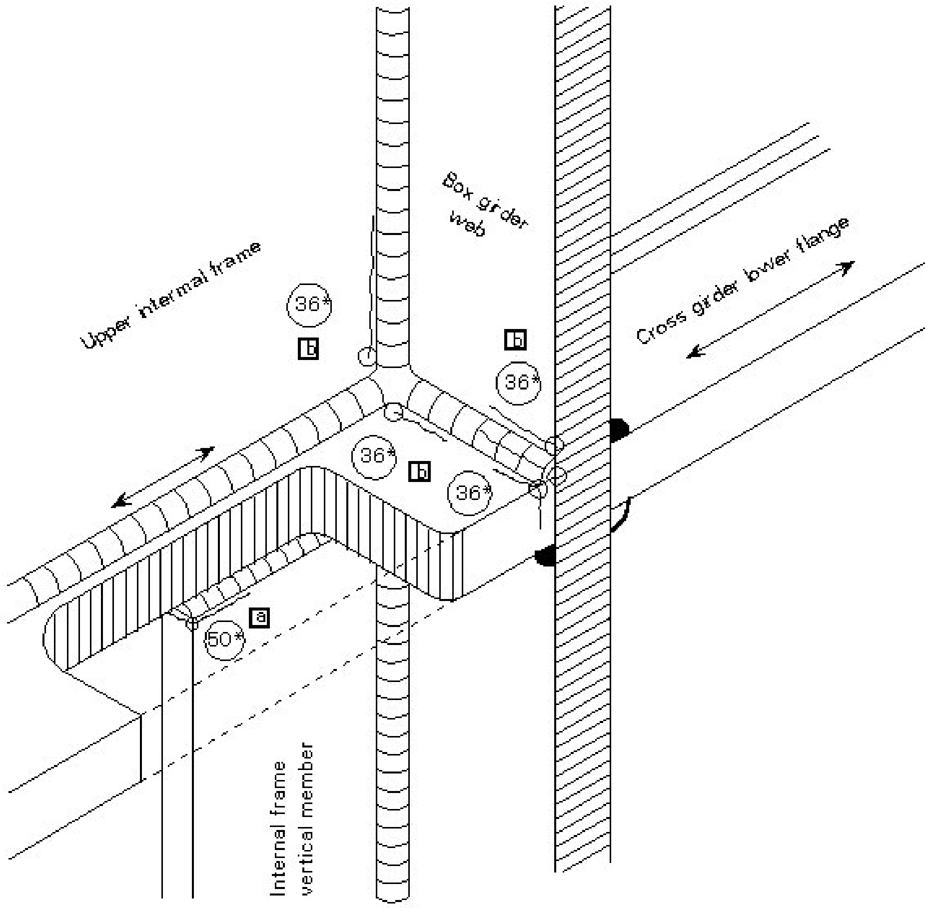


Figure 7b

Notes on Fig. 7b:

a. This is a detail which is not explicitly classified in Eurocode 3 Part 1. It is close to the cruciform detail but probably rather less severe. An appropriate category is 50*.

b. These welds are all effectively the worst possible cruciform details. Note that if the welds are made of sufficiently large section to avoid root cracking there are other mechanisms which may govern.

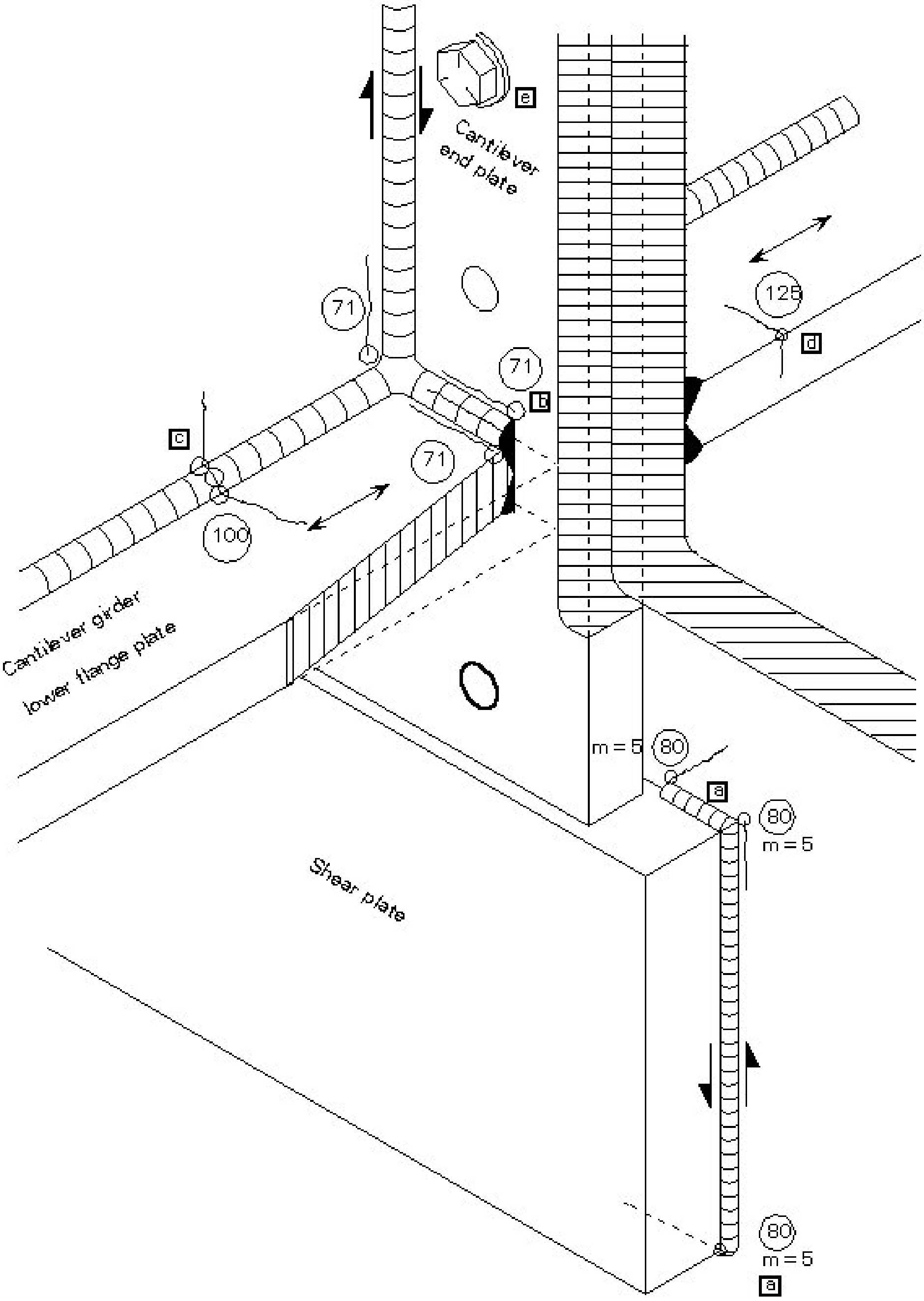


Figure 7c

Notes on Fig. 7c:

a. As for Figure 4a, note a. Note crack propagating across direction of principal tensile stress.
b. These welds are similar to those associated with cracking in the parent plate from the toe of the weld in cruciform joints.
c. This weld is likely to be placed manually - see comments at note b for Figure 2.
d. As for Figure 2a, note d.
e. This detail is intended to represent what happens with a bolt in tension through an endplate. The category for the bolt itself is the low one of 36*, and

the stress in tension in it should be calculated using its stress area. Account should also be taken of any prying action resulting from flexing of the endplate; it should be noted, however, that the stress range in the bolt may be reduced substantially by appropriate preloading. The crack position in the endplate shown on Figure 7c should also be checked under the flexural stresses resulting from prying action.

Notes on Fig. 7d:

a. This is a straightforward instance of the detail for the ends of a continuous weld at a cope hole.
b. This detail is a straightforward instance of the end of an intermittent fillet weld. Note that where it occurs close to (but not actually at) a cope hole, it permits use of the higher category of 80, compared with detail a above where terminating the weld actually at the cope hole requires use of category 71.
c. This detail is not explicitly covered in Eurocode 3 Part 1. The weld is non-load carrying, and hence there are some similarities with the detail shown in Table 9.8.4 (3). However, the "transverse attachment" is a load carrying plate, hence the detail is not fully appropriate. Tests have indicated a somewhat lower category (50) is reasonable.
d. This detail is equivalent to the standard one for cracking in the main plate at the end of a fillet welded lap joint. Note the specified rule for the calculation of the stress in the main plate.
e. This detail is equivalent to the standard one for cracking in the lap plates in a fillet welded lap joint. Note that the weld termination should be held back at least 10mm from the plate edge, and that shear cracking in the weld should also be checked.
f. As for Figure 3, detail g.
g. Whilst this detail belongs in the relatively high category of 140 for a machine gas cut edge with all edge discontinuities removed, the stresses should be calculated using the appropriate stress concentration factor for the radius which is used.
h. This is the standard category for web stiffeners where the thickness of the stiffener does not exceed 12mm and the welds do not come within 10mm of a plate edge.
i. Whilst this detail is not explicitly covered in Eurocode 3 Part 1, it shows a number of similarities to the "wide cover plate" detail. It is clear that a low category is appropriate, and 45* is proposed.

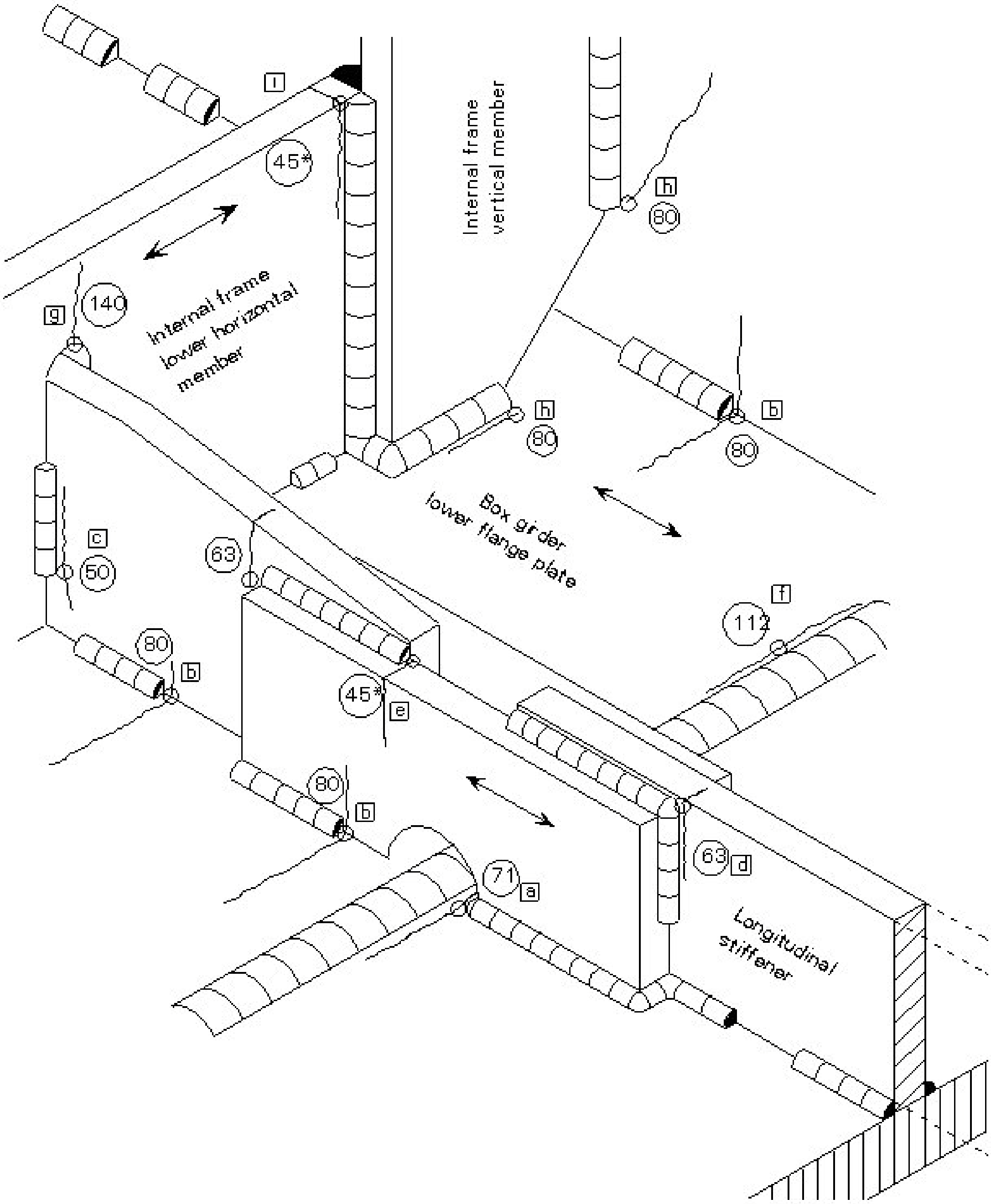

Internal frame
vertical member
Internal frame
lower horizontal
member
Box girder
lower flange plate
Longitudinal
stiffener
45*
140
80
50
63
112
71
i
g
h
b
c
e
f
d
a

Figure 7d

3.4 ALLOWABLE STRESSES FOR WELDED JOINTS

3.4.1 Allowable Stresses for Butt (Groove) Welds

The complete joint penetration groove weld is of the same strength on the effective area as the piece being joined. For permissible stresses two values are considered; the first for good welds fulfilling the requirements of the specifications, the second value for excellent welding where all welds are examined to guarantee the efficiency of the joint:

i- Permissible Stresses for Static Loading:

Table (3.9) Permissible Stresses for Static Loading in Groove (Butt) Welds

Type of Joint	Kind of Stress	Permissible Stress For	
		Good Weld	Excellent Weld
Butt and K- weld	Compression	1.0 F_c	1.1 F_c
	Tension	0.7 F_t	1.0 F_t
	Shear	1.0 q_{all}	1.1 q_{all}

Where F_c , F_t , and q_{all} are the minimum allowable compression, tension, and shear stresses of the base metals.

ii- Permissible stresses for Fatigue loading:

See section 3.3.

3.4.2 Allowable Stresses for Fillet Welds

The stress in a fillet weld loaded in an arbitrary direction can be resolved into the following components:

$f_{\perp}$ = the normal stress perpendicular to the axis of the weld.
$q_{//}$ = the shear stress along the axis of the weld.
$q_{\perp}$ = the shear stress perpendicular to the axis of the weld.

These stresses shall be related to the size (s) of the legs of the isosceles triangle inscribed in the weld seam if the angle between the two surfaces to be welded is between 60^O and 90^O . When this angle is greater than 90^O the size of the leg of the inscribed rectangular isosceles triangle shall be taken.

The permissible stresses F_{pw} for all kinds of stress for fillet welds must not exceed the following:

All kind of stresses $F_{pw} \leq 0.2\ \tilde{F}_u$ ……………………………... 3.41

Where F_u is the ultimate strength of the base metal (see section 1.3.1.2).

In case where welds are simultaneously subject to normal and shear stresses, they shall be checked for the corresponding principal stresses. For this combination of stresses, an effective stress value f_{eff} may be utilized and the corresponding permissible weld stress is to be increased by 10 % as follows:

$$\mathbf{f_{eff} = \sqrt{f_\perp^{\ 2} + 3(q_\perp^{\ 2} + q_{//}^{\ 2})}} \quad \text{……………………………………………..} \quad 3.42$$

The effective length of a fillet weld is usually taken as the overall length of the weld minus twice the weld size (s) as deduction for end craters.

3.5 ALLOWABLE STRESSES FOR BOLTED JOINTS

3.5.1 STRENGTH OF NON-PRETENSIONED BOLTED CONNECTIONS OF THE BEARING TYPE

In this category ordinary bolts (manufactured from low carbon steel) or high strength bolts, from grade 4.6 up to and including grade 10.9 can be used. No pre- tensioning and special provisions for contact surfaces are required. The design load shall not exceed the shear resistance nor the bearing resistance obtained from Clauses 6.4.1 and 6.4.2.

3.5.1.1 Shear Strength R_{sh}

i- The allowable shear stress $\mathbf{q_b}$ for bolt grades 4.6, 5.6 and 8.8 shall be taken as follows:

$$\mathbf{q_b = 0.25\ F_{ub}} \quad \text{…………………………………………………..} \quad \mathbf{3.43}$$

ii- For bolt grades 4.8, 5.8 , 6.8, and 10.9, the allowable shear stress $\mathbf{q_b}$ is reduced to the following:-

$$\mathbf{q_b = 0.2\, F_{ub}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.44}$$

iii- For the determination of the design shear strength per bolt ($\mathbf{R}_{sh}$) , where the shear plane passes through the threaded portion of the bolt:-

$$\mathbf{R_{sh} = q_b \,.\, A_s \,.n} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.45}$$

Where :

$\mathbf{A_s}$ = The tensile stress area of bolt.
n = Number of shear planes.

iv- For bolts where the threads are excluded from the shear planes the gross cross sectional area of bolt (A) is to be utilized.

v- The values for the design of shear strength given in Equations 6.43 and 6.44 are to be applied only where the bolts used in holes with nominal clearances not exceeding those for standard holes as specified in Clause 6.2.2 of Code.

3.5.1.2 Bearing Strength R_b

i- The bearing strength of a single bolt shall be the effective bearing area of bolt times the allowable bearing stress at bolt holes:-

$$\mathbf{R_b} = \mathbf{F_b\, .d.}\, \min \sum \mathbf{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.46}$$

Where:

$\mathbf{F_b}$ = Allowable bearing stress.
d = Shank diameter of bolt.
Min $\sum$ **t** = Smallest sum of plate thicknesses in the same direction of the bearing pressure.

ii- For distance center- to center of bolts not less than 3d, and for end distance in the line of force greater than or equal to 1.5 d, the allowable bearing stress $\mathbf{F_b}$ (t/cm^2):

$$\mathbf{F_b} = \alpha\, \mathbf{F_u} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.47}$$

Where:

$\mathbf{F_u}$ = The ultimate tensile strength of the connected plates.

As the limitation of deformation is the relevant criteria the α-values of Equation 6.5 are given in Table 3.10

Table (3.10) Values of α for Different Values of End Distance

	End distance in direction of force			
	$\geq 3d$	$\geq 2.5d$	$\geq 2.0d$	$\geq 1.5d$
α	1.2	1.0	0.8	0.6

3.5.1.3 Tensile Strength R_t

When bolts are externally loaded in tension, the tensile strength of a single bolt (R_t) shall be the allowable tensile bolt stress (F_{tb}) times the bolt stress area (A_s)

$$R_t = F_{tb} \cdot A_s \quad \ldots\ldots \quad 3.48$$

$$\text{With } F_{tb} = 0.33\, F_{ub} \quad \ldots\ldots \quad 3.49$$

3.5.1.4 Combined Shear and Tension in Bearing–Type Connections

When bolts are subjected to combined shear and tension, the following circular interaction Equation is to be satisfied:

$$\left(\frac{R_{sh.a}}{R_{sh}}\right)^2 + \left(\frac{R_{t.a}}{R_t}\right)^2 \leq 1 \quad \ldots\ldots \quad 3.50$$

Where:

$R_{sh.a}$ = The actual shearing force in the fastener due to the applied shearing force.

$R_{t.a}$ = The actual tension force in the fastener due to the applied tension force.

R_{sh} and R_t = The allowable shear and tensile strength of the fastener as previously given in Equations (6.45) and (6.48) respectively.

3.5.2 HIGH STRENGTH PRETENSIONED BOLTED CONNECTIONS OF THE FRICTION TYPE

3.5.2.1 General

In this category of connections high strength bolts of grades 8.8 and 10.9 are only to be utilized. The bolts are inserted in clearance holes in the steel components and then pretensioned by tightening the head or the nut in accordance with Clause 6.5.3 where a determined torque is applied. The contact surfaces will be firmly clamped together particularly around the bolt holes.

Any applied force across the shank of the bolt is transmitted by friction between the contact surfaces of the connected components, while the bolt shank itself is subjected to axial tensile stress induced by the pretension and shear stress due to the applied torque.

3.5.2.2 Design Principles of High Strength Pretensioned Bolts

a) The Pretension Force

The axial pretension force T produced in the bolt shank by tightening the nut or the bolt head is given by:-

$$\mathbf{T = (0.7)\, F_{yb}\; A_S} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{3.51}$$

Where:

$\mathbf{F_{yb}}$ = Yield (proof) stress of the bolt material, (see section 1.3.3).

$\mathbf{A_s}$ = The bolt stress area.

b) The Friction Coefficient or the Slip Factor "μ"

i- The friction coefficient between surfaces in contact is that dimensionless value by which the pretension force in the bolt shank is to be multiplied in order to obtain the frictional resistance P_S in the direction of the applied force.

ii- The design value of the friction coefficient depends on the condition and the preparation of the surfaces to be in contact. Surface treatments are classified into three classes, where the coefficient of friction **μ** should be taken as follows:-

μ = 0.5 for class A surfaces.
μ = 0.4 for class B surfaces.
μ = 0.3 for class C surfaces.

iii- The friction coefficient μ of the different classes is based on the following treatments:

In class A:
- Surfaces are blasted with shot or grit with any loose rust removed, no painting.
- Surfaces are blasted with shot or grit and spray metallized with Aluminum.
- Surfaces are blasted with shot or grit and spray metallized with a Zinc based coating.

In class B:
- Surfaces are blasted with shot or grit and painted with an alkali-zinc silicate painting to produce a coating thickness of 50-80 μm.

In class C:
- Surfaces are cleaned by wire brushing, or flame cleaning, with any loose rust removed.

iv- If the coatings other than specified are utilized, tests are required to determine the friction coefficient. The tests must ensure that the creep deformation of the coating due to both the clamping force of the bolt and the service load joint shear are such that the coating will provide satisfactory performance under sustained loading.

c) The Safe Frictional Load (P_s)

The design frictional strength for a single bolt of either grade 8.8 or 10.9 with a single friction plane is derived by multiplying the bolt shank pretension T by the friction coefficient μ using an appropriate safety factor γ as follows:-

$$P_S = \mu T / \gamma \quad \text{.......} \quad 3.52$$

Where :

T = Axial pretensioning force in the bolt.

μ = Friction coefficient.

γ = Safety factor with regard to slip .

= **1.25** and **1.05** for cases of loading **I** and **II** respectively for ordinary steel work.

= **1.6** and **1.35** for case of loading **I** and **II** respectively for parts of bridges, cranes and crane girders which are subjected mainly to dynamic loads.

Table 3.11 gives the pretension force (**T**) and the permissible frictional load (**P**$_s$) per one friction surface for bolts of grade 10.9.

3.5.3 Design Strength In Tension Connections

Where the connection is subjected to an external tension force (T_{ext}) in the direction of the bolts axis, the induced external tension force per bolt ($\mathbf{T}_{ext,b}$) is to be calculated according to the following relation:-

$$\mathbf{T_{(ext,b)} = T_{(ext)} / n \leq 0.6\ T} \quad \ldots\ldots\ldots\ldots \quad \mathbf{3.53}$$

Where :

n = The total number of bolts resisting the external tension force $\mathbf{T_{(ext)}}$.

Table (3.11) Properties and Strength of High Strength Bolts (Grade 10.9*)

Bolt Diameter (d) mm	Bolt Area (A) cm^2	Stress Area (A_s) cm^2	Pretension Force (T) tons	Required Torque (M_a) kg.m	Permissible Friction Load of One Bolt Per One Friction Surface (P_s) tons							
					Ordinary Steel Work				Bridges and Cranes			
					St.37&42-44 (μ=0.4)		**St. 50-55 (μ=0.5)**		**St.37&42-44 (μ=0.4)**		**St. 50-55 (μ=0.5)**	
					Cases of Loading				Cases of Loading			
					I	II	I	II	I	II	I	II
M12	**1.13**	**0.84**	**5.29**	**12**	**1.69**	**2.01**	**2.11**	**2.52**	**1.32**	**1.56**	**1.65**	**1.95**
M16	**2.01**	**1.57**	**9.89**	**31**	**3.16**	**3.37**	**3.95**	**4.71**	**2.47**	**2.92**	**3.09**	**3.66**
M20	**3.14**	**2.45**	**15.4**	**62**	**4.93**	**5.90**	**6.17**	**7.36**	**3.85**	**4.56**	**4.82**	**5.71**
M22	**3.80**	**3.03**	**19.1**	**84**	**6.10**	**7.27**	**7.63**	**9.10**	**4.77**	**5.65**	**5.96**	**7.06**
M24	**4.52**	**3.53**	**22.2**	**107**	**7.11**	**8.45**	**8.89**	**10.6**	**5.55**	**6.58**	**6.94**	**8.22**
M27	**5.73**	**4.59**	**28.9**	**157**	**9.25**	**11.0**	**11.6**	**13.8**	**7.22**	**8.55**	**9.03**	**10.7**
M30	**7.06**	**5.61**	**35.3**	**213**	**11.3**	**13.5**	**14.1**	**16.8**	**8.83**	**10.5**	**11.1**	**13.1**
M36	**10.2**	**8.17**	**51.5**	**372**	**16.5**	**19.6**	**20.6**	**24.5**	**12.9**	**15.2**	**16.1**	**19.1**

* For HSB grade 8.8 , the above values shall be reduced by 30%

In addition to the applied tensile force per bolt $T_{(ext,b)}$, the bolt shall be proportioned to resist the additional induced prying force (P) (Fig. 3.6).

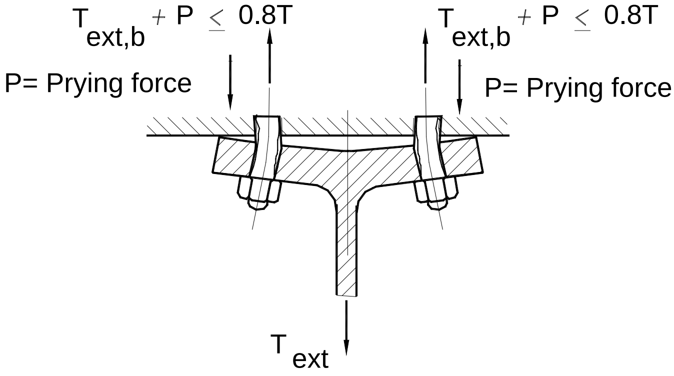


Figure 3.6 Prying Force

The prying force (P) depends on the relative stiffness and the geometrical configuration of the steel element composing the connection. The prying force should be determined according to Clause 6.9 of ECP 2001 and hence the following check is to be satisfied:

$$\mathbf{T_{(ext,b)} + P \leq 0.8\, T} \qquad \textbf{3.54}$$

3.5.4 Design Strength in Connections Subjected to Combined Shear and Tension

In connections subjected to both shear (Q) and tension (T_{ext}), the design strength for bolt is given by the following formulae:-

$$\left.\begin{aligned} \mathbf{Q_b} &\leq \frac{\mu\,(T - T_{ext,b})}{\gamma} \\ \mathbf{T_{(ext,b)}} &+ \mathbf{P \leq 0.8\, T} \end{aligned}\right\} \qquad \textbf{3.55}$$

3.5.5. Design Strength in Connections Subjected to Combined Shear and Bending Moment

In moment connections of the type shown in Fig. 3.7, the loss of clamping forces in region “A” is always coupled with a corresponding increase in contact pressure in region “B”. The clamping force remains unchanged and there is no decrease of the frictional resistance as given by the following :-

$$\mathbf{P_S = \mu\, T / \gamma} \qquad \textbf{3.56}$$

The induced maximum tensile force $T_{(ext,b,M)}$ due to the applied moment (M) in addition to the prying force P that may occur, must not exceed the pretension force as follows:-

$$\mathbf{T_{(ext,b,M)}} + P \leq \mathbf{0.8T} \quad \ldots\ldots \quad \mathbf{3.57}$$

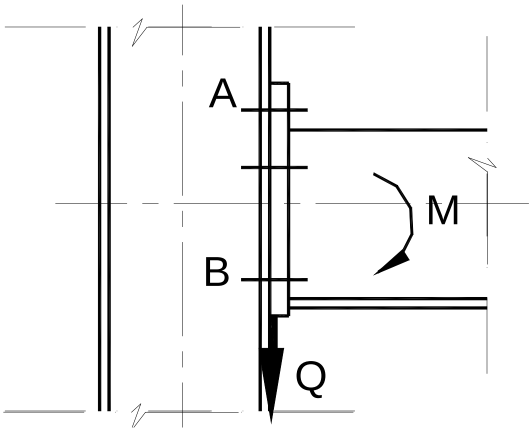
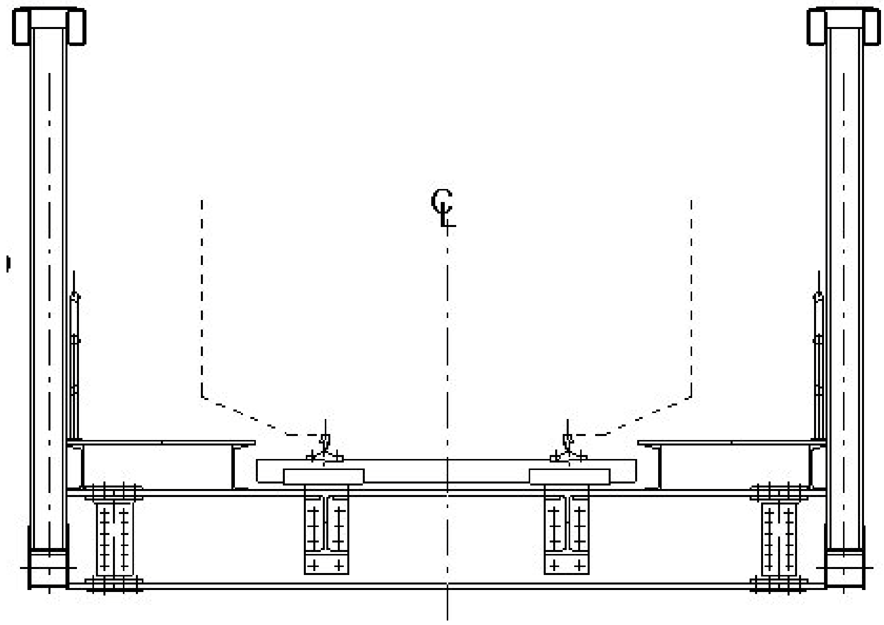


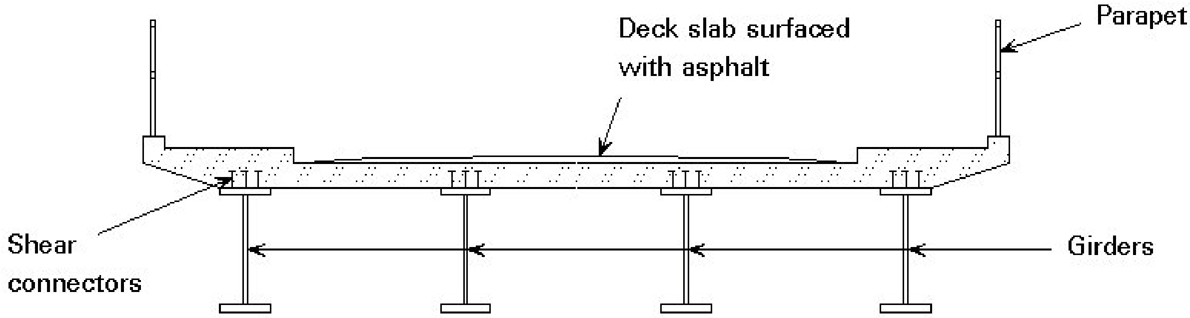
Figure 3.7 Connections Subjected to Combined Shear and Bending Moment

3.5.6 Design Strength in Connections Subjected to Combined Shear, Tension, and Bending Moment

When the connection is subjected to shearing force (Q), a tension force (T_{ext}) and a bending moment (M), the design strength per bolt is to be according to the following formulae:-

$$\left.\begin{aligned} \mathbf{Q_b} &\leq \frac{\mu\,(\mathbf{T} - \mathbf{T_{ext,b}})}{\gamma} \\ T_{(ext,b)} + T_{(ext,b,M)} &+ \mathbf{P} \leq 0.8T \end{aligned}\right\} \quad \ldots\ldots \quad \mathbf{3.58}$$





CHAPTER 4

BRIDGE FLOORS

CHAPTER 4

BRIDGE FLOORS

4.1 INTRODUCTION

The principal function of a bridge deck is to provide support to local vertical loads (from highway traffic, railway or pedestrians) and transmit these loads to the primary superstructure of the bridge, Figure 4.1. In addition to this, the overall structural actions may include:

2. Contributing to the top flange of the longitudinal girders
3. Contributing to the top flange of cross girders at supports and, where present in twin girder and cross girder structures, throughout the span,
4. Stabilizing stringers and cross girders in the transversal direction,
5. Acting as a diaphragm to transmit horizontal loads to supports,
6. Providing a means of distribution of vertical load between longitudinal girders.

It may be necessary to take account of these combined actions when verifying the design of the deck. This is most likely to be the case when there are significant stresses from the overall structural actions in the same direction as the maximum bending moments from local deck actions, e.g. in structures with cross girders where the direction of maximum moment is along the bridge.

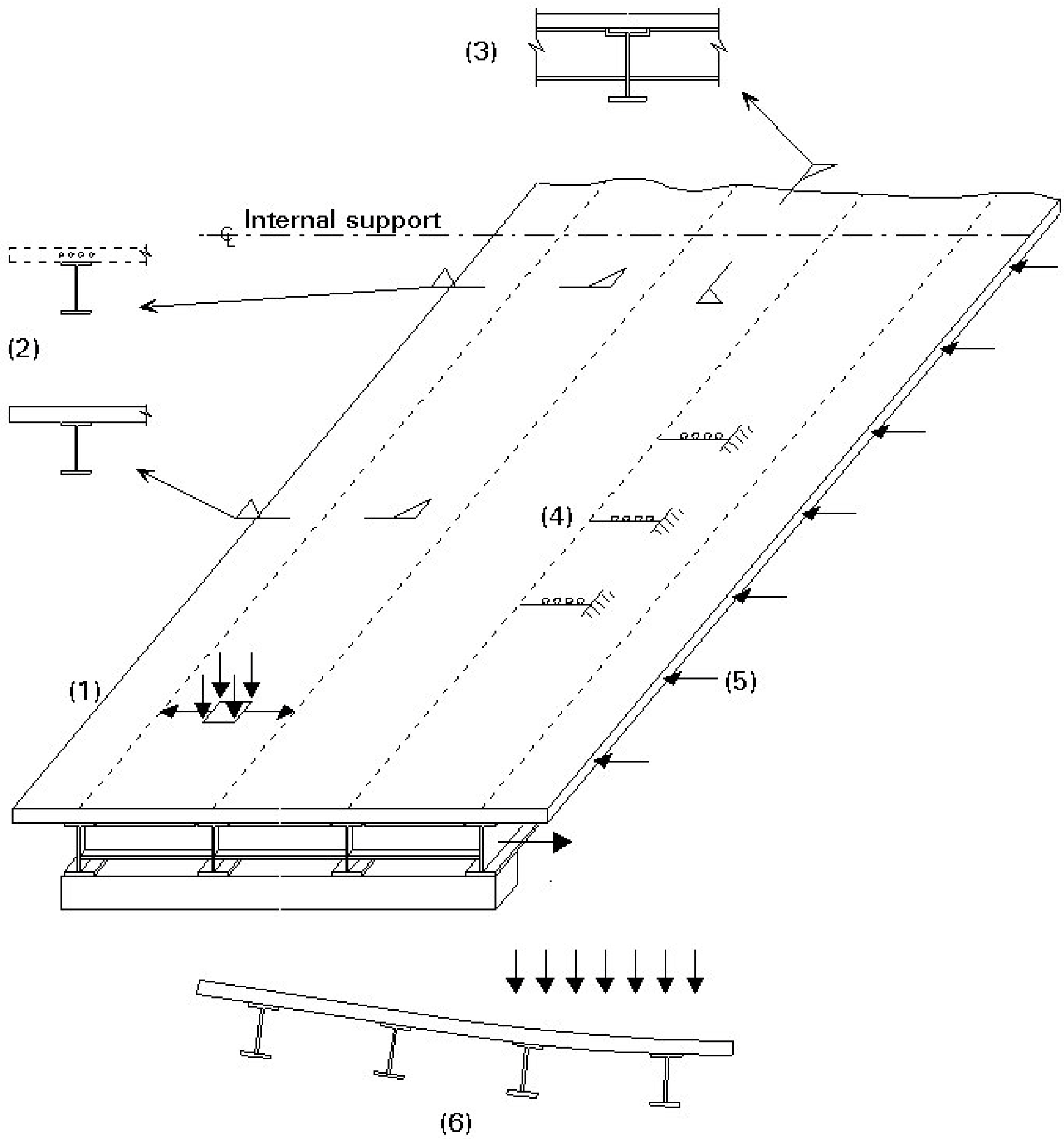


Fig 4.1 Structural Actions of a Roadway Bridge Deck

4.2 STRUCTURAL SYSTEMS OF BRIDGE FLOORS

Structural systems used in bridge floors vary according to the bridge usage as follows:

4.2.1 ROADWAY BRIDGE FLOORS

Three main types of transverse structural systems may be used in roadway bridge floors:

a) Slab
b) Beam-Slab (slab with floor beams)
c) Orthotropic plate floor

a) In the ***Slab*** cross-sections, Fig. 4.2a, a reinforced concrete deck slab about 20 to 30 cm thick is supported directly on the bridge main girders. This system is economical for small spans, generally below 25m, where multiple girders are used for the longitudinal structural system at spacing of 2.5 – 4 m.

b) In the ***Beam-Slab*** cross-sections, Fig 4.2b, the deck slab is supported on longitudinal floor beams (called stringers) and /or transversal floor beams (called cross-girders). This system is generally adopted for medium spans below 80 m where the spacing of main girders exceeds about 4 m.

In both cases, the slab may act independently of the supporting beams (a very uneconomic solution for medium and large spans) or it may work together with the supporting beams (composite bridge deck). The composite action requires the shear flow between the slab and the girders to be taken by shear connectors as shown in Fig. 4.2a.

c) In the ***Orthotropic Plate Deck,*** Fig. 4.2c, a stiffened steel plate covered with a light wearing surface is welded on top of the main girder webs to provide a deck surface. The deck plate, acting as the top flange of the main girders, gives a very efficient section in bending. The steel plate is longitudinally stiffened by ribs, which may be of open or closed section. Transversally, the ribs are connected through the transverse floor beams (cross girders) yielding a complex grillage system where the main girders, the steel plate, the ribs and the floor beams act together.

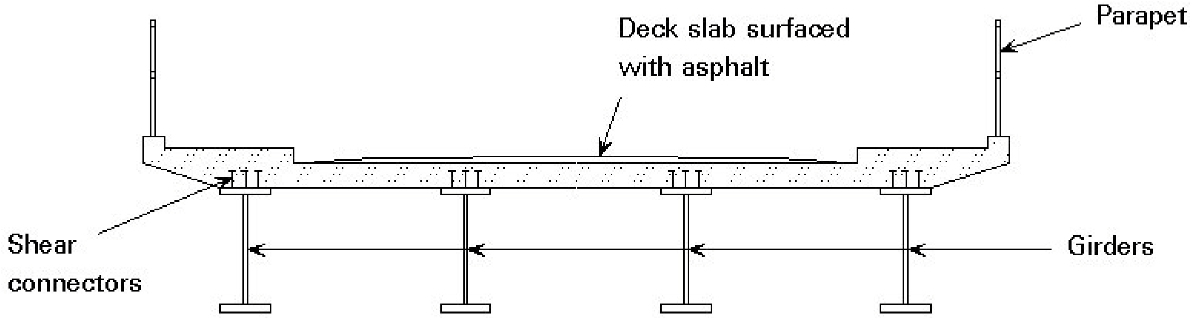


Fig. 4.2 Roadway Bridge Floors: a) Slab Type Floor

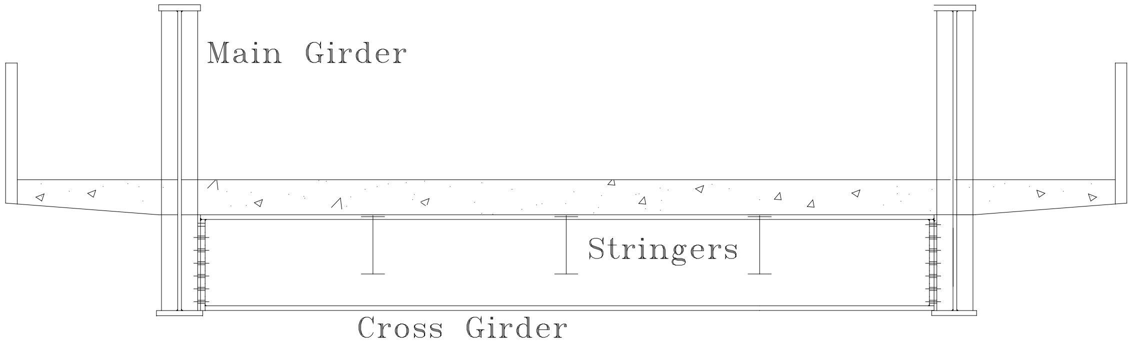


Fig. 4.2 Roadway Bridge Floors: b) Beam Slab Type Floor

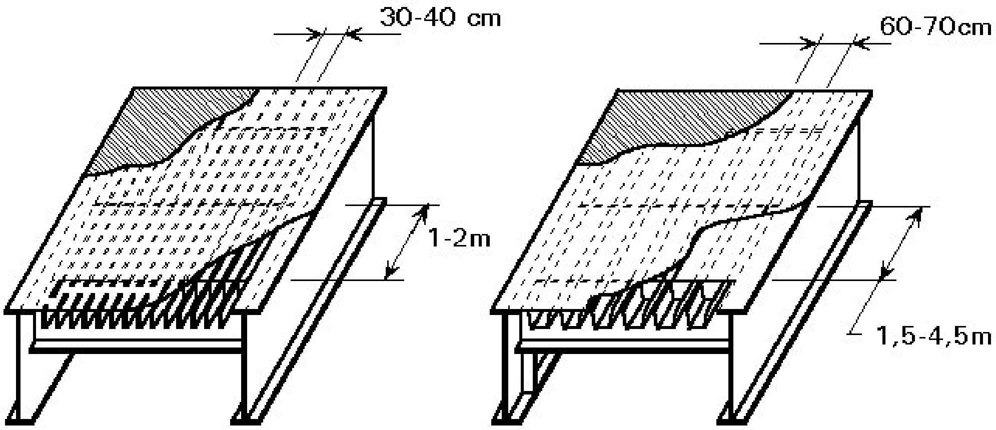


Fig. 4.2 Roadway Bridge Floors: c) Orthotropic Plate Floors

When compared to concrete slab decks, the biggest disadvantage of orthotropic steel plate decks is their high initial cost and the maintenance required. Concrete decks are therefore usually more economic than orthotropic steel plates. The latter are only adopted when deck weight is an important component of loading, i.e. for long span and moveable bridges.

3.2.2 RAILWAY BRIDGE FLOORS

Tracks of railway bridges are normally carried on timber sleepers which are 260 cm long and spaced at not more than 50 cm between centers. The sleepers are then supported on the bridge floor system, which may be of the open timber floor type, Fig. 4.3a, or of the ballasted floor type, Fig. 4.3b:

a) The ***Open Floor*** type consists of longitudinal beams, called stringers, spaced at 1.5 to 1.8 meters, and transversal beams, called cross girders, spaced at 4.0 to 6.0 meters.

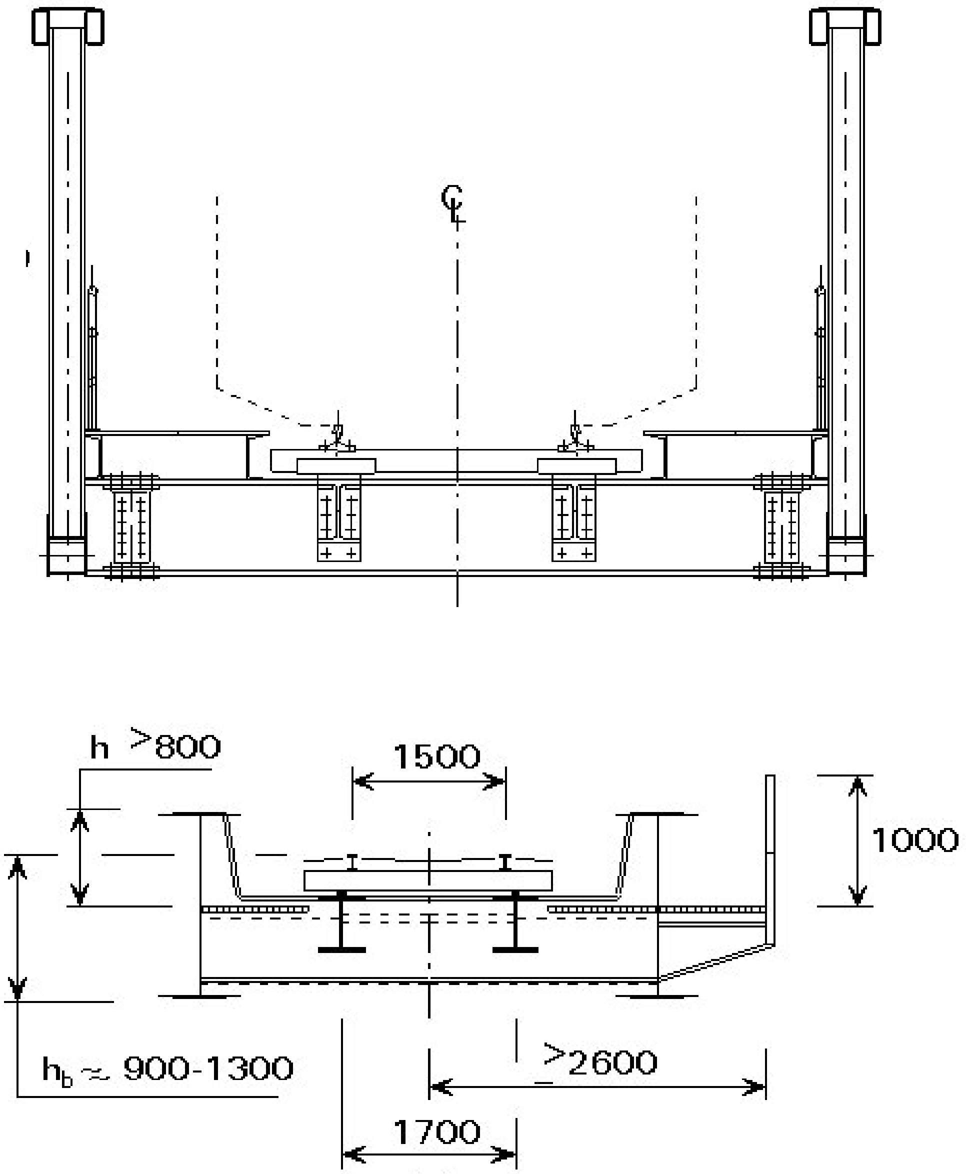


Fig. 4.3 Railway Bridge Floors: a) Open Timber Floor

b) The ***Ballasted floor*** type consists of a 20 cm layer of ballast carried on an R.C. slab which is supported on steel floor beams, e.g.; stringers and/or cross girders as shown in Fig. 4.3b:

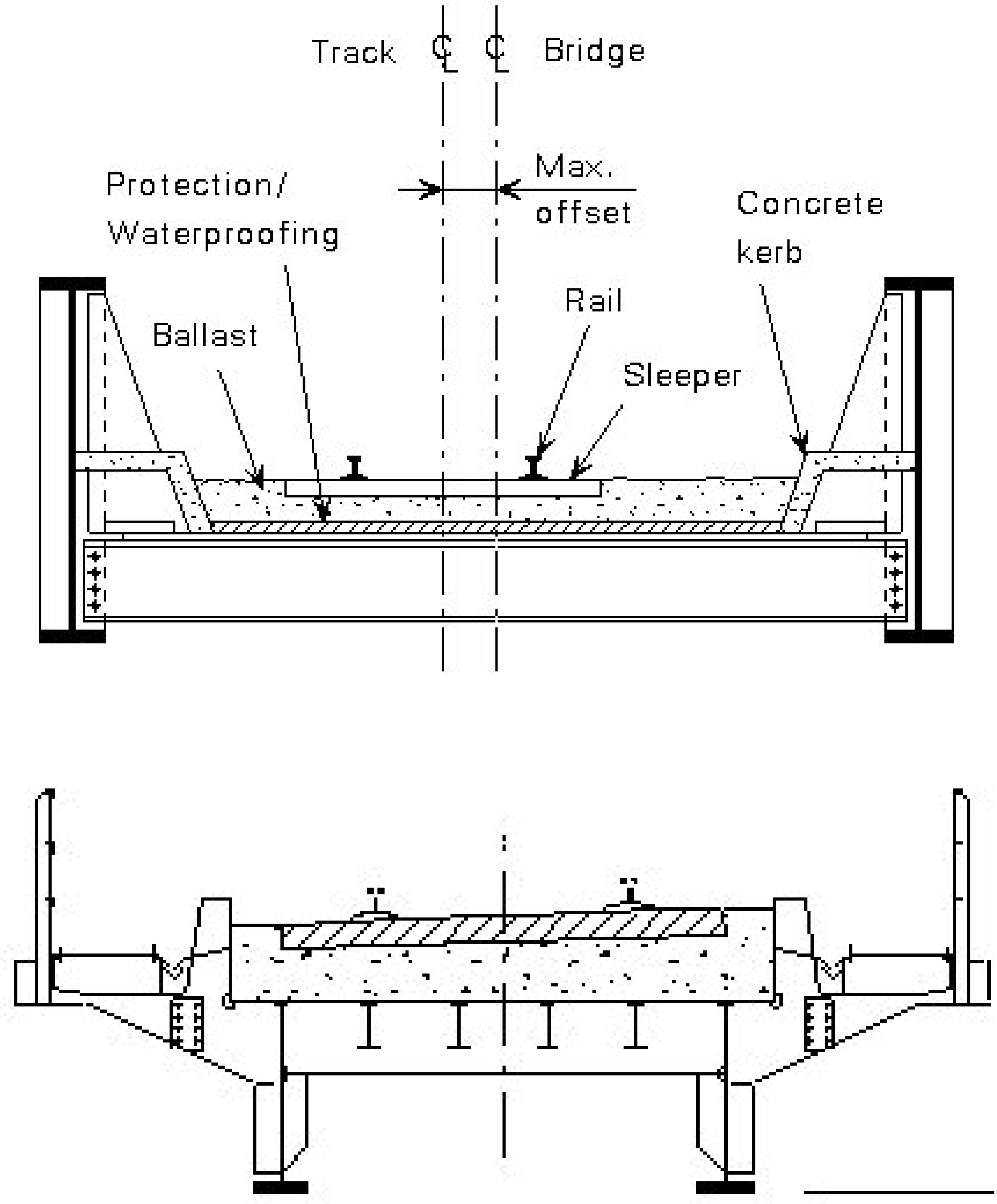


Fig. 4.3 Railway Bridge Floors: b) Ballasted Floor

4.3 DESIGN CONSIDERATIONS

4.3.1) ALLOWABLE STRESSES FOR STEEL St 52 (ECP 2001)

4.3.1.1) Allowable Stress in Bending F_b

1- Tension and compression due to bending on extreme fibers of **"compact"** sections symmetric about the plane of their minor axis:

$$F_{bx} = 0.64\ F_y = 2.304\ t/cm^2$$

In order to qualify under this section:

i- The member must meet the compact section requirements of Table 2.1 of ECP.
Note that most rolled sections satisfy these requirements.

ii- The laterally unsupported length (L_u) of the compression flange is limited by

$$L_{u1} \le \frac{20 b_f}{\sqrt{F_y}} \qquad\qquad L_{u2} \le \frac{1380 A_f}{d\,F_y} C_b$$

2- Compression on extreme fibers of flexural members meeting the **"non-compact"** section requirements of Table 2.1of ECP:

$$F_{bx} = F_{ltb} < 0.58\ F_y = 2.1\ t/cm^2$$

Usually F_{ltb} is governed by: $$F_{ltb1} = \frac{800}{L_u . d / A_f} C_b \le 0.58\ F_y$$

3- Tension and compression due to bending on extreme fibers of doubly symmetrical I-shape members meeting the **"compact"** section requirements of Table 2.1(c) of ECP, and **bent about their minor axis:**

$$F_{by} = 0.72\ F_y = 2.592\ t/cm^2$$

4.3.1.2) Allowable Stress In Shear:

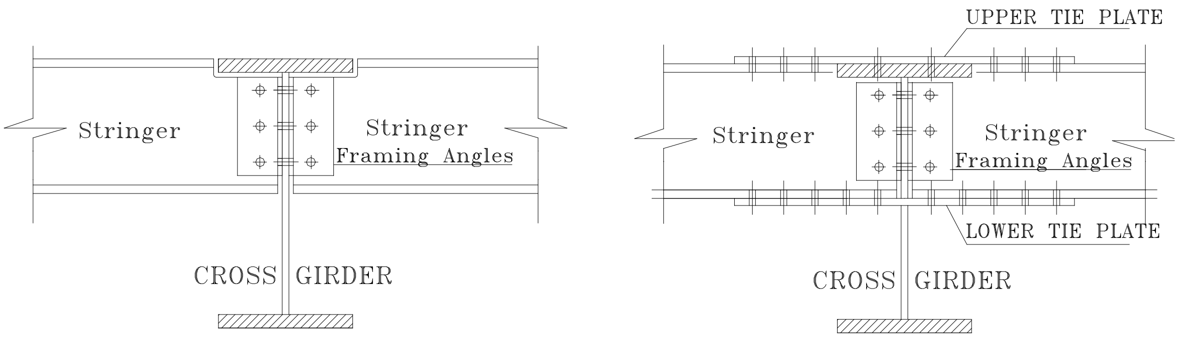
$$q_{all} = 0.35\ F_y = 1.26\ t/cm^2$$

The effective area in resisting shear of **rolled shapes** shall be taken as the **full height** of the section times the web thickness while for fabricated shapes it shall be taken as the web height times the web thickness.

4.3.2) DESIGN OF STRINGER CROSS SECTION

Stringers are usually designed as beams simply supported on the cross girders. The maximum straining actions for design are computed from the load positions and load combinations producing the maximum effect on the member considered. The maximum bending stress in the flanges or the maximum shear stress in the web usually governs the cross section size. The stringers are usually connected at their ends to the cross girder by two framing angles which are designed to transmit the maximum end reaction of the stringer to the cross girder, Fig. 4.4a.

Stringers may also be designed as continuous beams. In this case the connection between the stringer and the cross girder is designed to carry also the negative moment at the stringer supports, Fig. 4.4b.



a) Simple Stringer **b) Continuous Stringer**

Fig. 4.4 Connection between Stringer and Cross Girder

In addition to the effect of vertical loads, stringers in open railway bridge floors should be designed to carry the effect of the horizontal loads caused by the lateral shock of the running wheels, see section 2.2 (f). This lateral load is transmitted from the rails to the sleepers and then to the upper flange of the stringer. This effect causes double bending of the stringer cross section. Alternatively, a system of horizontal bracing, called lateral shock bracing, can be arranged between the stringers upper flanges to reduce the effect of lateral loads, Fig. 4.5.

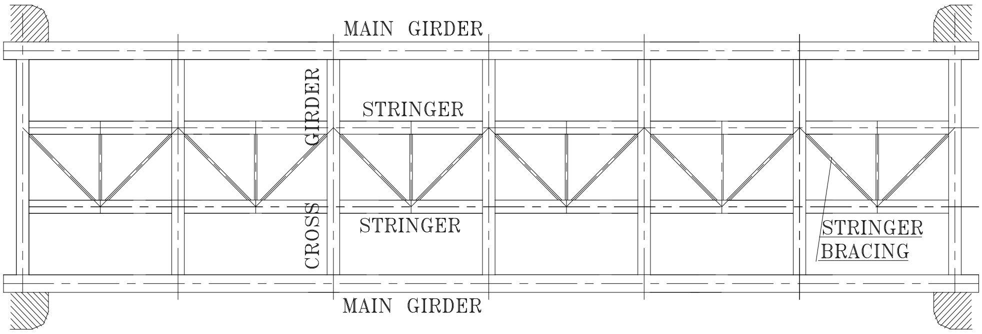


Fig. 4.5 Stringer (Lateral Shock) Bracings

4.3.3) DESIGN OF CROSS GIRDER CROSS SECTION

Similarly, cross girders are usually designed as beams simply supported on the main girders. The maximum bending stress in the flanges or the maximum shear stress in the web usually governs the cross section size. The cross girders are usually connected at their ends to the main girder by two framing angles which are designed to transmit the maximum end reaction of the cross girder to the main girder.

Cross girders of open railway bridge floors are designed to carry the effect of the horizontal loads caused by the longitudinal braking forces. This lateral load is transmitted from the rails to the sleepers and then to the upper flange of the stringers and the cross girders. This effect causes double bending of the cross girder section. Alternatively, a system of horizontal bracing, called braking force bracing, can be arranged between the stringers and the cross girders to eliminate the effect of longitudinal loads, see Fig. 4.6

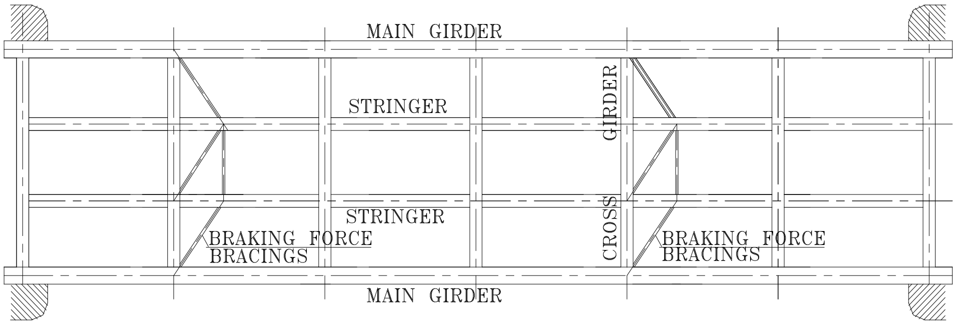
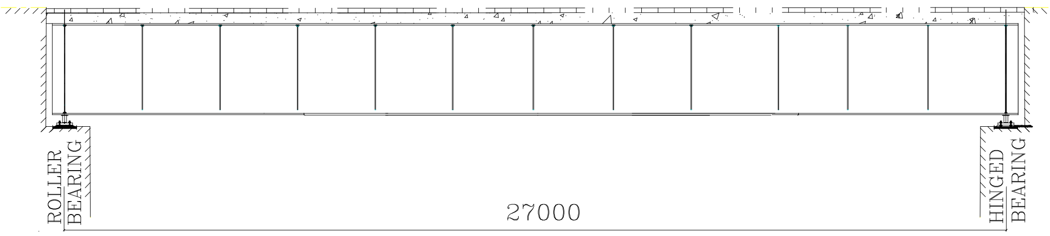


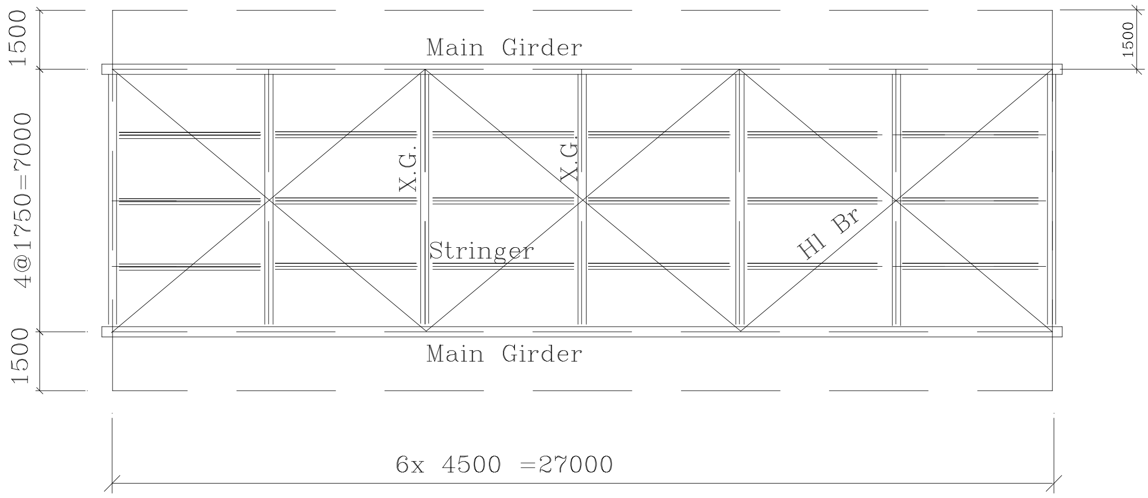
Fig. 4.6 Cross Girder (Braking Force) Bracings

4.4 DESIGN EXAMPLES

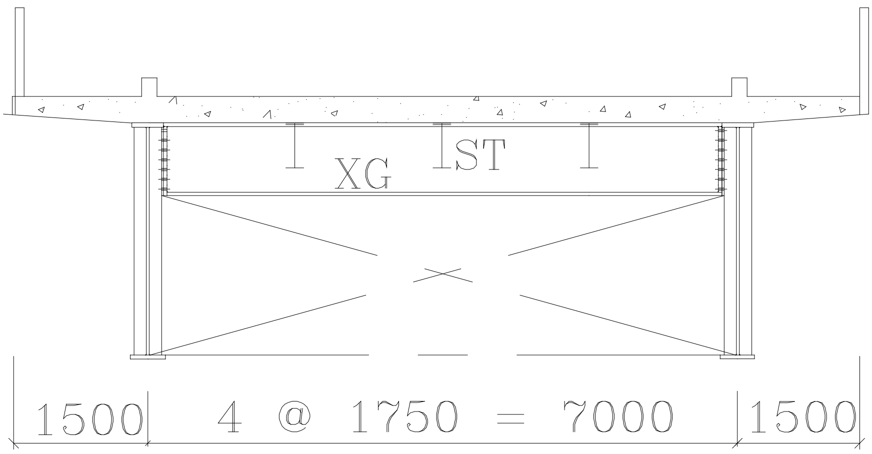
4.4.1) EXAMPLE 1: ROADWAY BRIDGE FLOOR



BRIDGE ELEVATION



BRIDGE FLOOR PLAN



BRIDGE CROSS SECTION

4.4.1.1) STRINGER

Structural System: Beam supported on cross girders,
Span = 4.50 m, Spacing = 1.75 m.

1) Straining Actions:

1.1) Dead Load:

- 22 cm Deck Slab = $0.22 \times 2.5 = 0.55$ t/m^2
- 5 cm Asphalt = $0.05 \times 2.0 = 0.10$ t/m^2

Total D.L = 0.65 t/m^2

Own wt of stringer (assumed) = 0.10 t/m$^/$
uniform load on stringer = $0.65 \times 1.75 + 0.1 = 1.238$ t/m$^/$
Dead Load Actions: $\mathbf{Q_{DL} = 1.238 \times (4.5) / 2 = 2.784}$ **t**

$$\mathbf{M_{DL} = 1.238 \times (4.5)^2 / 8 = 3.132 \text{ mt}}$$

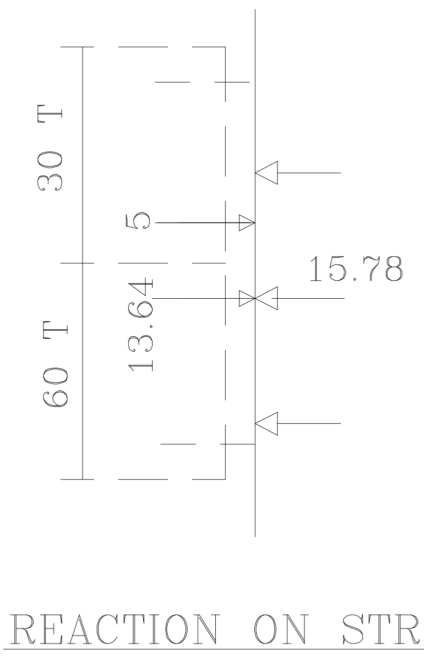
1.2) Live Load & Impact:

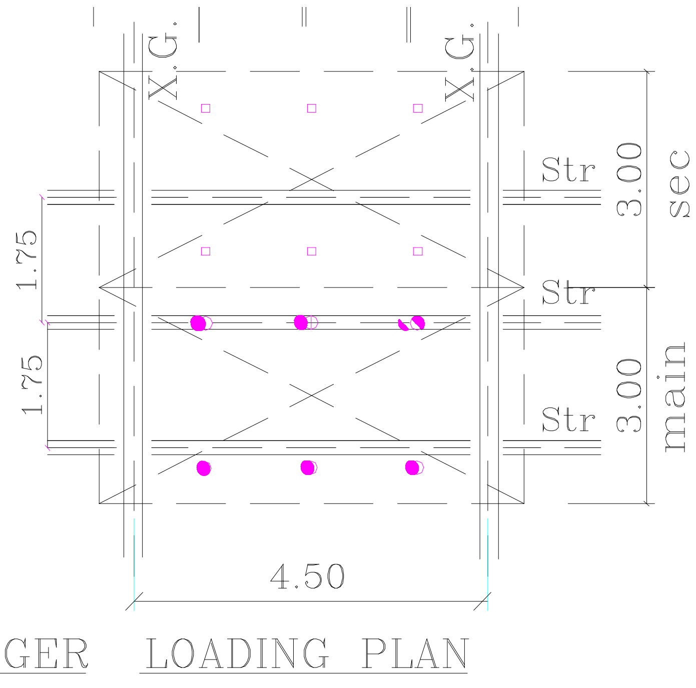
- **Impact factor I = 0.4 - 0.008 * L = 0.4-0.008*4.5= 0.364**

(L = Loaded Length of main traffic lane = 4.5m)
Impact is applied to Main Lane Loads only

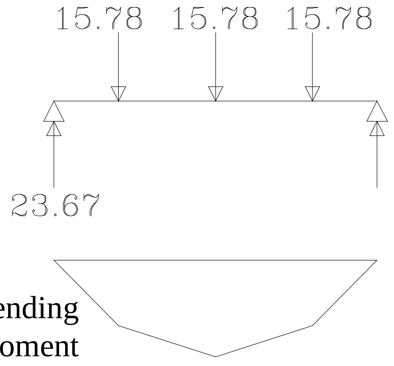
- **Maximum LL Reaction on Intermediate Stringer:**

Place 10^t on stringer and add effect of 5 ^{t} @ 1 m:
P = 10 * (1 + 0.364) + 5 × (1.75-1)/1.75 = 15.783 t





Loads position for Max Moment:



Note:

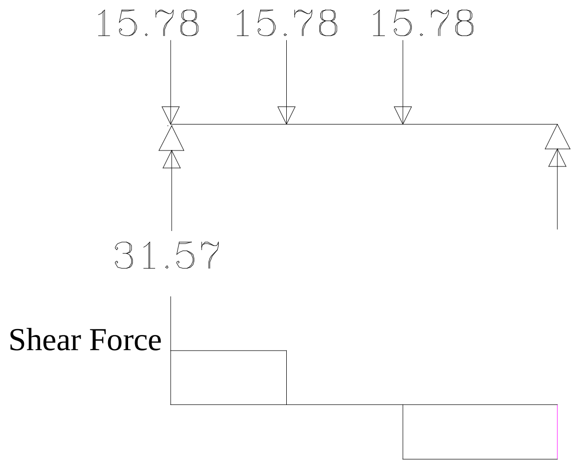
For L_{st} < 2.6 m: M_{max} occurs at the middle section with one LL reaction load acting in the middle.

For L_{st} > 3.4 m: M_{max} occurs at the middle section with all three loads acting as shown;

For 2.6 < L_{st} < 3.4 m: M_{max} occurs with two LL reaction loads placed such that the stringer centerline bisects the distance between the resultant and one load.

$$\mathbf{M_{LL\ \&\ I} = 23.674 \times 2.25 - 15.783 \times 1.5 = 29.592\ mt}$$

- Loads position for Max Shear:



Notes:
For L > 3.0 m: Q_{max} occurs at support with two loads acting on span
For L > 3.0 m: Q_{max} occurs at support with all three loads acting as shown:

$$\mathbf{Q_{LL\&I} = 15.783 + 15.783 \times (3/4.5) + 15.783 \times (1.5/4.5) = 31.566\ t}$$

1.3) <u>Design Straining Actions:</u>

<u>The total design moment on an intermediate stringer is:</u>

at middle section: $\mathbf{M_{design}}$ = 3.132 + 29.592 = 32.724 mt

<u>The total design shear on an intermediate stringer is:</u>

At support: $\mathbf{Q_{design}}$ = 2.784 + 31.566 = 34.35 t

2) <u>Design of Cross Section:</u>

2.1) <u>Case of Simple Stringer:</u>

<u>Straining Actions</u>: ***M_x = 32.724 m.t. (Maximum near middle)***
Q_y = 34.504 t (Maximum at support)

Design for Bending then check shear.

Section is **compact** w.r. to both **local buckling** requirements (being a rolled section), and **lateral torsional** buckling requirements (**compression flange supported by deck slab**); i.e.

$$F_{bx} = 0.64\ F_y = 2.304\ t/cm^2$$

Req. $Z_x = M_x / F_{bx}$ 32.72 x 100 / 2.304 = 1420 cm^3 --------- use **<u>IPE 450</u>**

Bending Stress: f_{bx} = 32.72 x 100 / 1500 = 2.18 t/cm^2 < 2.304 t/cm^2 OK

Check of Fatigue: Actual Stress Range = f_{sr} =(0.5 x 29.592)x100/1500
= **<u>0.9984</u>** t/cm^2
< Allowable Stress Range = F_{sr} = **<u>1.26</u>** t/cm^2
(Assuming Class B detail under $2x10^6$ cycles (Case 1.2 of Group 1 ECP)

Check Shear: $q_y = Q / A_{w\ net}$ = 34.504 / (0.85 x 45 x 0.94) = 0.95 t/cm^2
< 1.26 t/cm^2 OK

2.2) <u>Case of Continuous Stringer:</u>

<u>i)</u> <u>Section at mid span</u>:

$$\boldsymbol{M_x = 0.80\ x32.72 = 26.20\ m.t.,}$$

Section is compact (see above): Req. $Z_x = M_x / F_{bx}$ = 26.20 x 100 / 2.304
= 1137 cm^3

Use <u>IPE 400</u>

Check is similar to case 2.1 above.

<u>ii)</u> ***<u>Section at support</u>:***

$$Mx = 0.75 \times 32.72 = 24.54 \text{ m.t.}$$

Compression flange (being at the bottom) is **laterally unsupported**, therefore the section is assumed ***non-compact*** for simplicity. (Usually $L_u > L_{u1}, L_{u2}$) i.e., $F_{bx} = 0.583\, F_y = 2.10$ t/cm^2.

Use <u>IPE 450:</u>

$$f_{bx} = M_x / Z_{x\,net} = 24.54 \times 100 / (0.85 \times 1500) = 1.925 \text{ t/cm}^2 < 2.10 \text{ t/cm}^2 \quad \text{OK}$$

(Note: Net section properties were used to account for the moment bolted connection)

Check Shear:

$$q = Q / A_{w\,net} = 34.504 / (0.85 \times 45 \times 0.94) = 0.95 \text{ t/cm}^2 < 1.26 \text{ t/cm}^2 \quad \text{OK}$$

Equivalent Stresses due to combined shear and bending:

$$f_e = \sqrt{f^2 + 3q^2} \leq 1.1\ F_{all} = 2.436 \text{ t/cm}^2$$
> 1.1 x 2.1 = 2.31 N.G., Use **<u>IPE 500</u>**.

<u>4.4.1.2) CROSS GIRDER</u>

<u>Structural System:</u> Beam supported on main girders
Span = 7 m, Spacing = 4.5 m

1) Straining Actions:

1.1) <u>Dead Load Effect</u>:

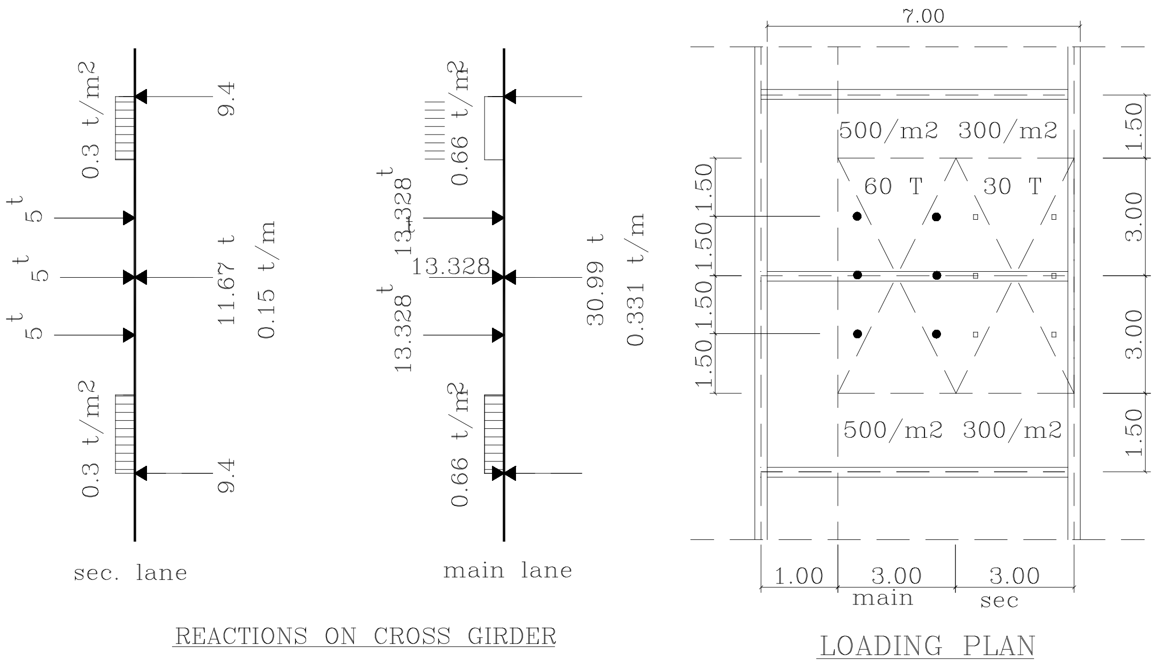
Concentrated reaction from stringers = 2 × 2.784 = 5.568 t
Own weight of Cross Girder (assumed) = 0.3 t/m/

Q_{DL} = 3 × 5.568/2 + 0.3 × 7 / 2 = 9.402 t
M_{DL} = 11.70 × 3.5 – 5.568 ×1.75 - 0.3 × $(3.5)^2$/2 = 21.326 mt

1.2) Live Load & Impact Effect:

Impact **I = 0.4 - 0.008 L = 0.4 × 0.008 (2×4.5) = 0.328**
{L = larger of 2 L_{st} (directly loaded members)
or L_{xg} (indirectly loaded member)}
Impact is applied to Main Lane only

- **Max LL Reactions on Cross Girder:**

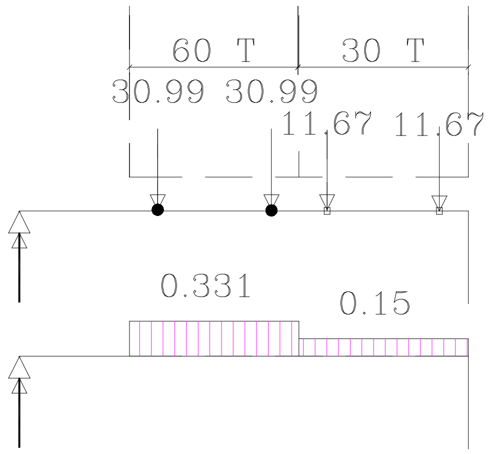


a) From Main Truck:
P_{60} = 10 x (1+0.328) + 2 x 13.28 x 3/4.5
= 30.987 t
b) From Main Lane Uniform Load:
w_{60} = 2 [0.5x1.328 x 0.75/4.5] = 0.331 t/m/
c) From Secondary Truck:
P_{30} = 5+ 2 x 5x 3/4.5 = 11.667 T
d) From Secondary Lane Uniform Load:
w_{30} = 2 [0.3x 1.5 x 0.75/4.5] = 0.15 t/m/

N.B.: Uniform load on Lane Fractions on both sides of trucks is to be neglected.

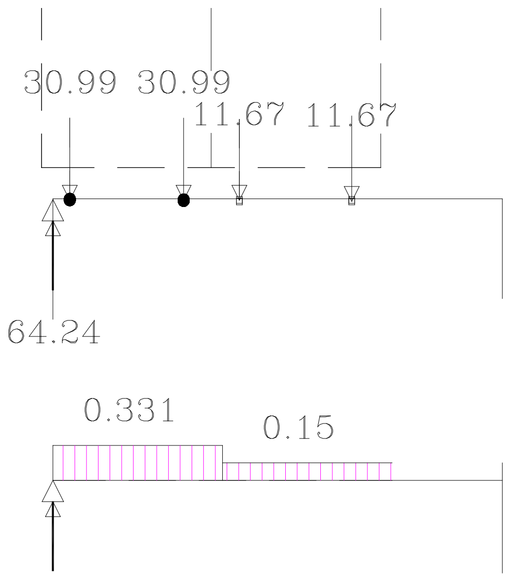
- **Loads position for Max Moment:**
  M_{max} occurs at the middle with loads placed as shown:
  $M_{LL\ \&\ I}$ = 96.88 mt

LOADS FOR MAX MOMENT



- **Loads position for Max Shear:**
  Q_{max} occurs at support with loads placed as shown:
  $Q_{LL\&I}$ = 64.24 t

LOADS FOR MAX SHEAR



1.3) <u>Design Straining Actions:</u>

<u>The total design moment on an intermediate cross girder is:</u>

At the middle section: M_{design} = 21.326 + 96.88 = 118.206 mt

<u>And the total design shear on an intermediate cross girder is:</u>

At the support: Q_{design} = 9.402 + 64.24 = 73.74 t

2) ***Design of Cross Section:***

Straining Actions:

Mx = 118.206 m.t. (Maximum near middle)
Qy = 73.74 t (Maximum at support)

Section is **compact** w.r. to both **local buckling** requirements (being a rolled section), and **lateral torsional buckling** requirements (**comp flange supported by deck slab**); i.e.

$$F_{bx} = 0.64\ F_y = 2.304\ t/cm^2$$

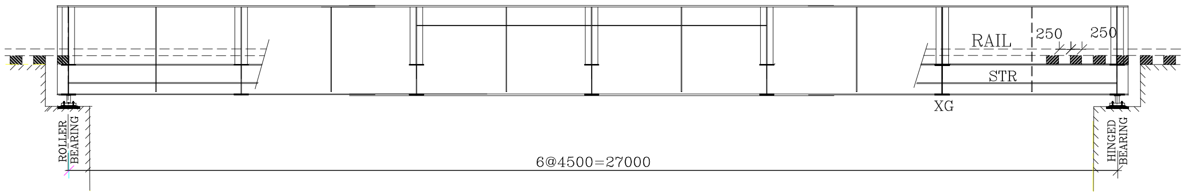
Req. $Z_x = M_x / F_{bx} = 118.206 \times 100 / 2.304 = 5121\ cm^3$ --------- use HEA 650

$f_{bx} = 118.206 \times 100 / 5470 = 2.157\ t/cm^2 < 2.304\ t/cm^2$ **OK**

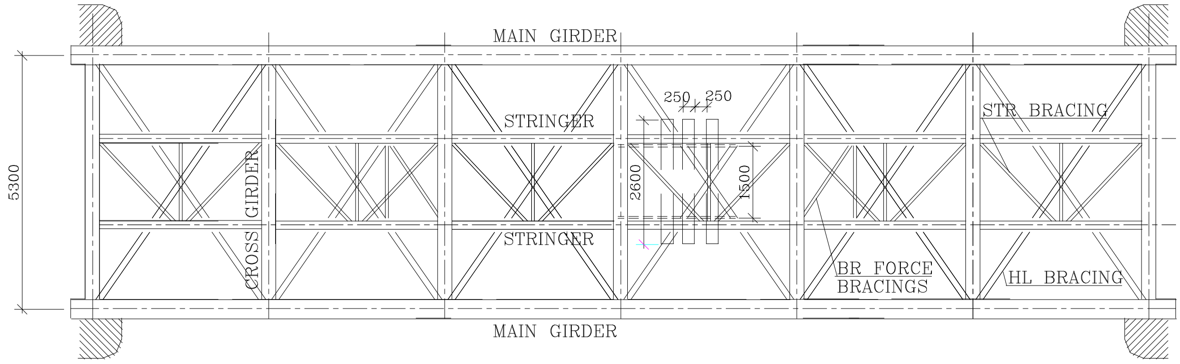
Check of Fatigue: Actual Stress Range = f_{sr} =(0.5 x 96.88)x100/5470
= **0.886** t/cm^2
< Allowable Stress Range = F_{sr} = **1.26** t/cm^2
(Assuming Class B detail under 2×10^6 cycles (Case 1.2 of Group 1 ECP)

Check Shear: $q = Q / A_{w\ net} = 73.74 / (0.85 \times 64 \times 1.35) = 1.00\ t/cm^2$
$< 1.26\ t/cm^2$

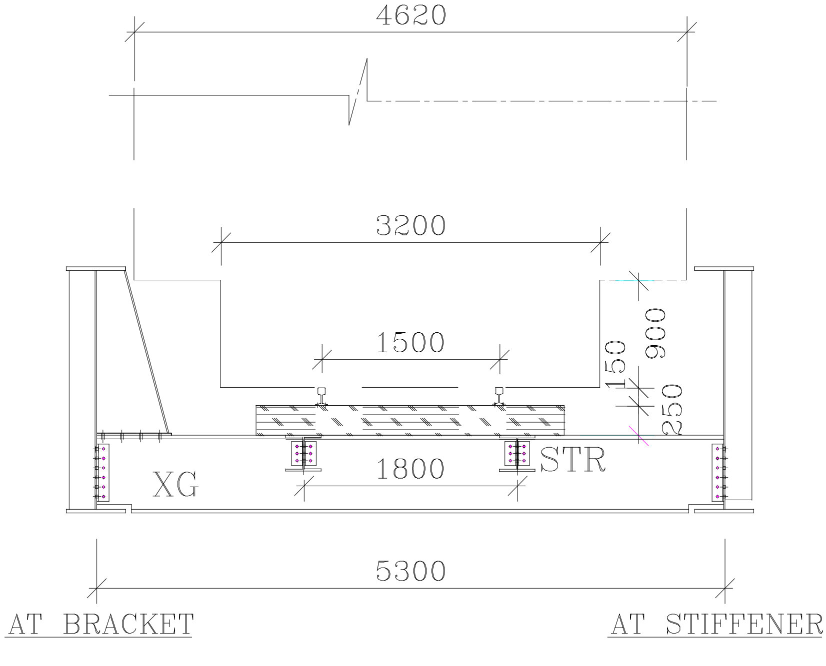
4.4.2) EXAMPLE 2: RAILWAY BRIDGE FLOOR



BRIDGE ELEVATION



BRIDGE FLOOR PLAN



BRIDGE CROSS SECTION

4.4.2.1) STRINGER

Structural System:

Beam supported on cross girders,
Span = 4.50 m, Spacing = 1.80 m.

1) Straining Actions:

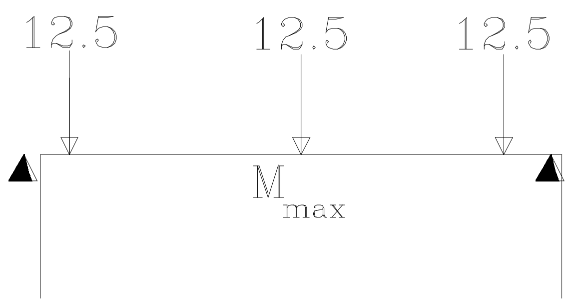
1.1) Dead Load:

Track (rails, sleepers, conn.) = 0.6 t/m/ of track
= 0.3 t/m/ of stringer

Own wt of stringer (assumed) = 0.15 t/m/
uniform load on stringer =0.3 + 0.15 = 0.45 t/m/
Dead Load Actions: $\mathbf{Q_{DL} = 0.45 \times (4.5) / 2 = 1.013\ t}$
$\mathbf{M_{DL} = 0.45 \times (4.5)^2 / 8 = 1.139\ mt}$

1.2) Live Load & Impact:

- Impact factor **I = 24/(24+ L) = 24/(24+4.5)= 0.842 (max 0.75)**
  (L = Loaded Length of track = 4.5 m)



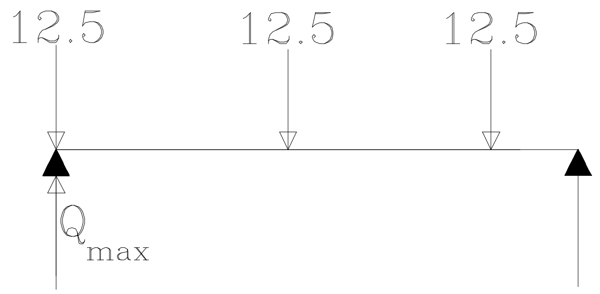
- Loads position for Max Moment:

For L < 3.4 m: M_{max} occurs with single wheel on stringer
For 3.4 < L < 4.4 m: M_{max} occurs with two wheels on stringer

For L > 4.4 m: M_{max} occurs with three wheels on stringer

$$\mathbf{M_{LL\ \&\ I} = ((3x12.5/2)x2.25-12.5x2)\ x(1+I) = 30.08\ mt}$$

- <u>Loads position for Max Shear:</u>



For L > 3.0 m: Q_{max} occurs at support with three loads acting as shown:

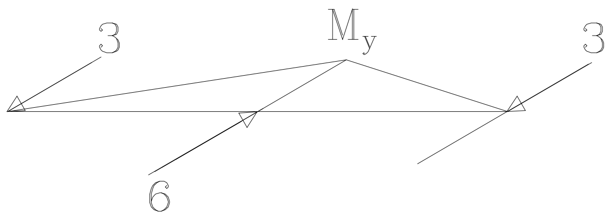
$$\mathbf{Q_{LL\&I} = (12.5+12.5x2.5/4.5+12.5x.5/4.5) \ x \ (1+I) = 36.46 \ t}$$

1.3) <u>Lateral Shock Effect:</u>

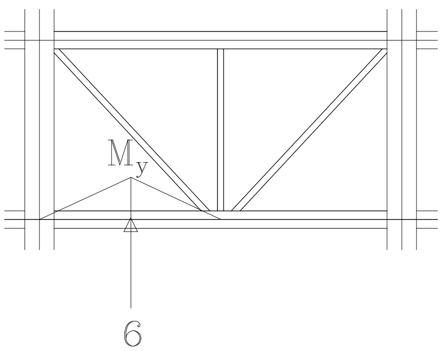
a) If No Stringer Bracing is used:

$\mathbf{M_y = 6 \ x \ 4 \ /4 = 6 \ m.t.}$ **(at middle)**

(Corresponding M_x = 25.51 m.t.)



b) If Stringer Bracing is used:



$\mathbf{M_y = 6 \ x \ 2 \ / \ 4 = 3 \ m.t.}$ **(at quarter point)**

Corresponding M_x = 0.9 + 21.875 = 22.775 m.t.)

1.4) <u>Design Straining Actions:</u>

The total <u>design moment</u> on an intermediate stringer is:

a) If No Stringer Bracing is Used: Critical Section at Middle

$\mathbf{M_x = 1.139 + 30.08 = 31.219}$ **mt**

My = 6.75 mt

b) If Stringer Bracing is Used: Critical Section at Quarter Point

$\mathbf{M_x = 25.38}$ **mt**

My = 3.375 mt

And the total <u>design shear</u> on an intermediate stringer is:

At support: $\mathbf{Q_y = 1.013 + 36.46 = 37.473}$ **t**

1.5) <u>Design of Cross Section:</u>

1.5.1) <u>Case of Simple Stringer without Lateral Shock (stringer) Bracing:</u>

<u>Straining Actions</u>: $M_x = 31.94$ ***m.t.,*** $M_y = 6.75$ ***m.t.***

(carried by top flange only)

$Q_y = 37.47$ ***t***

Section is **compact** w.r. to **local buckling** requirements (being a **rolled section**), but **not compact** w.r. to **lateral torsional buckling** requirements (**comp flange unsupported for L_{un}= 4.5m**); i.e., Fbx = 0.583 F_y = 2.10 t/cm^2

$$\mathbf{F_{by} = 0.72\ F_y = 2.592\ t/cm^2}\ \textbf{(Minor axis bending)}$$

<u>Section HEB 400</u>

f_{bx} = 31.94 x 100 / 2880 = 1.1 t/cm^2 < 2.10 t/cm^2 OK

f_{by} = 6.75x 100 / (721/2) = 1.87 t/cm^2 < 2.592 t/cm^2 OK

Combined Bending: $f_{bx} / F_{bx} + f_{by} / F_{by}$ = 1.1/2.10 + 1.87/2.592= 1.24

< 1 x **1.2**

(Factor **1.2** accounts for additional stress of Case II loads)

<u>Unsafe → then use HEB 450</u>

Check of Fatigue:

From Mx : Actual Stress Range = f_{sr} =(30.08)x100/3551 = **<u>0.847</u>** t/cm^2

From My : Actual Stress Range = f_{sr} =(6.75)x100/(781/2) = **<u>1.728</u>** t/cm^2

Total Stress Range = 0.847 + 1.728 = 2.575 t/cm2

> Allowable Stress Range = F_{sr} = 1.2x 1.26 = **<u>1.512</u>** t/cm^2

(Assuming Class B detail under 2x10^6 cycles (Case 1.2 of Group 1 ECP)

<u>Fatigue Check is UNSAFE: increase cross section</u>

Check Shear: $q = Q / A_{w\ net} = 37.47 / (0.85 \times 45 \times 1.4) = 0.7\ t/cm^2$
$< 1.26\ t/cm^2$

1.5.2) <u>Case of Simple Stringer with Lateral Shock (stringer) Bracing:</u>

<u>Straining Actions</u>: ***Mx = 28.25 m.t., M_y = 3.375 m.t. (at quarter point)***
Qy = 37.47t at support & Qy = 22.325t at quarter point

Section is **compact** w.r. to both **local buckling** requirements (being a **rolled section**), and w.r. to **lateral torsional buckling** requirements (**comp. flange supported at L_{un}= 2.25 m by stringer bracing**);
i.e., $F_{bx} = 0.64\ F_y = 2.304\ t/cm^2$

$$\mathbf{F_{by} = 0.72\ F_y = 2.592\ t/cm^2}\ \text{(Minor axis bending)}$$

<u>Section HEA 360</u> $f_{bx} = 28.25 \times 100 / 1890 = 1.495\ t/cm^2 < 2.304\ t/cm^2$ OK

$f_{by} = 3.375 \times 100 / (526/2) = 1.283\ t/cm^2 < 2.592\ t/cm^2$ OK

Combined Bending: $f_{bx} / F_{bx} + f_{by} / F_{by} = 1.495/2.304 + 1.283/2.592$
$= 1.144$
$< 1 \times \mathbf{1.2}$
(Factor **1.2** accounts for additional stress of Case II loads)

Check of Fatigue is similar to case above.

Check Shear at support: $q = Q / A_{w\ net} = 37.47 / (0.85 \times 35 \times 1.0)$
$= 1.259\ t/cm^2$
$< 1.26\ t/cm^2$ OK

<u>1.5.3) Case of Continuous Stringer without Lateral Shock Bracing:</u>

i) <u>Section near mid span:</u> Mx = 0.8 x 31.94= 25.552 m.t., M_y = 6.75 m.t.
(not affected by continuity)

Section is **compact** w.r. to **local buckling** requirements (being a **rolled section**), but **not compact** w r to **lateral torsional buckling** requirements (**comp flange unsupported for L_{un}= 4.5 m**); i.e., $F_{bx} = 0.583\ F_y = 2.10\ t/cm^2$

$$\mathbf{F_{by} = 0.72\ F_y = 2.592\ t/cm^2}\ \text{(Minor axis bending)}$$

<u>Section HEB 400</u>

$f_{bx} = 25.552 \times 100 / 2880 = 0.887\ t/cm^2 < 2.10\ t/cm^2$ OK

$f_{by} = 6.75 \times 100 / (721/2) = 1.872\ t/cm^2 < 2.592\ t/cm^2$ OK

Combined Bending: $f_{bx} / F_{bx} + f_{by} / F_{by} = 0.887/2.10 + 1.872/2.592 = 1.145 < 1 \times \mathbf{1.2}$

(Factor **1.2** accounts for additional stress of Case II loads)

ii) Section at support ***: $M_x = 0.75 \times 31.94 = 23.955$ m.t., $M_y = 0$, $Q_y = 37.47$ t***

Compression flange (being at the bottom) is laterally unsupported, therefore the section is *non-compact;*

Use **HEB360**: for $L_u \le \dfrac{20 b_f}{\sqrt{F_y}} = 316$ **cm,**

therefore: $F_{bx} = 0.583\ F_y = 2.10\ t/cm^2$

$f_{bx} = M_x / Z_{x\,net} = 23.955 \times 100 / (0.85 \times 2400) = 1.174\ t/cm^2 < 2.10\ t/cm^2$ **OK**

$q = Q / A_{w\,net} = 37.47 / (0.85 \times 36 \times 1.25) = 0.980\ t/cm^2 < 1.26\ t/cm^2$ OK

Equivalent Stresses: $f_e = \sqrt{f^2 + 3q^2} \le 1.1\ F_{all} = 1.993\ t/cm^2 < 1.1 \times 2.1 = 2.31\ t/cm^2$

1.5.4) Continuous Stringer with Lateral Shock (stringer) Bracing:

i) Section at mid span: ***Mx = 0.8 x 28.25= 22.60 m.t.***
M_y = 3.375 m.t (at quarter point)

Section is **compact** w.r. to both **local buckling requirements** (being a **rolled section**), and **lateral torsional buckling** requirements (**comp. flange supported at L_{un}=2 m by stringer bracing**); i.e., $F_{bx} = 0.64\ F_y = 2.304\ t/cm^2$

$F_{by} = 0.72\ Fy = 2.592\ t/cm^2$ **(Minor axis bending)**

Section HEB 320

$f_{bx} = 22.60 \times 100 / 1930 = 1.171\ t/cm^2 < 2.304\ t/cm^2$ OK
$f_{by} = 3.375 \times 100 / (616/2) = 1.096\ t/cm^2 < 2.592\ t/cm^2$ OK

Combined Bending: $f_{bx} / F_{bx} + f_{by} / F_{by} = 1.171/2.304 + 1.096/2.592 = 0.931 < 1.2$

(Factor **1.2** accounts for additional stress of Case II loads)

Check Shear: $q = Q / A_{w\ net} = 37.47 / (0.85 \times 32 \times 1.15) = 1.198\ t/cm^2$
$< 1.26\ t/cm^2$

ii) Section at support : See ((1.3) ii) above.

4.4.2.2) CROSS GIRDER

Structural System:

Beam supported on main girders
Span = 5.30 m, Spacing = 4.50 m

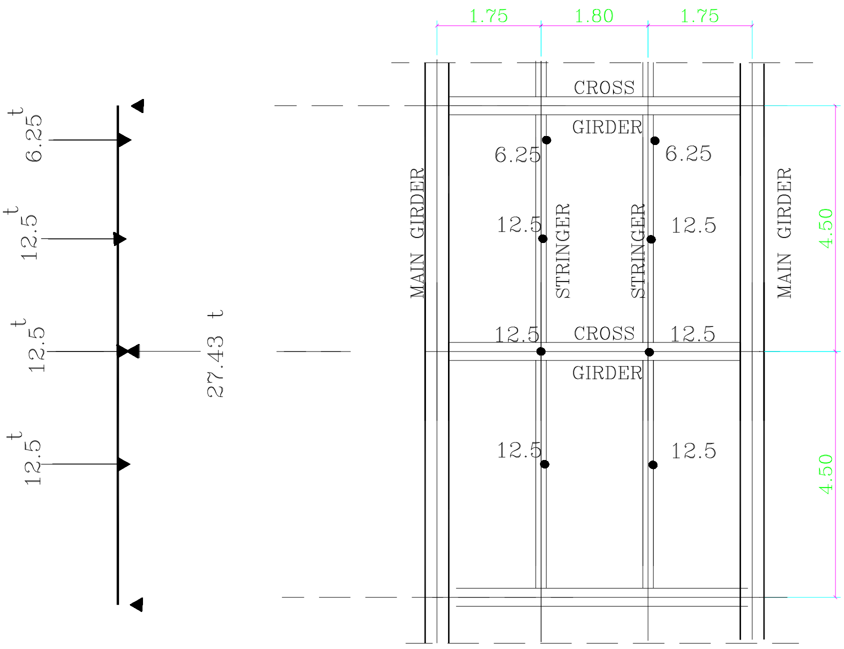
2.1) Dead Load Effect:

Concentrated reaction from stringers = 2 × 1.013 = 2.026 t
Own weight of X.G. (assumed) = 0.3 t/m/
$\mathbf{Q_{DL} = 2.026 + 0.3 \times 5.3 / 2 = 2.821\ t}$
$\mathbf{M_{DL} = 2.821 \times 5.3/2 - 2.026 \times 0.9 - 0.3 \times (2.65)^2/2 = 4.6\ mt}$

2.2) Live Load & Impact Effect:

- Impact factor **I = 24/(24+ L) = 24/(24+ 9)=0.727**

  (L = Loaded Length of tracks= 2 x 4.5 = 9 m)

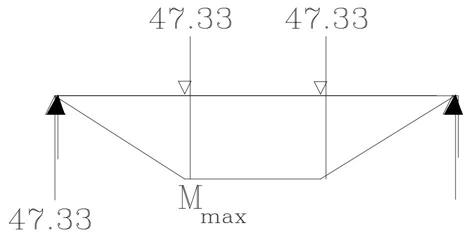


REACTIONS ON CROSS GIRDER

LOADING PLAN

- **Reactions on Cross Girder:**

P = {12.5 + 2 x 12.5 x 2.5 / 4.5 + 6.25x0.75/4.5}x (1+I)
= {12.5 + 2 x 12.5 x 2.5 / 4.5 + 6.25x0.75/4.5}x (1.727)
= 47.37t



M_{max} occurs at the stringer location

$$\mathbf{M_{LL\ \&\ I} = 47.37 \times 1.75 = 82.90\ mt}$$

Q_{max} occurs at support:

$$\mathbf{Q_{LL\&I} = 47.37\ t}$$

So the total design moment on an intermediate XG is:

$$\mathbf{M_{design} = 4.6 + 82.90 = 87.5\ mt}$$

And the total design shear on an intermediate XG is:

$$\mathbf{Q_{design} = 2.821 + 47.33 = 50.191\ t}$$

2.3) <u>Braking Force Effect</u>:

a) If Braking Force Bracing is used:
Braking force is carried by the braking force bracing without any bending in the Cross Girders. (i.e. My=0)

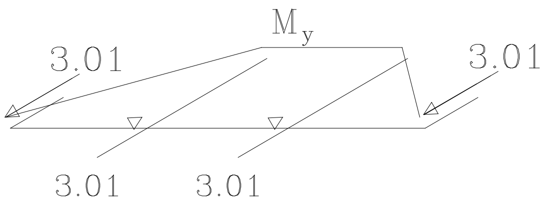
b) If No Braking Force Bracing is used:

Total Braking Force on the bridge:

B = Sum of train loads on bridge / 7 = 295 / 7 = 42.1 t

Braking force is equally divided between cross girders:

Braking force/ XG = 42.1 /no of XGs = 42.1/7= 6.02 t



M_y = (6.02 / 2) x 1.75 = 5.26 mt

2.4) <u>Design of Cross Section:</u>

<u>2.4.1) Without Braking Force Bracing:</u>

***<u>Straining Actions</u>*: M_x = 87.50 m.t., M_y = 5.26 m.t. (at stringer location)**
Q_y = 50.191 t (at support)

Section is **compact** w.r. to **local buckling** requirements (being a **rolled section**), but **not compact** w.r. to **lateral torsional buckling** requirements (**comp. flange unsupported for L_{un}=5.3 m**); i.e., F_{bx} = 0.583 F_y = 2.10 t/cm^2

F_{by} = 0.72 F_y = 2.592 t/cm^2 (Minor axis bending)

Section <u>HEB 600</u>:

f_{bx} = 87.50 x 100 / 5700 = 1.535 t/cm^2 < 2.10 t/cm^2 OK
f_{by} = 5.26 x 100 / (902/2) = 1.166 t/cm^2 < 2.592 t/cm^2 **OK**

Combined Bending: f_{bx} / F_{bx} + f_{by} / F_{by} = 1.535/2.10 + 1.166/2.592 = 1.074
<**1.2** OK

Check Fatigue as before.

Check Shear: q = Q / $A_{w\ net}$ = 50.192 / (0.85 x 79 x 1.50) = 0.498 t/cm^2
< 1.26 t/cm^2 OK

<u>2.4.2) With Braking Force Bracing:</u>

M_y is carried by axial forces in the braking force bracing with the XG subjected to M_x only:

Req. Z_x = M_x / F_{bx} = 7.50 x 100 / 2.304 = 3798 cm^3 --------- use <u>HEB 550</u>

Check Shear: q = Q / $A_{w\ net}$ = 50.192 / (0.85 x 55 x 1.5) = 0.716 t/cm^2
< 1.26 t/cm^2

4.4.3) CONNECTIONS OF BRIDGE FLOOR BEAMS

4.4.3.1 Calculations of Bolt Resistance:

High strength bolts of Grade 10.9 or 8.8 are normally used in bridge constructions. Connections may be designed as **Bearing Type** (easier in execution) or **Friction Type** (when slip is not allowed).

For Bearing Type Connections:

Bolt Resistance = Smaller of R_{shear} and $R_{bearing}$;

R_{shear} = n x (Bolt area x Allowable bolt shear stress) = n x (π d^2 /4) x 2

$R_{bearing}$ = Bolt diameter x Allowable bearing stress x t_{min} = d x (0.8 F_{ult}) x t_{min}

(n = no. of shear planes, d= bolt diameter, Edge distance $\geq$ 2 d, F_{ult} = 5.2 t/cm^2)

For Friction Type Connections: Bolt Resistance = n x P_s

Bolt Diameter	Bearing Type Connections			Friction Type Connections	
	$R_{S.Shear}$	$R_{bearing}$	$R_{D.\ Shear}$	Bolt 8.8	Bolt 10.9
M20	6.28	8.32 t_{min}	12.56	3.37	4.82
M22	7.60	9.15 t_{min}	15.20	4.17	5.96
M24	9.04	9.98 t_{min}	18.08	4.85	6.94

4.4.3.2 Design of Connection between Stringer and Cross Girder:

Railway Bridge Floor Data:

Stringer: Shear Force = 37.47 ^{t}, -ve Moment = 23.955 m.t.,
section is HEB 360.

Cross Girder: HEB 600

2.1) Case of Simple Stringer:

Connection is designed for the max shear of stringer using framing angles as shown:.

Using M20 HSB Grade 10.9 (Bearing Type):

a) Bolts between stringer web (t_w = 1.25 cm) and angle legs:

Double shear bolts $R_b = 8.32 \times 1.25 = 10.4^{t} = R_{least}$
Number of bolts = Q / R_{least} = 37.47 / 10.4 = 3.3 bolts

Fatigue Considerations: Case 27.1 of Group 3 of ECP: for Class C;
Allowable stress range = 0.91 t/cm2
$R_{sh} = \mathbf{2} \times 2.45 \times 0.91 = 4.46$ ton $= R_{sr}$

Number of bolts = Q_{sr} / R_{sr} = 36.46 / 4.46 = 8.17 too many
Either use bolts with larger diameter or use Friction Type Joint:
i) Use M24 bolts: $R_{sh} = 2 \times 3.53 \times 0.91 = 6.43$ ton $= R_{sr}$
Number of bolts = Q_{sr} / R_{least} = **36.46** / 6.43 = 5.8
Use 6 bolts M24 (Bearing Type)
ii) Friction Type Joint: $R_{sh} = 2 \times 4.82 = 9.64$ ton $= R_{least}$
Number of bolts = Q_t / R_{least} = **37.47** / 9.64 = 3.88
Use 4 bolts M24 (Friction Type)

b) Bolts between angle legs and cross girder web:

Single shear bolts on two sides: Fatigue governs the design
Number of bolts = Q_{sr} / R_{sr} = 36.46 / (3.53x0.91) = 11.35

Use 12 bolts M24 (6 bolts each side)

(Alternative Using Friction Type Bolts: No. of Bolts = 37.47/4.82= 7.77,
i.e.; 4 bolts M20 (Friction Type) Each Side)

2.2) Case of Continuous Stringer:

Continuity is achieved by using top and bottom plates designed to transmit the flange force:

$$C = T = M_{-ve} / h_{str} = (0.75 \times 30.08) / 0.36 = 62.667 \text{ t}$$

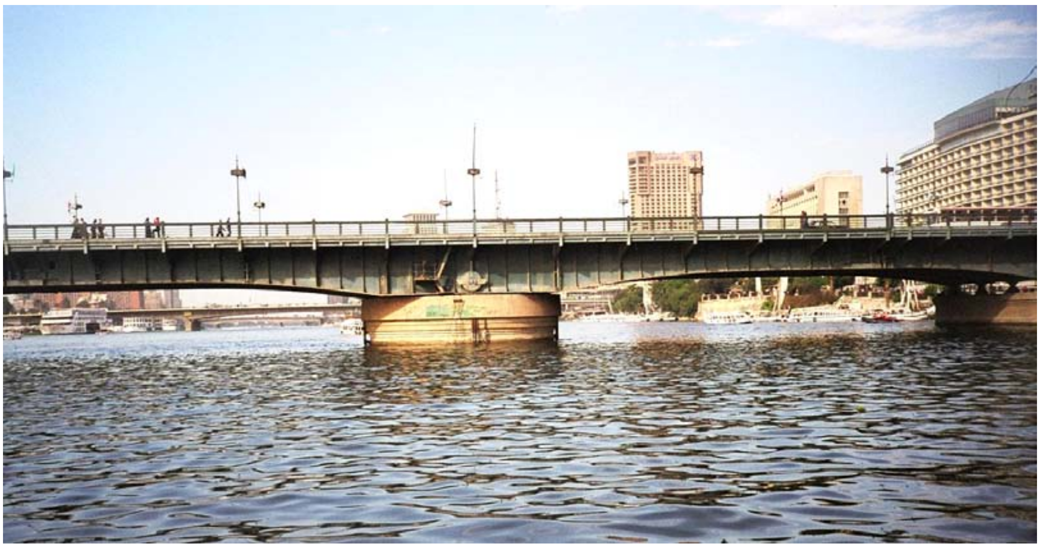
Number of bolts = 62.667 / 6.43 = 9.746
use 10 bolts M24 (Bearing Type) each side.

(Alternative Using Friction Type Bolts: No. of Bolts = 66.54/4.82=13.8,
i.e.; 14 bolts M20 (Friction Type) Each Side)

Compute Plate Thickness from:

$$(30 - 4 \times 2.6) \times t \times 2.1 = 66.54 \quad \text{which gives } t_{pl} = 1.617 \text{ cm}$$

Use t_{pl} = 1.80 cm



CHAPTER 5

PLATE GIRDER BRIDGES

CHAPTER 5

PLATE GIRDER BRIDGES

5.1 INTRODUCTION

5.1.1 General

In section 1.4, bridges were classified according to the structural systems of the main girder in the longitudinal direction into several types such as beam and frame bridges, arch bridges, cable stayed bridges, and suspension bridges. The cross section of the main girder used in any of these bridge types may be a solid web girder or a truss girder depending on the values of the design actions. For short and medium spans, solid web girders in the form of I-section or box section are usually used. These girders are usually fabricated from welded plates and thus are called "Plate Girders". Plate girders may be defined as structural members that resist loads primarily in bending and shear. Although shaped similarly to the commonly used hot-rolled steel I-beams, plate girders differ from them in that they are fabricated from plates, and sometimes angles, that are joined together to form I-shapes. They are characterized by thin webs, which are usually deeper than those of the deepest available rolled shapes. Such girders are capable of carrying greater loads over longer spans than is generally possible using standard rolled sections or compound girders. Plate girders may also be used as long-span floor girders in buildings, as crane girders in industrial structures, and as bridge girders in all types of bridges.

5.1.2 Cross Sections of Plate Girders

Several cross sections may be used for plate girders as shown in Fig. 5.1. Early plate girders were fabricated by riveting, Fig. 5.1(a). Their flanges consisted of two angles riveted to the web ends and cover plates riveted to the outstanding legs of the angles. Structural welding, which began to be widely used in the 1950s, has significantly simplified the fabrication of plate girders. Modern plate girders are normally fabricated by welding together two flange plates and a web plate as shown in Fig. 5.1(b). Although not commonly used, other variations are possible as shown in Fig. 5.1(c).

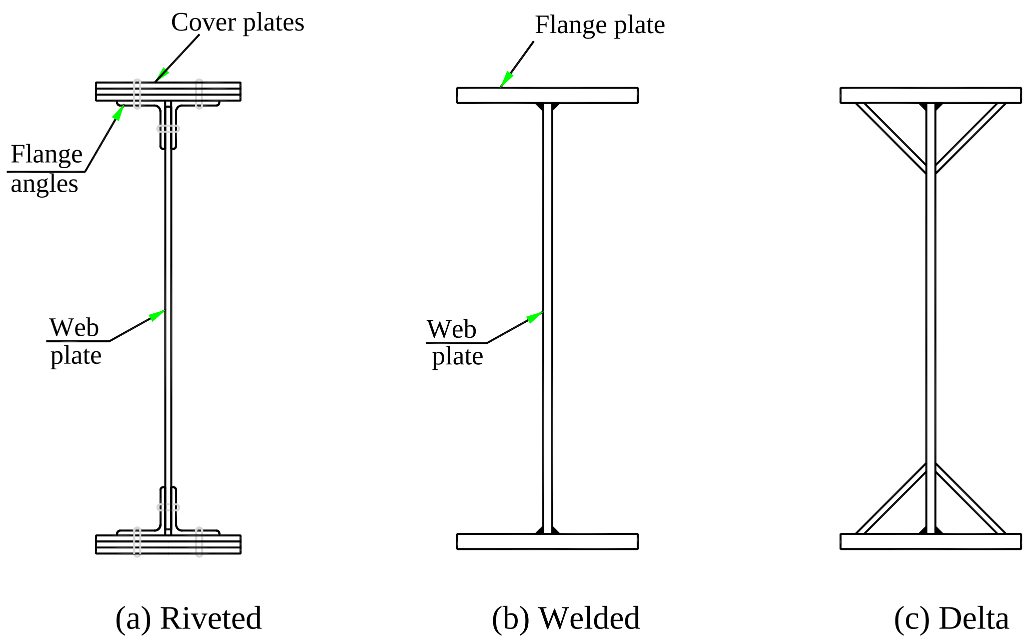
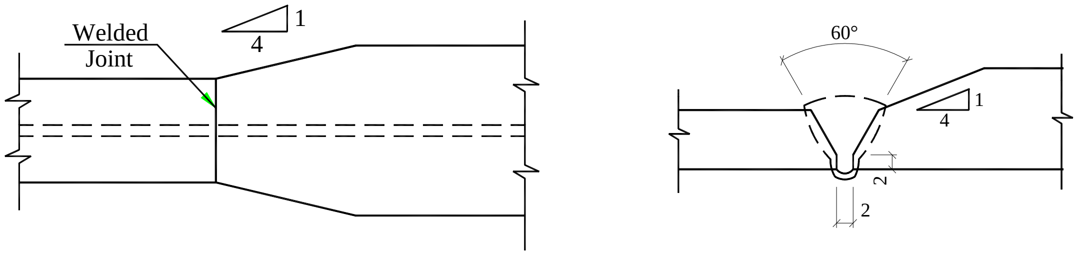


Fig. 5.1 Cross Sections of Plate Girders

Because a plate girder is fabricated from individual elements that constitute its flanges and web, a significant advantage offered by a plate girder is the freedom a designer can have in proportioning the flange and web plates to achieve maximum economy through more efficient arrangement of material than is possible with rolled beams. This freedom gives a considerable scope for variation of the cross-section in the longitudinal direction. For example, a designer can reduce the flange width or thickness in a zone of low applied moment as shown in Fig. 5.2. Equally, in a zone of high shear, the designer can thicken the web plate. Furthermore, the designer has the freedom to use different grades of steel for different parts of the girder. For example, higher-grade steel St. 52 might be used for zones of high applied moments while standard grade steel St. 37 would be used elsewhere. Also, "hybrid girders" with high strength steel in the flange plates and low strength steel in the web offer another possible means of more closely matching resistance to requirements. More unusual variations are adopted in special circumstances, e.g., girders with variable depth, see Fig. 5.3.



(a) Width (b) Thickness

Fig. 5.2 Transition of Flange Plate Width and Thickness

Fig. 5.3 Plate Girder Bridge with Variable Depth

5.2 GENERAL DESIGN CONSIDERATIONS

5.2.1 GIRDER DESIGN

Any cross-section of a plate girder is normally subjected to a combination of shear force and bending moment. The primary function of the top and bottom flange plates of the girder is to resist the axial compressive and tensile forces arising from the applied bending moment. The primary function of the web plate is to resist the applied shear force. Under static loading, bending and shear strength requirements will normally govern most plate girder design, with serviceability requirements such as deflection or vibration being less critical.

The first step in the design of plate girder section is to select the value of the web depth. For railway bridges, the girder depth will usually be in the range $L_o/12$ to $L_o/8$, where L_o is the length between points of zero moment. However, for plate girder roadway bridges the range may be extended to approximately $L_o/20$ for non-composite plate girders and to $L_o/25$ for composite plate girders.

Having selected the web plate depth, the effective flange area to resist the applied moment can be computed from the relation, see Fig. 5.4(b):

$$\mathbf{M = F_e \ A_e \ h_e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{(5.1)}$$

Where: F_e = allowable bending stress at flange centroid,
h_e = effective depth for flange,
A_e = equivalent flange area.

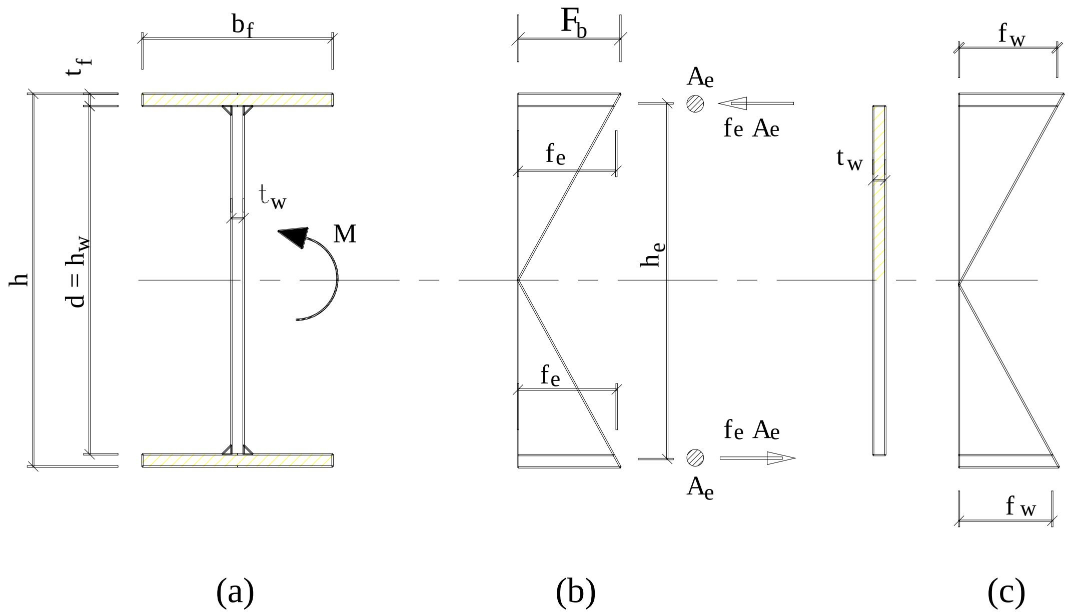


Fig. 5.4 Proportioning of Plate Girder Flanges

Flange Stress: According to ECP 2001, girders with laterally supported compression flanges can attain their full elastic strength under load, i.e., F_b = $0.64*F_y$ for compact sections and $F_b = 0.58 * F_y$ for non-compact sections. If the compression flange is not supported laterally, then appropriate reduction in the allowable bending stresses shall be applied to account for lateral torsional buckling as set in the Code.

The equivalent flange area A_e is made up of the actual area of one flange, plus the part of the web area that contributes in resisting the applied moment. The moment resistance M_w of the web can be defined by; Fig. 5.4 (c):

$$M_w = (0.5\ F_w)\ (0.5\ A_w)\ (2h_w/3) = F_w\ h_w\ A_w/6 \ldots\ldots\ldots\ldots\ldots\ldots (5.2)$$

where A_w = area of web and F_w = maximum bending stress for web. From the above equation it can be seen that one sixth of the total web area can be considered as effective in resisting moment M_w with lever arm h_w and stress F_w. Consequently, the area required for each flange will be:

$$A_f = A_e - A_w / 6 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.3)$$

Substituting for A_e from Eqn. 5.1 gives:

$$A_f = (M / F_b\ d) - A_w / 6 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.4)$$

5.2.2 OPTIMUM GIRDER DEPTH

An optimum value of the plate girder depth d which results in a minimum weight girder can be obtained as follows:

Express the total girder area as: $$A_g = d\ t_w + 2\ A_f \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.5)$$

The moment resistance of the girder can be expressed as

$$M = F_b\ Z_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.6)$$

Where Z_x is the section modulus of the girder. Substituting from Eqn. 5.6 into Eqn. 5.4 gives:

$$A_f = Z_x / d - A_w / 6 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.7)$$

Substituting from Eqn 5.7 into Eqn. 5.5 gives:

$$A_g = 2\ Z_x / d + 2\ A_w / 3 = 2\ Z_x / d + 2\ d\ t_w / 3 \ldots\ldots (5.8)$$

By introducing a web slenderness ratio parameter, $\beta = d/t_w$, Eqn 5.8 can be expressed as

$$A_g = 2\ Z_x / d + 2\ d^2 / 3\ \beta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.9)$$

A_g is minimum when $\partial A_g / \partial d = 0$ which gives:

$$d^3 = 1.5\ \beta\ Z_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.10)$$

Substituting $Z_{\mathbf{x}} = M / F_{\mathbf{b}}$, Eqn 5.10 gives:

$$\mathbf{d = \sqrt[3]{1.5\,\boldsymbol{\beta}\, M / F}} \quad \text{.......................................} \mathbf{(5.11)}$$

The value of β will normally lie in the range 100 to 150. With M expressed in meter-ton units and F in t/cm^2 units, the above equation gives the optimum girder depth in meters as:

$$\mathbf{d = (0.25 \sim 0.3)\sqrt[3]{M / F}} \quad \text{.................................} \mathbf{(5.12)}$$

For steel St. 52 with $F_{\mathbf{b}} = 0.58\ F_{\mathbf{y}}$ this equation gives:

$$\mathbf{d = (0.2 \sim 0.24)\sqrt[3]{M}} \quad \text{.......................................} \mathbf{(5.13)}$$

Design Considerations:

For efficient design it is usual to choose a relatively deep girder, thus minimizing the required area of flanges for a given applied moment. This obviously results in a deep web whose thickness t_w is chosen equal to the minimum required to carry the applied shear. Such a web may be quite slender, i.e. has a high d/t_w ratio, and may be subjected to buckling which reduces the section strength. A similar conflict may exist for the flange plate proportions. The desire to increase weak axis inertia encourages wide, thin flanges, i.e. flange with a high b/t_f ratio. Such flanges may also be subjected to local buckling.

Design of plate girders therefore differs from that of rolled sections because the latter generally have thicker web and flange plates and thus are not subjected to buckling effects. In contrast, the freedom afforded in material selection in plate girder design makes buckling a controlling design criterion. Thus, in designing a plate girder it is necessary to evaluate the buckling resistance of flange plates in compression and of web plates in shear and bending. In most cases various forms of buckling must be taken into account. Figure 5.5 lists the different buckling problems associated with plate girder design. A brief description of each form is given below:

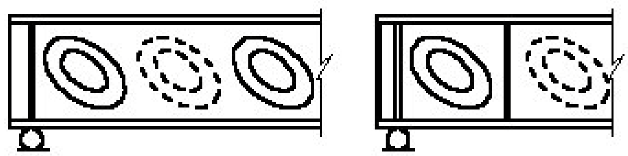
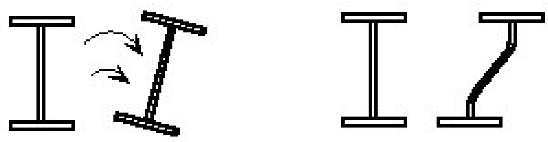
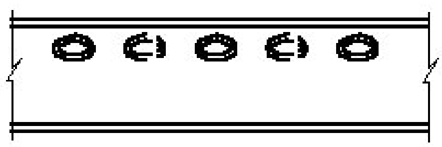
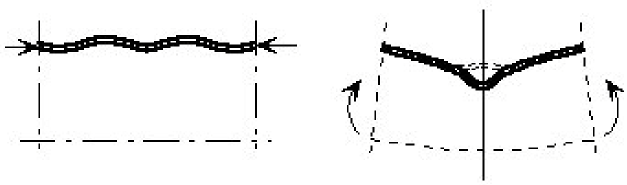
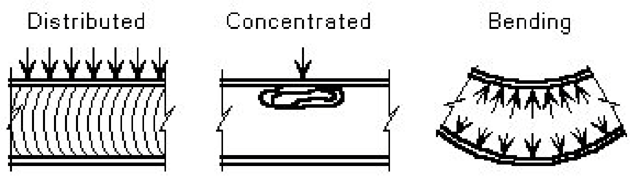
Buckling type	Illustration
a- Shear buckling of web	
b- Lateral-torsional buckling of girder	
c- Local buckling of compression flange	
d- Compression buckling of web	
e- Flange induced buckling of the web	
f- Local buckling of web (due to vertical load)	

Fig. 5.5 Plate Buckling Problems Associated with Plate Girders

a) Shear Buckling of the Web Plate

If the web width-to-thickness ratio d/t_w exceeds a limiting value, the web will buckle in shear before it reaches its full shear capacity. Diagonal buckles, of the type shown in Fig.5.5a, resulting from the diagonal compression associated with the web shear will form. This local buckling reduces the girder shear strength.

b) Lateral Torsional Buckling of girder

If the compression flange is not supported laterally the girder is subjected to lateral torsional buckling which reduces the allowable bending stresses, see Fig. 5.5b.

c) Local Buckling of the Compression Flange

If the compression flange width-to-thickness ratio exceeds a limiting value, it will buckle before it reaches its full compressive strength as shown in Fig. 5.5c. This local buckling will reduce the girder's load carrying resistance.

d) Compression Buckling of the Web Plate

If the web width-to-thickness ratio d/t_w exceeds a limiting value, the upper part of the web will buckle due to bending compression as shown in Fig. 5.5d. Consequently, the moment resistance of the cross section is reduced.

e) Flange Induced Buckling of the Web Plate

If particularly slender webs are used, the compression flange may not receive enough support to prevent it from buckling vertically rather like an isolated strut buckling about its minor axis as shown in Fig. 5.5e. This possibility may be eliminated by placing a suitable limit on d/t_w.

f) Local Buckling of the Web Plate

Vertical loads may cause buckling of the web in the region directly under the load as shown in Fig. 5.5f. This buckling form is known as web crippling. The level of loading that may safely be carried before this happens will depend upon the exact way in which the load is transmitted to the web and the web proportions.

Detailed considerations of these buckling problems will be presented in the following sections.

5.3 INFLUENCE OF BUCKLING ON PLATE GIRDERS DESIGN

5.3.1 General

In the previous section, it was shown that plate girders might be subjected to different forms of local plate buckling. In order to study the effect of local buckling on the strength of the cross-section, knowledge of the theory of buckling of rectangular plates is essential. Flanges can be modeled as long plates under uniform compression with one long edge assumed simply supported and the other long edge free. Webs can be modeled as long plates with the two long edges as simply supported. The compression on the plate edge may be uniform, as in the girder flange, see Fig. 5.6 a, or non-uniform, as in the girder web, see Fig. 5.6 b. In addition, the web plate may be subjected to shear stresses as shown in Fig. 5.6 c.

In the following sections, a brief treatment of the buckling of plates is given. The results are then used to study the effect of plate buckling on the strength of plate girders.

5.3.2 Buckling of Plates under Uniform Edge Compression

5.3.2.1 Theoretical Buckling Resistance

Consider a **uniformly compressed** plate of thickness t, width b, and length a simply supported along its four edges as shown in Figure 5.7. Up to a certain load, the plate remains compressed in its own plane. However, as the load increases and reaches a critical value, the plane state of the plate becomes unstable. Further increase in load causes the plate to deflect laterally, resulting in the out-of-plane configuration shown in Fig. 5.7. This phenomenon is referred to as plate buckling, and the stress that causes it is called the critical buckling stress.

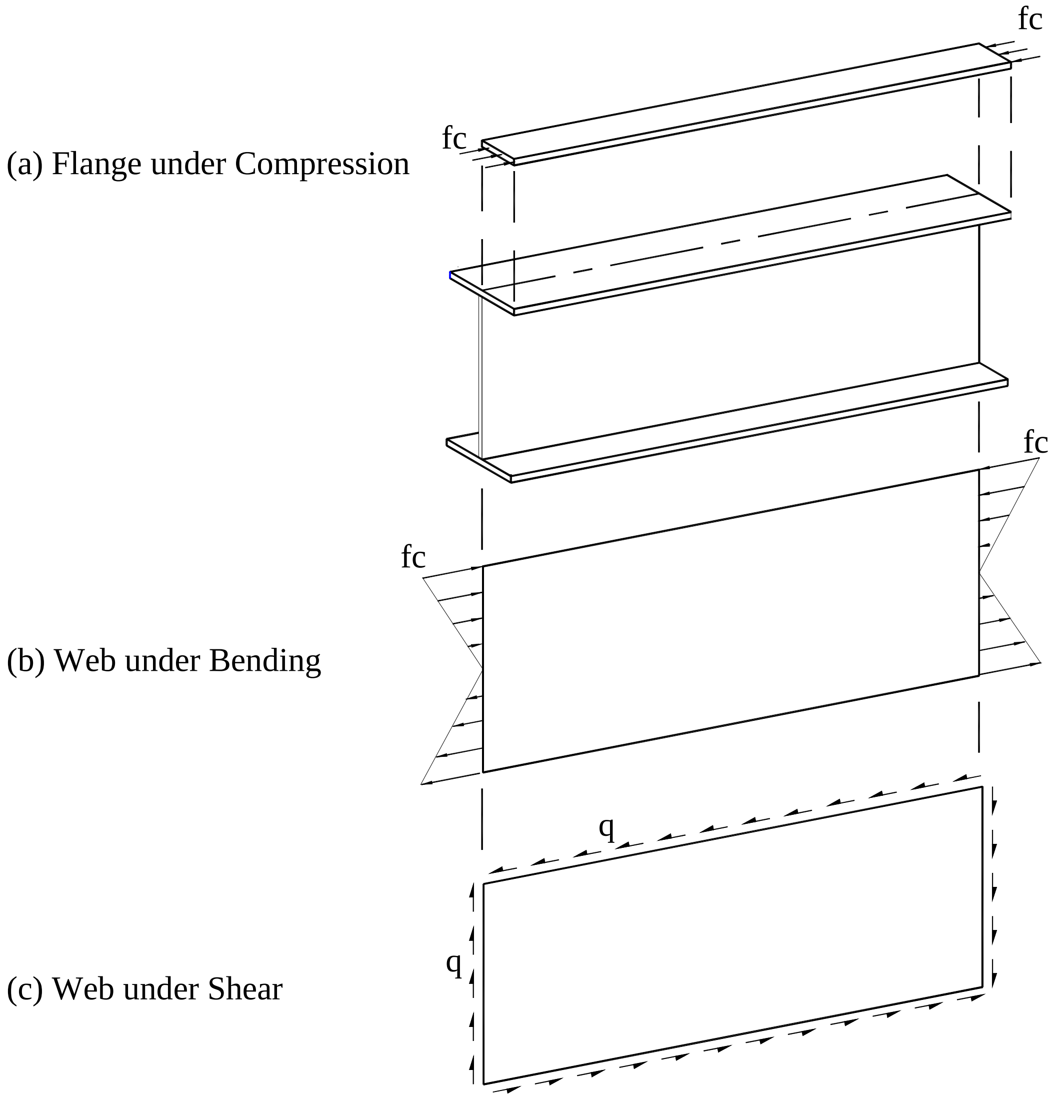


Fig. 5.6 Modeling of Plate Girder Components

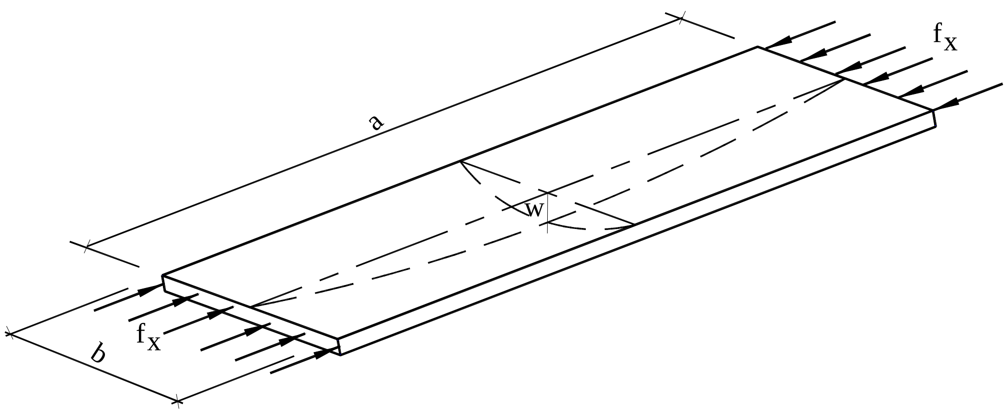


Fig. 5.7 Buckling of Simply Supported Plate under Uniform Compression

The value of this buckling stress can be determined by applying structural mechanics theories to study the behavior of the plate. The assumptions used to solve this stability problem are those used in thin plate theory (Kirchhoff's theory):

1. Material is linear elastic, homogeneous and isotropic.
2. Plate is perfectly plane and initially stress free.
5. Thickness "t" of the plate is small compared to its other dimensions.
4. In-plane actions pass through its middle plane.
5. Transverse displacements w are small compared to the plate thickness.
6. Slopes of the deflected middle surfaces are small compared to unity.
7. Deformations are such that straight lines, initially normal to the middle plane, remain straight lines and normal to the deflected middle surface.
8. Stresses normal to the thickness of the plate are of a negligible order of magnitude.

Based on these assumptions, the governing differential equation of the plate buckling is expressed as:

$$\mathbf{D\left(\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2 \partial y^2}+\frac{\partial^4 w}{\partial y^4}\right)+t f_x \frac{\partial^2 w}{\partial x^2}=0} \quad \ldots\ldots\ldots\ldots\ldots\ldots \mathbf{(5.14)}$$

where f_x = normal stress
D = plate bending rigidity = $E\, t^3 / 12(1 - \nu^2)$
E = Elastic Modulus = 2100 t / cm^2
t = Plate thickness
ν = Poisson's ratio = 0.3

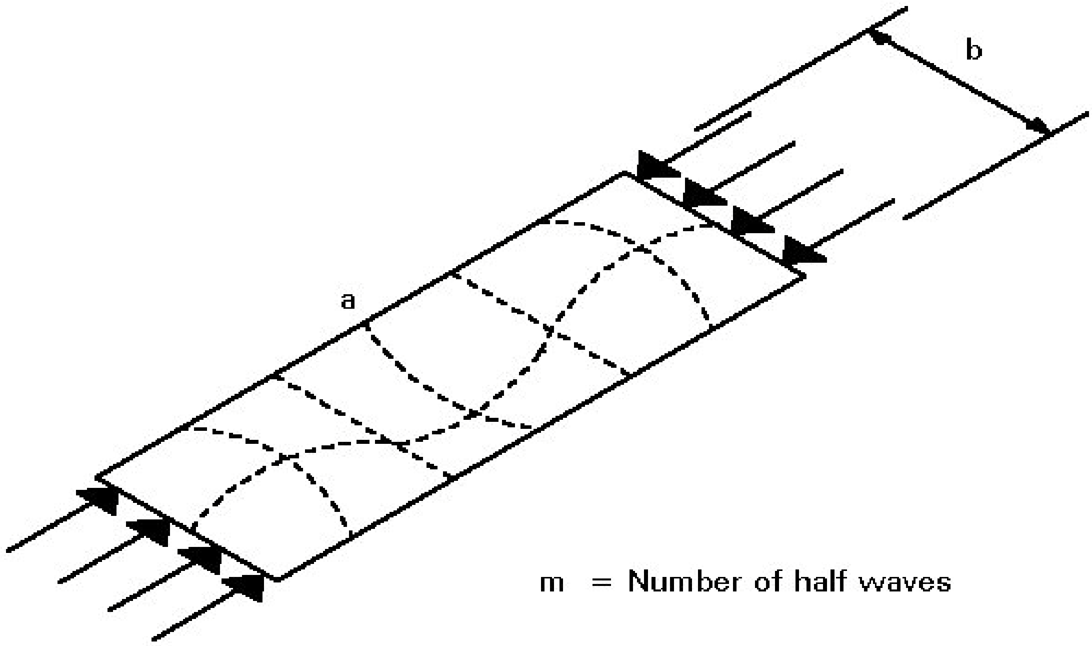
The solution of this equation gives the elastic buckling stress F_{cr} of the plate as:

$$\mathbf{F_{cr}=k_c \frac{\pi^2 E}{12(1-\upsilon^2)}\left(\frac{t}{b}\right)^2 = 1898\, k_c\, (t/b)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \mathbf{(5.15)}$$

where k_c = plate buckling factor which depends on the type of stress distribution, the edge support conditions, and the plate aspect ratio $\alpha = a/b$. For the case considered it can be expressed as:

$$\mathbf{k_c = (m / \alpha + \alpha / m)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \mathbf{(5.16)}$$

where m = number of buckling half-waves in the longitudinal direction. For each value of m, there exists a corresponding buckling stress and a buckled configuration. Fig. 5.8 shows the dependence of k_c on the ratio α for various values of m. The buckling mode for values of $\alpha < \sqrt{2}$, has one half wave, for values $\sqrt{2} < \alpha < \sqrt{6}$, two half waves, etc.



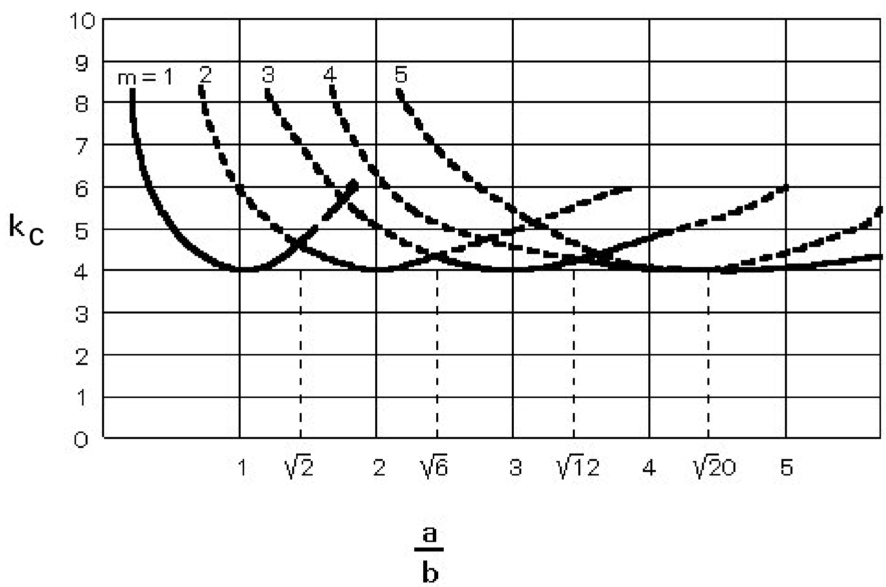


Fig. 5.8 Plate Buckling Coefficient k_c under axial compression

Referring to the curve for m = 1, it is seen that kc is large for small values of α and decreases as α increases until α = 1 (i.e., square plate) when k reaches its minimum value of 4. The value of k_c increases again as α increases. Similar behavior is obtained for other values of m. Therefore, $k_c = 4$ may be considered as valid for all values of m and is used as the basis for design.

Fig. 5.9 shows examples of buckled configurations of the plate for m=1,2,5. In a physical sense, Eqn. 5.16 can be interpreted to mean that a plate, simply supported on all four edges and uniformly compressed along the shorter sides, buckles in half – waves whose lengths approach the width of the plate, see Fig. 5.9 d.

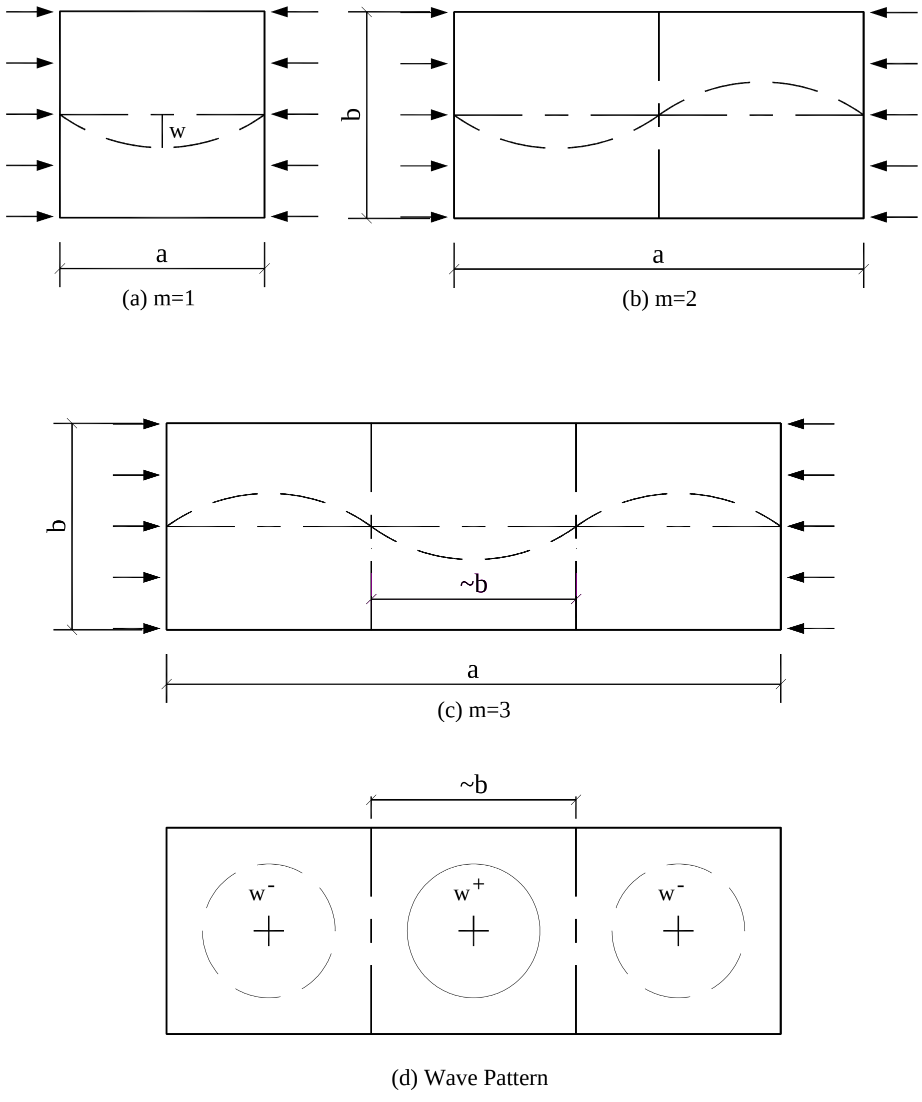


Fig. 5.9 Buckling Configurations

The above discussion applies to plates simply supported along their four edges. Plate girder sections may comprise plates, which are free along one longitudinal edge, and supported along the other edges, e.g., flange plates. Solution of the governing differential equation under these boundary conditions yields the value of the plate buckling factor $k_c = 0.425$.

Substituting the values of k_c into Eqn. 5.5, the critical buckling stress is obtained as:

1- For a plate with simple supports, (k_c =4) $F_{cr} = 7592 / (b/t)^2$

2- For a plate with a free edge, (k_c =0.425) $F_{cr} = 807 / (b/t)^2$

Fig. 5.10 shows the relationship between F_{cr} and (b/t) according to these equations.

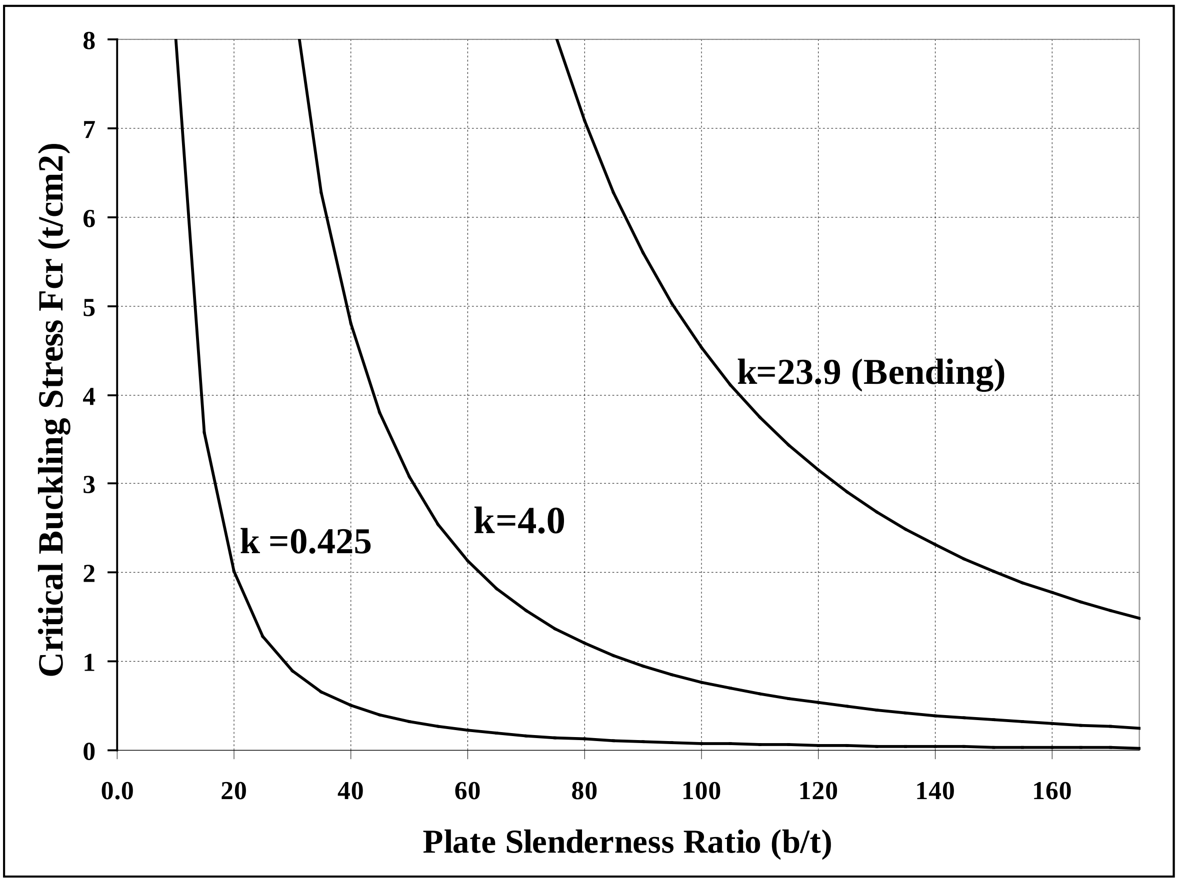


Fig. 5.10 Critical Buckling Stress F_{cr} under Compression and/or Bending

Analogy with buckling of Axially Loaded Columns:

At this point, it is instructive to compare and understand the differences between this buckling behavior of simply supported plates and that of axially loaded simply supported columns for which the critical load is given by: $P_{cr} = \pi^2 E I / L^2$. The simply supported column buckles into one half-wave of length L and the value of the critical buckling load is inversely proportional to L^2 and is independent of the column width. By contrast, because of the supports along the unloaded edges, a plate buckles into multiple half-waves the length of which approach the plate width b. The critical stress in the simply supported plate is inversely proportional to b^2 and independent of its length a.

5.3.2.2 Buckling of Plates under Linearly Varying Edge Compression

The above results can be extended to cover the general case of a plate subjected to a **linearly varying compressive stress**, e.g., due to bending moment in the plane of the plate as shown in Fig. 5.11 .

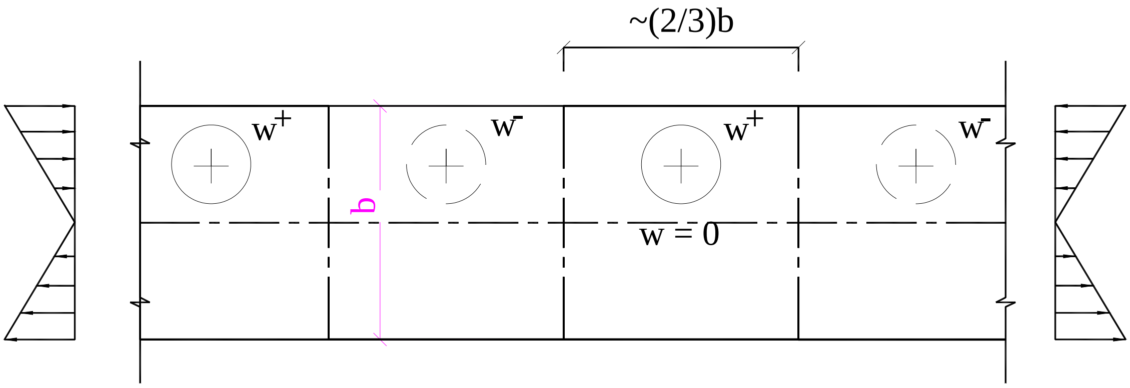


Fig. 5.11 Buckling of Plate due to Bending

The critical elastic buckling stress for this case is expressed as:

$$F_{cr} = k_\sigma \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t}{b}\right)^2 \quad \text{...} \mathbf{(5.17)}$$

where the value of the plate buckling factor k_σ is given by:

$$k_\sigma = \frac{16}{\sqrt{(1+\psi)^2 + 0.112\,(1-\psi)^2} + (1+\psi)} \qquad \textbf{(for } 1 > \psi > -1\textbf{)}\text{..........}\mathbf{(5.18)}$$

And $\mathbf{k_\sigma = 5.98\ (1 - \psi)^2}$ **(for -1 > ψ >-2)**..........**(5.19)**

where $\psi = \sigma_2/\sigma_1$ = ratio of smaller stress σ_2 to larger stress σ_1 . The value ψ = 1 corresponds to uniformly distributed compressive stress, where as the value ψ = -1 corresponds to the case of pure bending **($\sigma 2$ = - $\sigma 1$)**. The intermediate values, $-1 < \psi < +1$, correspond to combined bending and compression.

For the special case of pure bending, i.e., ψ = -1, the value of k_σ is equal to 25.9, giving:

$$F_{cr} = 23.9 \; \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t}{b}\right)^2 = 45362 \left(\frac{t}{b}\right)^2 \quad \text{..............................} (5.20)$$

A plot of the relationship between F_{cr} and (b/t) according to Eqn. 5.20 is shown in Fig. 5.10.

Fig. 5.11 shows a typical buckling pattern of a plate initiated by bending of the plate. This bend-buckling is somewhat different than the buckling of a uniformly edge-compressed plate in that the out-of-plane deformation in the tensile zone of the plate is zero (shown by w = 0). The plate buckles in a single half-wave transversally (i.e., depth wise) and in multiple half-waves longitudinally (i.e., length wise). As shown in Fig. 5.11, the lengths of the buckling waves approach 2/3 b.

5.3.2.3 Buckling of Plates under Edge Shear

Fig. 5.12(a) shows a plate under the action of edge shear stresses. These stresses are equivalent to tension and compression stresses that are equal in magnitude to the shear stresses but inclined at 45^o. The compressive stresses may cause the plate to buckle as shown in Fig. 5.12 (a). The buckling mode is composed of multiple wave forms which are skewed with respect to the edges. The half-wave length is equal to about 1.25 b for long simply supported plates.

According to the elastic buckling theory, the critical buckling shear stress can be expressed as:

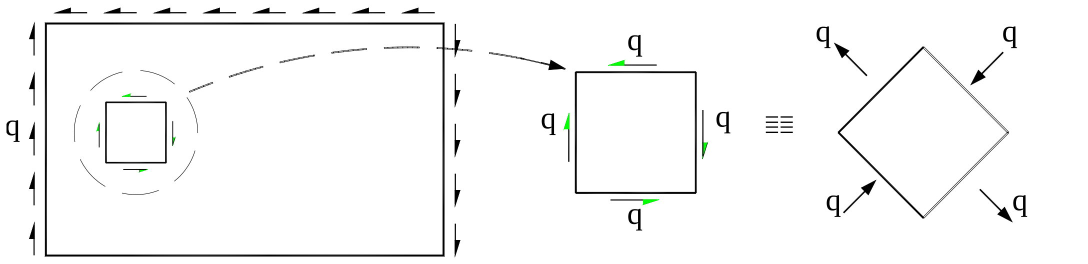
$$q_{cr} = k_q \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t}{b}\right)^2 = 1898 \; k_q \left(\frac{t}{b}\right)^2 \quad \text{............................} (5.21)$$

where k_q is a shear buckling factor calculated from elastic buckling theory according to the plate aspect ratio α = a/b as follows:

$$k_q = 4.00 + 5.34 / \alpha^2 \qquad \alpha < 1 \text{.........................} (5.22\ a)$$

$$k_q = 5.34 + 4.00 / \alpha^2 \qquad \alpha > 1 \text{.........................} (5.22\ b)$$

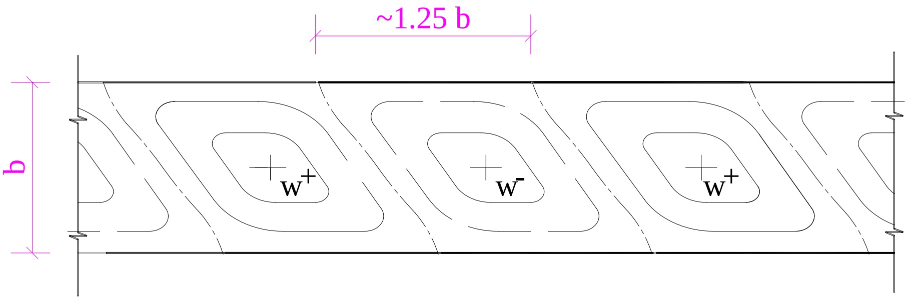
A plot of the relationship between q_{cr} and (b/t) according to Eqns. 5.21 and 5.22 is shown in Fig. 5.13 for different values of α.



(a) Plate under Pure Shear

(b) Element in pure shear

(c) Principal Stresses



(d) Wave Pattern

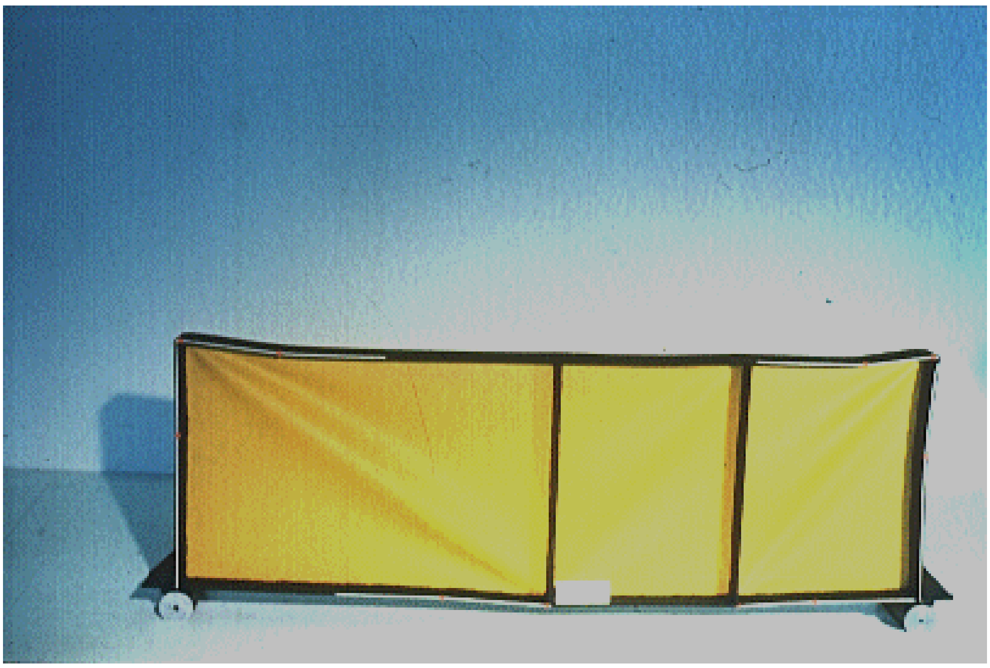
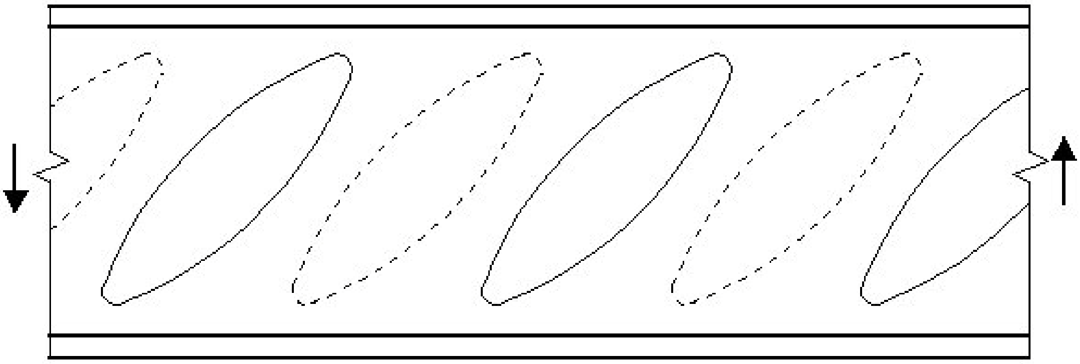
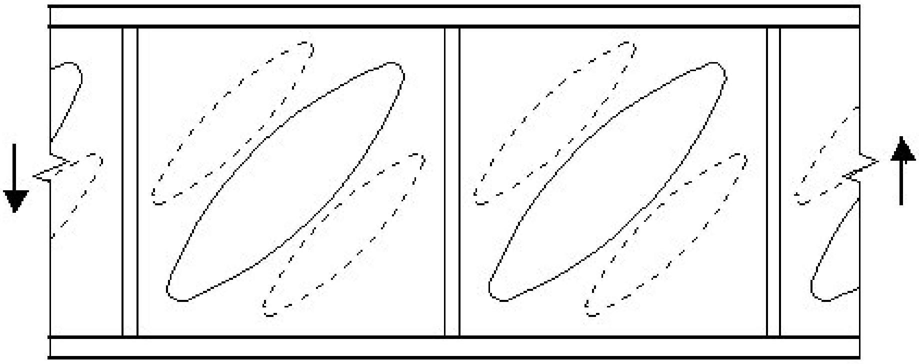


Fig. 5.12 Shear Buckling of Plates



(a) Unstiffened web



(b) stiffened web

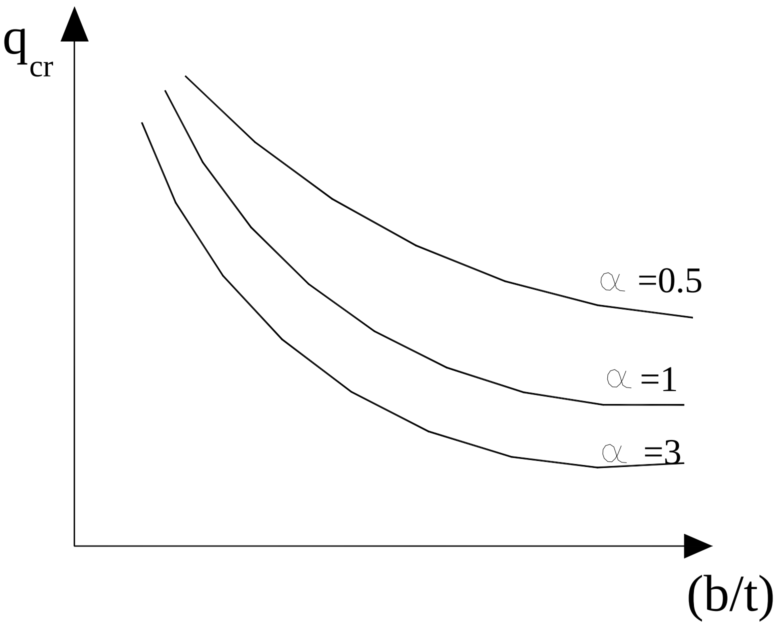


Fig. 5.13 Critical Buckling Stress due to Shear

5.3.3 Resistance of Actual Plates

The buckling theory described in the previous section is based on assumptions (1) to (8) of section 5.3.2.1 that are never fulfilled in real structures. The consequences for the buckling behavior when each of these assumptions is not valid are now discussed.

5.3.3.1 Effect of Inelastic Behavior

The first assumption of linear elastic behavior of the material is obviously not valid when the value of F_{cr} according to these equations exceeds the material yield strength F_y. This behavior is typical for thick plate panels having low (b/t) ratios. In this case failure is governed by yielding rather than buckling. If the material is considered to behave as linear elastic-ideal plastic, the buckling curve must be cut off at the level of the yield stress F_y as shown in Figure 5.14.

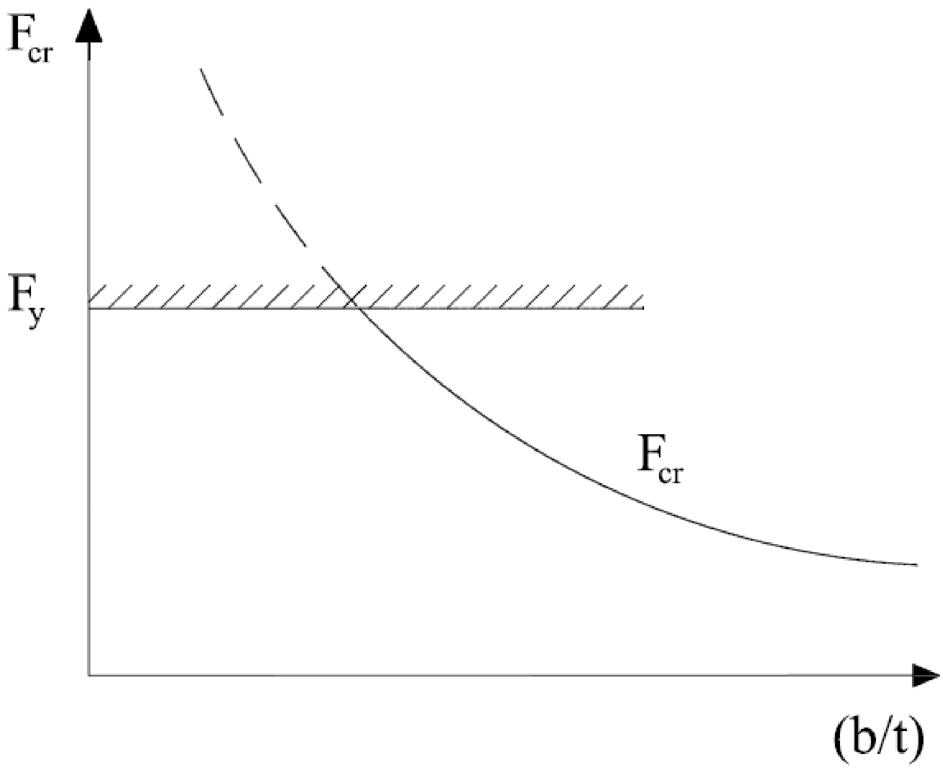


Fig. 5.14 Effect of Inelastic Behavior on Plate Buckling

Accordingly, two regions must be considered for establishing strength:

1. for large (b/t) values: $F_{cr} < F_y$, i.e., ***Elastic buckling*** governs the design.
2. for low (b/t) values: $F_{cr} \geq F_y$, i.e, ***Yielding*** governs the design.

For design purposes, it is convenient to define a plate slenderness parameter λ in terms of the ratio of the yield stress to the critical stress F_y/F_{cr} as:

$$\lambda = \sqrt{\frac{F_y}{F_{cr}}} = \sqrt{\frac{F_y}{k\,(1898)\left(\frac{t}{b}\right)^2}} = \left(\frac{b}{t}\right)\sqrt{\frac{F_y}{1898\,k}} \quad \text{.............................} \mathbf{(5.23)}$$

A plot of F_{cr}/F_y versus λ is shown in Fig. 5.15. Curve (a) represents the theoretical buckling curve defined by Eqn. 5.18, while the horizontal line at $F_{cr} = F_y$ represents the yield condition. The value $\lambda = 1$ represents the limit between elastic buckling and yield. Consequently, the plate fails due to elastic buckling when $\lambda > 1$ and due to yield when $\lambda < 1$.

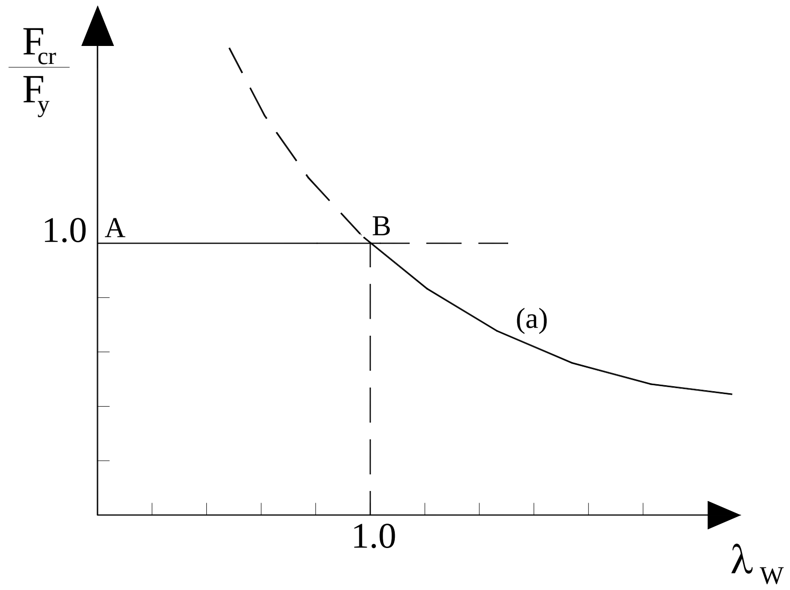


Fig. 5.15 Non-dimensional Buckling Curve

5.3.3.2 Effect of Imperfections and Residual Stresses

The second and fourth assumptions of a plate without geometrical imperfections and residual stresses are also never fulfilled in real structures. Plates in fabricated structures are likely to have some initial out-of-plane deviations. When the plate is loaded, these deviations will start growing in depth and thus cause additional stresses on the cross section. Furthermore, steel plates as well as rolled sections contain residual stresses. Residual stresses in rolled sections are mainly caused by uneven cooling after hot rolling. Plates in welded plate girders are subjected to high temperatures during flame-cutting and welding. Shrinkage due to cooling of the hot areas is resisted by the remaining cold parts of the cross section. As a result, the areas adjacent to the weld or flame cut are subjected to high tensile strains which may be several times the yield strain, and the rest of the cross-section is subjected to compression. As compressive and tensile residual stresses in the cross-section balance, residual stresses do not cause any resultant axial force or bending moment on the cross-section. However, those parts of the cross section where the residual stress is of the same nature as the applied stress will reach yield earlier. With further loading these yielded parts will not contribute any resistance to the cross section and thus the effective stiffness, and consequently the plate buckling strength, will be reduced. Residual stresses are less important for plates subjected to shear or bending stress than plates under compression because the applied stresses and the residual stresses are likely to be of a different nature in different parts of the plates.

Tests have shown that the reduction in plate buckling strength due to imperfections and residual stresses is most pronounced for plates with intermediate values of (b/t). For design purposes, this effect is considered by using a reduced value of the limit plate slenderness $\lambda_0 < 1$. Because of statistical variations in material properties and imperfections which are not sufficiently well known to be quantified accurately, the appropriate value of λ_0 differs substantially from country to country. A review of the international design codes shows that λ_o varies approximately from 0.6 to 0.9. ECP has adopted the following limiting values for the plate slenderness parameter:

1. $\lambda_o = 0.74$ for Class 2 elements in compression.
2. $\lambda_o = 0.90$ for Class 2 elements in bending.
3. $\lambda_o = 0.80$ for elements under pure shear.

These values can be used to calculate the limiting slenderness ratios of different parts in a plate girder section as follows:

a)Limiting b/t Ratio for Flanges under Uniform Compression:

The flange plate in a plate girder cross-section is essentially a uniformly compressed long narrow plate. As shown in sec. 5.3.2.1, the elastic buckling stress may be calculated from Eqn. 5.15 using the appropriate value for the plate buckling factor k =0.425. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.74$ is used. Substituting a value of k = 0.425 and $\lambda_o = 0.74$ in Eqn. 5.23 gives:

$$\mathbf{0.74 = \sqrt{\frac{F_y}{F_{cr}}} = \left(\frac{b}{t}\right)\sqrt{\frac{F_y}{1898\,k}}} \quad \text{..................................} \mathbf{(5.24)}$$

which gives: $$\left(\frac{\mathbf{b}}{\mathbf{t}}\right)_{\mathbf{lim}} \leq \mathbf{21} \;/\; \sqrt{\mathbf{F_y}} \quad \textbf{(5.25)}$$

= 11 for St. 52,
= 15.5 for St. 37.

Whenever the width–to–thickness ratio of the plate girder compression flange exceeds the a.m. limit, the flange is considered a "slender" element whose strength is affected by local buckling as explained in the next section.

b)Limiting (d/t) Ratio for Webs under Pure Bending:

The web plate in a plate girder cross-section is essentially subjected to a linearly varying normal stress due to bending. As shown in sec. 5.3.2.2, the elastic buckling stress may be calculated from Eqn. 5.17 using the appropriate value for the plate buckling factor k = 25.9. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.90$ is used. Substituting a value of k = 25.9 and $\lambda_o = 0.90$ in Eqn. 5.23 gives:

$$\mathbf{0.90} = \sqrt{\frac{\mathbf{F_y}}{\mathbf{F_{cr}}}} = \left(\frac{\mathbf{d}}{\mathbf{t}}\right)\sqrt{\frac{\mathbf{F_y}}{\mathbf{1898 \, x \, 23.9}}} \quad \textbf{(5.26)}$$

which gives: $$\left(\frac{\mathbf{d}}{\mathbf{t}}\right)_{\mathbf{lim}} \leq \mathbf{190} \;/\; \sqrt{\mathbf{F_y}} \quad \textbf{(5.27)}$$

= 100 for St. 52,
= 122 for St. 37

Whenever the width–to–thickness ratio of the plate girder web exceeds the a.m. limit, the web is considered a "slender" element whose strength is affected by local buckling as explained in the next section.

c)Limiting (d/t) ratio for Webs under Pure Shear:

As shown in sec. 5.3.2.3, the elastic buckling stress for a plate under pure shear may be calculated from Eqn. 5.21 using the value for the plate buckling factor k defined by Eqns. 5.22. For a narrow long plate, $\alpha >> 1$ which gives k_q =5.34. Furthermore, to account for the reduction in buckling strength due to residual stresses and imperfections a reduced value of $\lambda = \lambda_o = 0.80$ is used.
Defining the plate slenderness parameter in shear λ_q as:

$$\lambda_q = \sqrt{\frac{F_y / \sqrt{3}}{q_{cr}}} = \sqrt{\frac{F_y / \sqrt{3}}{k_q \, (1898) \left(\frac{t}{d}\right)^2}} \quad \text{..............................} \textbf{(5.28)}$$

Substituting a value of k_q = 5.34 and λ_o = 0.80 in Eqn. 5.23 gives:

$$\left(\frac{d}{t}\right)_{lim} \leq 105 \;/\; \sqrt{F_y} \quad \text{..} \textbf{(5.29)}$$

= 55 for St. 52,
= 67 for St. 37

Whenever the width–to–thickness ratio of unstiffened plate girder webs exceeds the a.m. limit, the web is considered a “slender” element whose shear strength is affected by local buckling as explained in the next section.

5.3.3.3 Effect of Large Displacement:

Fig. 5.16 shows typical behavior of a compressed plate loaded to its ultimate load. As shown in the figure, the stress distribution remains uniform as the loading increases until the elastic buckling stress F_{cr} is reached. Unlike one dimensional structural members, such as columns, compressed plates will not collapse when the buckling stress is reached. Further increase in load beyond the elastic buckling load corresponding to the stress F_{cr} can be achieved before failure takes place. However, the portion of the plate farthest from its side supports will deflect out of its original plane. This out-of-plane deflection violates assumption (5) of small displacements and causes the stress distribution to become non-uniform. The stresses redistribute to the stiffer edges and the redistribution becomes more extreme as buckling continues. The additional load carried thus by the plate beyond its elastic buckling stress F_{cr} is termed the “post-buckling” strength. Tests have shown that the post-buckling strength is high for large values of (b/t) and very small for low values of (b/t).

In order to estimate the post-buckling strength, the non-uniform stress distribution can be replaced in design calculations by equivalent rectangular stress blocks over a reduced "effective width" b_e as shown in Fig. 5.17.

This equivalent uniform stress has the same peak stress and same action effect of the non-uniform stress distribution. The effective width of the element is computed from the condition that if the maximum stress is considered uniform over that width, the total section capacity will be the same.

By applying this model, an "effective cross-section" is obtained from the original cross-section by deducting the ineffective areas where local buckling occurred. This design procedure is then the same used for sections not subjected to local buckling effect provided that the stresses are calculated using the effective section properties.

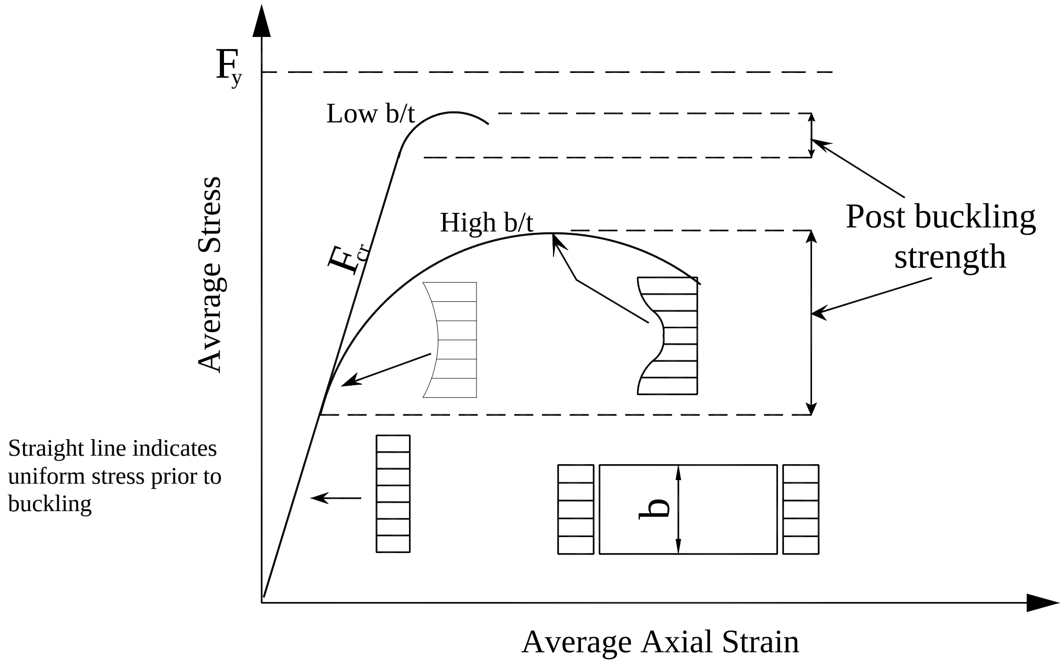


Fig. 5.16 Actual Plate Buckling Strength

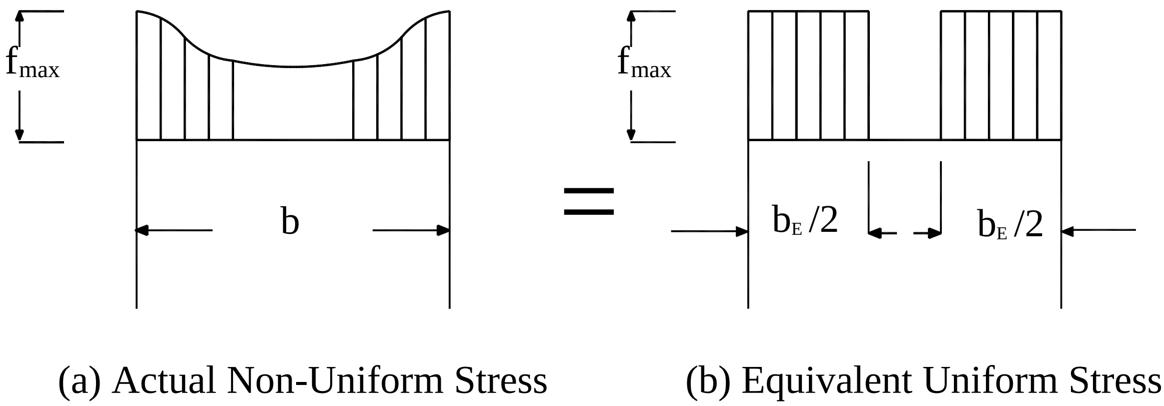


Fig. 5.17 Effective Width Concept

Definition of the "Effective Width":

According to this procedure, the effective width can be expressed in terms of the plate slenderness λ_p defined by Eqn. 5.23 as:

$$\mathbf{b_e = \rho * b} \quad \text{...} \quad \mathbf{(5.30)}$$

where ρ = **reduction factor** = $(\lambda_p - 0.2) / \lambda_p^2$

For the general case where the plate is subjected to a linearly varying compression, e.g., due to bending, the reduction factor can be expressed in terms of the stress ratio ψ as:

$$\boldsymbol{\rho = (\lambda_p - 0.15 - 0.05 * \psi) / \lambda_p^2} \quad \text{.....................................} \quad \mathbf{(5.31)}$$

Tables 5.1 and 5.2 give the effective width of compression elements for the case of stiffened elements, e.g., girder webs, and unstiffened elements, e.g., girder flange, respectively.

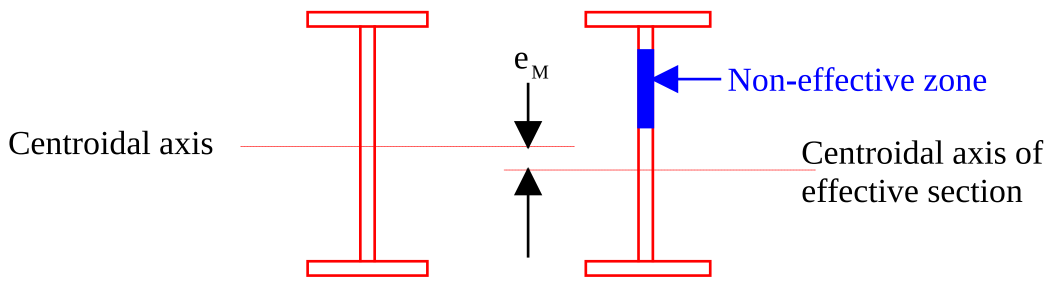


Figure 5.18 Effective Cross Section for Girder in Bending

For members in bending test results have shown that the effective widths may be determined on the basis of stress distributions calculated using the gross section modulus, Z_x , even though the formation of "effective holes" in the compression parts will shift the neutral axis of the effective cross-section as shown in Fig. 5.18. An iterative process is not, therefore, necessary to compute the effective section properties.

Table (5.1) Effective Width and Buckling Factor for Stiffened Compression Elements

For $1 > \psi \geq -1$: $k_\sigma = \dfrac{16}{[(1+\psi)^2 + 0.112(1-\psi)^2]^{0.5} + (1+\psi)}$						$-1 > \psi > -2$
$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	
Buckling Factor k_σ	4.0	$\dfrac{8.2}{1.05+\psi}$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98(1-\psi)^2$

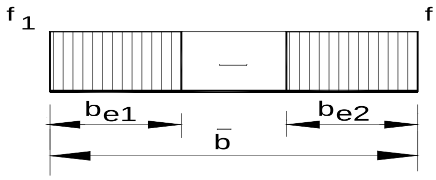
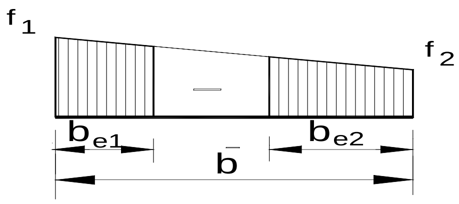
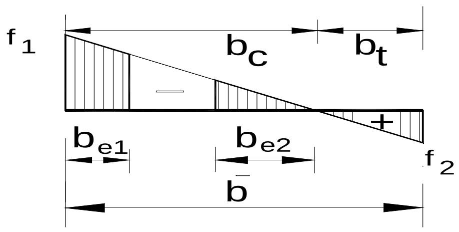
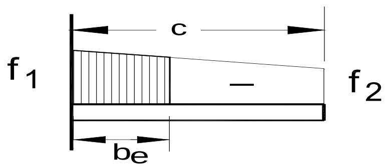
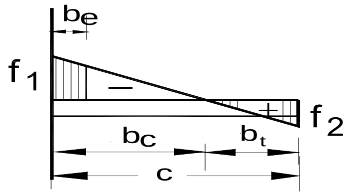
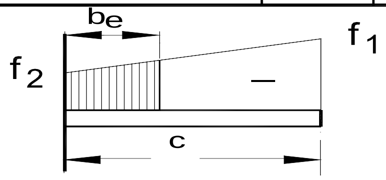
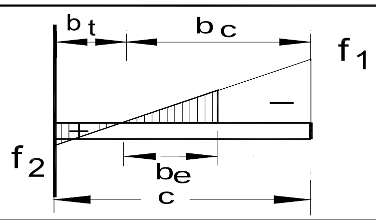
Stress Distribution	Effective Width b_e for $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi)/\bar{\lambda}_p^2 \leq 1$
	$\underline{\psi = 1:}$ $b_e = \rho \bar{b}$ $b_{e1} = 0.5\, b_e$ $b_{e2} = 0.5\, b_e$
	$\underline{1 > \psi \geq 0:}$ $b_e = \rho \bar{b}$ $b_{e1} = 2\, b_e/(5-\psi)$ $b_{e2} = b_e - b_{e1}$
	$\underline{\psi < 0:}$ $b_e = \rho b_c = \rho \bar{b}/(1-\psi)$ $b_{e1} = 0.4\, b_e$ $b_{e2} = 0.6\, b_e$

Table (5.2) Effective Width and Buckling Factor For Unstiffened Compression Elements

<table>
<tr><td colspan="2">Stress Distribution</td><td colspan="4">Effective Width b_e for
$\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi) / \bar{\lambda}_p^2 \leq 1$</td></tr>
<tr><td>$\psi = f_2 / f_1$</td><td>1</td><td>$1 > \psi > 0$</td><td>0</td><td>$0 > \psi > -1$</td><td>-1</td></tr>
<tr><td>Buckling factor k_σ</td><td>0.43</td><td>$\frac{0.578}{\psi + 0.34}$</td><td>1.70</td><td>$1.7 - 5\psi + 17.1\psi^2$</td><td>23.8</td></tr>
<tr><td colspan="2"></td><td colspan="4">$1 > \psi > 0$:
$b_e = \rho c$</td></tr>
<tr><td colspan="2"></td><td colspan="4">$\psi < 0$
$b_e = \rho b_c = \rho c / (1-\psi)$</td></tr>
<tr><td>$\psi = f_2 / f_1$</td><td>1</td><td>0</td><td>-1</td><td colspan="2">$1 > \psi > -1$</td></tr>
<tr><td>Buckling factor k_σ</td><td>0.43</td><td>0.57</td><td>0.85</td><td colspan="2">$0.57 - 0.21\psi + 0.07\psi^2$</td></tr>
<tr><td colspan="2"></td><td colspan="4">$1 > \psi > 0$:
$b_e = \rho c$</td></tr>
<tr><td colspan="2"></td><td colspan="4">$\psi < 0$:
$b_e = \rho b_c = \rho c / (1-\psi)$</td></tr>
</table>

In determining the effective width of compression elements in a given cross-section, the following assumptions can be made:

1. To determine the effective width of flange plate, the stress ratio ψ may be based on the properties of the gross cross -section.
2. To determine the effective width of the web plate, the stress ratio ψ may be obtained using the effective area of the compression flange but the gross area of the web.
3. Generally the centroidal axis of the effective cross section will shift by a distance, e, measured from the centroidal axis of the gross cross section, see Figure 5.18. This eccentricity should be considered when calculating the properties of the effective cross-section.
4. When the cross section is subjected to an axial force, N, the stress calculations shall take into account the additional moment $\Delta M = N * e_N$,

where e_N = eccentricity of the centroidal axis when the effective cross section is subjected to uniform compression.

The actual plate girder strength is therefore represented by:

1- For plates with low values of (b/t); i.e., $\lambda < \lambda_o$, the strength is computed directly from the yield strength divided by the appropriate safety factor.

2- For plates having higher values of (b/t), $\lambda > \lambda_o$, the strength is computed from the yield strength or the elastic buckling strength by applying the effective width concept to account for the stress reduction due to residual stresses and imperfections and the stress increase due to post-buckling.

Fig. 5.19 summarizes the strength of actual plates of varying slenderness. It shows the reduction in strength due to residual stress and imperfections for intermediate slender plates, region BC, and the increase due to post-buckling strength for slender plates, region CD .

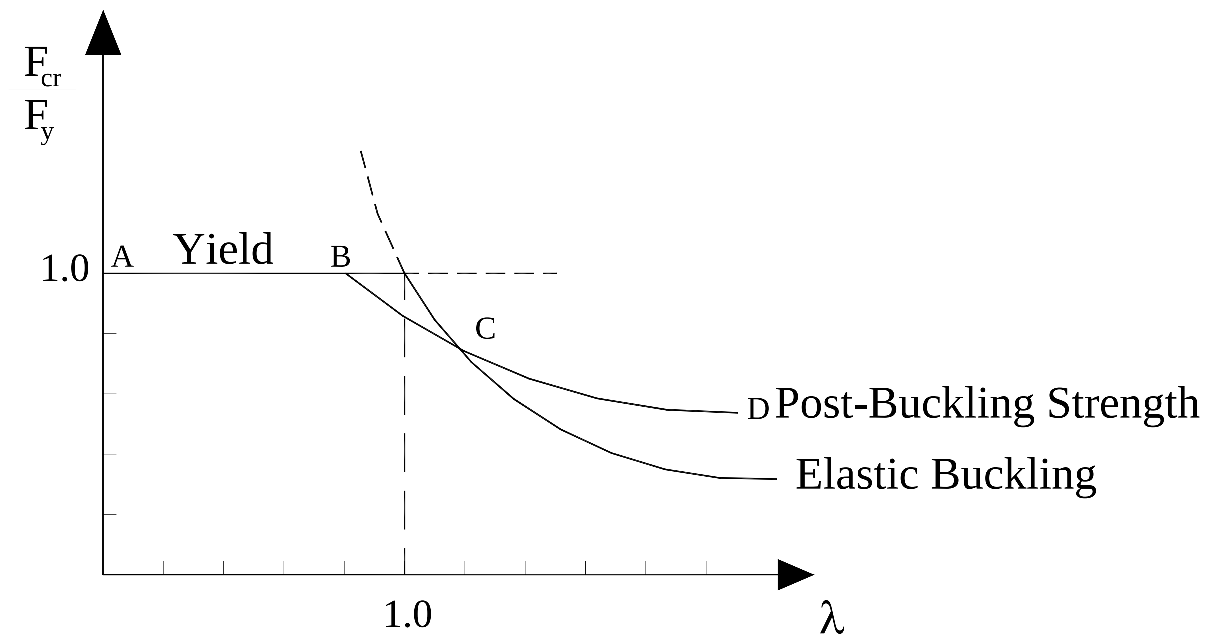


Fig. 5.19 Actual Plate Buckling Strength in Compression

5.4 ACTUAL STRENGTH OF PLATE GIRDER ELEMENTS

5.4.1 General

It has been shown in the preceding section that the strength of plates is affected by local buckling when the plate slenderness ratio exceeds a limiting value. These limiting values are:

i) For flange plate under uniform compression: $\left(\frac{b}{t}\right)_{lim} \leq 21 \ / \sqrt{F_y}$

ii) For web plate under pure bending: $\left(\frac{d}{t}\right)_{lim} \leq 190 \ / \sqrt{F_y}$

iii) For web plate under pure shear: $\left(\frac{d}{t}\right)_{lim} \leq 105 \ / \sqrt{F_y}$

Whenever the width–to–thickness ratio of the girder web plate or flange plate exceeds the a.m. limit, the plate is considered a "slender" element whose strength is affected by local buckling. This effect is considered in the design of plate girder sections as follows:

5.4.2 Plate Girders Under Bending Moment:

Plate girders subjected to the action of bending moment should be designed using the section modulus determined for the effective cross-sections as shown in section 5.3.3.3 and Table 5.1 and 5.2. This means that the bending stress computed from the familiar bending formula $f_b = Mx / Z_{eff}$ should not exceed the allowable bending stress value:

a) For the compression flange: the allowable bending stress is equal to $0.58\ F_y$ if the flange is laterally supported otherwise lateral torsional buckling governs the design.

b) For the tension flange: Two checks have to be made:
   i) the maximum tensile stress should not exceed $0.58\ F_y$
   ii) the maximum stress range due to live load application should not exceed the allowable fatigue stress range, as shown in section 3.3.

According to ECP 2001; ***for plate girders without longitudinal stiffeners***:

a- The web plate thickness of plate girders without longitudinal stiffeners (with or without transverse stiffeners) shall not be less than that detemined from:

$$\mathbf{t_w \geq d\sqrt{f_{bc}}\ / 145 > d/120} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{(5.32)}$$

b- Where the calculated compressive stress $\mathbf{f_{bc}}$ equals the allowable bending stress $\mathbf{F_{bc}}$, the thickness of the web plate shall not be less than:

$$\mathbf{t_w \geq d\sqrt{Fy}\ /\ 190} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{(5.33)}$$

Grade of	$t_w \geq$	
	t ≤ 40 mm	**40 mm < t ≤ 100**
St 37	**d/120**	**d/130**
St 44	**d/110**	**d/120**
St 52	**d/100**	**d/105**

If the assumed web thickness is not sufficient to resist buckling due to bending, the section strength can be increased by providing a thicker web. In plate girders with practical proportions, the flanges carry most of the applied bending moment, ~ 85 %, while the web carries all the shear force and a small part of the moment, ~ 15 %. Therefore, increasing the web thickness to resist bend-buckling is not effective. A more economic solution is usually achieved by limiting the web plate thickness to the minimum value required to resist the applied shear force. If this thickness is not sufficient for bend-buckling, the plate buckling strength is increased by providing the web plate with longitudinal stiffeners as shown in Fig. 5.20.

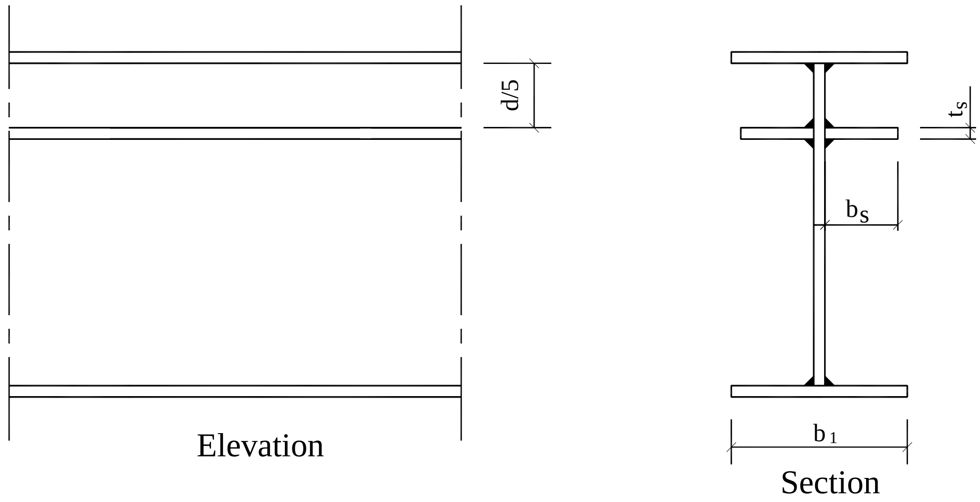


Fig. 5.20 Web Plate with Longitudinal Stiffeners

A longitudinal stiffener essentially forces the web to buckle in a higher mode by forming a nodal line in the buckled configuration, with waves much shorter than those of the longitudinally unstiffened plate. Analytically, the stiffener subdivides the plate into smaller sub-panels, thus increasing considerably the stress at which the plate will buckle. Theoretical and experimental studies have shown that the optimum location of one longitudinal stiffener is at 0.2d from the compression flange. The presence of this stiffener increases the plate buckling coefficient to 42.5 as compared to 23.9 for a longitudinally unstiffened web, i.e., about 280 % increase in the elastic buckling stress.

The corresponding slenderness limit for this case becomes:

$$\left(\frac{d}{t}\right)_{lim} \leq 320 \;/\; \sqrt{F_y} \quad \text{...} \quad \textbf{(5.34)}$$

According to ECP 2001; ***for Girders Stiffened Longitudinally***:

a- The web plate thickness of plate girders with longitudinal stiffeners (with or without transverse stiffeners), placed at **d/5** to **d/4** from compression flange, shall not be less than that determined from:

$$t_w \geq d\sqrt{f_{bc}} \;/\; 240 > d/240 \quad \text{.........................} \quad \textbf{(5.35)}$$

b- Where the calculated compressive stress $\mathbf{f_{bc}}$ equals the allowable bending stress $\mathbf{F_{bc}}$, the thickness of the web plate shall not be less than:

$$t_w \geq d\sqrt{F_y} \;/\; 320 \quad \text{......................................} \quad \textbf{(5.36)}$$

Grade of	$t_w \geq$	
	t ≤ 40 mm	**40 mm < t ≤ 100 mm**
St 37	**d/206**	**d/218**
St 44	**d/191**	**d/200**
St 52	**d/168**	**d/175**

For deep webs, e.g., depth larger than ~ 2.5 meters, a single longitudinal stiffener is usually not sufficient to prevent web buckling due to bending. The buckling strength of such webs is further increased by providing multiple longitudinal stiffeners in the region between the neutral axis and the compression flange.

5.4.3 Web Plates Under Pure Shear:

The effect of residual stresses and imperfections on the shear buckling stress of plate girder webs is treated in a different manner. Instead of considering an effective section for the buckled plate, the critical buckling stress in shear as calculated from Eqn. 5.21 is divided by a suitable factor of safety to give the allowable buckling shear stress. This stress is empirically modified to allow for residual stresses and imperfections. For plate girders with practical proportions, an economic solution can be obtained in most cases by using a thin web stiffened transversally by stiffeners as shown in Fig. 5.21.

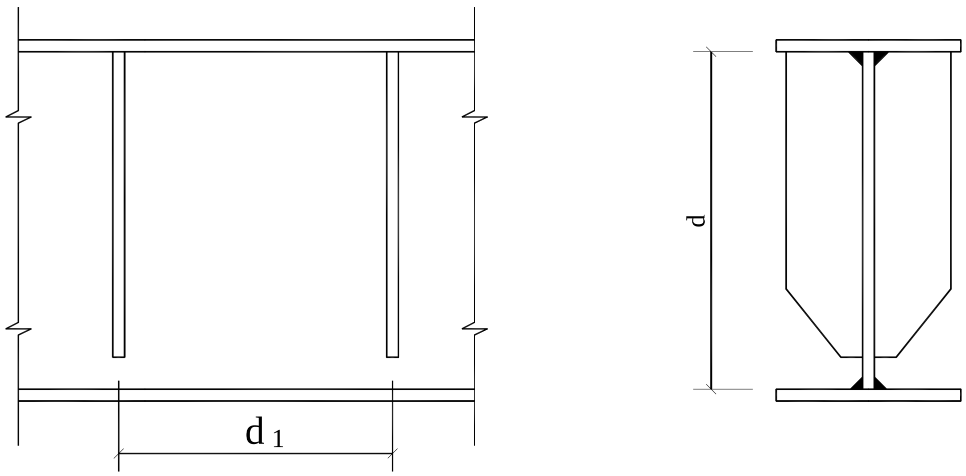


Fig. 5.21 Web Plate with Transverse Stiffeners

Post Buckling Stress in Shear: For transversely stiffened girders where the transverse stiffener spacing lies within the range 1 < a/d < 3, full account may be taken of the considerable reserve of post-buckling resistance. This reserve arises from the development of "tension field action" within the girder.

Figure 5.22 shows the development of tension field action in the individual web panels of a typical girder. Once a web panel has buckled in shear, it loses its resistance to carry additional compressive stresses. In this post-buckling range, a new load-carrying mechanism is developed, whereby any additional shear load is carried by an inclined tensile membrane stress field. This tension field anchors against the top and bottom flanges and against the transverse stiffeners on either side of the web panel, as shown. The load-carrying action of the plate girder than becomes similar to that of the N-truss in Figure 5.22 b. In the post-buckling range, the resistance offered by the web plates is analogous to that of the diagonal tie bars in the truss. The total shear buckling resistance for design is calculated by adding the post-buckling resistance to the initial elastic buckling resistance.

In this case, the shear buckling factor k_q, is computed from Eqn. 5.21 according to the value of $\alpha = d_1/d$ and the slenderness parameter in shear λ_q as determined from Eqn. 5.28.

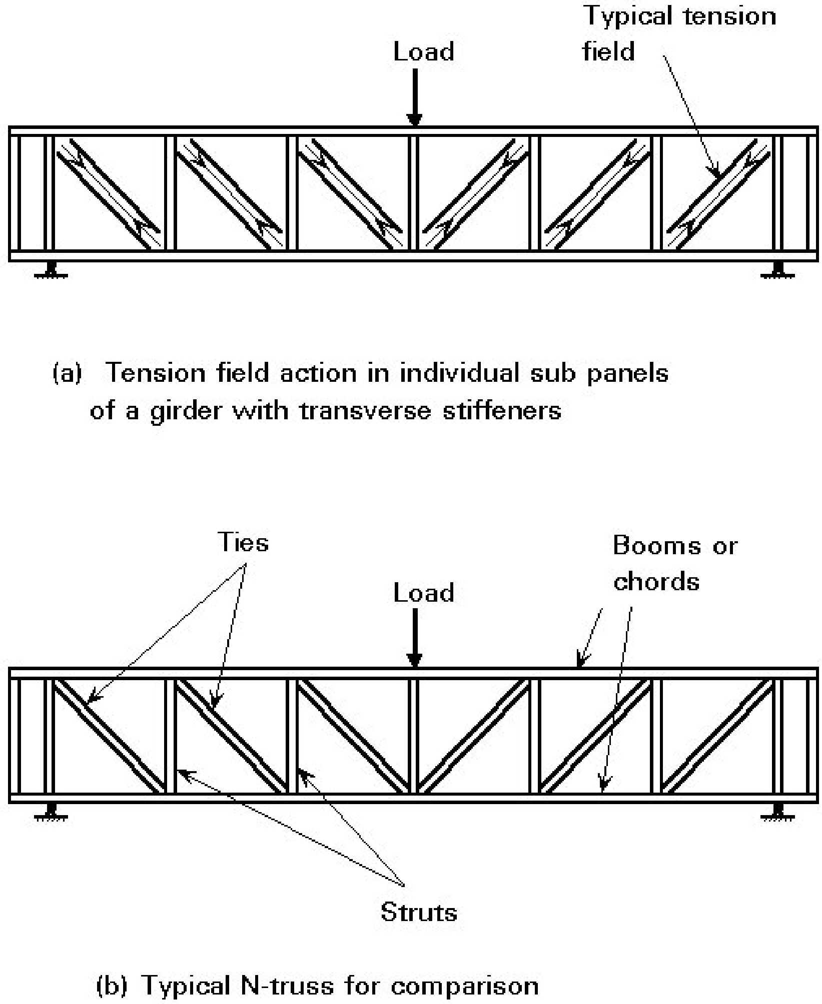


Fig. 5.22 Tension Field Action in Plate Girders

The calculation of the allowable shear buckling stress then depends, as illustrated in Figure 5.23, upon whether the web is:

1. **thick** ($\lambda_q < 0.8$, region AB in Fig. 5.23) in which case the web will not buckle and the shear stress at failure will reach the shear yield stress of the web material:

$$\mathbf{q_b = 0.35 * F_y} \quad \text{....................................} \quad \mathbf{(5.37)}$$

2. **intermediate** ($0.8 < \lambda_q < 1.2$, region BC in Fig. 5.23) which represents a transition stage from yielding to buckling action with the shear strength being evaluated empirically from the following:

$$\mathbf{q_b = (1.5 - 0{,}625\ \lambda_q)\ (0.35*F_y)} \quad \text{...................} \quad \mathbf{(5.38)}$$

3. **slender or thin** ($\lambda_q > 1.2$, region CD in Figure 5.23) in which case the web will buckle before it yields and a certain amount of post-buckling action is taken into account empirically:

$$\mathbf{q_b = (0.9 / \lambda_q)\,(0.35{*}F_y)} \text{ .... } \mathbf{(5.39)}$$

In all cases the calculated shear stress q_{act} should not exceed the allowable buckling shear stress q_b .

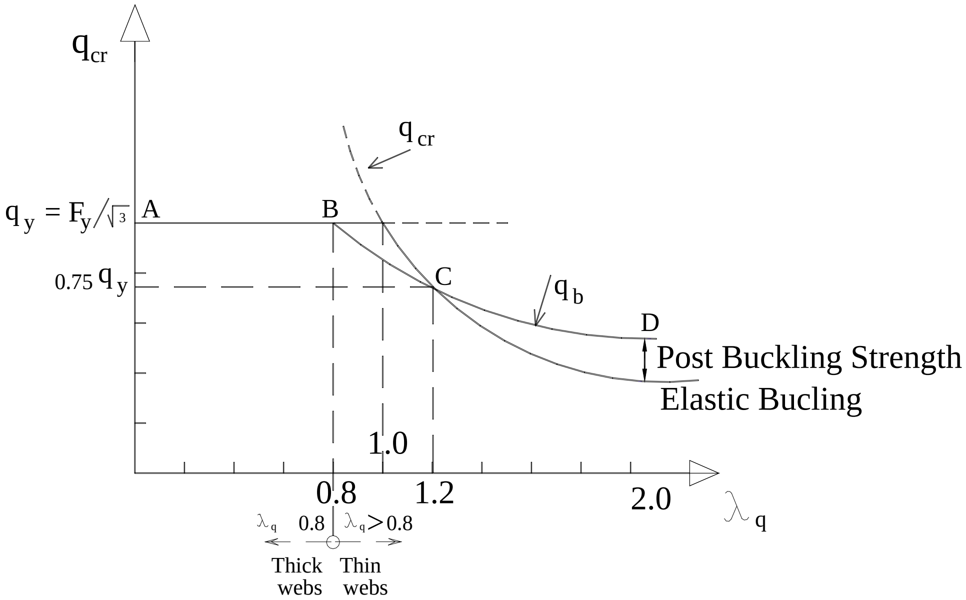


Fig. 5.23 Buckling Shear Stress

***Web plate without transversal stiffeners*:** The web plate of a typically unstiffened plate girder has a large aspect ratio α. For such a case, the allowable buckling shear stress q_b is obtained from the Eqn. 5.21 using a value of $k_q = 5.34$ as:

$$\textbf{For } (d/t) \le 159/\sqrt{Fy} : q_b = [1.5 - (d/t)\sqrt{Fy}/212]\,[0.35\,Fy] \le 0.35Fy \text{.....} \mathbf{(5.40)}$$

$$\textbf{For } (d/t) > 159/\sqrt{Fy} : q_b = \{119 / [\,(d/t)\sqrt{Fy}\,]\}\,\{0.35\,Fy\} \text{.....................} \mathbf{(5.41)}$$

The forgoing equations may require relatively thick webs making the resulting design uneconomic.

Effect of Longitudinal Stiffeners on Shear Buckling

Both shear and bending strengths of a plate girder are increased by the presence of a longitudinal stiffener. Its location is, therefore, a key factor that affects both. Theoretical and experimental studies have shown that the optimum location of one longitudinal stiffener is at 0.2d from the compression flange for bending and 0.5d for shear. It is important to note that these criteria for location of the stiffeners are based on elastic buckling considerations. The longitudinal stiffener may be more effective in contributing to the ultimate strength of the plate girder under combined bending and shear if placed somewhere between 0.2d and 0.5d from the compression edge of the web. In bridge design practices, 0.2d has been adopted as the standard location for a longitudinal stiffener. Theoretical and experimental studies have shown that the contribution of the longitudinal stiffener placed at 0.2d to the shear buckling stress is relatively small and is usually neglected, see Fig. 5.24.

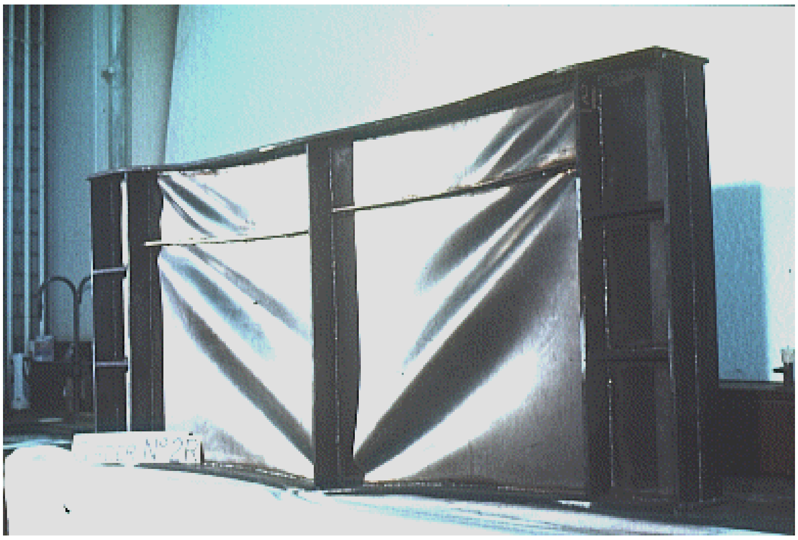


Fig. 5.24 Effect of Longitudinal Stiffeners on Shear Buckling

5.4.4 INTERACTION BETWEEN SHEAR AND BENDING

In general, any cross-section of a plate girder will be subjected to bending moment in addition to shear. This combination makes the stress conditions in the girder web considerably more complex. The stresses from the bending moment will combine with the shear stresses to give a lower buckling load. The interaction between shear and bending can be conveniently represented by the diagram shown in Fig. 5.25, where the allowable bending stress is plotted on the vertical axis and the allowable buckling shear stress of the girder is plotted horizontally. The interaction represents a failure envelope, with any point lying on the curve defining the co-existent values of shear and bending that the girder can just sustain. The equation representing this interaction diagram is:

$$\mathbf{F_b = [\,0.8 - 0.36\,(q_{act} / q_b)]\,F_y} \quad \textbf{(5.42)}$$

The interaction diagram can be considered in 3 regions. In region AB, the applied shear stress q_{act} is low ($< 0.6\ q_b$) and the girder can sustain the full bending stress F_b based on the effective width b_{eff} for the compression flange At the other extreme of the interaction diagram in region CD, the applied shear stress is high ($= q_b$) then the allowable bending stress is reduced to 0.44 F_y to allow for the high shear. In the intermediate region BC the allowable bending stress is reduced linearly from 0.58 F_y to 0.44 F_y.

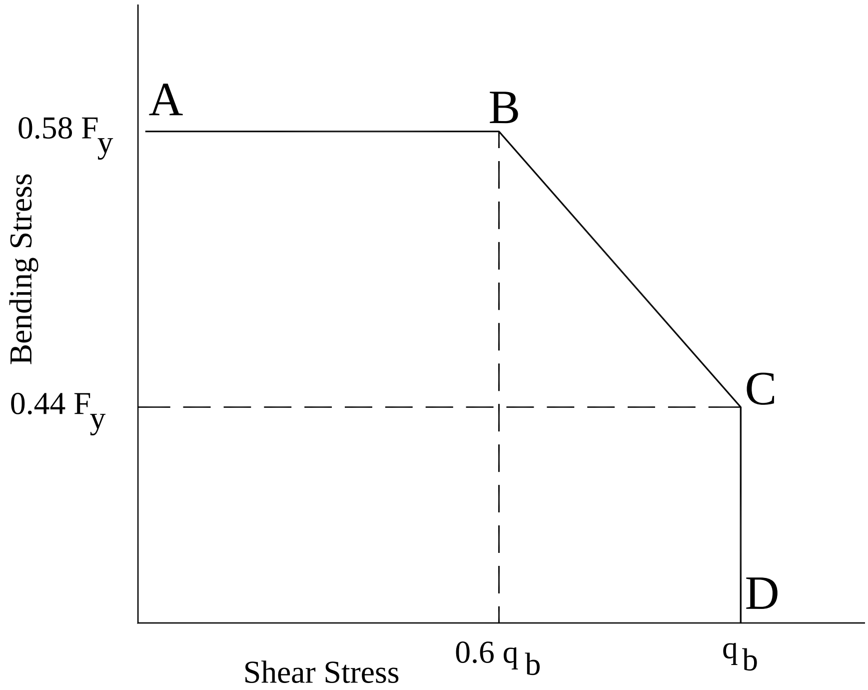


Fig. 5.25 Interaction between Shear and Bending

5.5 Flange Plate Curtailment:

Welded girders offer more flexibility than design with rolled sections. Since the total design moment varies along the girder span, flange plates of varying thicknesses, and sometimes of varying widths, may be butt welded to provide a section strength that closely approximates the variation in bending moment. Theoretical locations at which flange-plate thickness or width may be changed along the girder length can be determined as follows; Fig. 5.26(a):

1. The resisting moments of the girder with several selected flange plate areas are calculated.
2. The above values of the resisting moments are super-imposed on the graph of the total design moment. This plot is then used to determine the required length of each size flange plate.

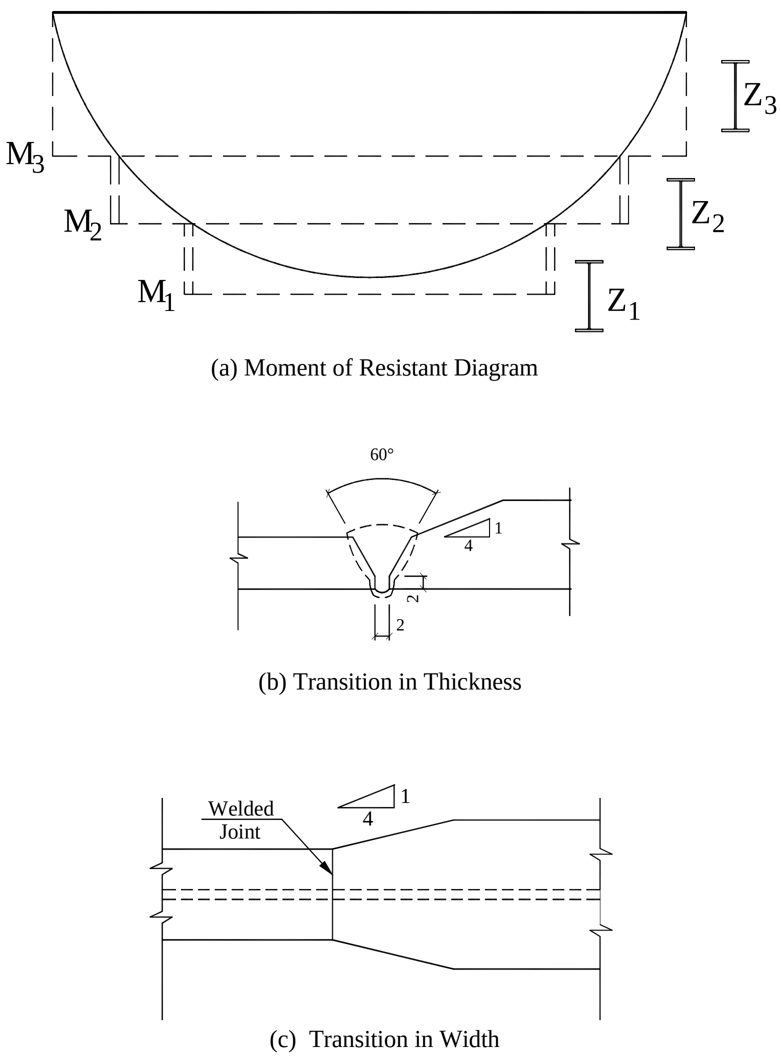


Fig. 5.26 Curtailment of Flange Plates

The actual changes in flange plate thickness or width are made near theoretical locations. Although a minimum steel weight results from such changes, an excessive number of changes should be avoided since the cost of making and testing the necessary butt welds increases the over-all cost of the fabricated girder. For a simple span, the flange is usually made from three plates of two sizes; a center plate covering 40 - 60 % of the span, and two plates butt-welded to the center plate.

When flange plates of different thicknesses are butt-welded, design codes require a uniform transition slope between the offset surfaces not exceeding 1 in 4, Fig. 5.26(b). If plates of different widths are joined, the wider plate must taper into the narrower plate with the same slope or with a radius of 60 cm, Fig. 5.26(c).

5.6 DESIGN DETAILS

Having designed the main girder to resist the action of applied loads, the next step in the design of plate girder bridges is to design various details needed to arrive at a complete bridge. These details include:

1. Connection between web and flange plates.
2. Stiffeners.
3. Splices
4. Lateral Bracings.
5. Bearings.

These details are governed in the next sections.

5.7 FLANGE-TO-WEB CONNECTION:

The connection between the flange plate and the web plate is usually executed using fillet welds on both sides of the web plate. This weld should be designed to transmit the horizontal shear flow between web and flange plate at any point along the girder plus any load applied directly to the flange.

Shear Effect:

The effect of horizontal shear flow between the web and the flange can be considered with reference to Fig. 5.27 as follows:

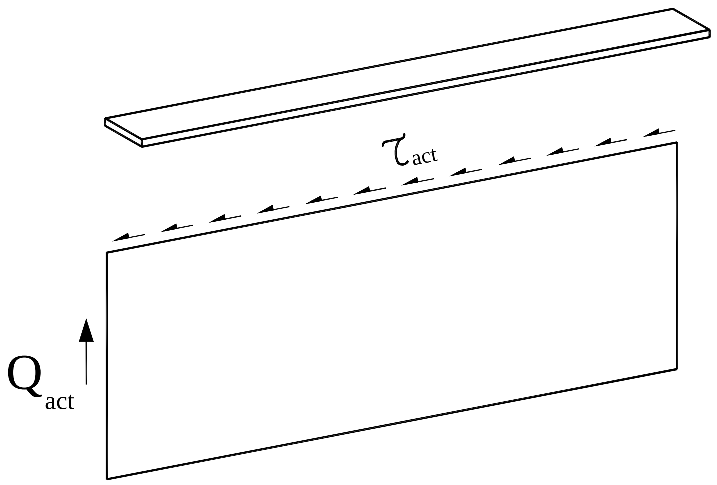


Fig. 5.27 Horizontal Shear Flow between Web and Flange

Horizontal shear/unit length = shear flow

$$\boldsymbol{\tau_{act} = Q_{act}\, S_f / I} \quad \textbf{(5.43)}$$

Where

Q_{act} = shear force,
S_f = first moment of area of flange about neutral axis,
I = moment of inertia about neutral axis.

If the allowable shear stress in welds is q_w, then the weld size s can be calculated from the equation:

$$\text{weld strength} = \boldsymbol{q_w * (2\, s) \geq \tau_{act}} \quad \textbf{(5.44)}$$

$$\text{i.e., weld size } \boldsymbol{s \geq \tau_{act} / 2\, q_w} \quad \textbf{(5.45)}$$

Direct Load Effect:

In deck bridges where the wheel loads are transmitted to the girder web through the direct contact between the girder flange and the web, the flange-to-web weld is also subjected to a vertical load in addition to the horizontal shear stress. The direct load in railroad deck bridges, where sleepers rest on the top flange, is taken as the train wheel load (12.5 ton) plus impact distributed over one meter. For flanges carrying ballasted decks, the train wheel load may be assumed distributed over 1.5 meter. In roadway deck bridges, the truck wheel load (10 ton) plus impact is distributed over a length of 1 meter.

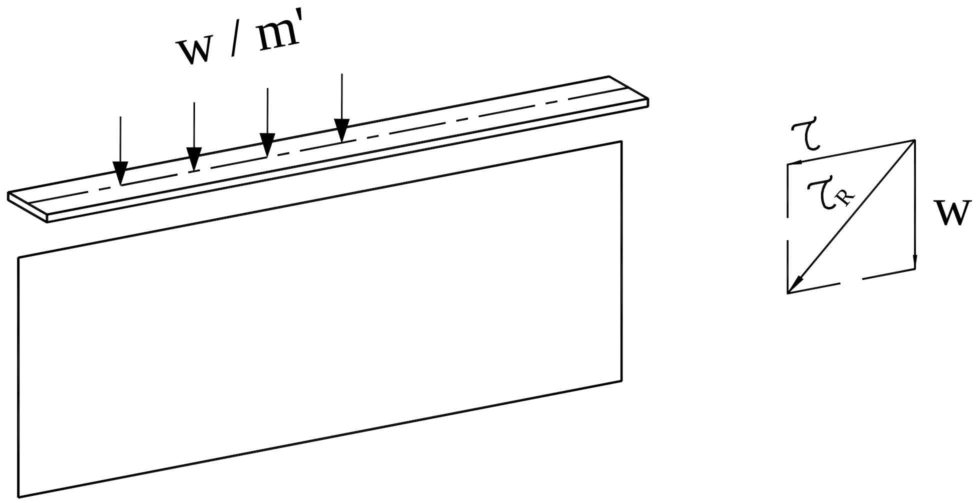


Fig. 5.28 Direct Load Effect on Flange-to-Web Weld

The effect of these external loads should be superimposed on the a.m. shear stresses. If the external direct load per unit length of flange is w, the resultant shear on the weld shall be, see Fig. 5.28:

$$\boldsymbol{\tau_R = \sqrt{\tau^2 + w^2}} \quad \text{..} \mathbf{(5.46)}$$

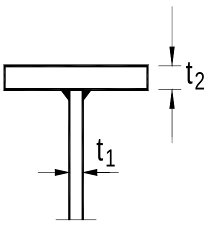
and the weld size is computed from**:**

$$\mathbf{s > \tau_R \;/\; 2\, q_w} \text{..} (\mathbf{5.47})$$

The calculated weld size (**s**) should satisfy the following requirements:

1. The maximum size of fillet weld should not exceed the thickness of the thinner plate to be welded.
2. The minimum size of fillet welds as related to the thickness of the thicker part to be joined is shown in the following table:

t (max. of t_1 or t_2) (mm)	Size s (mm)
≤ 10	≥ 4
10-20	≥ 5
20 - 30	≥ 6
30-50	≥ 8
50-100	≥ 10



Cover-plated sections:

For economic design, the cross-section of the main girder is usually changed along the bridge length according to the structural requirements.

A flange may comprise a series of plates joined end-to-end by full penetration welds. Three schemes can be used to accomplish changes in the flange plate areas:

a) varying the thickness of the flange plates, Fig. 5.26b
b) varying the width of the flange plates, Fig. 5.26c, or
c) adding cover plates at regions of high moment, Fig. 5.29.

Proper connection in the region of cover plate cut-off presents a some what special case of the previous procedure. Welds connecting a cover plate to a flange should be continuous and capable of transmitting the horizontal shear between the cover plate and the flange. The “theoretical end” of the cover plate is the section at which the stress in the flange without that plate equals the allowable stress. The "terminal distance" is the extension of the cover plate beyond the theoretical end. Welds connecting the cover plate to the flange within the terminal distance should be of sufficient size to develop the computed stress in the cover plate at its theoretical end. This distance can be calculated as follows; see Fig. 5.29:

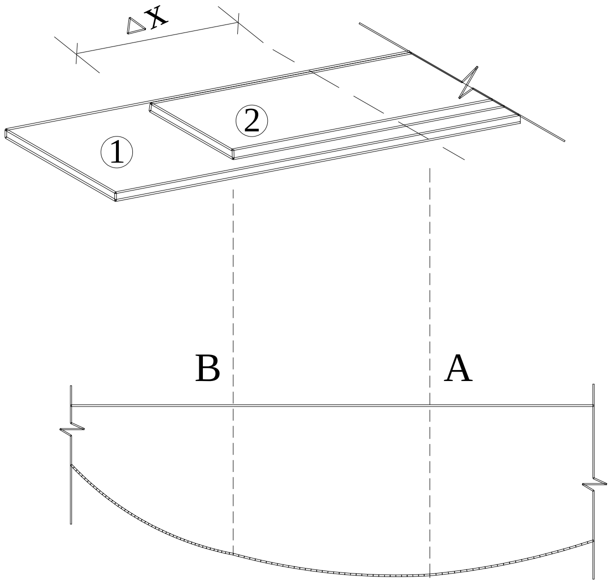


Fig. 5.29 Weld at Cover Plate End

Let point A be the theoretical end of the cover plate A_2 with a girder having a continuous flange A_1. The size of weld connecting the cover plate to the flange plate can be computed from shear flow considerations as:

$$\text{Horizontal shear / unit length} = q_c = Q \text{ x } S_c / I \quad \textbf{(5.48)}$$

where S_c= first moment of area of cover plate about neutral axis.

$$\text{Weld strength} = q_w \text{ x}(2\text{ s}) > q_c \quad \textbf{(5.49)}$$

$$\text{i.e., weld size } = s > q_c / 2\, q_w \quad \textbf{(5.50)}$$

Let Δx be the terminal distance of the cover plate extending from point A to point B. The shear force between the cover plate and the flange is equal to the resultant force in the cover plate, i.e.,

$$\mathbf{\Delta P_2 = P_2 = \mathit{f}_A * A_2 \leq q_w * s * (2\Delta x)} \quad \textbf{(5.51)}$$

i.e.
$$\mathbf{\Delta x = \frac{\mathit{f}_A\ A_2}{2\, s\, q_w}} \quad \textbf{(5.52)}$$

5.8 STIFFENERS:

In section 5.4, it was found that different types of stiffeners were needed to increase the buckling strength of plate girder webs. In particular, longitudinal stiffeners were used to increase the bend-buckling strength while transverse stiffeners were used to increase the shear buckling strength. In order for these stiffeners to effectively perform these functions, they should be adequately designed as shown in the following sections:

5.8.1 Longitudinal Stiffeners

Where required, $(\mathbf{(d/t) > 190/\sqrt{Fy}})$, a longitudinal stiffener, Fig. 5.30, should be attached to the web at a distance d/5 from the inner surface of the compression flange measured to the center of the stiffener when it is a plate or to the gage line when it is an angle. Longitudinal stiffeners are usually placed on one side of the web. They need not be continuous, and they may be cut at their intersection with the transverse stiffeners when both are provided on the same side of the web.

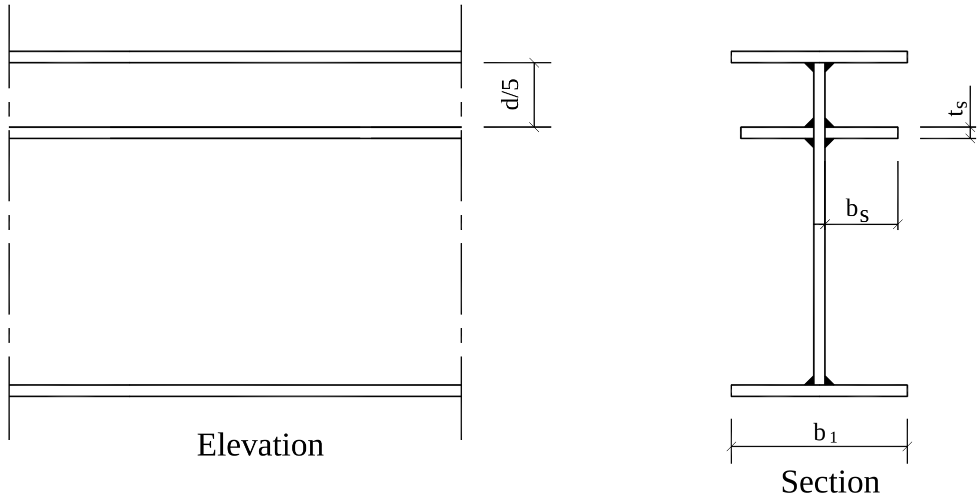


Fig. 5.30 Web Plate with Longitudinal Stiffeners

The primary function of the longitudinal stiffener is to increase the bend buckling strength of the web plate. To perform this function efficiently, a longitudinal stiffener must meet the following requirements:

1. Because the resistance to bend buckling is increased as a consequence of higher buckling mode owing to the presence of a longitudinal stiffener, it should be sufficiently stiff to maintain a longitudinal node in the buckled web. For this reason, the stiffener should be proportioned so that it has the following minimum value of its inertia:

$$\mathbf{I \geq 4d_w\, t_w^3} \quad \text{.......} \quad \mathbf{(5.53)}$$

where I = moment of inertia, cm^4, of longitudinal stiffener about the edge in contact with web,

If a second longitudinal stiffener is needed at the neutral axis; i.e., when (**$(d/t) > 320/\sqrt{Fy}$**), its inertia should not be less than $d_w\, t_w^{3}$.

2. To avoid local buckling of the stiffener, it must meet the width-thickness limit of non-compact compression elements; i.e., $b_s/t_s \leq 21 / \sqrt{Fy}$

3. The computed bending stress in the stiffener should not exceed the allowable bending stress for the stiffener steel.

5.8.2 Transverse stiffeners

Transverse stiffeners, Fig. 5.31, should be used where d_w / t_w exceeds the value given in Eqn. 5.22; i.e.

$$\left(\frac{d}{t}\right)_{lim} \leq 105 \ / \ \sqrt{F_y} \quad \text{...} \ \mathbf{(5.54)}$$

or when the actual shear stress exceeds the allowable shear stress given by Eqn. 5.29,5.30:

For $(d/t) \leq 159/\sqrt{Fy}$: $q_b = [1.5 - (d/t)\ \sqrt{Fy}\ / 212]\ [0.35\ Fy] \leq 0.35F_y$ … **(5.55)**

For $(d/t) > 159/\sqrt{Fy}$: $q_b = \{119 / [\ (d/t)\ \sqrt{Fy}\ ]\ \}\ \{0.35\ F_y\}$ ……….…… **(5.56)**

5.8.2.1 Cross Sections

Transversal stiffeners are usually fabricated of plates welded to the girder web. They may be used in pairs (one stiffener welded on each side of the web) with a tight fit at the compression flange. When a concentrated load is applied on the plate girder flange, transverse stiffeners in pairs are required to prevent crippling in the web immediately adjacent to the concentrated load. These stiffeners are designed as bearing stiffeners, see 5.8.3.

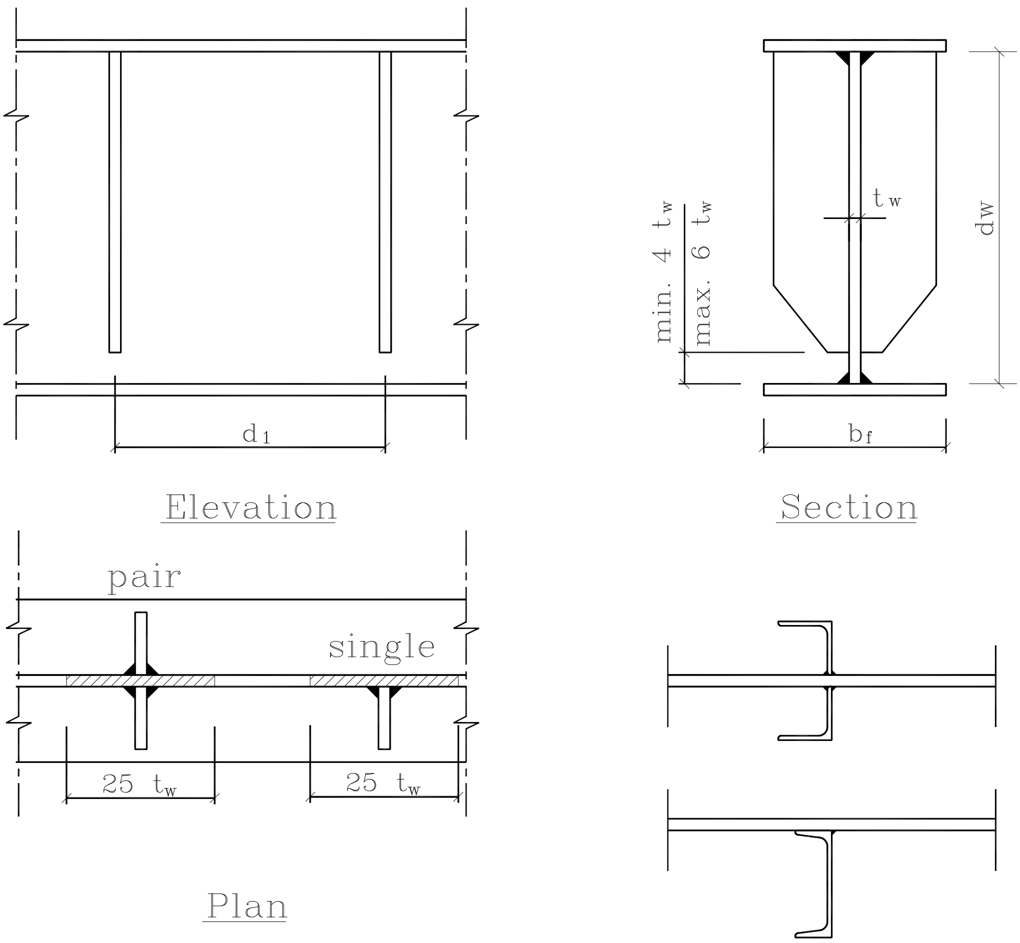


Fig 5.31 Transverse Stiffeners

Alternatively, transverse stiffeners may be made of single plates welded to only one side of the web plate. In this case they must be in bearing against the compression flange (to prevent its twisting) but need not be attached to the compression flange to be effective. When only single stiffeners are used, it is usual to place them on the inside face of the web for aesthetic reasons. In some cases a stiffener may be used as a connecting plate for a cross frame or a lateral support (see sec. 5.10 bridge bracings), which could result in out-of-plane movement in the welded flange-to-web connection. In such cases, attachment of the stiffener to the compression flange may be necessary and the connection should be adequately designed to transmit the lateral force developed at the connection. Transverse stiffeners need not be in bearing with the tension flange, but they should be terminated within a distance of four to six times the web thickness from the tension flange. Transverse stiffeners should not be welded to the tension flange to avoid fatigue problems, see sec. 5.7.

5.8.2.2 Design Considerations:

The primary function of the transverse stiffener is to increase the shear buckling strength of the web plate. To perform this function efficiently, the stiffener must meet the following requirements:

1. Stiffeners should project a distance $\underline{\mathbf{b_s}}$ from the web of:

   a) at least b_f / 4, where b_f is the flange width and

   b) at least (d_w / 30 + 5) cm for stiffeners on both sides of the web, or (d_w/30 + 10) cm for stiffeners on one side only, where d_w is the girder depth, cm.

2. To avoid local buckling of the stiffener, it must meet the width-thickness limit of compression elements; i.e**.,** $\mathbf{b_s/t_s \le 21 / \sqrt{Fy}}$

3. Intermediate transverse stiffeners should be designed to resist a force $\mathbf{C_s}$ equal to:

$$\mathbf{C_s = 0.65 \left(\frac{0.35\ F_y}{q_b} - 1\right) Q_{act.}} \qquad \mathbf{(5.57)}$$

   where q_b = allowable buckling stress , Q_{act} actual shear force at stiffener location. A part of the web equal to 25 times the web thickness may be considered to act with the stiffener area in the design of the intermediate stiffener.

4. Transverse stiffeners should be designed as a compression member with a buckling length of $0.8d_w$.

5. The connection between the transverse stiffener and the web should be designed on the stiffener design force such that the weld in either the upper or the lower thirds of the stiffeners should transform the design force.

Welding of the stiffener across the compression flange provides stability to the stiffener and holds it perpendicular to the web. In addition, such welding provides restraint against torsional buckling of the compression flange of the girder. For situations where the stiffener serves as the attachment for lateral bracing, the weld to the compression flange should be designed to transmit a force that equals 1 percent of the compression force in the flange.

Welding of stiffeners perpendicular to the tension flange should be avoided because a severe fatigue condition may be created, see section 3.3. In situations where the stiffener has to be connected to the tension flange, the weld is made parallel to the tensile stress direction as shown in Fig.5.32. Intermediate stiffeners should be terminated not closer than 4 times the web thickness from the tension flange. To prevent web crippling, however, the distance between stiffener-web connection and face of tension flange should not exceed 6 times the web thickness.

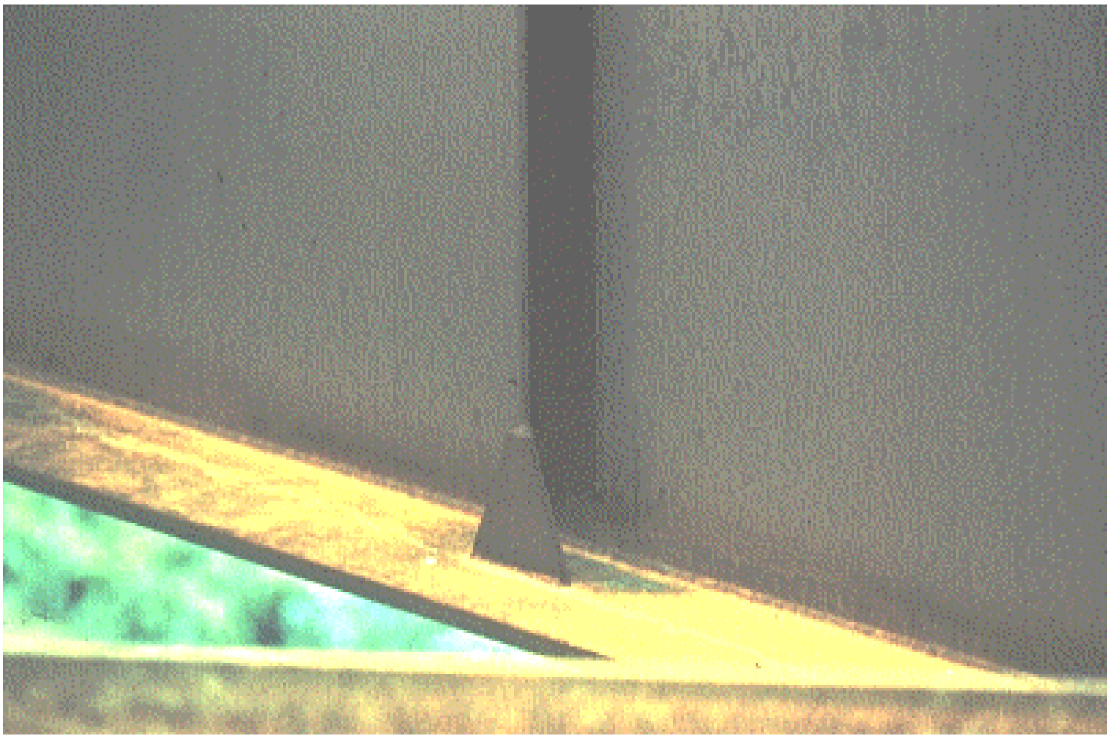


Fig 5.32 Welding of Transverse Stiffener to Tension Flange

5.8.3 Bearing Stiffeners

Bearing stiffeners, Fig. 5.33, are required where concentrated loads are to be transmitted to the web through flanges. Such locations are:

a) end bearings and intermediate supports of plate girders where the bottom flanges receive the reactions,
b) points of concentrated loads applied to the top girder flange.

The function of these stiffeners is to distribute reactions or concentrated loads into the web to create web shear. Additionally they prevent the possibility of local crippling and/or vertical buckling of the web.

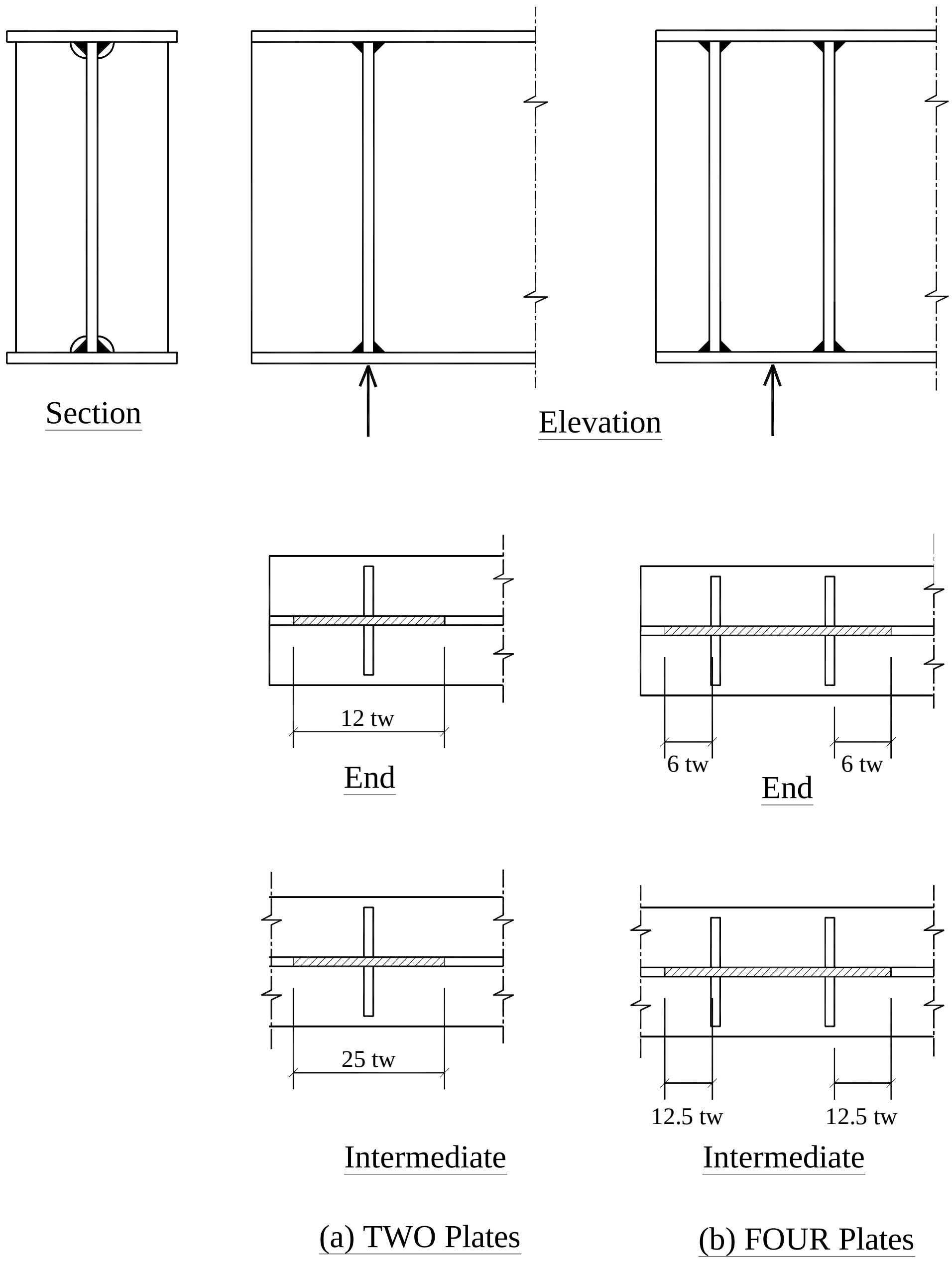


Fig 4.33 Bearing Stiffeners

To effectively perform these functions, bearing stiffeners should be sufficiently stiff against buckling. Therefore, it is preferred to have bearing stiffeners consisting of plates provided in pairs (i.e., placed on both sides of the web), and their connection with the web should be designed to transmit the entire reaction to the bearings. They must bear firmly on the flanges (i.e., fit tightly against the loaded flanges) through which they receive the reaction (or the concentrated load), and extend as far possible to the outer edges of the flanges. The ends of bearing stiffeners must be milled to fit against the flange through which they receive their reactions.

To provide space for continuous fillet welds at the girder web-flange connection, the side corner on one edge of the stiffeners must be clipped to ensure tight fit against the flange. This results in a reduced contact area between the stiffener and the loaded flange. This reduced contact area of the stiffener should be adequate to transmit the reaction without exceeding the permitted bearing stress on either the flange material or the stiffener material.

Design Considerations:

Bearing stiffeners are designed as concentrically loaded columns. A portion of the web extending longitudinally on both sides of the bearing stiffeners is considered participating in carrying the reaction. Depending on the magnitude of the reaction to be transmitted, the design may require two (one on each side of the web) or four or more stiffeners (symmetrically placed about the web). The cross sectional area of the fictitious column is defined as follows:

1. when two stiffener plates are provided, the column section consists of the two stiffener plates and a centrally loaded strip of the web equal to 12 t_w for bearing stiffeners at girder ends and 25 t_w for bearing stiffeners at interior supports

2. If there are four or more stiffener plates, the column section consists of the areas of all stiffener plates and a centrally loaded strip of the web plate whose width is equal to that enclosed by the stiffener plates plus a width equal to 12 t_w for bearing stiffeners at girder ends and 25 tw for bearing stiffeners at interior supports.

***a) Buckling Check*:** The actual compressive stress in the fictitious column should not exceed the allowable buckling stress of the stiffener cross section considered to act as a column with a buckling length of 0.8 d_w. The radius of gyration of the section is computed about the axis through the center of the web.

***b) Compression Check**:* The compressive stress in the stiffener plate alone should be less than the allowable stress in compression for the stiffener steel.

***c) Bearing Check*:** The calculated stress on the actual contact area between the stiffener and the bottom flange should not exceed the allowable bearing stress. According to ECP: $F_{bearing} = 2\ F_t$, where F_t is the allowable tensile stress of the material.

Connections of bearing stiffeners to the web should be designed to transmit the concentrated load, or reaction, to the web

5.9 SPLICES:

Apart from the simplest of bridges, with relatively short spans, the main girders of bridges are made up of elements connected together in the fabricating shop. For example, a plate girder is normally fabricated by welding together top and bottom flanges, web plates and stiffeners. Normally, as much of the fabrication as possible is carried out in the fabricating shop as shown in Fig. 5.34.

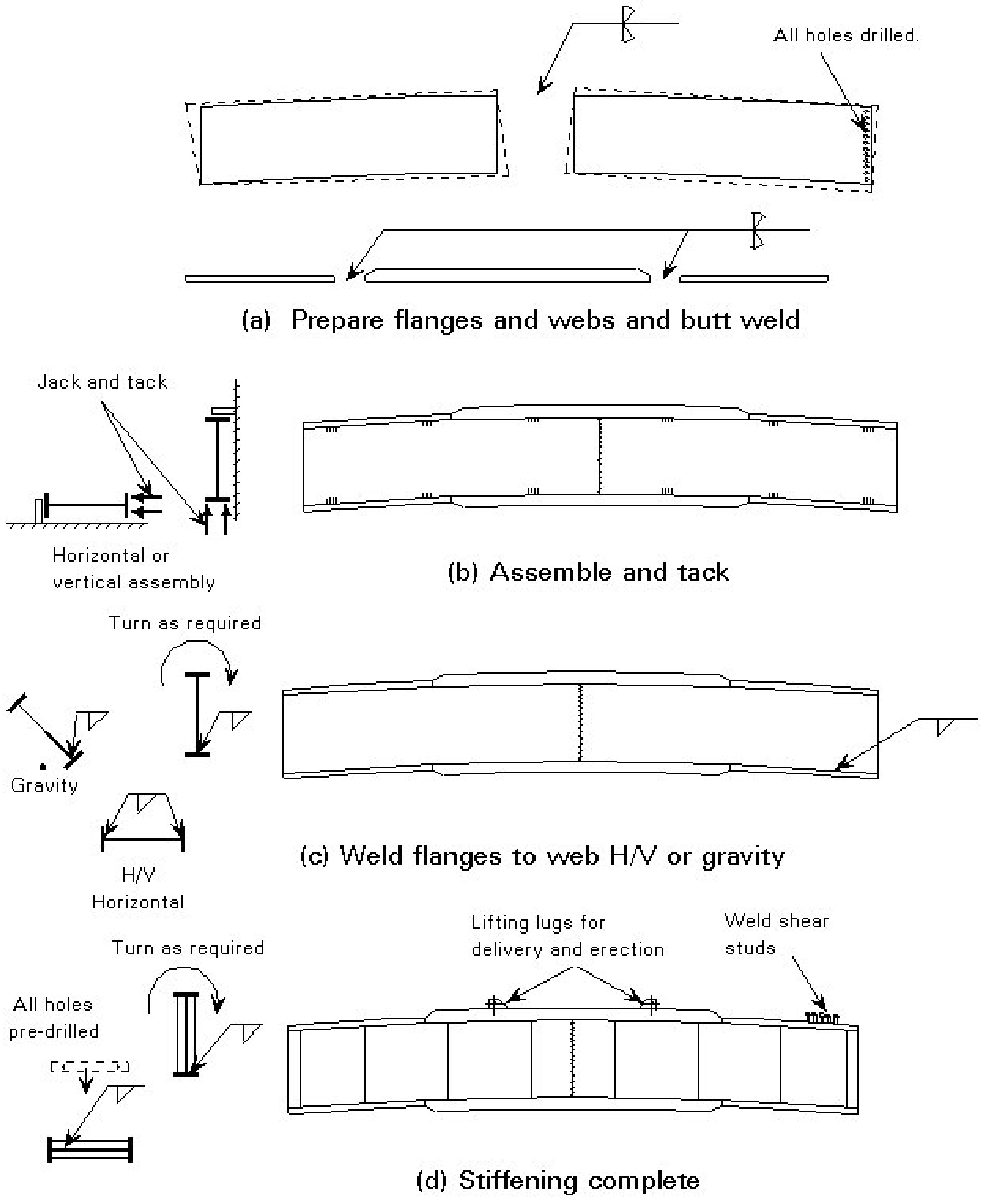


Fig. 5.34 Plate Girder Assembly Sequence

However, due to the reasons given below, most bridges consist of a number of sub-assemblies connected together at site. Consequently, site connections, referred to as splices, are required between sections of the main girders where these cannot be delivered to site and erected in one piece.

Splices for girders should be avoided whenever possible. However, there are conditions when splicing of girders is unavoidable. One is the available length of plates and shapes; another is the length limit imposed by the transportation facilities from the fabricating shop to the site of the structure. Occasionally, the capacity of the erecting crane may set the maximum weight of one piece to be handled. The maximum length of plates obtainable from local mills is 6 meters while the maximum length of rolled shapes is 12 meters. Transportation facilities vary greatly with local conditions. Where good highways lead from the fabricating shop to the site, special arrangement can be made to transport long and heavy pieces. Where direct railroad transportation is used, the length of the pieces is governed by tunnel and bridge clearances, especially on curves. Sometimes it is a matter of balancing the extra cost of splice against the additional cost of transporting heavier and longer pieces.

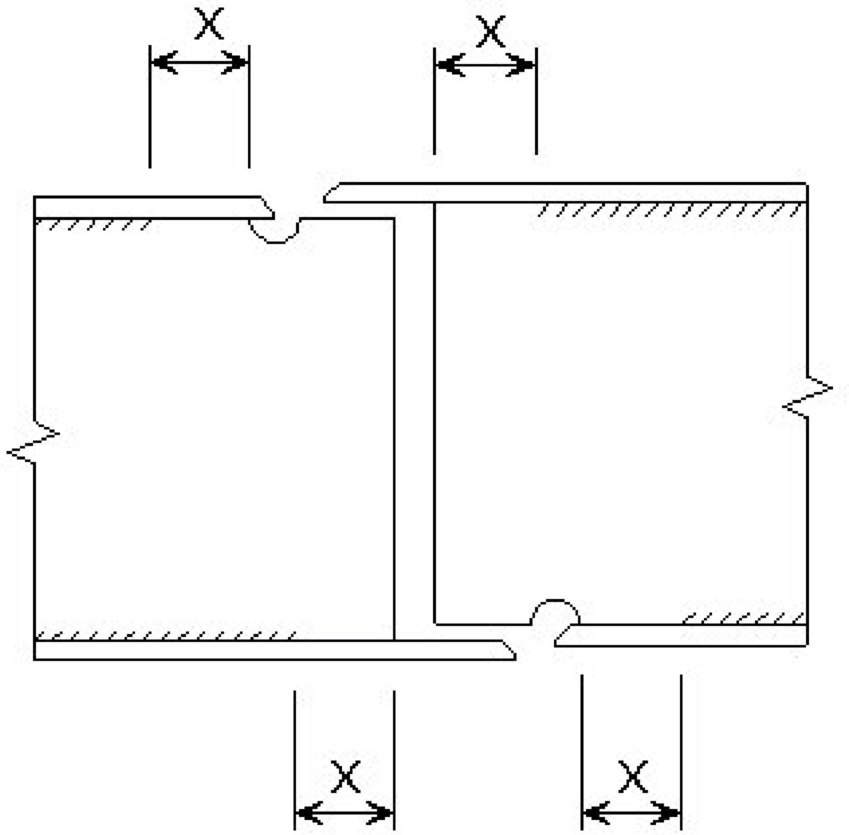
The location of splices has a major influence on the economics of the design, fabrication and erection of bridges. In addition, the detailing of splices influences the fatigue and corrosion resistance of a bridge.

The designer must always, from initial concept through design and analysis to final detailing of the bridge, keep the connections in mind. At all stages he must know where the connections will be, how they will be designed and detailed, how they will be fabricated and when they will be fitted together.

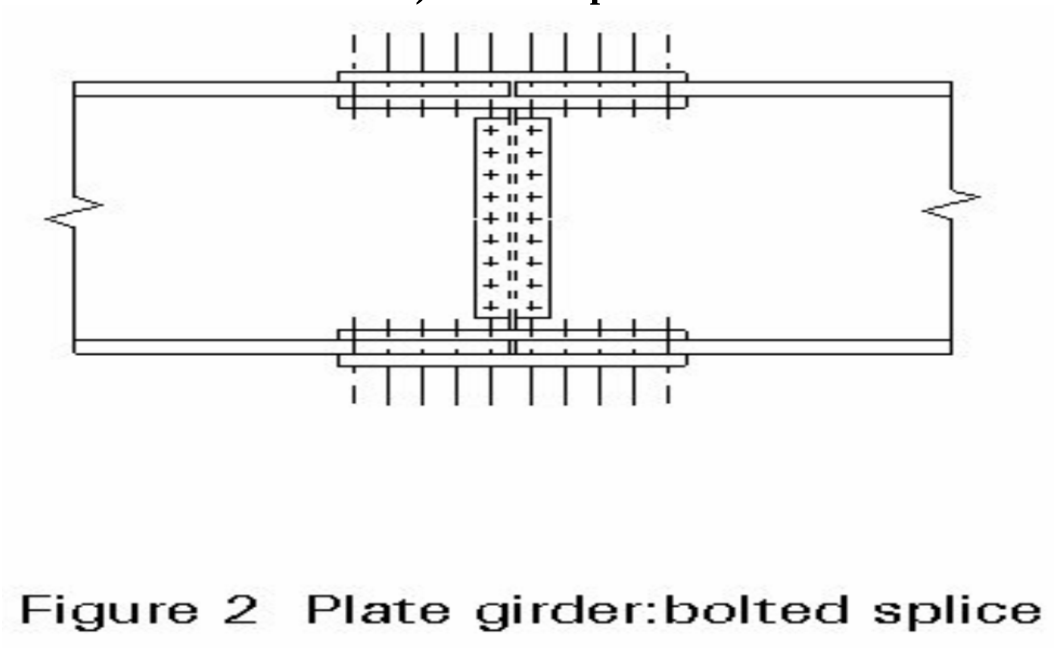
The relative position and orientation of the elements to be joined can make the difference between a straightforward, effective connection and one that is difficult to design, detail, fabricate and erect. It is for this reason that the connections should be considered at an early stage in the design process.

5.9.1 TYPES OF SPLICE

There are two basic methods of making splices. ***Welding***, using butt welds or fillet welds, and ***bolting***, see Fig. 5.35. Where the main elements of the splice can be connected together with full strength butt welds, the design is simple and the effect of any loss of section due to the bolt holes does not arise.



a) Welded Splice



b) Bolted Splice

Fig. 5.35 Plate Girder Splices

When making a decision as to whether welding or bolting is to be used, the following are some of the points that should be considered:

Aesthetics: Butt-welded connections are normally less obtrusive than bolted connections.

Access: Adequate and safe access is required for both methods of connection; but protection from wind and rain is also required for satisfactory welding.

Temporary support**:** The support of the member while the connection is being made has to be considered. This is particularly significant in a welded splice, where the location and alignment of the elements to be spliced must be maintained during welding. This often requires the use of temporary erection cleats and, if these are welded, the effect of the welding needs to be taken into account when making any fatigue checks (even if they are removed after erection).

Corrosion: Particular care is required to ensure that the corrosion protection prevents rusting between the plates in a bolted connection and that the weld area is properly cleaned before painting in a welded connection. Both types of connection should then perform adequately as far as resistance to corrosion is concerned.

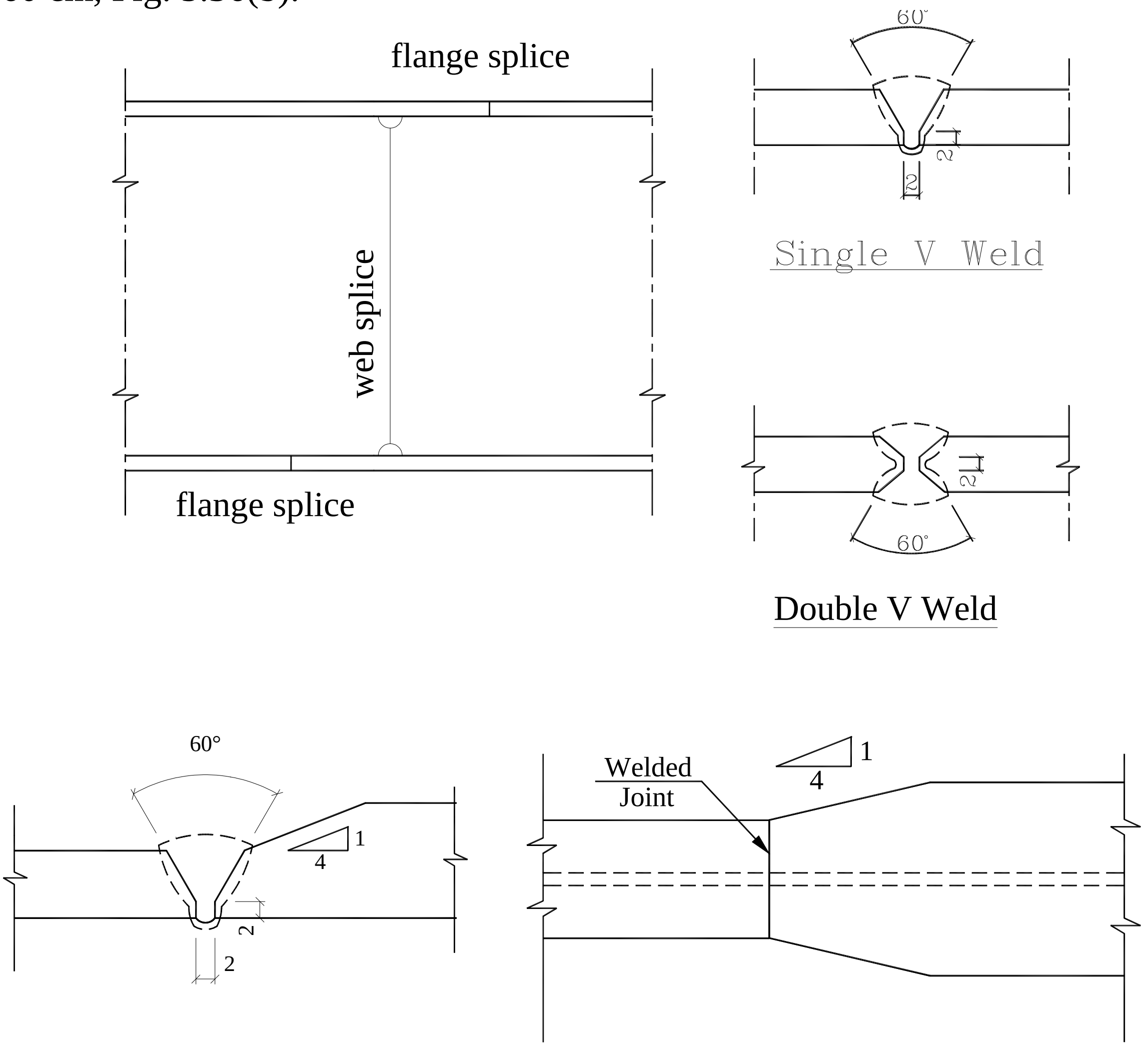
Details**:** Bolted cover plate splices take up additional space, compared with butt welded splices. This could be a problem, for example, where deck plates are fixed to top flanges, particularly when a relatively thin wearing surface is to be applied to the deck plates.

Cost**:** The cost of the various options should also be taken into account when making decisions regarding the type and position of connections.

5.9.2 Welded Splices:

Welded Splices are usually made in the fabricating shop and therefore are called ***Shop Splices***. The locations of these splices are usually dictated by the available plate lengths. Web and flange plates are usually spliced in the workshop by full penetration butt welds of the V-type, Fig. 5.36. For thicker plates, usually above 20 mm, a double V weld is used to reduce the amount of welding and to balance the welding on both sides and thus eliminating angular distortions.

In large girders, web and flange plates may be formed of plates of various widths or thicknesses that are butt-welded together along both transverse and longitudinal seams. When plates of different thicknesses are butt-welded, design codes require a uniform transition slope between the offset surfaces not exceeding 1 in 4, Fig. 5.36(a). If plates of different widths are joined, the wider plate must taper into the narrower plate with the same slope or with a radius of 60 cm, Fig. 5.36(b).



(a) Transition in Thickness (b) Transition in Width

Fig 5.36 Welded Splices

All details of welding procedures should be arranged to minimize distortion and residual stresses. All important welds, particularly field welds, should be inspected by one of the following weld inspection methods:

Table 5.3 Characteristics of Common Weld Inspection Methods

Inspection Method	Characteristics and Applications	Limitations
Visual (VT)	Most common, most economical. Particularly good for single pass.	Detects surface imperfections only.
Dye Penetrant (DPT)	Will detect tight cracks, open to surface.	Detects surface imperfections only. Deep weld ripples and scratches may give false indications.
Magnetic Particle (MT)	Will detect surface and subsurface cracks to ~ 2 mm depth with proper magnetization. Indications can be preserved on clear plastic tape.	Requires relatively smooth surface. Careless use of magnetization prods may leave false indications.
Radiographic (RT)	Detects porosity, slag, voids, irregularities, lack of fusion. Film negative is permanent record.	Detects must occupy more than ~ 1.2 % of thickness to register. Only cracks partial to impinging beam register. Radiation hazards.
Ultrasonic (UT)	Detects cracks in any orientation, Slag, lack of fusion, inclusions, lamellar tears, voids. Can detect a favorably oriented planar reflector smaller than 1mm. Regularly calibrate on 1 ½ mm dia. drilled hole. Can scan almost any commercial thickness.	Surface must be smooth, Equipment must be frequently calibrated. Operator must be qualified. Exceedingly coarse grains will give false indications. Certain geometric configurations give false indication of flaws.

5.9.3 Bolted Splices

Splices made in the field are called Field *Splices* and are usually made using bolts because of the difficulty sometimes encountered in field welding. The location of field splices is usually dictated by length limits imposed by the available transportation facilities, or by weight limits imposed by the capacity of the erecting cranes.

Untorqued, bearing bolts in normal (2mm) clearance holes are not generally used for splices in bridges. In most splices the deformation associated with slip into bearing would be unacceptable. To avoid the slip, fitted bolts, in close tolerance holes, or High Strength Friction Grip (HSFG) bolts are required, Fig. 5.37. Generally HSFG bolts are used, since this avoids the need to match and ream the holes. The pretensioning of the bolts also improves their fatigue life and prevents the nuts working loose due to vibration.

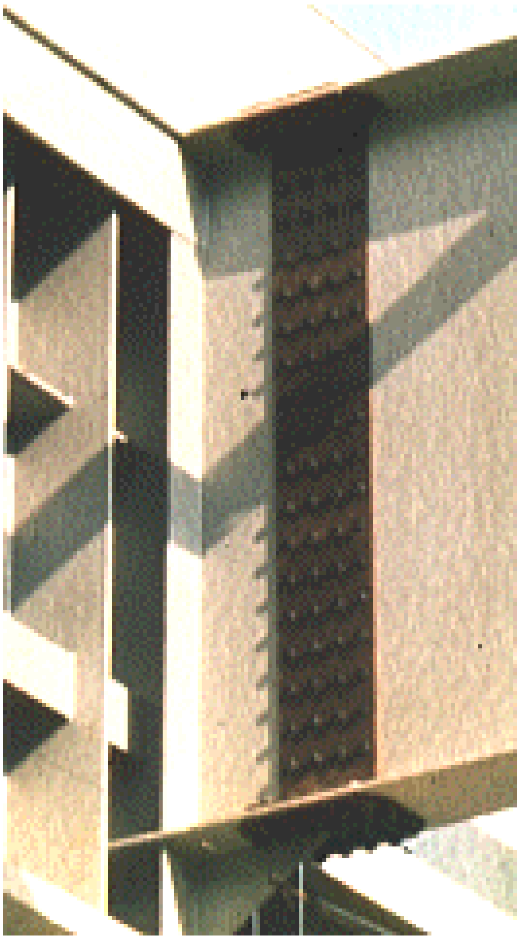


Fig. 5.37 Example of Bolted Splice

5.9.4 DESIGN

The most straightforward procedure for the design of a splice is to consider the load paths by which the forces are transmitted through the splice. The load paths must be sufficient to carry all the applied forces, moments and shears. The load paths must be complete and in equilibrium, i.e., there must be no weak or missing links. They should be as direct as possible.

Splices should be designed to carry the maximum bending resistance of the girder section and the actual shear force at the splice location.

The following points must be considered in the design and detailing of splices:

1- Care is required to ensure that the worst combinations of moments and forces that can occur at the splices are used for their design. They are not necessarily the moment and forces used for the design of the members. It follows that the moments and forces supplied by a computer program for the design of the members may not be sufficient for the design of the connections.
2- The centroidal axes of members (and elements of members) should intersect wherever possible. If it is not possible, the effects of any eccentricity should be taken into account in the design.
3- Wherever practicable, the centroidal axis of the splice material should coincide with the centroidal axis of the elements joined. If this is not possible, the effect of any eccentricity should be considered in the design.
4- When a section changes at a splice location, the smaller section should be used in calculations.
5- Avoid severe stress concentrations. This is particularly important where fatigue could be a problem.
6- When friction type bolts are to be used, adequate clearances must be provided to allow the use of suitable tightening tools.
7- When shims or packs are needed, for example, where there is a change of flange plate thickness, it is essential that the surfaces of the packs or shims comply with the requirements assumed for the faying surfaces in the design.
8- When the bridge girder is continuous, the splices are usually positioned near to where the inflection point (zero moment) would be if the bridge were subjected to uniform loading. The maximum moment (and shear) that the splice can be subjected to under the possible loading patterns must be determined.

5.9.4.1 Bolted Web Splice

The girder web transmits primarily shearing stresses, and web splices are most efficiently located at points of small shear, although practical requirements may dictate otherwise. In general, the shear force to be spliced in the web is much smaller than the shear capacity of the web. Most bolted web splices, except those for very heavy girders, are controlled by minimum dimension requirements rather than stress computations. For example, two splice plates are usually employed one on each side of the web; the splice plates must have not less than the minimum thickness; and must be extended the entire depth of the girder from flange to flange. In all cases the net section through the splice plates must provide the required area to resist the shear and the required section modulus to resist the bending moment safely.

When a web splice is to transmit a pure shear **Q** (without any moment at the splice location), the bolts should be designed to resist a force **Q** applied at the centroid of the bolt group, Fig. 5.38. This means that the bolts should be designed to transmit load **Q**, with an eccentricity ***e***. When the depth d of the web is much greater than the eccentricity ***e***, the design is often made for a direct shear **Q**, neglecting the eccentricity. In this case for a given bolt diameter, the bolt resistance ***R*** is known, and the required number of bolts is simply **Q/*R***.

If, In addition to shear **Q**, there is a moment ***M*** at the splice section, then the portion of the total moment carried by the web must be transmitted by the web splice. This moment ,M_w , is obtained as: $M_w = M\ I_w / I_g$, where I_w and I_g = net moments of inertia of web and girder, respectively.

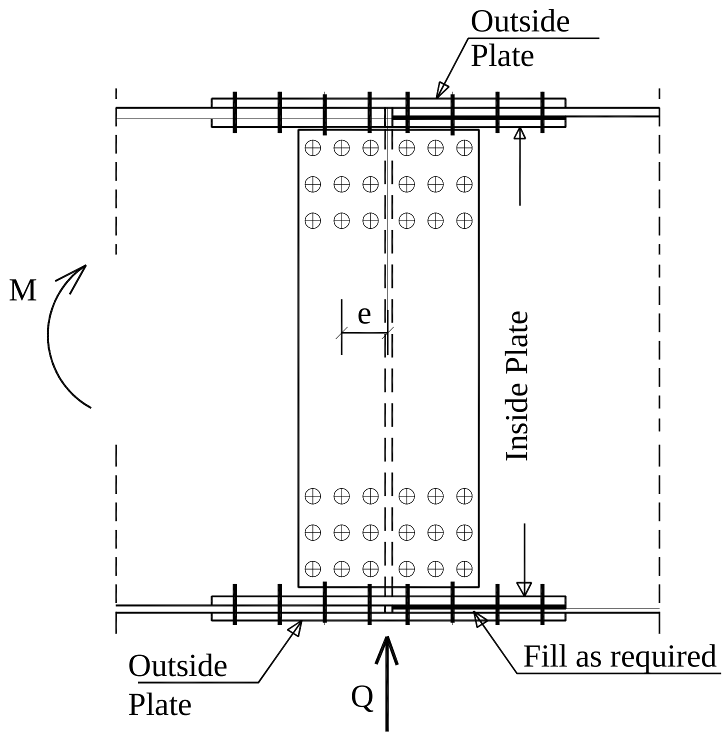
The splice is then designed to resist a shear force **Q** plus a bending moment $M_s = Q * e + M_w$. A check is then made of the resulting force in the extreme bolt and the bending stress in the splice plate.

5.9.4.2 Bolted Flange Splice

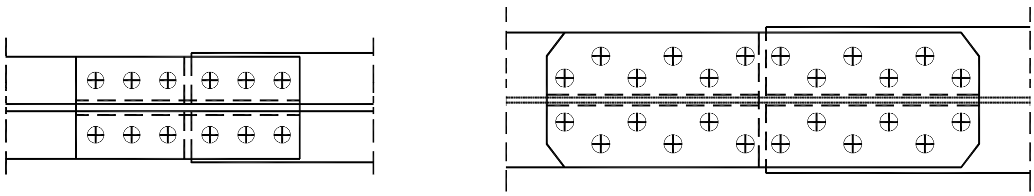
Girders flanges carry normal stresses due to bending moment, and therefore whenever possible, for economy of material, flange splices should be located at sections other than those of maximum moment. The flange splice is designed to carry that portion of the total design moment not carried by the web splice. The flange splice plates transmit the moment couple across the splice in axial tension or compression, and into the girder flange by double shear on bolts, see Fig. 5.38b:

Bolt Design: Flange splice moment = $M_f = M - M_w$ .... **(5.58)**
Flange couple C = **T** $= M_f / d$ .... **(5.59)**
Number of splice bolts = T / R_{least} .... **(5.50)**
where R_{least} is the bolt resistance.

Splice Plate Design:
The net section of the splice plates is designed to carry the flange force T.



Splice Elevation



(a) One Gauge Line

(b) Two Gauge Lines

Flange Splice

Fig 5.38 Bolted Splice of Plate Girder

5.10 BRIDGE BRACINGS

A bridge is actually a ***space structure*** that not only carries the vertical gravity loads to the supporting piers and abutments, but also resists:

a) Transversal loads caused by wind, seismic and centrifugal loads,
b) Longitudinal loads caused by braking and thermal effects.

The analysis and design of the bridge is usually simplified by breaking it down into planar and linear components, such as stringers, cross girders, main girders and bracing systems. The effect of vertical loads on bridge elements has been presented in the preceding sections. In this section, the effect of transversal loads due to wind on bridge elements is presented.

5.10.1 Transmission of Wind Loads

The horizontal wind pressure on the bridge is assumed to be transmitted to the bridge supports using suitable systems of bracings. In general it may consist of Upper, Lower, and Transversal bracing as shown in Fig. 5.39. The wind load is assumed to be carried to the bridge supports as follows:

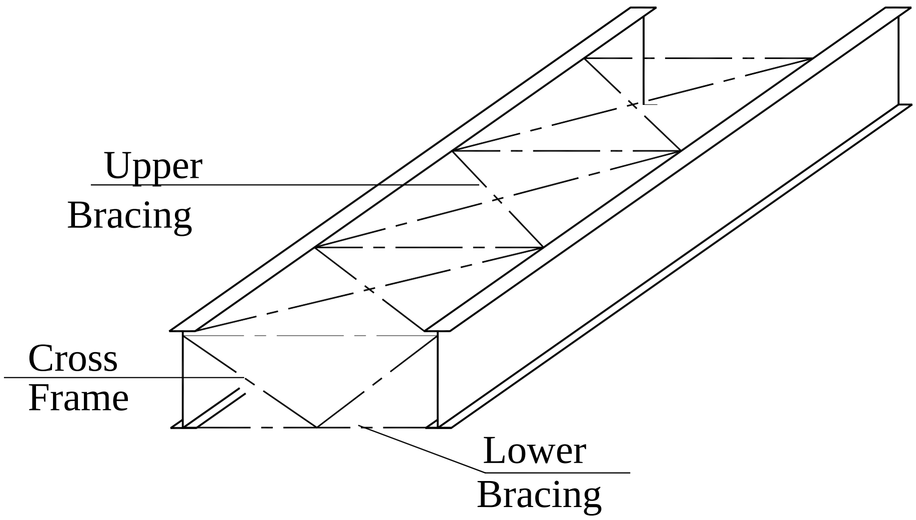
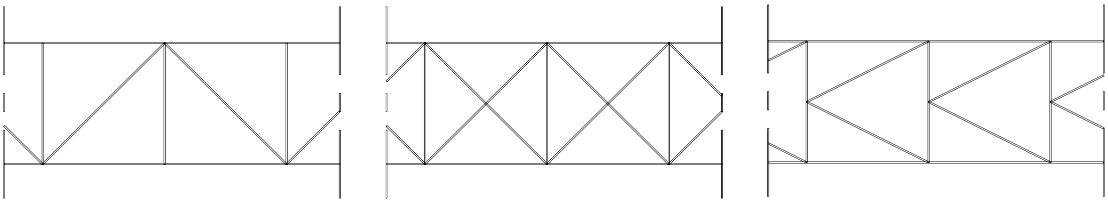


Fig. 5.39 Bridge Bracings

5.10.2) In deck bridges

a) The wind load on the upper half of the web of the exterior girder as well as that on the live load on the bridge is assumed to be carried by a horizontal bracing truss in the plane of the top flange to the span ends. The flanges serve as the chords of the lateral bracing truss, and are connected together by the cross girders plus a system of diagonal members. The diagonal members may be single or double diagonals, or may be of the K- type, see Fig. 5.40. In a deck bridge provided with a deck slab, the slab may be assumed to act as a horizontal diaphragm transmitting wind loads to the span ends. In this case the bracing truss is needed only temporarily during erection before the slab hardens.



(a) single diagonals (b) double diagonals (c) K-bracing

Fig. 5.40 Lateral Bracing Truss Systems

b) The wind load on the lower half of the web of the exterior girder of a deck bridge is usually much smaller in value than that on the top flange (being the unloaded chord) and thus may not need a complete lateral truss. Instead, wind load on the bottom flange may be transmitted to the upper plane using:

a) intermediate cross frames (Fig. 5.41),
b) intermediate inverted U-frames (Fig.5.42), or
c) intermediate diaphragms (Fig. 5.43)

In addition, these intermediate systems facilitate erection and serve also to brace the compression flange of the girder. According to ECP 2001, lateral bracing of the compression flange of deck girders should be designed for a transverse shear in any panel equal to 2.0 % of the total axial stress in the flange in that panel, in addition to the shear from specified lateral forces.

Intermediate cross frames and diaphragms should be spaced at intervals up to 8 meters. They should be placed in all bays. Cross frames should be as deep as practicable. The angle of cross frame diagonals with the vertical should not exceed 60 degrees.

In order to transmit the end reactions of upper bracings to the bridge supports, end cross frames are provided at the bridge ends and over interior supports.

5.10.3) In Through bridges

Neither cross bracings nor top lateral bracing can be used in most cases of through plate girder bridges. Furthermore, the top flange is subjected to compression in regions of positive moments and therefore must be braced to prevent its lateral buckling. Lateral bracings are normally located near the bottom flanges. These flanges thus also serve as the chords of the lateral bracing truss, and are connected together by the floor beams plus a system of diagonals. In such regions, the top compression flange should be stiffened against lateral deformation with solid web ***knee brackets*** as shown in Fig. 5.44. The brackets should be attached securely to the top flanges of the bridge cross girders and to stiffeners on the main girders. They should be as wide as clearance permits and should be extended to the top flange of the main girder.

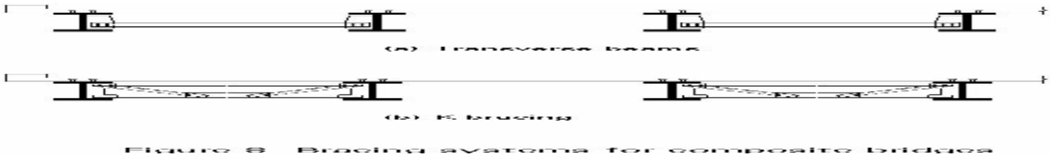


Fig. 5.41 Cross Bracing for Deck Bridges - Intermediate Cross frames

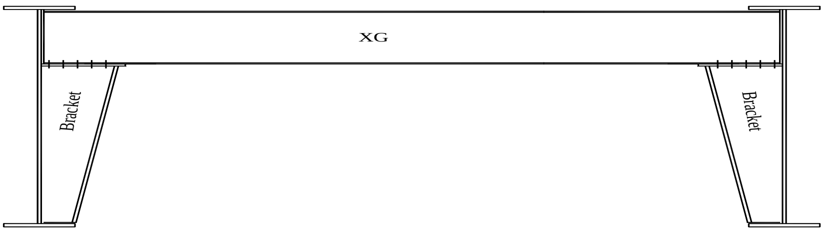


Fig. 5.42 Cross Bracing for Deck Bridges - Intermediate U- frames

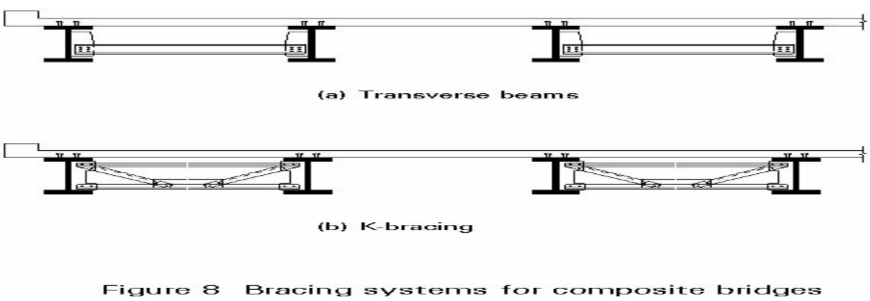


Fig. 5.43 Cross Bracing for Deck Bridges - Intermediate Diaphragms

These knee brackets are designed to carry the share of wind loads on the bridge main girder and the moving live load (truck or train). When the bracket is also used to support the compression flange against lateral torsional buckling, it should be designed to carry additionally a stability force that is equal to 2 % of the flange compression force.

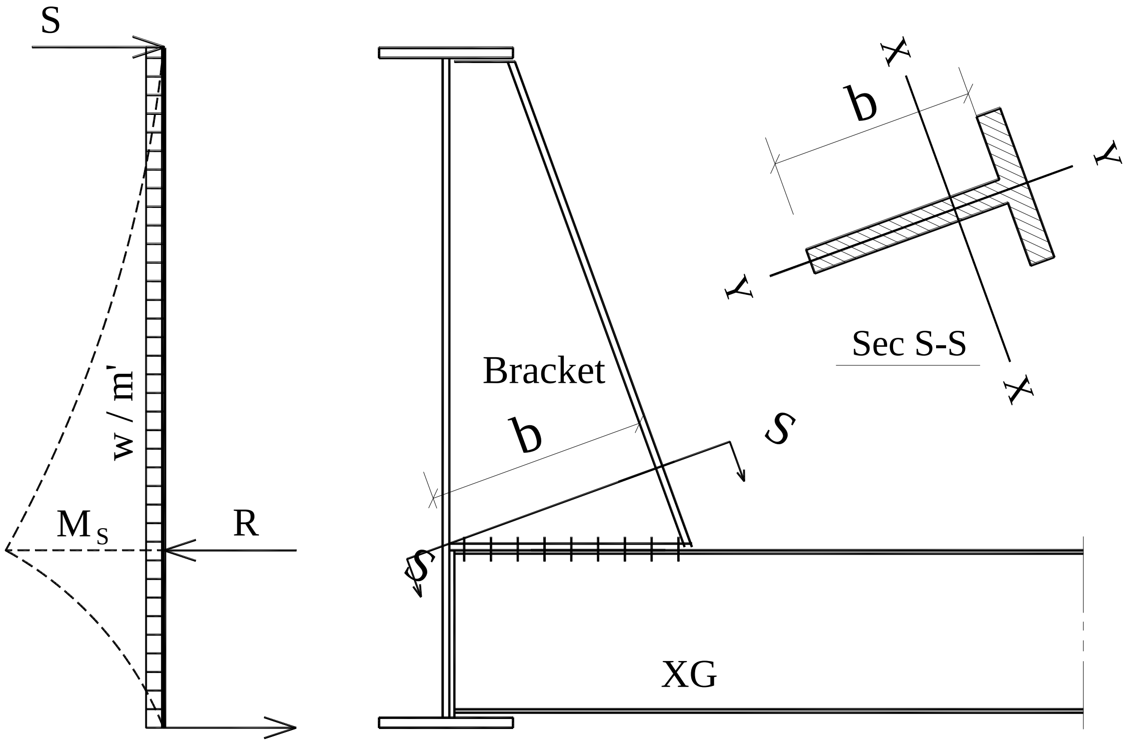


Fig. 5.44 Knee Bracket for Through Bridges

5.11 BRIDGE BEARINGS

Bearings are needed in bridges to fulfill the following functions:

1- transfer forces from one part of the bridge to another, usually from the superstructure to the substructure;

2- allow movement (translation along and/or rotation about any set of axes) of one part of the bridge in relation to another;

The main sources of movements in a bridge are due to temperature changes and axial and bending strains arising from applied loads. In general, it is not recommended to fully restrain a bridge against temperature movements. This becomes obvious if we consider a steel plate girder of cross sectional area 500 cm^2. If this girder is subjected to a 30^{o} C rise in temperature and is restrained from expanding axially, an axial stress of $E\ \alpha\ \Delta T = (1.2*10^{-5})*2100*30 = 0.756\ t/cm^2$ is induced in the girder. The corresponding restraining force required is 0.756*500 = 378 ton. Neither the girder nor its supporting structure can carry such a force. Movements caused by bending strains arise from the rotation of the member around the hinged bearing that is always located at a distance from the neutral axis of the member.

To achieve the required degrees of freedom of movement, a complete bearing usually consists of several components, each permitting a particular movement, the sum of which is the total freedom required. This can be achieved by using any of the following bearing types:

5.11.1 TYPES OF BEARINGS

Bearings may be classified according to their ***deformation behavior*** into three basic types: a) fixed bearings,
b) hinged bearings,
c) expansion bearings.

a) A *fixed-end bearing* completely restrains the member end from translation and rotation. It is capable of supplying a vertical and a horizontal reaction plus a restraining moment. Considering the expense of fixing a heavy steel member at the ends, the use of such a bearing is usually limited to sites having very strong rocky soils. Typical applications of this type of bearings are found at supports of arch bridges and sloping leg frames bridges.

b) A *hinged bearing* will permit rotation of the member ends, and this is usually accomplished by a pin, see Fig. 5.45. Hinges carrying heavy vertical loads are normally provided with lubrication systems to reduce friction and ensure free rotation without excessive wearing.

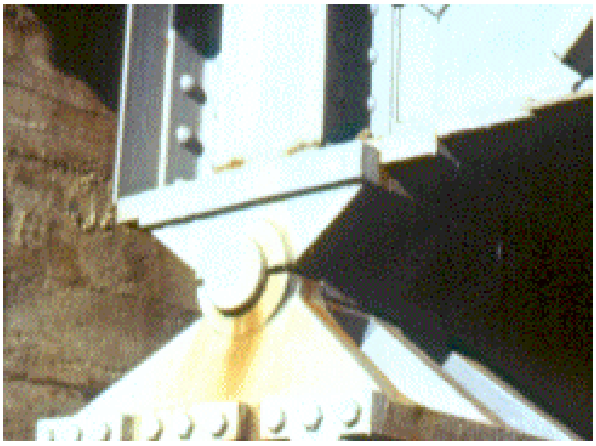


Fig. 5.45 Examples of Hinged Bearing

c) ***Expansion bearings*** permit movement as well as rotation of the superstructure. They are usually provided in two forms: the sliding type and the rolling type. ***Sliding type*** bearings are used only for short spans and small loads since they cause high friction forces between the sliding plates. ***Rolling-type*** bearings achieve their translational movement by using cylindrical rollers, see Fig. 5.46.

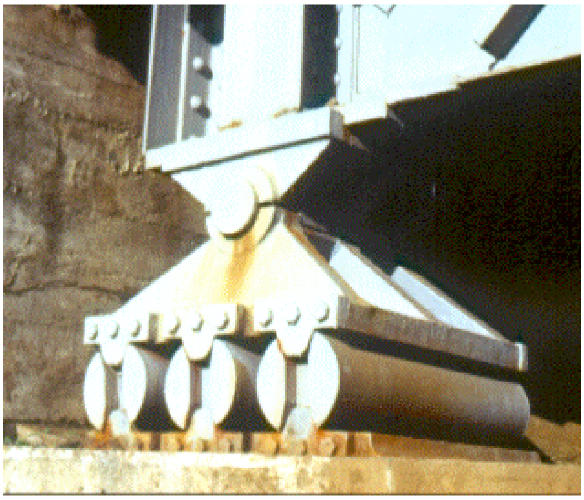


Fig.5.46 Roller Bearings

Bearings may also be classified according to the ***material*** used in their fabrication into:

a) steel bearings,
b) elastomeric bearings,

A brief description of each of these types is given next.

5.11.2 STEEL BEARINGS

i) Roller Bearings

Roller bearings consist essentially of one or more steel cylinders between parallel upper and lower plates, see Fig. 5.47. Gearing or some other form of guidance should be provided to ensure that the axis of the roller is maintained in the desired orientation during the life of the bearing. Roller bearings with a single cylinder can permit translation parallel to the longitudinal bridge axis and rotation about a horizontal axis in the transversal direction. Multiple cylinders on the other hand require another element such as a rocker or a knuckle bearing to permit rotation.

ii) Rocker Bearings

Rocker bearings consist primarily of a curved surface in contact with a flat or curved surface and constrained to prevent relative horizontal movement, see Fig. 5.47. The curved surface may be cylindrical or spherical to permit rotation about one or more axes. Rocker bearings on their own do not permit translation and are usually used at the fixed end of a bridge to complement roller bearings. They can also permit rotation by the rolling of one part over another.

iii) Knuckle Pin Bearing

Knuckle pin bearings consist of a steel pin housed between an upper and a lower support each having a curved surface which mates with the pin, see Fig. 5.47. Transversal lateral loads may be transmitted by flanges on the ends of the pin. This type of bearing permits rotation by the sliding of one part on the other.

iv) Leaf Pin Bearing

Leaf bearings consist of a pin passing through a number of interleaved plates fixed alternatively to the upper and lower bearing plates, see Fig. 5.47. They permit only rotational movements, but can be used in conjunction with roller bearings to provide rotation and translation.

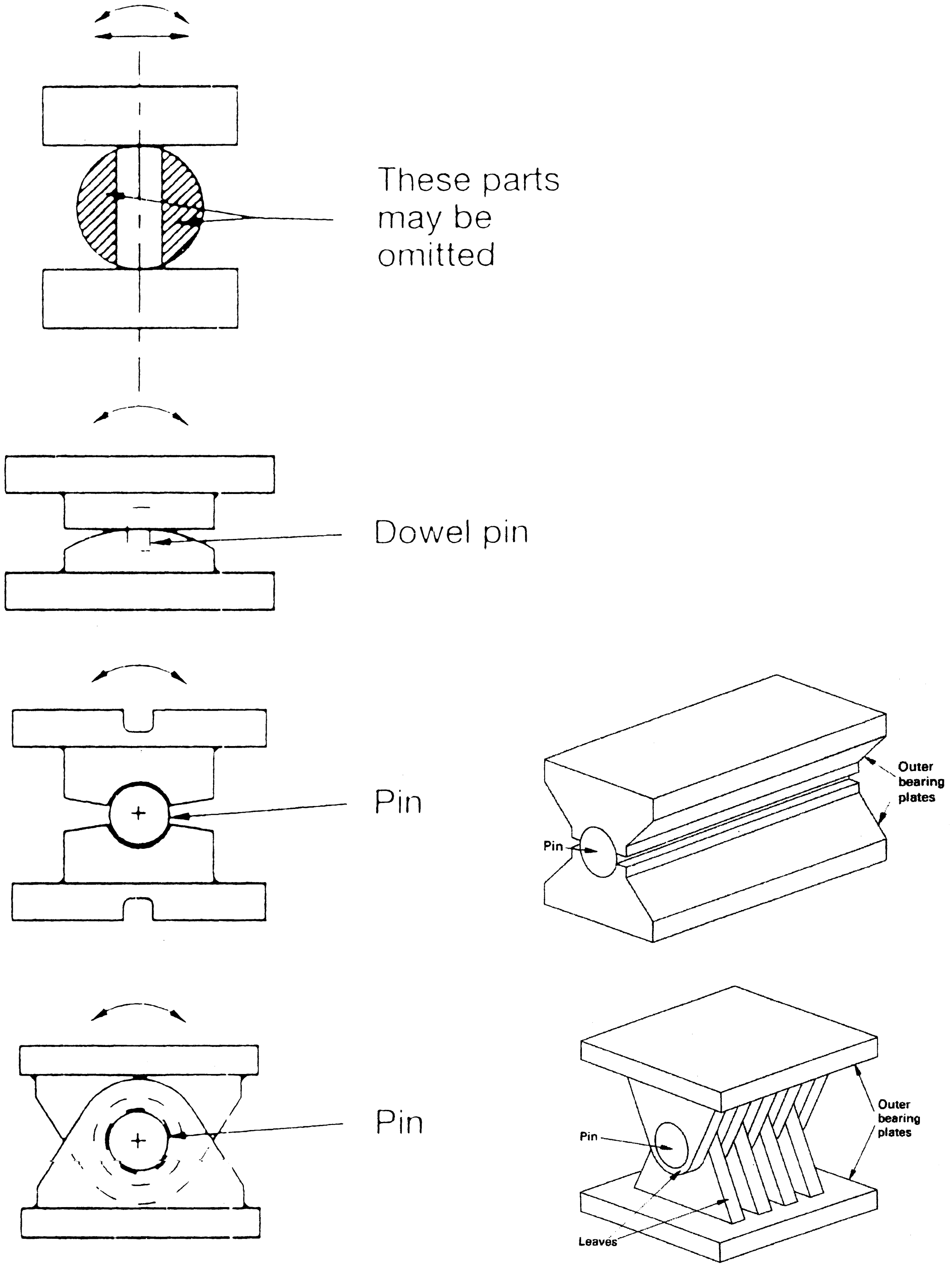
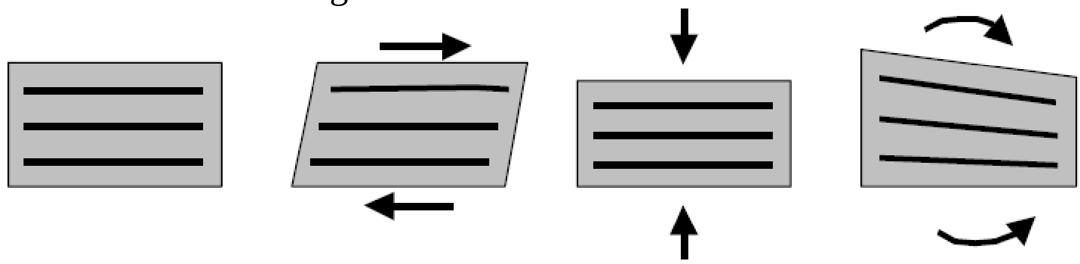


Fig. 5.47 Steel Bearings: Roller , Rocker, Knuckle Pin, and Leaf Pin

5.11.3 ELASTOMERIC BEARINGS

The main component of elastomeric bearings is a rubber pad that distributes the loads from the superstructure to the substructure and uses its material flexibility to accommodate the rotation and longitudinal movement of the superstructure. Translational movement is accommodated by shear in the elastomer, one surface of which moves relative to the other. Rotational movement is accommodated by the variation in compressive strain across the elastomer as shown in Fig. 5.48.



a) Original b) Shear c) compression d) rotation

Fig. 5.48 Deformation of Elastomeric Bearing Under Load

The rubber used is either natural rubber or synthetic rubber (neoprene). Because of their relative simplicity and minimal fabrication effort, elastomeric bearings are now widely used in new bridges.

Elastomeric bearings are available in two basic forms;

1- ***Plain elastomeric pads*** which are single unreinforced pads of elastomer of relatively thin section;

2- ***Reinforced elastomeric pads*** comprising one or more layers of elastomer bonded to reinforcing plates in sandwich form. Two main types of reinforcements are used:
   i) steel,
   ii) polytetraflouroethylene (PTFE) also known as Teflon.

A **steel reinforced elastomeric** bearing consists of discrete steel thin plates strongly bonded between adjacent layers of elastomer as shown in Fig. 5.49, and 5.50. The design of this type of bearings consists of finding the plan dimensions, number of elastomeric layers and their corresponding thicknesses, and steel plate thicknesses. Because these calculations depend largely of the properties of the rubber used, the design of these bearing types is usually taken from their manufacturer's certified design tables.

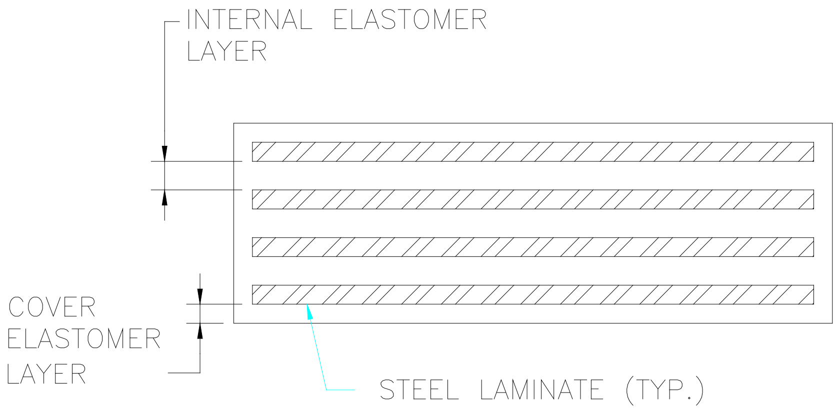


Fig. 5.49 Steel Reinforced Elastomeric Pad

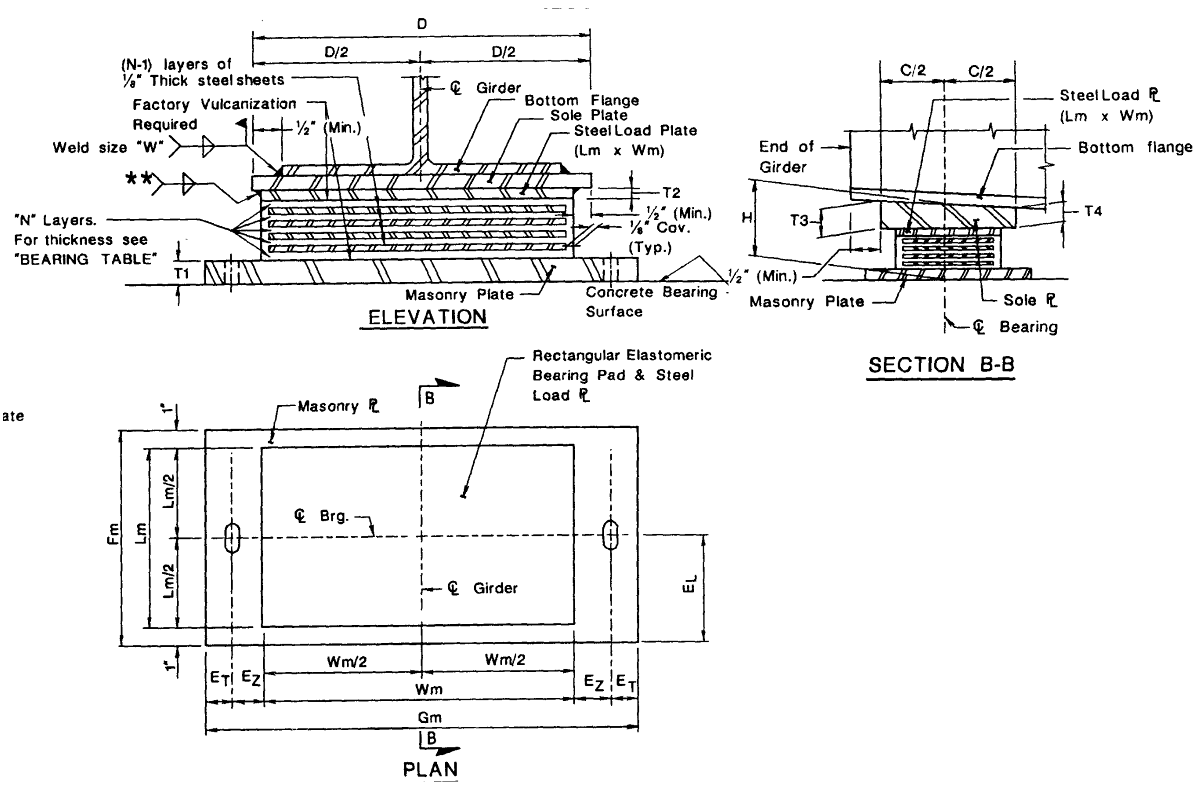


Fig. 5.50 Steel Reinforced Elastomeric Bearing

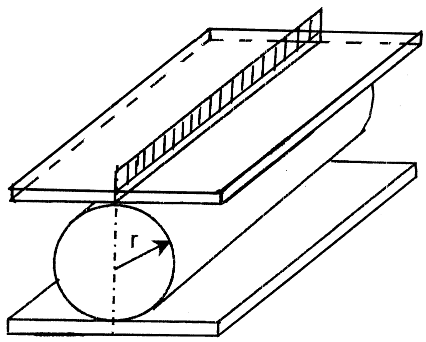
The other type of reinforced elastomeric bearings uses PTFE which is a fluorocarbon polymer, one of a group of plastics remarkable for their extreme chemical resistance, wide range of working temperatures and low coefficient of friction. These features make PTFE an ideal material for bearings. A **PTFE** bearing incorporates fabric pads with PTFE-stainless steel sliding interface to permit large translational movements. The design of these bearings also depends largely on the properties of the fabric used, and therefore is usually taken from their manufacturer's certified design tables.

5.11.4 DESIGN OF STEEL BEARINGS

5.11.4.1 ROLLER BEARINGS

i) Roller Design:

The maximum contact stress (f in t/cm^2) between a roller and a flat surface is given by the following *Hertz formula:*



$$\mathbf{f} = \sqrt{\frac{\mathbf{EV}}{\mathbf{2\pi r(1-\upsilon^2)L}}} = \mathbf{0.423}\sqrt{\frac{\mathbf{EV}}{\mathbf{L\,r}}} \qquad \textbf{(5.61)}$$

Where V = reaction in tons
r = radius of roller, cm
E = modulus of elasticity of steel, t/cm^2
υ = Poisson's ratio of steel
L = roller length, cm

Since the contact stress is confined and limited to a small area, it is permissible to use a high allowable stress, even exceeding the ultimate tensile strength of the material. For fixed, sliding and movable bearings with one or more rollers, the allowable contact stresses shall be as given below when the surface of contact between the different parts of a steel bearing is a line:

Material	**Allowable Contact Stress (t/cm^2)**
Cast Iron **Cl 14**	**5.00**
Rolled Steel **St 44**	**6.50**
Cast Steel **CST 55**	**8.50**
Forged Steel **FST 56**	**9.50**

Thus for bearing rollers made from structural steel St 44, resting on flat plates, using f = 6.5 t/cm^2, E = 2100 t/cm^2, and υ = 0.30 the above equation gives:

$$\mathbf{V = 0.0550\ d * L} \qquad \textbf{(5.62)}$$

where d is the diameter of roller in cms.

For other materials the following values are used:

Material	Allowable Reaction V(ton)
Rolled steel St 37	**0.040 d*L**
Cast Steel CST 55	**0.095 d*L**
Forged Steel FST 56	**0.117 d*L**

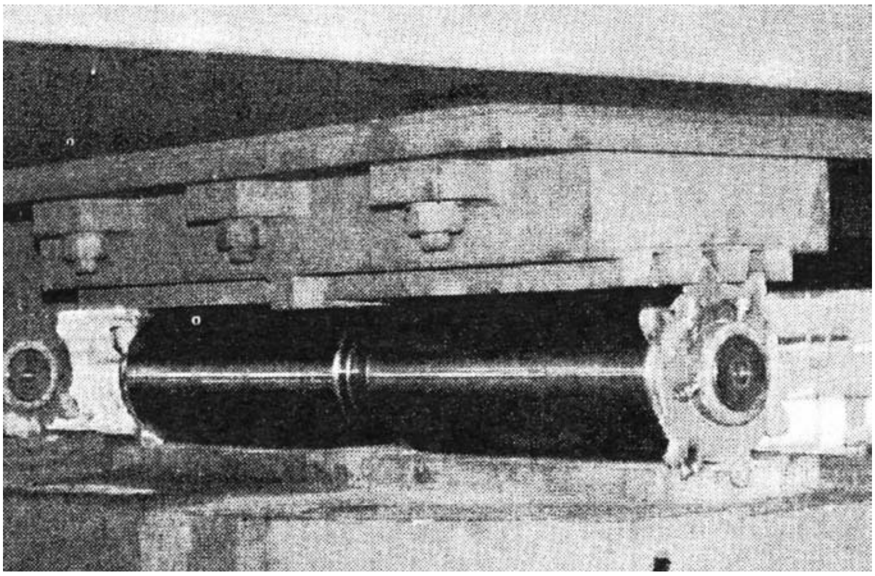


Fig. 5.51 Roller Bearing with Single Roller

It is recommended to use a single roller bearing made of special high tensile alloy steels, Fig. 5.51. However, bearings containing multiple cylinders of normal quality steels can be used, see Fig. 5.52. In bearings consisting of only one roller, the round surface accommodates the rotation as well as the longitudinal movement. With two or more rollers, an independent pin must be provided to allow end rotation of the bridge due to bending deflection, see Fig. 5.52. Furthermore, for bearings employing more than two rollers, the maximum permitted design loads given above for single rollers should be reduced by 20 % to allow for uneven loading of the rollers caused by dimensional differences.

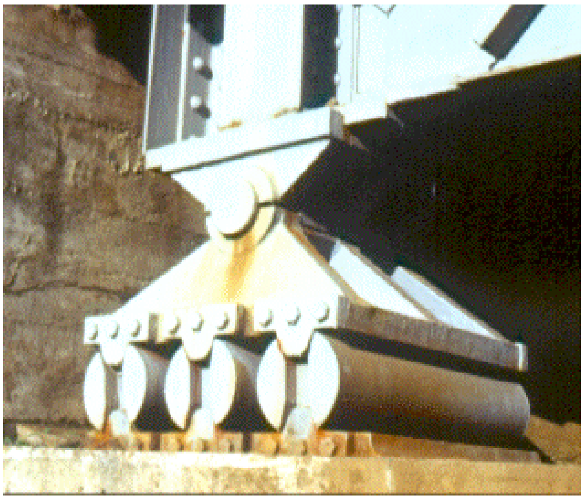


Fig. 5.52 Roller Bearing with Multiple Rollers

To save space between rollers, they can be flat sided, as shown in Fig. 5.53. Such rollers should be symmetrical about the vertical plane passing through the centre and the width should not be less than one-third of the diameter or such that the bearing contact doesn't move outside the middle third of the rolling surfaces when the roller is at the extreme of its movement.

ii) Base Plate Design:

The rollers are seated on a base plate which distributes the vertical load to the concrete abutment or pier. The area of this plate is computed from the allowable bearing stress on the concrete which is 70 kg/cm^2 for concrete C250 and 110 kg/cm^2 for concrete C350. The anchor bolts connecting the base plate to concrete are designed to transmit any transversal or longitudinal frictional forces resulting from movements. In some rare cases these bolts are designed to carry tension when the bearing is subjected to negative reactions.

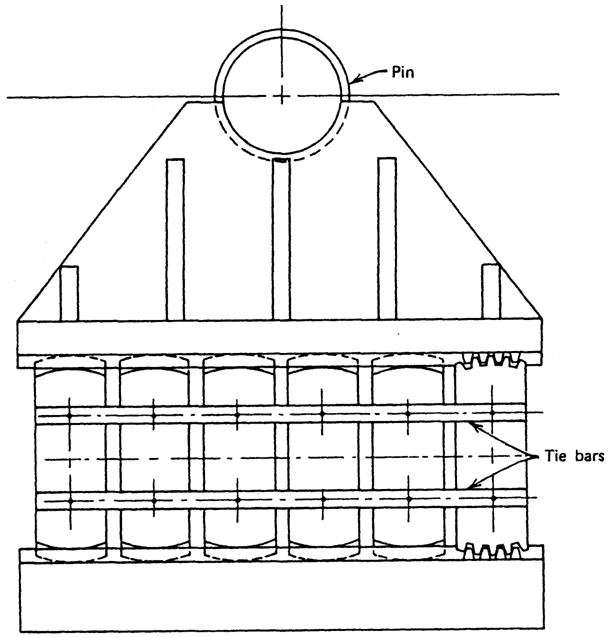


Fig. 5.53 Roller Bearing with Flat Sided Rollers

5.11.4.2 HINGED BEARINGS

The design of hinged bearings is similar to that for a roller bearing except that the contact stress used for the pin design is computed from Hertz formula for the case of bearing between two cylinders. If the pin is made of cast steel, the diameter is given by:

$$d = \mathbf{1.334\ V/L}$$

where d = diameter of pin in cm,
V = vertical load in ton,
L = length of pin in cm.

An example of a hinged bearing is shown on Fig. 5.54

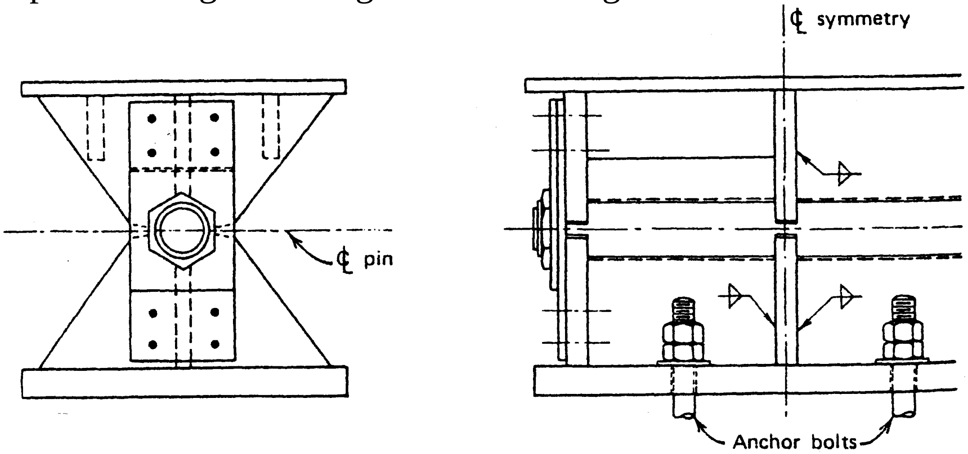


Fig. 5.54 Hinged Bearing

5.12 DESIGN EXAMPLE

5.12.1 GENERAL

The following example illustrates the application of the design principles presented in this chapter to the design of a two-lane plate girder roadway bridge. The span measures 27 m between the centers of bearings. The bridge cross section provides for a clear roadway having two 3-m-wide traffic lanes in addition to 1.00 m wide median and two 1.5 m side walks. The bridge is to be designed according to the Egyptian Code of Practice ECP2001 using steel grade ST. 52. An elevation, plan, and a cross-section of the bridge is shown in Fig. 5.55. The bridge deck consists of a 22 cm reinforced concrete slab covered by an 8 cm asphalt wearing surface. The deck is carried by four main girders spaced at 1.75 meters center-to-center.

The straining actions on an intermediate main girder due to dead loads and live loads plus impact at the critical sections are shown in the following table:

Action / **Load Case**	**At Support**		**6 m from support**		**Mid section**	
	Q (t)	M (m.t.)	Q (t)	M (m.t.)	Q (t)	M (m.t.)
Dead Load DL1	62	0	35	250	0	385
Add. Dead Load DL2	18	0	10	75	0	115
Live Load LL+I	100	0	60	460	25	700
Sum	180	0	105	785	25	1200

In this chapter, the main girder shall be designed as non-composite. A composite design of the main girder is presented at the end of chapter 6.

5.12.2) Main Girder Design

i) Girder depth, d:

An estimate of the girder depth is obtained from Eq. 5.12 as:

$$\mathbf{d = (0.25 \sim 0.3)\sqrt[3]{\frac{M}{F_b}}}$$

Assuming $F_b < 0.58\ F_y \cong (0.58 \times 3.6) = 2.1\ t/cm^2$, the required girder depth is

$$\mathbf{d = (0.25 \sim 0.3)\sqrt[3]{\frac{1200}{2}} = 2.10 \rightarrow 2.5\,m}$$

Use **d = 2.25** cm

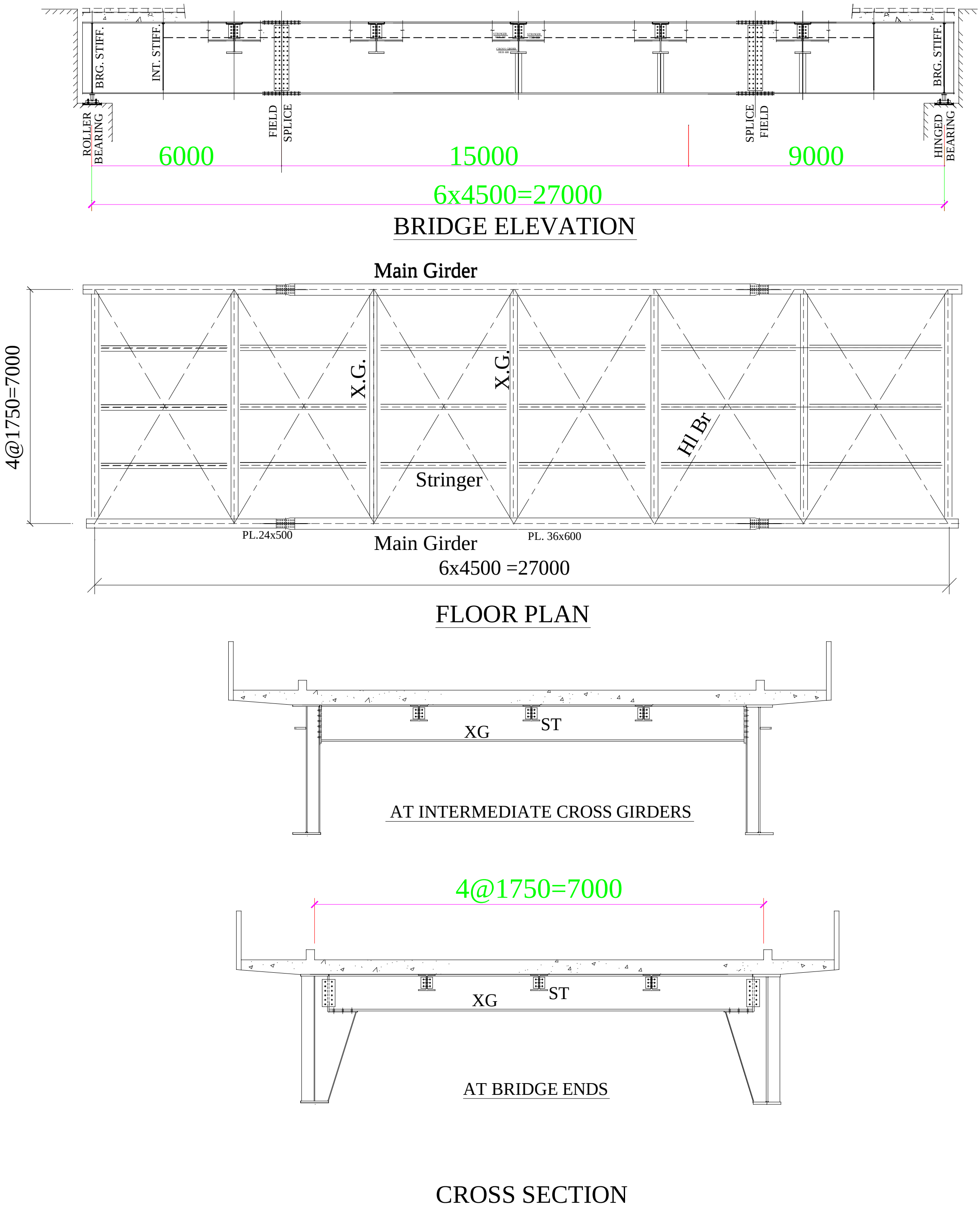


Fig. 5.55 Bridge Arrangement for the Design Example

ii) Web thickness, tw:

According to section 5.4.3, transverse stiffeners may be omitted if the actual shear stress does not exceed the allowable shear stress given by Eqns. 5.40, 5.41. Usually d/t > 1.59 $\sqrt{F_y}$ gives:

$$q_b = (119 / (d/t) \sqrt{\mathbf{F_y}}) (0.35 F_y)$$

With the actual shear stress given by $q_{act} = Q / (d * t)$, the minimum thickness for a web without transverse stiffeners is obtained from:

$$t^2 = Q / (41.65 \sqrt{F_y})$$

Substituting Q = 180 at support gives:

$$t^2 = 180 / 41.65 \sqrt{3.6}) = 2.27 \quad \text{i.e., } t = 1.51 \text{ cm}$$

Either use t = 16 mm (next even integer) without transverse stiffeners, usually an uneconomic solution, or a smaller value t = 14 mm with transverse stiffeners. Stiffener spacing d_1 is controlled by:

a) cross girder spacings = 4.50 m
b) aspect ratio $\alpha = d_1/d = \sim 1$

A suitable value for d_1 would be 2.25 m

iii) Check of web buckling due to shear:

Using transversal stiffeners at a distance d_1 = 2.25 m, then

Aspect ratio**:** $\alpha = d_1 / d = 225/225 = 1$

Buckling Coefficient $K_q = 4 + 5.34 / \alpha^2 = 9.34$

Plate Slenderness
$$\lambda_q = \sqrt{\frac{F_y / \sqrt{3}}{K_q\,(1898)\left(\frac{t}{d}\right)^2}} = \frac{d/t}{57.34}\sqrt{\frac{F_y}{K_q}}$$

$$= \frac{225/1.4}{57.34}\sqrt{\frac{3.6}{9.34}} = 1.74 > 1.2$$

Buckling Shear Stress: $\mathbf{q_b = \dfrac{0.9}{1.74} \times (0.35 \times 3.6) = 0.652\ t/cm2}$

Actual Shear Stress:

$$\mathbf{q_{act} = \frac{180}{225 \times 1.4} = 0.571\ \ t/cm^2\ <\ q_b\ (O.K.)}$$

i.e., Web Plate is safe against buckling due to shear at support

Since shear decreases away from support, the location where the transverse stiffeners are not needed can be found from the unstiffened web equation:

$$t^2 = Q / (41.65\ \sqrt{F_y}\ )$$

Substituting t = 1.4 cm gives Q = 154 t. This value is located at ~ 2 m from the support so that the transverse stiffener is only needed between the support and the first cross girder. Note that transverse stiffeners are always used at cross girder locations where the concentrated reaction of the cross girder is transmitted to the main girder.

iv) Flange Plate

From Eqn. 5.7:

$$A_f\ = (M / (F_b\ d)) - A_w / 6$$

$$= \mathbf{\frac{1200}{2 \times 2.25} - \frac{225 \times 1.4}{6} = 214.167\ \ cm^2}$$

Assume flange width $b_f = (0.2 \sim 0.3)\ d\ = (48 \sim 72)$ cm

Use $b_f = 60$ cm and calculate the required flange thickness as:

$$t_f = 214.167/\ 60 = 3.56 \text{ cm}$$

Provide two 600 × 36 mm flanges,

Check the b/t ratio for ***compression flange local buckling*** acc. to Eqn. 5.25 for st. 52:

$$b_f / 2f_t < 21 / \sqrt{3.6}\ = 11$$

actual $b_f / 2t_f = 600 / (2\times 36) = 8.333 < 11$ O.K. for non-compact flange

Check d/t ratio for ***web buckling due to bending*** acc. to Eqn. 5.27 for st. 52:

$$d / t < 190 / \sqrt{3.6} = 100$$

actual $d / t = 225 / 1.4 = 160.714 > 100$ i.e., web is <u>**slender**</u>

It is therefore necessary to use longitudinal stiffeners to prevent web buckling due to bending. First longitudinal stiffener at d/5 = 225/5 = 45 cm from compression flange (top).

Note that, according to Eqn. 5.34, no need for another longitudinal stiffener at d/2 since $d/t = 160.74 < 320 / \sqrt{3.6} = 168.65$.

v) Check of Bending Stresses:

Section properties:

Inertia $I_x = 1.4 (225)^3 / 12 + 2 * (60 \times 3.6) (114.3)^2 = 6973232$ cm^4
Modulus $Z_x = 6973232 / 116.1 = 60062$ cm^3

Actual bending stress $f_b = M_{max} / Z_x = 1200 \times 100 / 60062 = \underline{\mathbf{1.998}}$ t/cm^2

a) <u>Check of Bending Compression:</u>

Since the girder compression flange is supported laterally by deck slab, the allowable bending stress in compression $\underline{\mathbf{F_b = 0.583\ F_y = 2.10}}$ t/cm^2.

<u>***Girder is safe in bending compression***</u>

The lateral stability of the girder <u>during erection</u> (before the deck slab hardens) should be also checked. For this case:

Dead load bending stress $f_{DL} = 385*100/ 60062 = \underline{0.641}$ t/cm^2

The allowable lateral torsional buckling stress is computed as:

$L_u / r_t = 2700/15.5 = 177.42$ $F_{ltb} = 12000 / (177.42)^2 = \underline{0.381}$ t/cm^2

Since $f_{DL} > F_{ltb}$ then the girder must be supported laterally during erection using upper wind bracings.

b) <u>Check of Bending Tension:</u>

i) Allowable tensile stress = 0.583 F_y = 2.10 t/cm^2

<u>Girder is safe in bending tension</u>

ii) The girder tension flange should also be checked for ***Fatigue:***

Straining Actions for fatigue: According to ECP, live loads on roadway bridges are reduced by 50% for fatigue assessment; i.e.

$$M_{LL+I} = 0.5 \times 700 = 350 \text{ mt}$$

Actual stress range $f_{sr} = f_{max} - f_{min} = 0.5 * f_{LL+I}$

$$f_{sr} = 350 \times 100 / 60062 = \underline{0.583}\ \text{t/cm}^2$$

The allowable fatigue stress range (F_{sr}) is obtained as follows:

* From ECP Table 3.1.a: ADTT >2500, Number of cycles = 2 $\times 10^6$
Detail Class = B′ (case 4.2 of Table 3.3)

Table 3.2 gives $F_{sr} = \underline{1.02}$ t/cm^2 > f_{sr} (O.K.)

<u>Girder is safe against Fatigue</u>

vi) Curtailment of Flange Plates

The girder section has been designed to carry the maximum moment at the point of mid span. As the moment decreases away from that point, the girder section can be reduced accordingly. The section at maximum moment is usually taken to cover ~ 40 – 60 % of span, i.e., 10.8 – 16.2 meters. Assume that the section covers the middle 15 meters and find the reduced section to cover the end 6 meters from each support where the bending moment value is 785 m.t. and the shear value is 105 ton. By similar calculations the reduced section has a flange plate of 500*24 mm. The moment of inertia of the reduced section is 4431667 cm^4**.** Note that this section is subjected to the combined action of shear and bending. The actual shear stress is

$$\mathbf{q_{act} = \frac{105}{225 \times 1.4} = 0.333\ \ t/cm^2\ < 0.6 * q_b = 0.6 * 0.652 = 0.391}$$

Therefore no reduction of the allowable bending stress is required. The actual bending stress is

$$f_b = M_{max} / Z_x = 785 \times 100 / 38570 = \underline{2.035}\ \text{t/cm}^2$$

$$< F_b = 0.583\ F_y = 2.10\ \text{t/cm}^2.$$

5.12.3 FLANGE -TO-WEB WELD:

Each flange shall be connected to the web by a fillet weld on each side. These welds must be designed to resist the horizontal shear between the flange and the web as follows:

**** Shear Effect:***

Q = maximum shear force (at support) = 180 t
S = Static moment of flange = 50 * 2.4 * (112.5+1.2) = 13644 cm^3
Shear force / unit length = τ = QS/I = 180 * 13644 / 4431667 = 0.554 t/cm'

**** Direct Load Effect***

The effect of direct load on top flange of deck bridges is calculated assuming the maximum wheel load (10 t) plus impact (40 %) is distributed on a 1 m width;i.e.,

w = P / 1 m = 10*1.4 /1 = 1.4 t/m = 0.14 t/cm'

The resultant shear flow is thus given by

$$\boldsymbol{\tau_R = \sqrt{\tau^2 + w^2}} = \sqrt{\mathbf{(0.554)^2 + (0.14)^2}} = 0.572 \text{ t/cm'}$$

The allowable weld stress q_w is equal to 0.2 F_u, i.e., q_w = 0.2 * 5.2 = 1.04 t/cm^2. Using this value, the required weld size is computed from

$$\mathbf{s} = \tau_R / 2q_w = 0.572 / (2 * 1.04) = 0.274 \text{ cm}$$

**** Fatigue Considerations:***

The allowable fatigue stress range, F_{sr} according to ECP Table 3.2, for a weld detail D (case 23.1 of Table 3.3) and 2 * 10^6 cycles is 0.71 t/cm^2. The actual stress range is

$$\tau_{sr} = 0.5 * Q_{LL+I} * S / I = 0.5*100 * 13644 / 4431667 = 0.154 \text{ t/cm'}$$

The resultant shear stress range is thus given by

$$\boldsymbol{\tau_R = \sqrt{\tau^2 + w^2}} = \sqrt{\mathbf{(0.154)^2 + (0.14)^2}} = 0.208 \text{ t/cm'}$$

The required weld size from fatigue considerations is

$$s = \tau_{sr} / 2 F_{sr} = 0.208 / (2 * 0.71) = 0.146 \text{ cm}$$

This value is smaller than the value computed from shear stress considerations so that fatigue does not govern the design. Furthermore, both values are less than the ***6 mm minimum size*** permitted for a 24 mm thick flange plate according to ECP. Therefore, use 6 mm fillet weld.

5.12.4 STIFFENERS

5.12.4.1 End Bearing Stiffener at Support:

A pair of bearing stiffeners should be provided at each end to transmit the end reaction to the supports. The stiffener is designed as a compression member as follows:

Reaction = 180 t

Using a stiffener plate on each side of web:

$$\text{min. width: } x \geq \frac{h}{30} + 5 = \frac{225}{30} + 5 = 12.5\ \text{cm}$$

$$\text{max. width } x \leq \frac{50}{2} - \frac{1.4}{2} = 24.3\ \text{cm}$$

Try width b_s= 20 cm
minimum thickness from b/t < 21 / $\sqrt{Fy}$ = 11 for St. 52

$t \geq 20 / 11 = 1.82$ cm Try $t_s = 2$ cm

a) Check bearing area at stiffener ends:

Bearing area = 2 *(20 - 2) * 2 = 72 cm^2

Bearing stress = $\frac{180}{72}$ = 2.5 t/cm^2 < $f_{bearing}$ = 2 * F_b = 4.2 t/cm^2

b) Check column action:

Area A = 12 * 1.4^2 + 2 * 20 * 2 = 103.92 cm^2

Interia I = 2 $(2*20+1.4)^3$ / 12 = 11826 cm^4

i = $\sqrt{I/A}$ = 10.688 cm, $\frac{L_b}{i}$ = 0.8 * 225 / 10.688 = 16.84

F_{pb} = 2.1 - 0.000135 $(16.84)^2$ = 2.062 t/cm^2

f_{act} = 180 / 103.92 = 1.739 t/cm^2 < F_{pb}

c) Design of weld between stiffener and web:

Stiffener-web welds must be capable of carrying the end reaction of 180 t. With fillet welds on opposite sides of each stiffener, four lines of welds are used. They extend the total length of stiffeners.

Thus, Total weld length = 4 * (225 - 2 * 2) = 944 cm.

Average shear on weld = 180 / 944 = 0.191 t/cm'.

Weld size required to carry the end reaction is, with allowable weld stress of 0.2 * 5.2 = 1.04 t/cm^2,

$$s = \frac{\mathbf{0.191}}{\mathbf{1.04}} = 0.183 \text{ cm}$$

This, however, is less than the 5 mm minimum size of weld required for a 20 mm thick stiffener plate. Therefore, use 5 mm fillet weld.

5.12.4.2 Intermediate Transverse Stiffeners:

Intermediate transverse stiffeners will be provided at d_1 = 2.25 m as required to resist buckling due to shear. Using a single stiffener on the inside of each girder. The design of the first intermediate stiffener adjacent to the end bearing is as follows:

a) Stiffener Size:

min. width: $b_s \geq \frac{h}{30} + 10 = 17.5 \text{ cm}$

max. width: $b_s \leq$ 50 / 2 - 1.4/2 = 24.3 cm

Use b_s = 20 cm.

Min. thickness to resist local buckling = 20/11 = 1.818 cm

Use stiffener plate 200 * 20 mm.

b) Strength Requirements:

Shear force at the stiffener location = 150.7 t

Force carried by stiffener = $C_s = 0.65\ (\frac{0.35\ F_y}{q_b} - 1)\ Q_{act.}$

$$C_s = 0.65 \ (\frac{0.35 \ \ * 3.6}{0.635} - 1) \ 150.7 = 96.4 \text{ t}$$

Column area = 20 * 2 + 25 * 1.4^2 = 89 cm^2

Interia I = 2 $(20)^3$ / 3 = 5333 cm^4

$i = \sqrt{I / A}$ = 7.74 cm, $\frac{L_b}{i}$ =0.8 * 225 / 7.74 = 23.256

f_{pb} = 2.1 - 0.000135 $(23.256)^2$ = 2.027 t/cm^2

f_{act} = 96.4 / 89 = 1.083 t/cm^2 < f_{pb}

c) Weld between Stiffener and Web

Welding between the stiffener and the web plate in either the upper or lower thirds of the stiffener should be designed to transmit the design force C_s.

Weld length = (225/3) * **2** = 150 cm (**2** for weld on both sides)

Weld Force / unit length = 96.4 / 150 = 0.642 t/cm'

With allowable weld stress of 0.2 * 5.2 = 1.04 t/cm^2, the required weld size required to carry this force is,

Weld size = 0.642 / (1.04) = 0.617 cm use 7 mm weld.

Note that, for fatigue reasons, the weld and also the stiffener, is stopped at ~ 60 mm from the tension flange.

5.12.4.3 Longitudinal Stiffener:

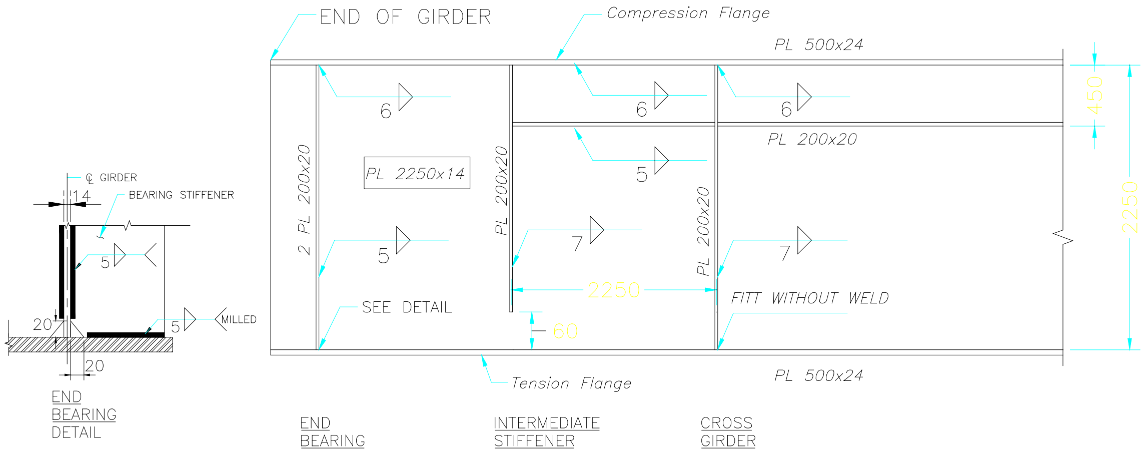
One longitudinal stiffener will be welded to the web at d/5 = 45 cm from the compression flange.

Assume: width = 20 cm and thickness = 2 cm as calculated for the transverse stiffener, then:

I_{act} = 2 * 20^3 / 3 = 5333 cm^4

I_{min} = 4 d_w t_w^3 = 2470 cm^4 < I_{act} O.K.

Therefore, use a 200 * 20 mm plate for the longitudinal stiffener.



Details of Stiffener Attachments

5.12.5 BOLTED SPLICE

A bolted field splice will be executed at 6m from the support. The design shear and moment at the splice location are:

Shear Force:

$Q_{DL} = 45$ t

$Q_{LL+I} = 60$ t

Total Shear = $\underline{105}$ t

Bending Moment:

$M_{DL} = 325$ mt

$M_{LL+I} = 460$ mt

Total Moment = $\underline{785}$ t

The value of the bending moment to be used for the design of splice is the moment capacity of the cross section, which is computed for the smaller section at the splice as follows:

Gross moment of inertia I_g = 4431667 cm^4

Gross section modulus Z_g = 38570 cm^3

Bending moment capacity M_{net} = 38570 * 2.1 /100 = $\underline{810}$ mt

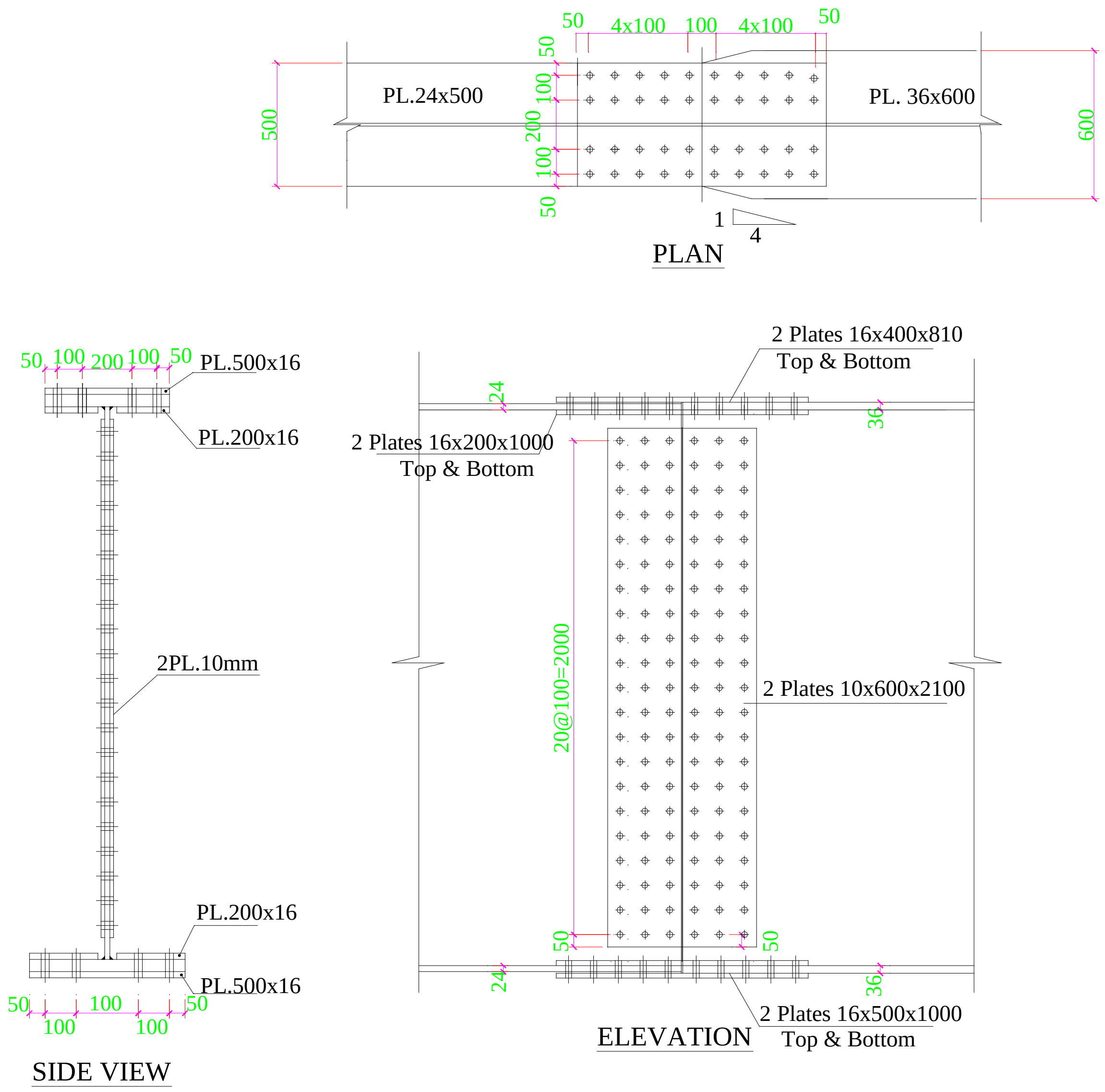


Fig. 5.56 BOLTED FIELD SPLICE

a) Web Splice:

The web splice carries a bending moment equal to the total design moment on the section multiplied by the ratio of the moment of inertia of the web to the moment of inertia of the entire section. In addition to this moment, the web splice also carries the design shear at the splice location and the moment due to the eccentricity of the shear force.

Using three columns of M 24 High Strength Friction Type bolts of grade 10.9 at a vertical pitch of 10 cm (21 rows) and a horizontal pitch of 10 cm, see Fig. 5.56 , the design is checked as follows:

Moment of inertia of web $I_w = 1.4*225^3 /12 = 1328906$ cm^4

Moment of inertia of girder section $I_g = 4431667$ cm^4

Moment carried by web $M_w = M_{des} * I_w / I_g = 242.89$ m.t.

Eccentricity moment $M_e = Q * e = 105 * 0.15 = 15.75$ m.t.

Total moment on web splice $M_s = 242.89 + 15.75 = 258.64$ m.t.

i) Check of Bolt Resistance:

Bolt force due to shear = Shear / Bolt Number = 105 / 21*3 = 1.667 t

Bolt force due to moment :

$$\sum x^2 = 21*(5^2+15^2+25^2)= 18375$$

$$\sum y^2 = 2*3*(10^2+20^2+30^2+40^2+50^2+60^2+70^2+80^2+90^2+100^2)= 231000$$

$$\sum d^2 =18375 + 231000 = 249375 \text{ cm}^2$$

Hl component $= M_s * y / \sum d^2 = 25864 * 100 / 249375 = 10.372$ t

Vl component $= M_s * x / \sum d^2 = 25864 * 5 / 249375 = 0.519$ t

Resultant shear force /bolt = $\sqrt{(1.667+0.519)^2 + 10.372^2}$ = **<u>10.6</u>** t

The allowable bolt resistance for M 24 friction type high strength bolt acting in double shear is equal to 2 * 6.94 = **<u>13.88</u>** t. Therefore, the design is safe.

ii) Design of Splice Plate:

Assume two splice plates 10 * 600 * 2100 mm,

Gross Inertia $I_g = 2 * (1 * 210^3 / 12) = 1774667$ cm^4

Bending Stress $= 25864 * (210/2) / 1774667 = 1.53$ $t/cm^2 < 2.1$ t/cm^2

Shear Stress $= 105 / (2*210*1) = 0.25$ $t/cm^2 < 1.26$ t/cm^2

b) Flange Splice:

The flange splice carries that portion of the total moment not carried by the web plate. The flange splice plates transmit the moment couple across the splice in axial tension (at bottom) or compression (at top), and into the girder flange by double shear on M 24 High Strength Friction Type bolts of grade 10.9.

i) Flange Bolts:

Design moment on flange splice = 810 – 242.89 = 567.11 m.t.
Flange force = 567.11 / 2.25 = 252 t

(This value can also be calculated from the flange strength as 50*2.4*2.1 = 252 t)

No. of M 24 double shear bolts = 252 / (2*6.94) = 18.15 bolts

Use 5 rows * 4 bolts each = 20 bolts.

ii) Splice Plates:

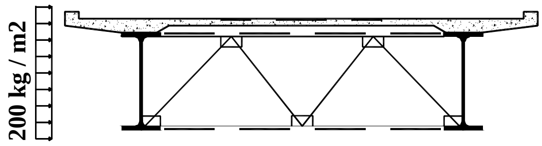
area required = 252 / 2.1 = 120 cm^2
assume one outside plate 16 * 500 = 80 cm^2
plus <u>two</u> inside plates 16 * 200 = 64 cm^2
Area provided = 80+64 = 144 cm^2

5.12.6 BRIDGE BRACINGS:

Wind loads on the Bridge

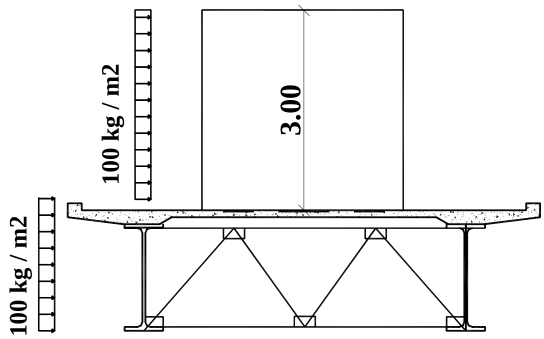
a) Unloaded Case:

wind pressure = 200 kg/m^2



b) Loaded Case:

wind pressure = 100 kg/m^2



<u>Bracing Systems and Transmission of Wind Loads:</u>

Since the bridge has a deck slab, it will carry all wind loads on the bridge after erection and thus no upper horizontal lateral bracings are needed. However, during erection, an upper bracing is needed to support the compression flange laterally and to carry wind loads on the bridge before the slab hardens.
The wind loads on the bridge are transmitted to the bridge supports as follows:

1- Wind loads on the lower half of the main girder can be carried directly by a system of lower horizontal lateral bracings to the bridge bearings. However, it is more economical to transmit these loads to the deck level using intermediate knee brackets at each cross girder working with the cross girders as inverted U-frames, see Fig. 5.56.
2- Wind loads on the upper half of the main girder and deck slab during erection are carried directly by an upper horizontal lateral bracings to the bridge ends.
3- After the slab hardens, wind loads on the moving trucks (height = 3 m) and the bridge main girder and deck slab (case of loaded bridge) are carried directly by the concrete slab to the bridge ends.

4- End brackets (or end cross frames) shall be provided at the bridge ends to transmit wind reactions at the deck level to end bearings located at the lower level, see Fig. 5.56.

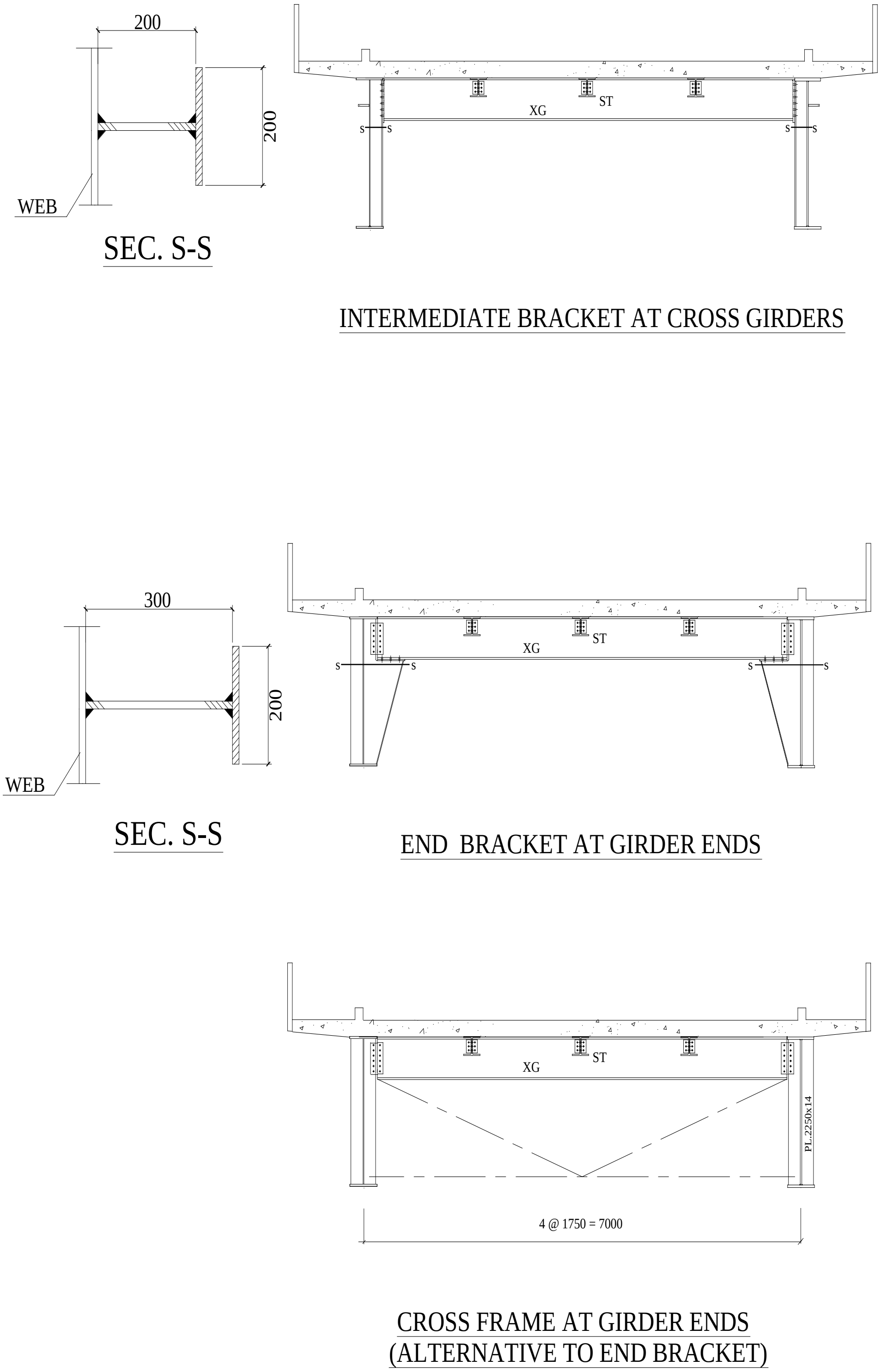


Fig. 5.56 Transversal Bracing Systems

1- Design of Intermediate Brackets (at each cross girder):

Wind pressure $q = 200\ kg/m^2$ (<u>Unloaded Bridge governs</u>)

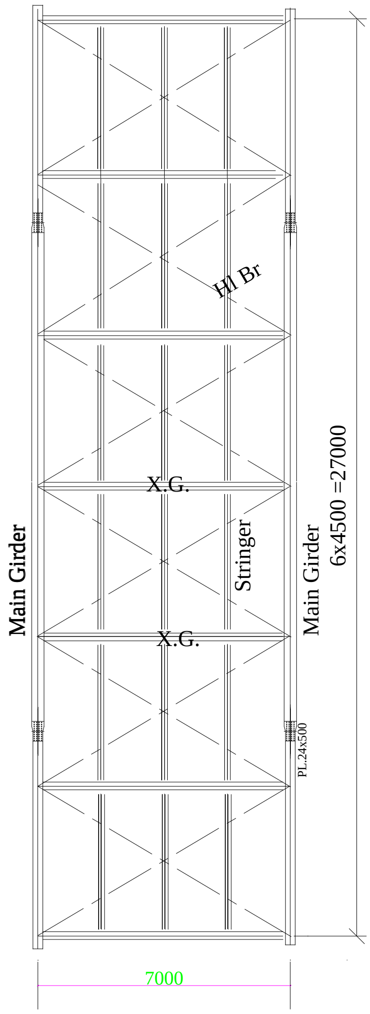
Wind load $\omega = 0.2 * 4.5 = 0.9\ t/m'$

Max moment on bracket section s-s (cross girder section HEB600)

$M_{max} = 0.9 * (2.25-0.6)^2 /2 = 1.225$ m.t.

Assume Section: Web: PL 200*16; Flange: Pl 200*12
Centroid at 14 cm from web, $I_y = 2624\ cm^4$
Bending Stress $f_b = 1.225*100* 14 / 2624$
$= 0.654\ t/cm^2 < 2.1\ t/cm^2$

2. Design of Upper Bracings:



PLAN OF UPPER BRACINGS

Upper bracing carries wind load on bridge during erection; i.e., case of unloaded bridge governs

Joint load (at each bracket) = 0.9 * (2.25+0.30) = 2.295 ton

End Reactions R_w = 3 * 2.295 = 6.885 ton

Tan θ = 7 / 4.5 = 1.5555 θ = 57.265

Max. Force (in first diagonal) = C = T = 6.885 / (2*sin θ) = 4.09 ton

Design for <u>Compression</u>; assume 2 L 100*100*10

Diagonal Length = $\sqrt{4.5^2 + 7^2}$ = 8.32 m

L_b / i = 0.75 * 832 / (0.45*10) = 138.67 < 140 for bridge bracings

F_{buck} = 0.85*{7500 / (138.67)^^{2} } = 0.332 t/cm^2

f_a = C/A = 4.09/(2*19.2) = 0.107 t/cm^2 < F_{buck} O.K.

3. Design of End Bracket (at Bearings):

End brackets carry wind reactions from upper level to bearings at lower level. Compare wind loads for the two cases of unloaded and loaded bridge and design for the critical case which is the loaded one here.

<u>For loaded bridge</u>:

Joint load W = 0.1*4.5*(2.25+0.30+3.00) = 2.5 t

End reaction R_w= 3 * 2. 5 = 7.5 t

Max moment on bracket section s-s:

M_{max} = (7.5/2) * (2.25-0.6)= 6.1875 m.t.

Assume Section: Web: Pl 300*16 ; Flange: Pl 200*12

Centroid at 19.8 cm from web, I_y = 7505 cm^4

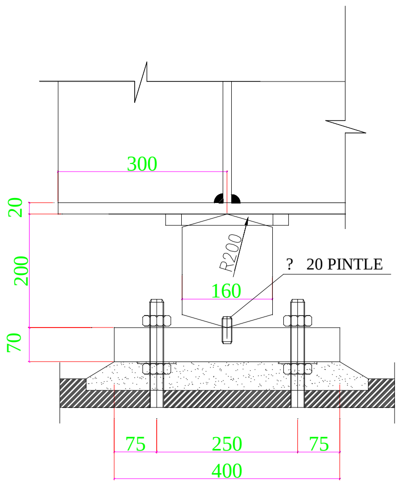
Bending Stress: f_b = 6.1875 *100* 19.8 / 7505 = 1.632 t/cm^2 < 2.1 t/cm^2 O.K

5.12.6 BEARINGS

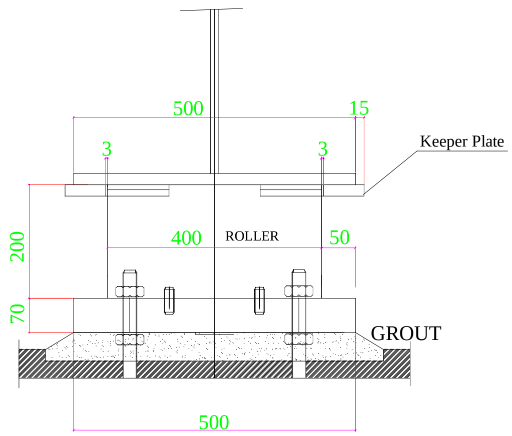
Each main girder transmits its end reactions to piers through one expansion bearing at one end and one hinged bearing at the other end. The roller bearing should be designed to permit movements resulting from variations of temperature between ± 30 ^{o}C, and to allow rotation of the girder ends under live loads. Both bearings are fabricated from forged steel.

i) Roller Bearing:

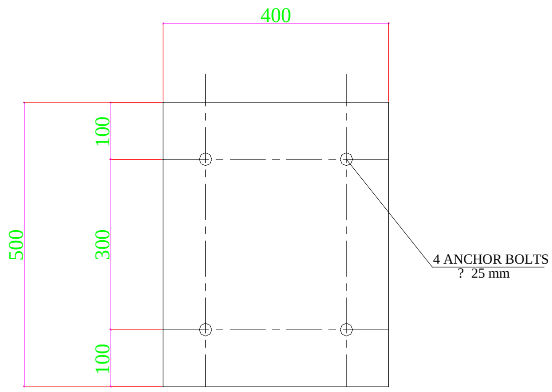
The expansion bearing incorporates a flat sided roller to permit the required movements and a base plate to distribute the load to the concrete foundation as shown:



ELEVATION



SIDE VIEW



BASE PLATE

Roller Design:

The roller length is taken equal to the bottom flange width minus 5 cm clearance each side. By Hertz formula for forged steel,

Reaction $V = 0.117 * d * l$

$180 = 0.117 * d * 40$

Roller Diameter $d \geq 38.46$ cm Use **<u>d = 40 cm</u>**

Use flat sided roller with width b > d/3 = 40/3 =13.33 cm to resist overturning. Select b = 16 cm

The 160-mm-thick roller web rests on a steel base plate while its curved top bears against the girder bottom flange. Thus, the compressive stress in the 40-cm-long web is:

$$f_p = \frac{\mathbf{180}}{\mathbf{16 * 40}} = 0.281 \text{ t/cm}^2 < 2.00 \text{ t/cm}^2$$

= Allowable compression for forged steel according to ECP

Base Plate:

The rocker is seated on a base plate, which distributes the 180 tons load to the concrete pier. Allowable bearing stress on the C350 concrete is 110 kg/cm^2. Hence,

Net area of the plate = 180 * 1000 / 110 = 1636 cm^2.

For a width of 40 cm , min. length of plate = 1636 / 40 = 40.9 cm.

Use **<u>base plate 40*50 cm</u>**

Thickness of plate must be large enough to keep bending stresses caused by the bearing pressure within the allowable. Under dead load and live load plus impact, the base pressure is:

$$p = \frac{\mathbf{180000}}{\mathbf{40 * 50}} = 90 \text{ kg/cm}^2$$

The bending moment in the middle of a 1 cm wide strip of plate (at the bearing point) is:

$$M = \frac{\mathbf{90(20)^2}}{\mathbf{2}} = 18000 \text{ kg.cm} = 18 \text{ t.cm}$$

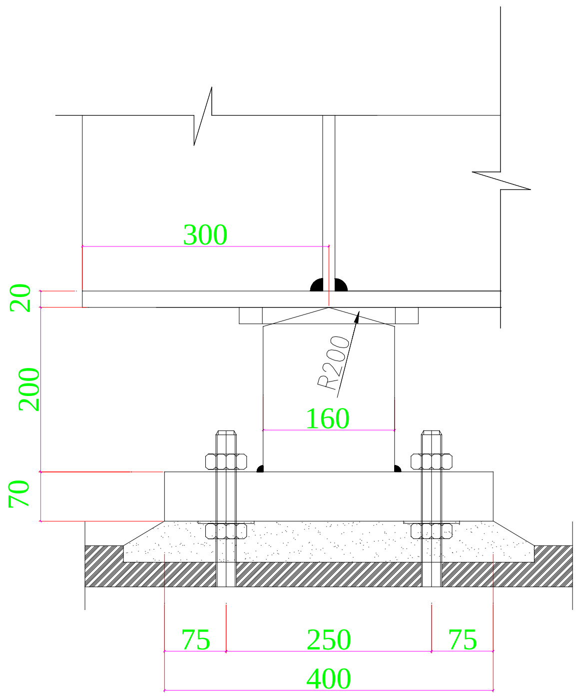
With the basic allowable stress F_b= 0.72*F_y = 0.72*3.35=2.412 t/cm^2 for ST 52 (F_y=3.35 t/cm^2 for thicknesses > 40 mm and F_b =0.72 F_y for rectangular section bent about their minor axis), the thickness of base plate required is:

$$\mathbf{t = \sqrt{\frac{6M}{F_b}} = \sqrt{\frac{6 * 18}{2.412}} = 6.69, \quad say\ 7\ cm}$$

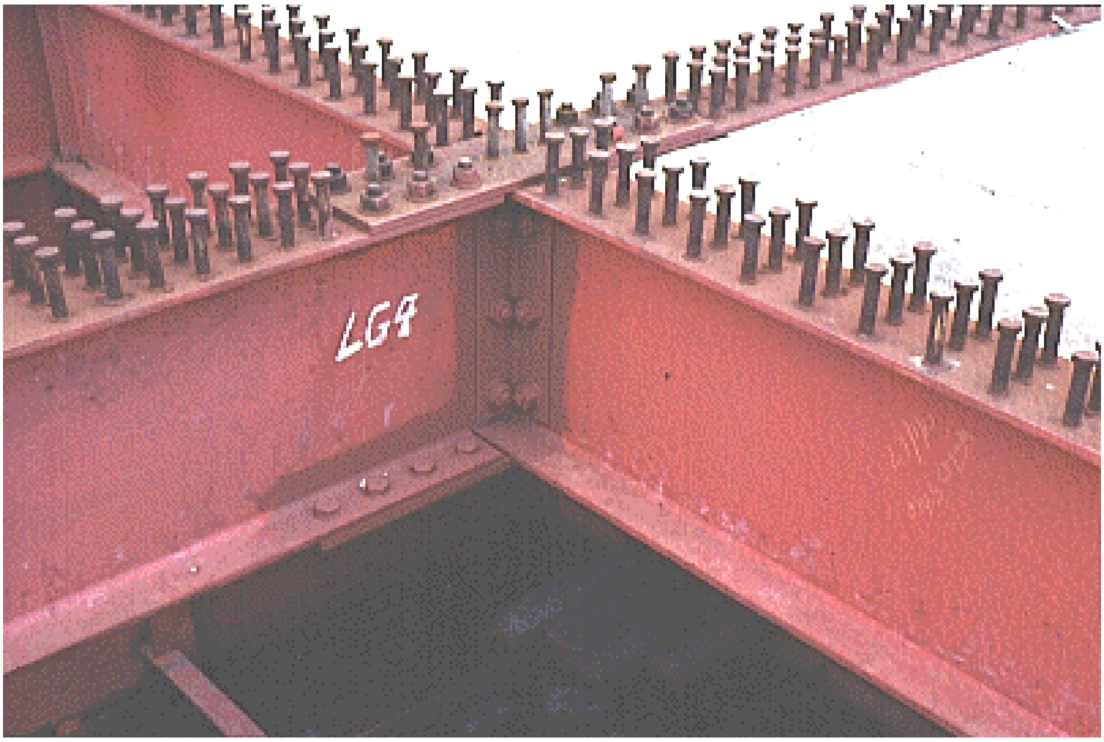
Use a base plate **400 * 70 mm by 500 mm** long.

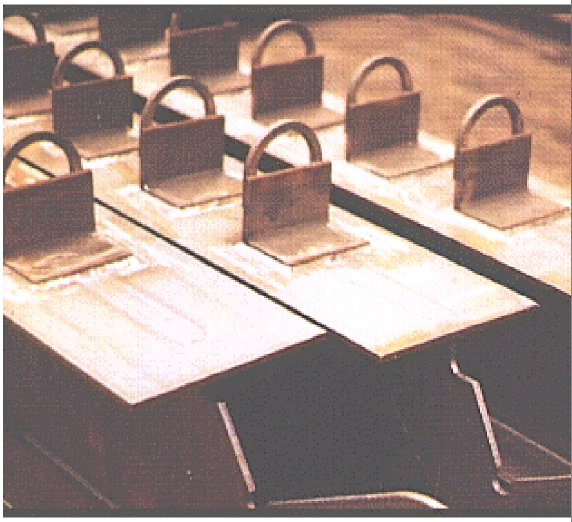
ii)Hinged Bearing:

The same detail is used for the hinged bearing except that the roller bottom is welded to the base plate to prevent translation as shown below:



HINGED BEARING





CHAPTER 6

COMPOSITE PLATE GIRDER BRIDGES

CHAPTER 6

COMPOSITE PLATE GIRDER BRIDGES

6.1 GENERAL

6.1.1 Composite Action

Steel structures supporting cast-in-place reinforced concrete slab construction were historically designed on the assumption that the concrete slab acts independently of the steel in resisting loads. No consideration was given to the composite effect of the steel and concrete acting together. This neglect was justified on the basis that the bond between the concrete deck and the top of the steel girder could not be depended upon. However, with the wide use of structural welding, it became practical to provide mechanical connectors between concrete and steel to resist the horizontal shear which develops during bending, Fig 6.1. Composite action is developed when the concrete deck and the supporting steel girder are integrally connected so that they deflect as a single unit.

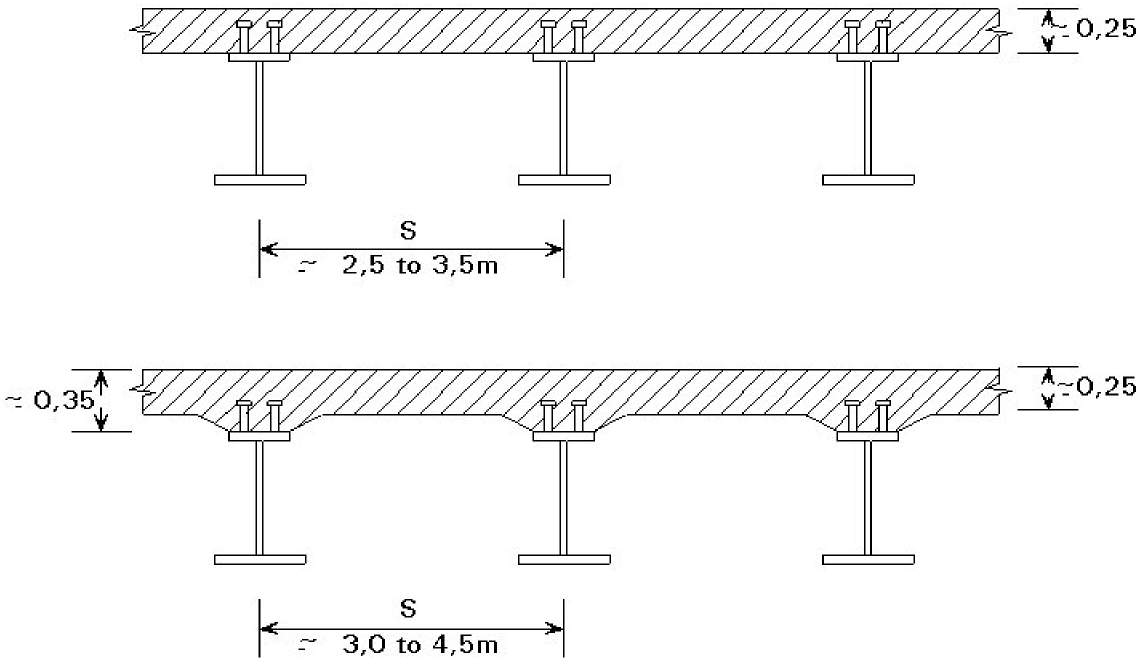


Fig. 6.1 Composite Deck Bridges

In developing the concept of composite behavior, consider first the non-composite girder of Fig. 6.2, wherein if friction between the slab and the girder is neglected the girder and slab each carry separately a part of the load. The strain distribution corresponding to this case is shown in Fig.6.2. There are two neutral axes; one at the slab mid surface and the other at the girder centroid. When the slab deforms under vertical loads, its lower surface is in tension and elongates; while the upper surface of the girder is in compression and shortens. Thus a discontinuity will occur at the plane of contact. Since friction is neglected, only vertical internal forces act between the slab and girder.

When complete interaction between the slab and girder is developed by the introduction of mechanical connectors, no relative slippage occurs between the slab and girder and the resulting strain diagram is shown in Fig. 6.3. Under this condition a single neutral axis occurs which lies below that of the slab and above that of the girder. Horizontal forces (shears) are developed that act on the lower surface of the slab to compress and shorten it, while simultaneously they act on the upper surface of the girder to elongate it.

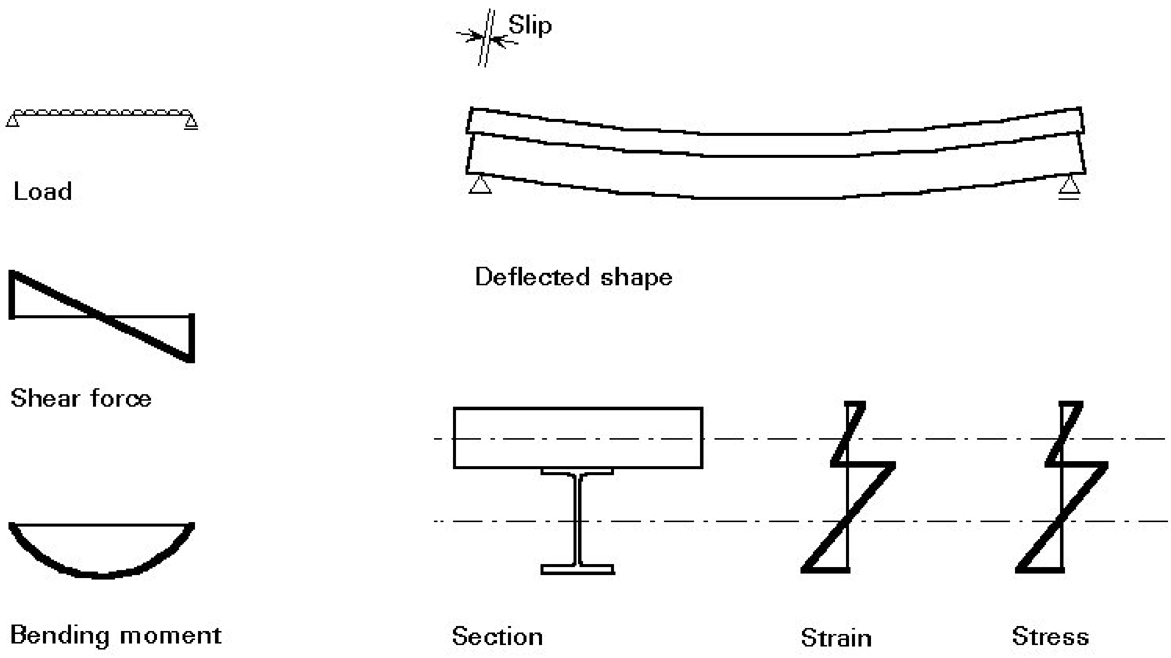


Fig. 6.2 Stress and Strain Distributions in Non-Composite Girders

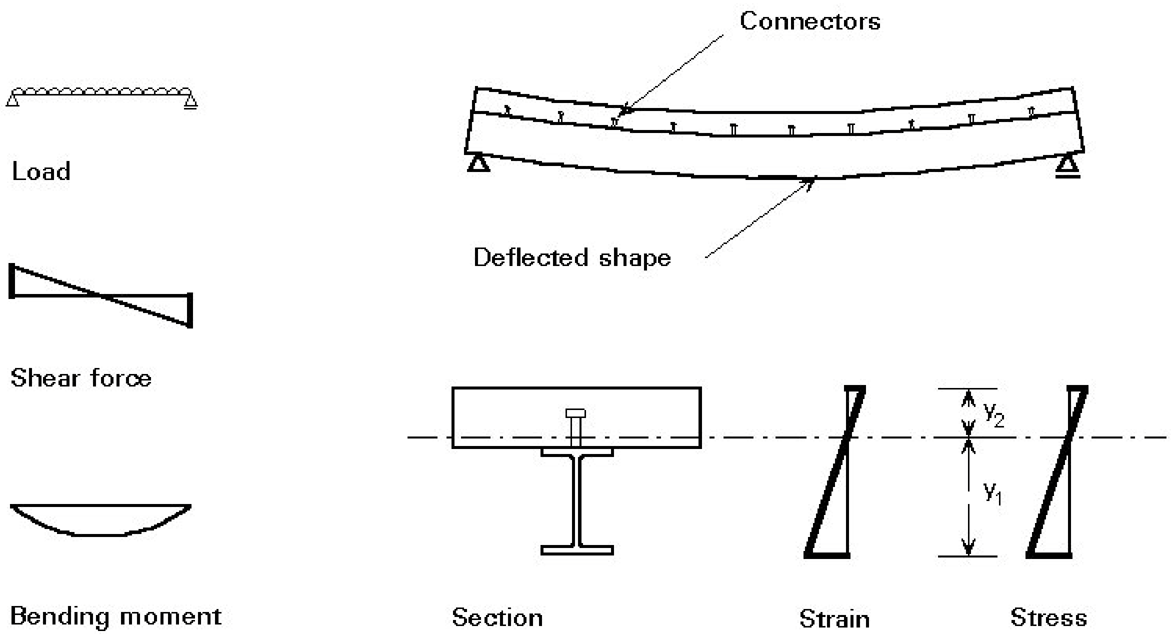


Fig.6.3 Stress and Strain Distributions in Composite Girders

6.1.2 Advantages and Disadvantages:

The basic advantages resulting from composite design are:

1. Reduction in steel weight.
2. Shallower steel girders.
3. Increased stiffness.

A weight saving in steel between 20 to 30 % is often possible by taking full advantages of composite action. Such a weight reduction in the supporting steel girders permits the use of a shallower section which is better from the traffic clearance point of view. The stiffness of the composite bridge is substantially greater than that of the concrete floor with its supporting steel girders acting independently. Normally the concrete slab acts as one-way plate spanning between the supporting girders. in composite design, an additional use is made of the slab by its action In a direction parallel to and in combination with the supporting steel girders. The net effect is to greatly increase the moment of inertia of the floor system in the direction of the steel girders. The increased stiffness considerably reduces the live load deflections. Assuming full composite action, the strength of the section greatly exceeds the sum of the strengths of the slab and the girder considered separately, providing high overload capacity.
While there are no major disadvantages to using composite construction, some limitations should be recognized. In continuous bridges, the negative moment region will have a different stiffness because the cracked concrete slab in tension is not participating. Also, long-term deflection caused by concrete creep and shrinkage could be important when the composite section resists a substantial part of the dead load, or when the live load is of long duration. These two points shall be dealt with in later sections.

6.2 COMPONENTS OF COMPOSITE GIRDERS

6.2.1 Steel Girder

Composite construction is more economic when the tension flange of the steel section is larger than the compression flange. The ratio of the girder span, L, to the girder overall depth including concrete slab, h, lies generally between 16 and 22. For limited girder depth, L/h may exceed 22 provided that deflection due to live load without impact does not exceed the allowable value specified by the Code as L/800.

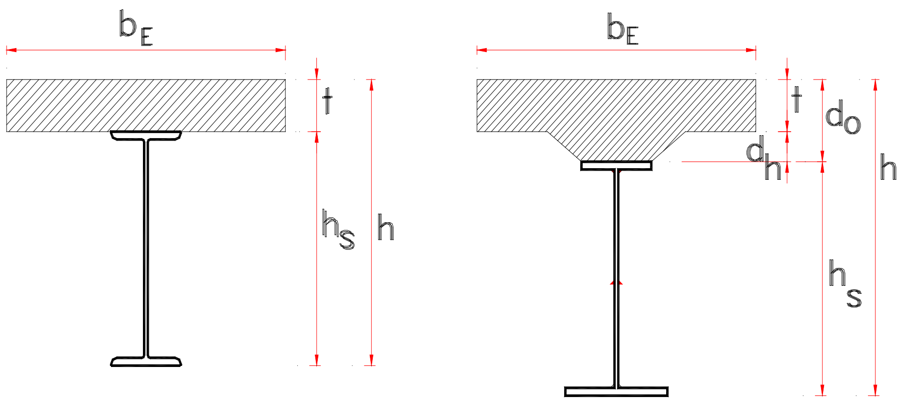


Fig. 6.4 Composite Section Parameters

6.2.2 Concrete Slab

Concrete**:** The concrete used for composite construction shall comply with the current Egyptian Code of Practice for Design of Reinforced Concrete Structures. The minimum accepted value for the characteristic cube concrete strength, f_{cu}, is 300 kg/cm^2 for bridges. For deck slabs subjected directly to traffic (without wearing surface) , the value of f_{cu} shall not be less than 400 kg/cm^2.

Thickness*:* The thickness for the deck slab shall not be less than 16 cm. If the slab is subjected directly to traffic with no wearing surface, the minimum thickness shall be 20 cm.
The slab may rest directly on the steel girder or on concrete haunch to increase the moment of inertia of the composite section, see Fig. 6.4. It is also possible to use formed steel deck with the deck ribs oriented parallel or perpendicular to the steel girder. The concrete slab may also be prestressed.

6.2.3 Shear Connectors

Since bond strength between concrete slab and steel girder is not dependable, mechanical shear connectors must be provided. They are connected to the top flange of the steel girder and embedded in the concrete slab to transmit the longitudinal shear and prevent any slippage between the concrete slab and the steel girder. There are several types of the shear connectors such as: anchors, hoops, block connectors, studs, channels and angle connectors as will be discussed in details in section 6.4.

6.3 DESIGN CONSIDERATIONS:

6.3.1 Effective Width

In ordinary girder theory the bending stress is assumed constant across the girder width and is calculated from the bending formula**,** $f = M * y / I$. Since the composite girder has a wide top flange, plate theory indicates that the stress in the concrete slab is not uniform across the girder width. Referring to Fig. 6.5, the stress is maximum over the steel girder and decreases non-linearly as the distance from the supporting girder increases. Similarly to the treatment of T-sections in reinforced concrete design, an effective width is used in place of the actual width, so that the ordinary girder theory can be used. The effective width of the slab b_E is computed from the condition that b_E times the maximum stress f_c equals the area under the nonlinear stress curve.

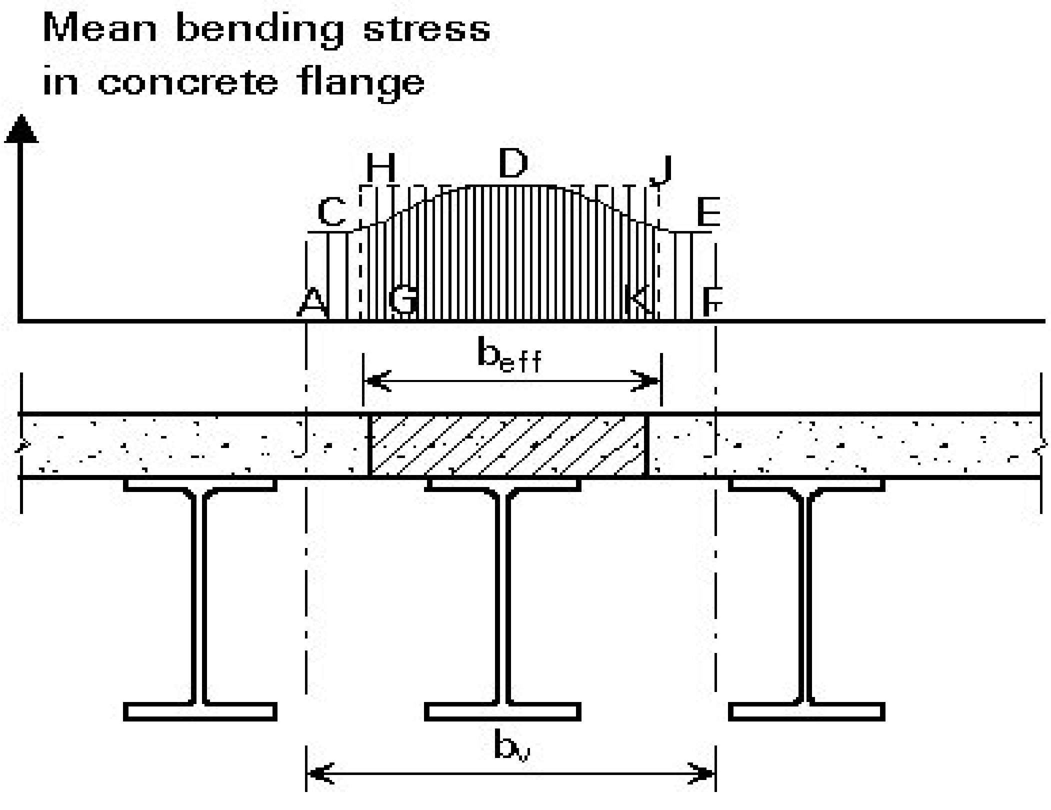


Fig. 6.5 Effective Width Concept

For design purposes, ECP defines the portion of the effective width of the concrete slab ***on each side*** of the girder centerline b_{EL} or b_{ER} as the smaller of the following values; see Fig. 6.6:

1. One-eighth of the girder span,(center-to-center of supports), L/8.
2. One half the distance to the center-line of the adjacent girder, b.
3. Six times the thickness (t) of the slab neglecting haunch.
4. the distance to the slab edge, b^* (for exterior girders)

For girders having a slab on one side only, the effective slab width b_E shall not exceed the smaller of 1/12 of the span length of the girder or six times the thickness (t) of the slab neglecting haunch.

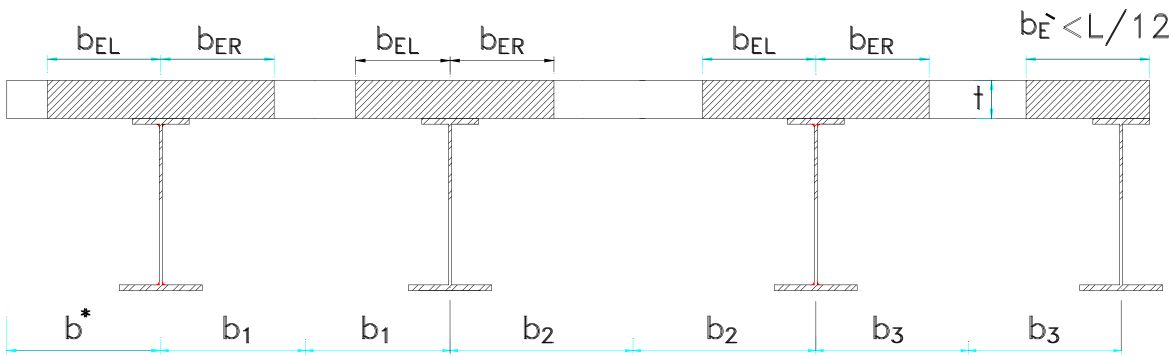


Fig. 6.6 Effective Width of Concrete Slab

The span lengths to be used in continuous beams are shown in Fig. 6.7. If the two adjacent spans in a continuous beam are unequal, the value of b_E to be used in calculating bending stresses and longitudinal shear in the negative moment regions shall be based on the mean of the values obtained for each span separately.

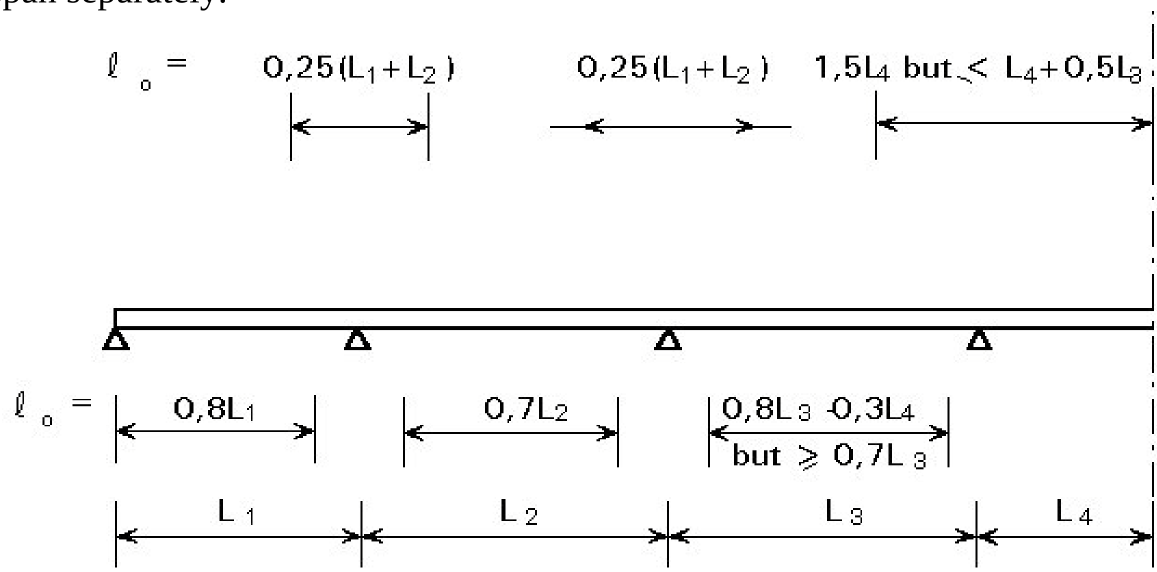


Fig. 6.7 Effective Spans for Continuous Beams

6.3.2 Computation of Section Properties

The section properties of the composite section can be computed by the transformed section method. In contrast to reinforced concrete design where the reinforcing steel is transformed into an equivalent concrete area, the concrete slab in the composite section is transformed into an equivalent steel area. As a result, the concrete area is reduced by using a reduced slab width equal to b_e/n, where n is ratio of the steel modulus of elasticity E_s to the concrete modulus of elasticity E_c. While as the steel modulus is equal to a constant value at 2100 t/cm^2, the concrete modulus varies according to the concrete grade. Consequently, the value of the modular ratio n varies with the concrete grade as follows:

Table (6.1) Recommended Values of the Modular Ratio (n)

Concrete Characteristic Cube Strength, f_{cu} (kg/cm^2)	**Modulus of Elasticity of Concrete, E_c (t/cm^2)**	**Modular Ratio, n**
250	220	10
300	240	9
400	280	8
≥ 500	310	7

With this transformation, the composite girder may be considered as a steel girder to which has been added a cover plate on the top flange. It should be noted that this cover plate, being concrete, is considered to be effective only when the top flange is in compression. In continuous girders, the concrete slab is in tension and thus composite action does not exist. Referring to Fig. 6.7, the following section properties can be calculated.,

A_s = Cross-sectional area of steel girder,
A_c = Cross-sectional area of slab only,
A_r = Cross-sectional area of steel reinforcement,
A_v = Cross-sectional area of transformed section = $A_s + A_c / n$,
I_s = Moment of inertia of steel girder @ its own central axis s – s,
I_c = Moment of inertia of effective slab @ its own central axis c – c,
I_v = Moment of inertia of transformed section @ its own central axis v – v,
= $I_s + A_s * e_v^2 + (I_c + A_c * y_c^2) / n$

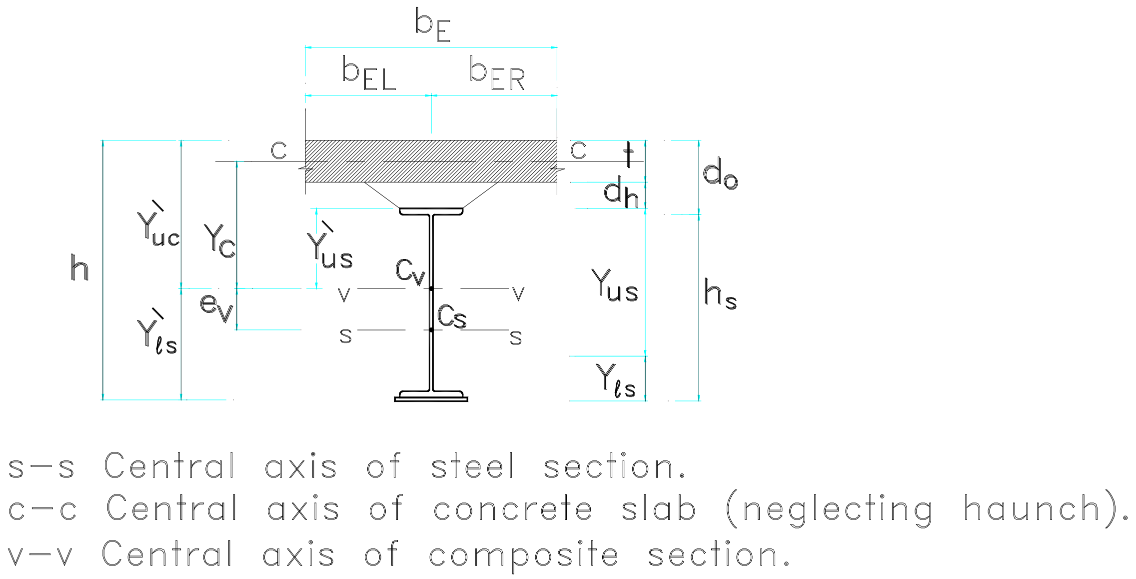


Fig. 6.7 Composite Section Properties

Section Moduli:

Steel Section: Upper Steel $Z_{us} = I_s / y_{us}$

Lower Steel $Z_{ls} = I_s / y_{ls}$

Composite Section: Upper Steel $Z'_{us} = I_v / y'_{us}$

Lower Steel $Z'_{ls} = I_v / y'_{ls}$

Upper Concrete $Z'_{uc} = I_v / y'_{uc}$

5.3.3 Stress Calculations

Bending stresses in the composite section (steel girder, concrete slab, and longitudinal reinforcement) shall be calculated in accordance with the elastic theory, ignoring concrete in tension and assuming no slippage between the steel girder and concrete slab. The actual stresses that result in the composite section due to a given loading depend on the manner of construction. Two different methods of construction may be used:

Case I: Without Shoring:

The simplest construction occurs when the steel girders are placed first and used to support the concrete slab formwork. In this case the steel girder, acting noncompositely, supports the weight of the forms, the wet concrete and 'its own weight. Once the forms are removed and concrete has cured, the section will act compositely to resist dead loads placed after concrete curing (e.g., wearing surface) and live loads. Such construction is said to be *without temporary shoring or* ***unshored.*** Referring to Figs. 6.8 and 6.10a; the stresses in steel and concrete are computed as:

$$f_{us} = M_D \;/\; Z_{us} + M_L \;/\; Z'_{us}$$
$$f_{lS} = M_D \;/\; Z_{ls} + M_L \;/\; Z'_{ls}$$
$$f_{uc} = M_L \;/\; (\, n * Z'_{uc})$$

where M_D= bending moment due to dead load and M_L =bending moment due to additional dead load and live load plus impact.

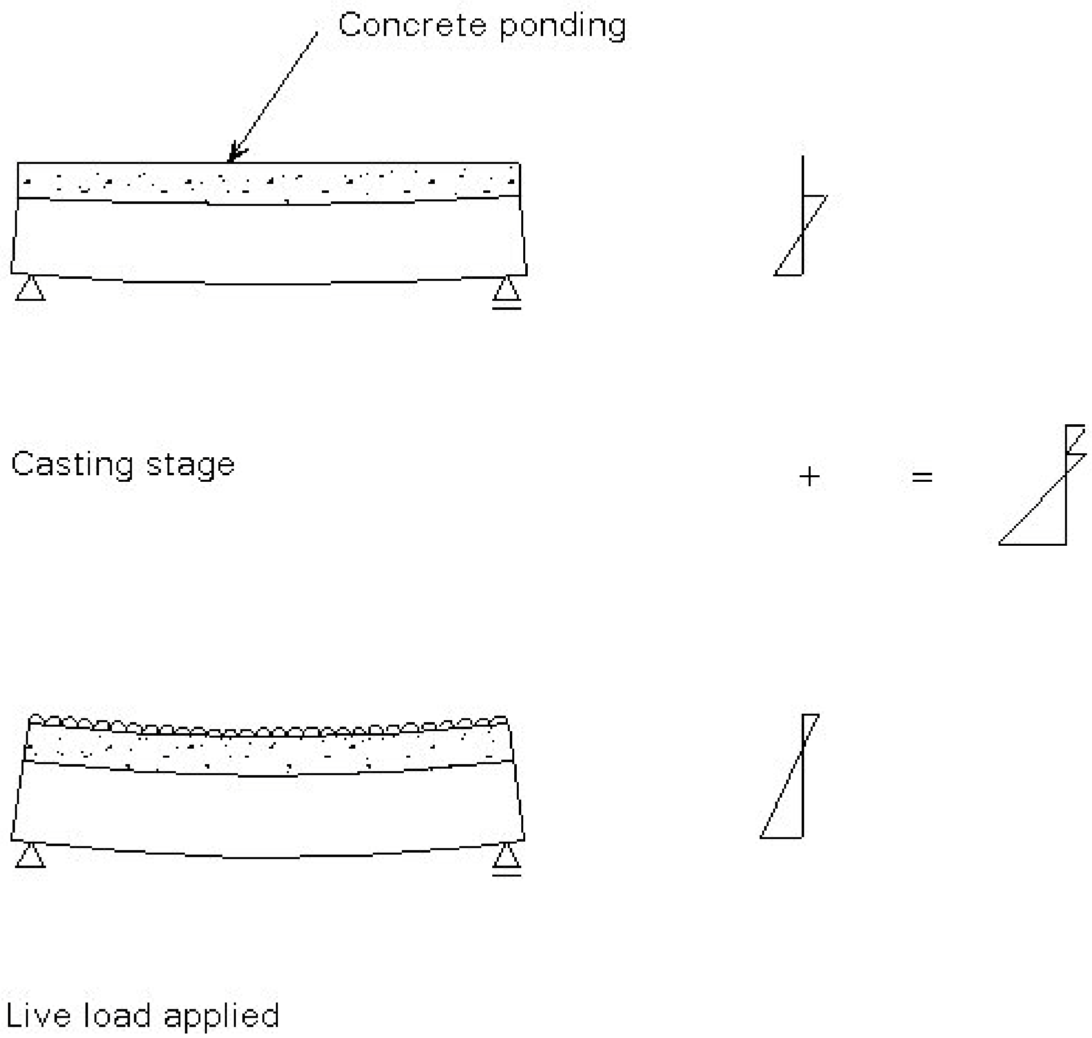


Fig. 6.8 Unshored Construction

Case II: With Shoring

Alternatively, the steel girders may be supported on ***temporary shoring***. In such a case, the steel girder, forms, and wet concrete are carried by the shores. After curing of concrete, the shores are removed and the section acts compositely to resist all dead and live loads. This system is called ***shored*** construction. Referring to Fig. 6.9 and 6.10b; the stresses in steel and concrete are computed as follows:

$$f_{us} = (M_D + M_L) / Z'_{us}$$

$$f_{ls} = (M_D + M_L) / Z'_{ls}$$

$$f_{uc} = (M_D + M_L) / n\ Z'_{uc}$$

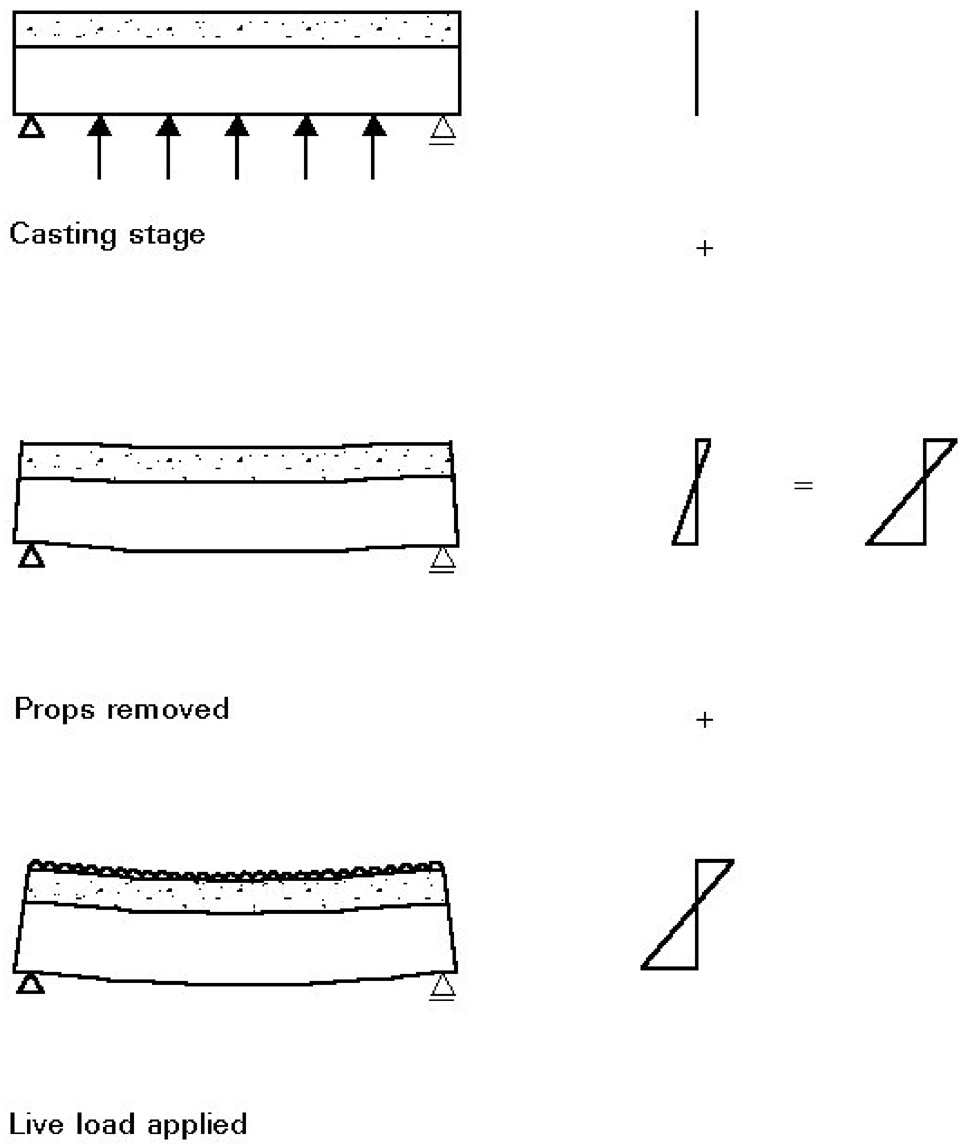


Fig. 6.9 Shored Construction

Figure 6.10 illustrates the distribution of bending stresses for composite girders constructed with or without shoring. Maximum bending stresses in the steel section shall comply with the allowable bending stresses of the used materials. The compression flange of the steel girder and its connection to the web must be designed for the shear flow calculated for the composite section. During construction, the compression flange must satisfy local buckling and lateral torsional buckling requirements. After construction, however, the composite section shall be exempt from such requirements.

The maximum bending stresses in the concrete slab shall not exceed the allowable limits permitted by the Egyptian Code of Practice for Design of Reinforced Concrete Structures. The steel web alone shall resist vertical shear stresses of composite girder, neglecting any concrete slab contribution.

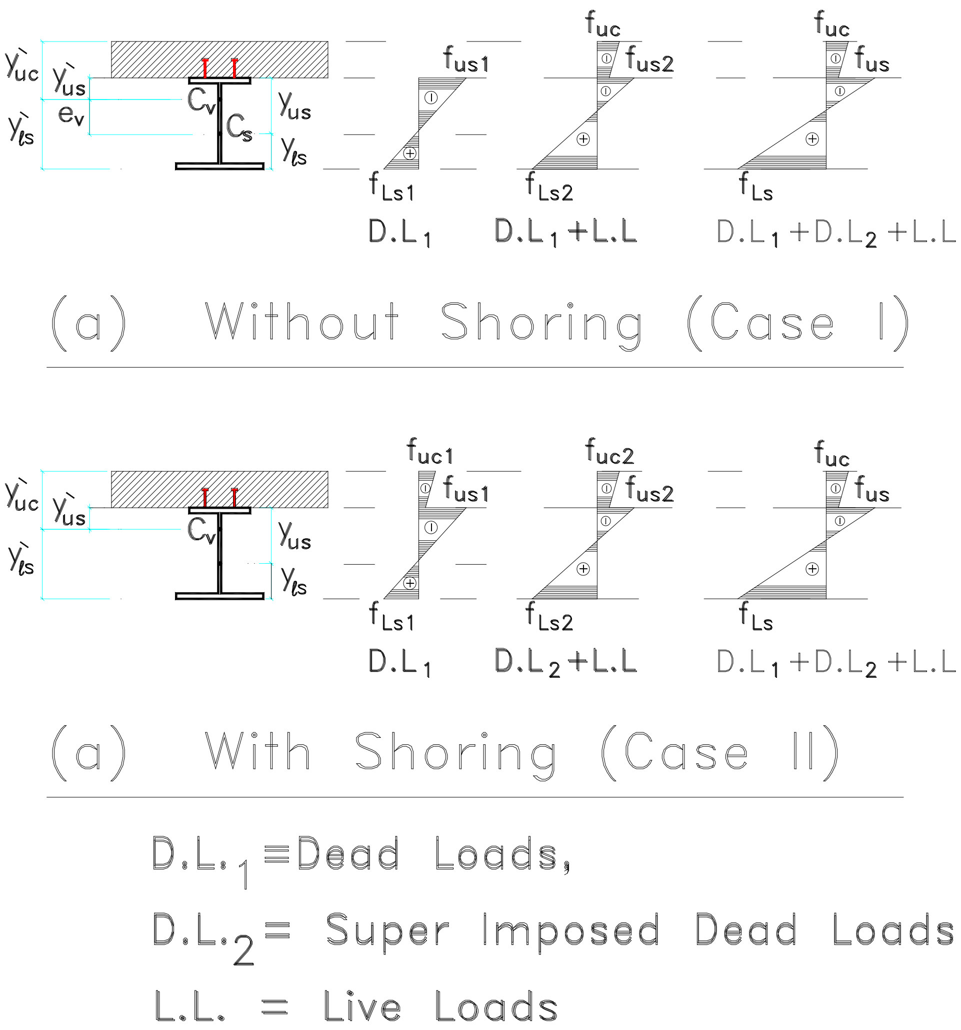


Fig. 6.10 Stress Distributions in Composite Sections

6.3.4 Design for Creep and Shrinkage

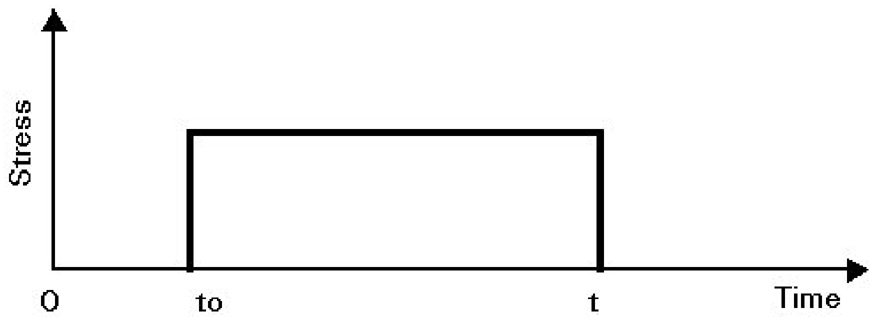
If shoring provides support during the hardening of concrete, i.e. Case II, the total deflection will be a function of the composite section properties. Account must be taken of the fact that concrete is subject to creep under long-time loading (i.e, dead load) and that shrinkage will occur, see Fig. 6.11.

6.3.4.1 Influence of Creep

i) General:

For the usual concrete dead loads, concrete does not behave as an elastic material. Actually, concrete is a plastic material subjected to progressive permanent deformation under sustained loads. This permanent deformation is known as creep. For a constant permanent load the creep will vary from 1 to 4 times the elastic deformation under the same load, see Fig. 6.11.

It is known that only permanent loads causing compressive stresses in concrete produce creep. Moving loads have little effect, as they do not last long. The amount of creep varies with the magnitude of the permanent compressive stresses. Low concrete stresses produce very little plastic flow, which may be neglected.



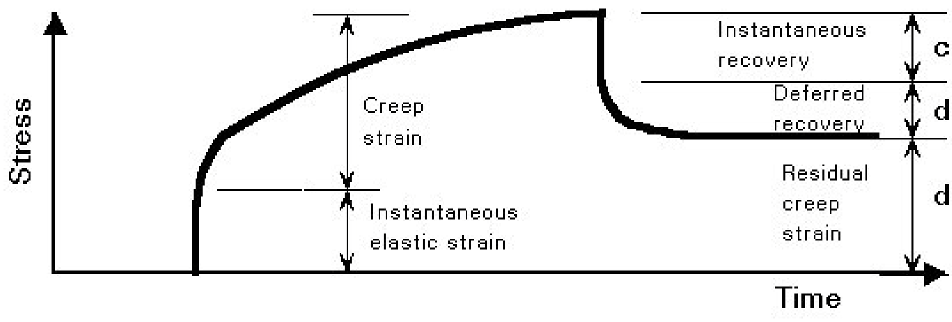


Fig. 6.11 Creep in Concrete

Creep in composite beams causes tensile stresses in concrete, compressive stresses in the top flange and relatively small tensile stresses in the bottom flange of the steel beam, see Fig. 6.12.

The creep of concrete depends on the curing conditions of the concrete at the time the stresses are applied, on the intensity and duration of their effect, on the quality of the concrete and the degree of humidity of its surroundings. Assuming that deformation of the concrete with creep is directly proportional to the prevailing stress and assuming a uniform modulus of elasticity E_C, the basic relationship between deformation & constant stress is :

$$\boldsymbol{\varepsilon} = (f_c / E_c) / (1 + \phi)$$

where ϕ = elastic expansion / creep expansion and $\boldsymbol{\varepsilon}$ = strain.

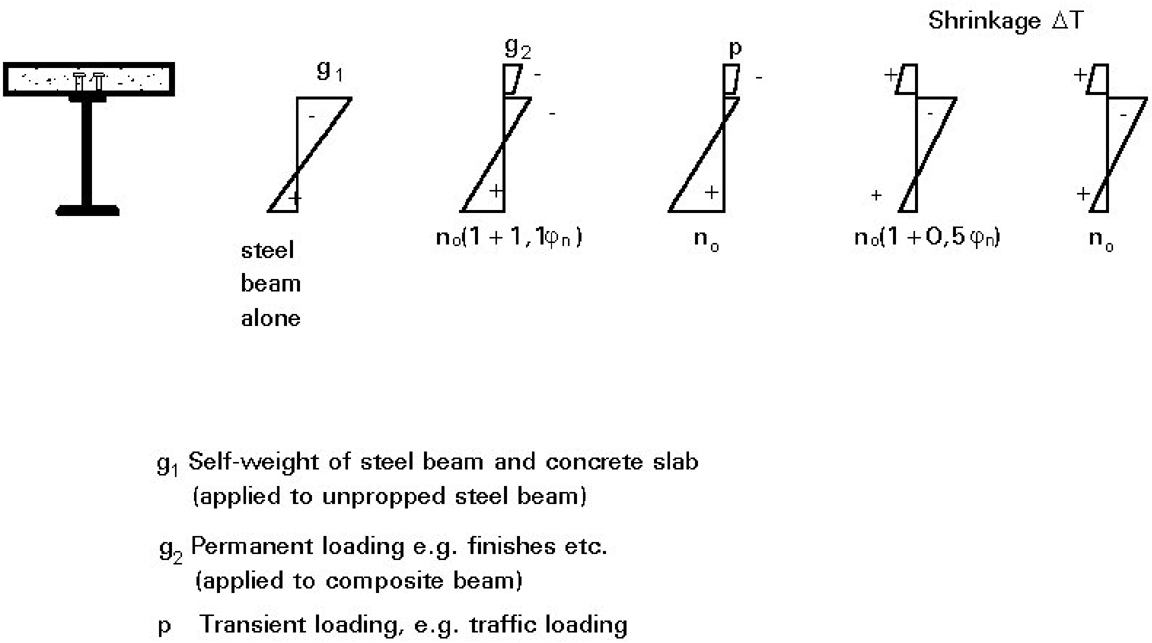


Fig. 6.12 Effect of Creep and Shrinkage on Composite Sections

ii) Design for Creep:

The influence of creep is different according to the method of erection of the composite beam.If the erection is done by case **I**, the concrete dead load is carried by steel alone thus no appreciable creep. If the erection is done by case **II**, the entire dead load is carried by the composite section causing creep.

Hence for case **I**, the stresses in the composite section may be computed neglecting creep. However, for case **II**, stresses in the composite section are computed using a modular ratio 3n for all dead loads and using a modular ratio of n for live loads, as to get more stresses in the steel section in agreement with the phenomena of creep.

Concrete stresses in composite beams are reduced by creep**.** Therefore the maximum concrete stress should be determined by ***neglecting creep***.

6.3.4.2 Shrinkage

If the concrete slab is restrained from shrinkage by the steel girders, internal stresses in concrete and steel independent of external loads will be produced. Similar to the effect of creep, the shrinkage of concrete creates internal tensile stresses in the concrete slab, compression in the top flange and tension in the bottom flange of the steel beams, similar to the effect of creep. The ultimate shrinkage strain in concrete shall be estimated to be equal to 0.0003.

6.3.5 Design For Temperature Effect

The variation of temperature shall be assumed according to the Egyptian Code of Practice for Calculating Design Loads and Forces on Structures. In general, a 30^{o}c uniform variation of the overall temperature of the structure is assumed. Due consideration shall be given for the fact that although the coefficient of thermal expansion for both steel and concrete is identical, the coefficient of thermal conductivity of concrete is only about 2 % of that of the steel. Therefore the top of the concrete slab and other levels through the depth of the girder shall be assumed as shown in Fig. 6.13c .
Such difference in temperature of steel and concrete will create internal stresses similar to those due to shrinkage and creep, Fig. 6.14. These stresses result from the jump of temperature at the area of contact between steel and concrete.
Due consideration of this phenomenon by appropriate method of calculation is recommended.

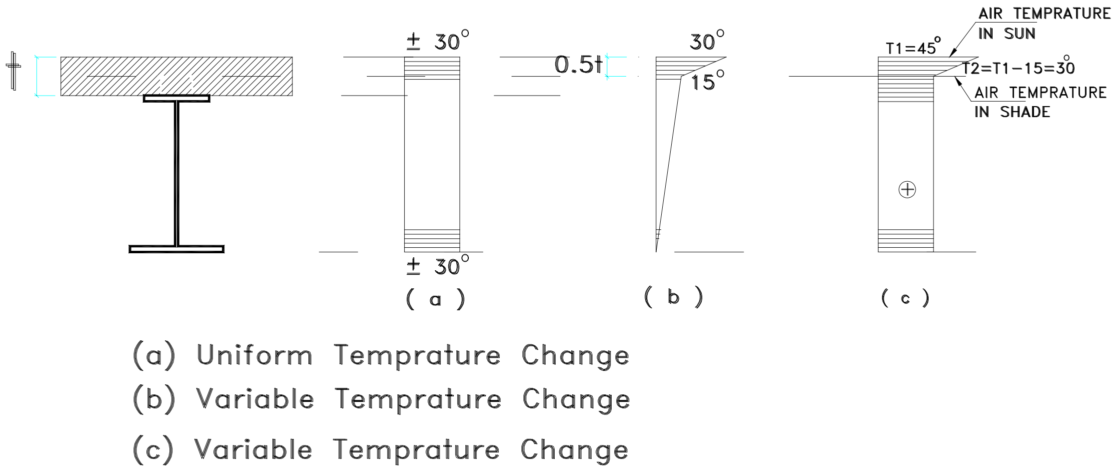


Fig. 6.13 Temperature Distribution

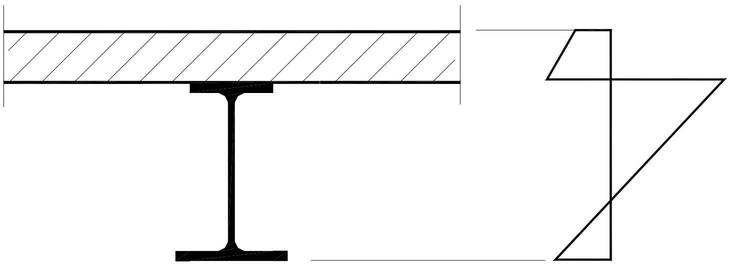


Fig. 6.14 Stress Distribution due to Effect of Temperature

6.3.6 Deflections

If the construction is shored during construction, Case II, the composite section will support both the dead load and the live load deflections. However, if the construction is not shored, Case I, the total deflection will be the sum of the dead load deflection of the steel girder and the live load deflection of the composite section. The deflection allowable limit due to live load without impact is equal to L/800.

6.3.7 Composite Construction in Continuous Span Bridges

i) General:

When the total bridge length is sufficiently long to require multiple spans, the designer can either select a series of simple spans or he can use continuous spans. Simple beam spans has the advantages of:

1) simpler analysis and design,
2) less field splices leading to faster erection,
3) no stresses due to support settlement.

Continuous span construction has the advantages of:

1) less steel weight,
2) less deflection,
3) Fewer number of bearings.

Before a system is selected for a particular bridge, the designer has to study the advantages of both systems and decide accordingly. A two-span continuous bridge has only slight economy over simple spans. The usual bridge structure has three or more spans with the intermediate spans 20 % to 30 % longer than the end spans.

In continuous span bridges, the top concrete slab is subjected to tensile stresses in the negative moment regions. Accordingly, the slab does not contribute to the resistance of the cross section. However, benefit can be taken from the presence of the slab by considering that the longitudinal reinforcement bars, see Fig. 6.15, remain active within an effective width of the slab.

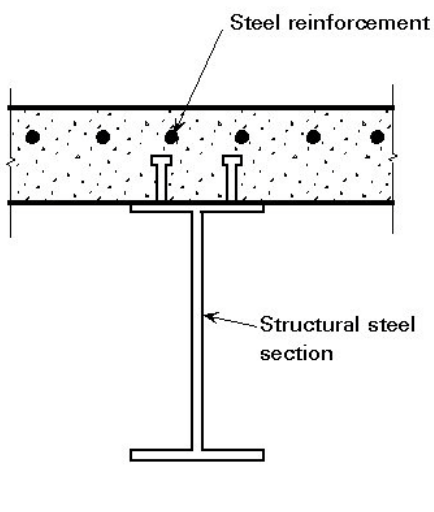


Fig. 6.15 Composite Cross-Section at Interior Supports

This effective width is related to the length of the negative moment region as shown in Fig. 6.7.

ii) Design Considerations:

The area of reinforcement bars within the effective width is added to the steel section of the negative moment region. Geometrical properties of both steel section only and the composite section are calculated and then used to check the bending stresses as explained before in section 6.3.3. In the negative moment regions, the lower flange of the steel girder is subjected to compression and therefore should be checked against lateral and local buckling provisions.

6.4 SHEAR CONNECTORS

6.4.1 Horizontal Shear Force

The horizontal shear force transferred by the connector shall be computed at the interface between concrete and upper flange of the steel girder utilizing the virtual section properties. This horizontal shear must be resisted so that the slip between both materials at the concrete-steel interface will be restrained. Friction between the concrete slab and the steel girder can not be depended upon to provide the required interface shear strength. Instead, the horizontal shear force at the interface between the concrete slab and the steel girder shall be transferred by shear connectors, as shown in Fig. 6.16 to Fig. 6.18, throughout simple spans and positive moment regions of continuous girders. In negative moment regions, shear connectors shall be provided when the reinforcing steel embedded in concrete is considered as part of the composite section, see section 6.3.7.

6.4.2 Connector Capacity

Ideally, to obtain a fully composite section, the connector should be stiff enough to provide the complete interaction; i.e., no slip at the interface. This, however, would require that the connectors be infinitely rigid. Also, since the shear force varies along the girder length, the distribution of the shear connectors should be such that more connectors are used at high shear locations.

6.4.3 Connector Design

If the dead load stresses are carried by the steel section, e.g., unshored construction, the connectors may be designed to carry the shearing forces due to live loads only. But to allow for shrinkage and creeping effects and to give better security against slip, it is recommended to design the connector to carry shearing forces due to half the dead load in addition to the live load. For shored construction, the connectors are to be designed to carry the shearing forces due to dead and live loads.

To design the connector, the longitudinal shearing force per unit length of the girder is calculated as:

$$\tau_c = Q\, A_c\, y_c / I_v$$

where:

A_c = Area of concrete section without haunches

y_c = Distance between central axis of concrete section and that of the composite section.

If the spacing between the connectors is equal to "e", then the total horizontal shear to be transmitted by one connector along a pitch "e" is :

$$e * \tau_c = e * (Q\, A_c\, y_c / I_v)$$

This value should be less than the allowable load the connector can carry, denoted by R_{sc}, i.e.,

$$e * (Q\, A_c\, y_c / I_v) < R_{sc}$$

From this equation, the connector spacing e can be calculated as:

$$e = R_{sc} * I_v / (Q\, A_c\, y_c)$$

Thus the pitch "e" is inversely proportional to Q and the connectors are to be arranged closer to each other at the supports and at bigger intervals near the middle of the girder.

In the following section, different types of shear connectors used in composite construction are described. In addition, it applies to the calculation of the allowable horizontal shear load, R_{sc}, for one connector. The value of R_{sc}

computed from the following formulas shall not exceed the allowable horizontal load, R_w, provided by the connector connection to the girder flange.

6.4.4 Connector Types

Various types of such connectors are shown in Figures 6.16 thru 6.18. The most common types are the anchor and hoop connectors, the block connector, the angle and channel connectors, and the stud connectors.

6.4.4.1 Anchors and hoops

a- Anchors and hoops (Fig. 6.16) designed for longitudinal shear should point in the direction of the diagonal tension. Where diagonal tension can occur in both directions, connectors pointing in both directions should be provided.
b- Hoop connectors (diameter = φ .. shall satisfy the following:

$r \geq 7.5\varphi$ $\quad$ $L \geq 4r$ $\quad$ concrete cover $\geq 3\ \varphi$

c- Development length and concrete cover of anchors shall be based on the allowable concrete bond stresses as per the Egyptian Code of Practice for the Design of Reinforced Concrete Structures.

d- The allowable horizontal load for each leg of anchors and hoops shall be computed as follows:

$$\mathbf{R_{sc} = 0.58\, A_s\, F_{ys}\, \cos\beta\ / (1 + \sin^2\alpha)^{1/2} \leq R_w}$$

Where

$\mathbf{A_s}$	=	Cross sectional area of anchor or hoop
$\mathbf{F_{ys}}$	=	Yield stress of anchor or hoop material
$\boldsymbol{\beta}$	=	Angle in horizontal plane between anchor and longitudinal axis of the girder.
$\boldsymbol{\alpha}$	=	Angle in the vertical plane between anchor or hoop and the girder upper flange.

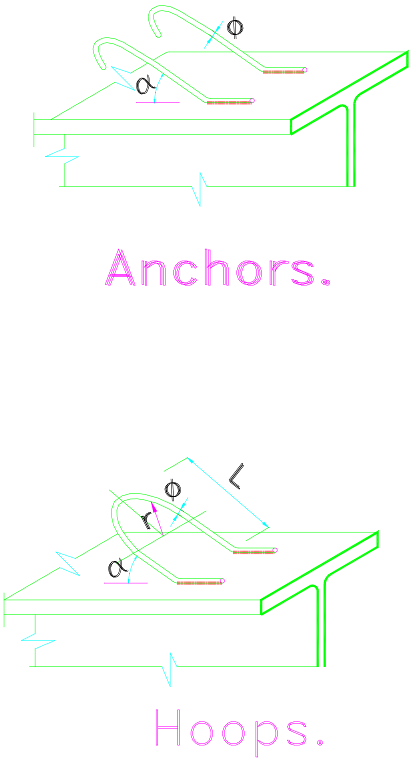


Figure (6.16) Anchor & Hoop Shear Connectors

6.4.4.2 Block Connectors

Block connectors (Fig. 6.17) such as bar, T-section, channel section and horseshoe can be used as shear connectors. The front face shall not be wedge shaped and shall be so stiff that uniform pressure distribution on concrete can be reasonably assumed at failure.

a- Block connectors shall be provided with anchoring devices to prevent uplift of concrete slab.
b- The height of bar connectors shall not exceed four times its thickness.
c- The height of T-sections shall not exceed ten times the flange thickness or 150 mm, whichever is the least.
d- Channel sections shall be hot rolled with a web width not exceeding 25 times the web thickness. The height of the connectors shall not exceed 15 times the web thickness nor 150 mm, whichever is the least.
e- The height of horseshoe connectors shall not exceed 20 times the web thickness nor 150 mm, whichever is the least.
f- The allowable horizontal load (R_{be}) transmitted by bearing can be computed from the following Equation:

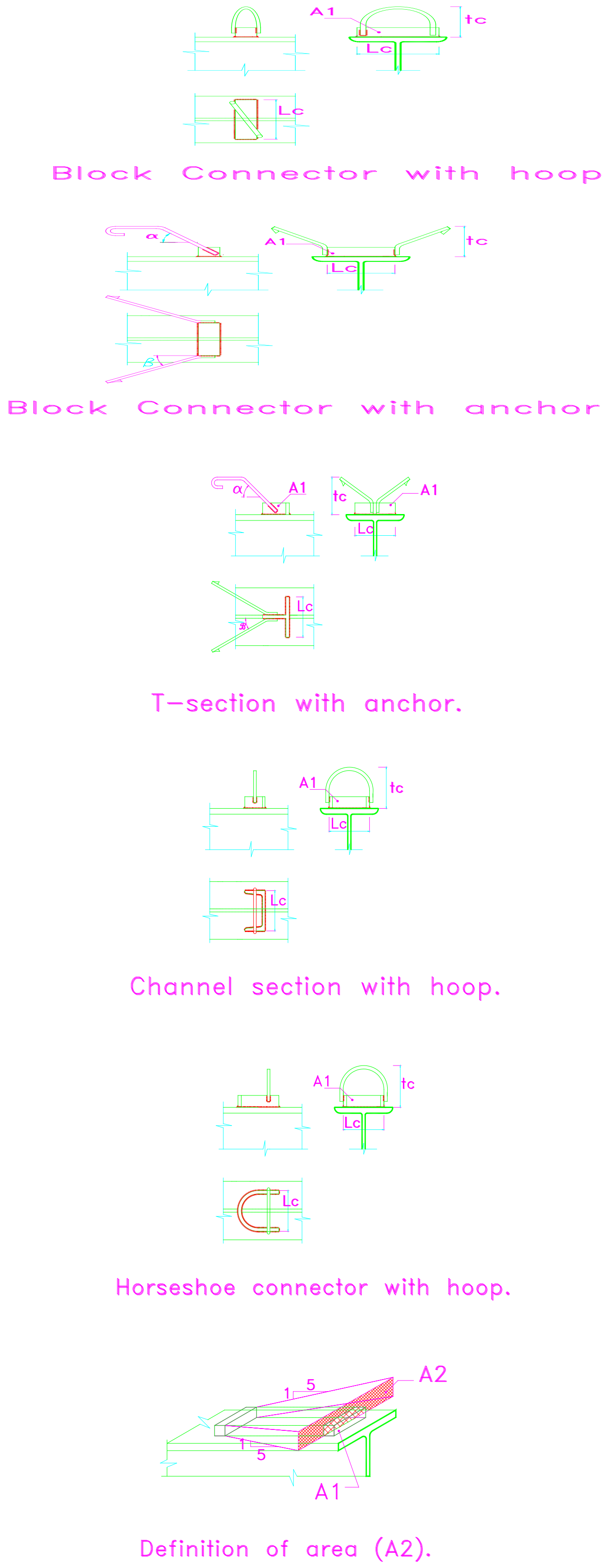


Fig. 6.17 Block Shear Connectors

$$R_{bl} = 0.3\ \eta\ A_1\ f_{cu}$$

Where

η	=	$(A_2/A_1)^{1/2} \le 2$
A_1	=	Area of connector front face
A_2	=	Bearing area on concrete, shall be taken as the front area of the connector, A_1, enlarged at a slope of 1:5 (see Fig. 6.17) to the rear face of the adjacent connector. Only parts of A_2 falling in the concrete section shall be taken into account.

g- Block connectors shall be provided with anchors or hoops sharing part of the horizontal load supported by the connector, provided that due account shall be taken of the differences of stiffness of the block connector and the anchors or hoops. The allowable horizontal load per connector can be computed from the following:

$$R_{sc} = R_{bl} + 0.5\ R_{an} \le R_w$$

And $$R_{sc} = R_{bl} + 0.7\ R_h \le R_w$$

Where

R_{an} = Horizontal load supported by anchor

R_h = Horizontal load supported by hoop

6.4.4.3 Channel Shear Connectors

The allowable horizontal load, R_{sc}, for one channel shear connector, Fig. 6.18, shall be computed from the following Equation:

$$R_{sc} = 0.12\ (t_f + 0.5 t_w)\ L_c\ (f_{cu}\ E_c)^{1/2} \le R_w$$

where: t_f , t_w = flange and web thicknesses, cm,

L_c = connector length, cm.

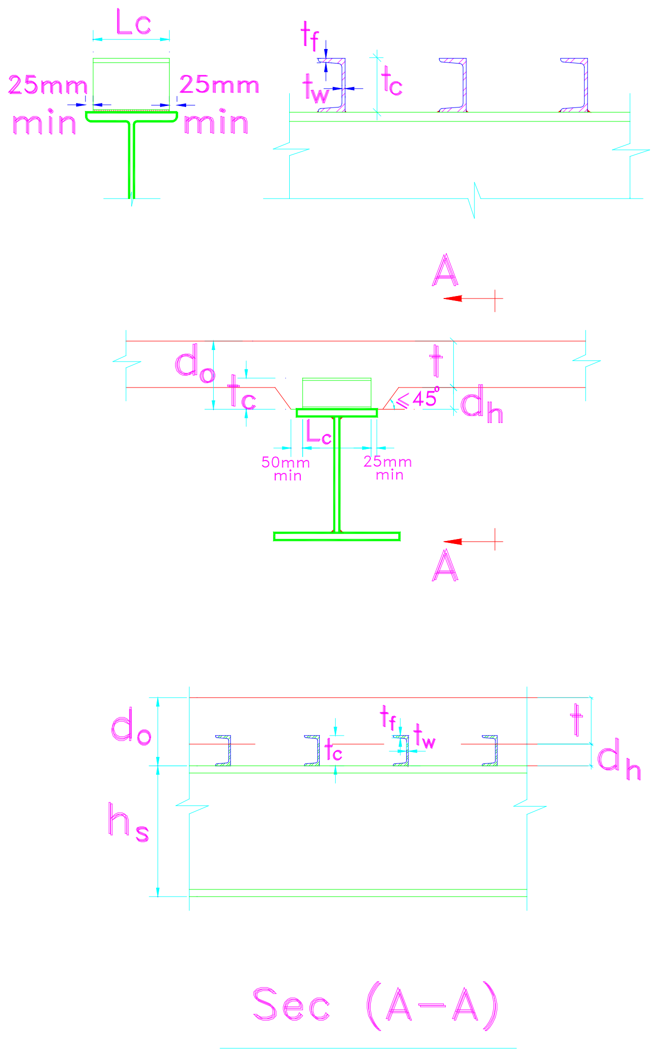


Fig. 6.18 Channel Shear Connectors

5.4.4.4 Angle Connectors

The height of the outstanding leg of an angle connector shall not exceed ten times the angle thickness or 150 mm, whichever is the smaller. The length of an angle connector shall not exceed 300 mm (Fig. 6.19).

The allowable horizontal load for an angle connector shall be computed as follows:

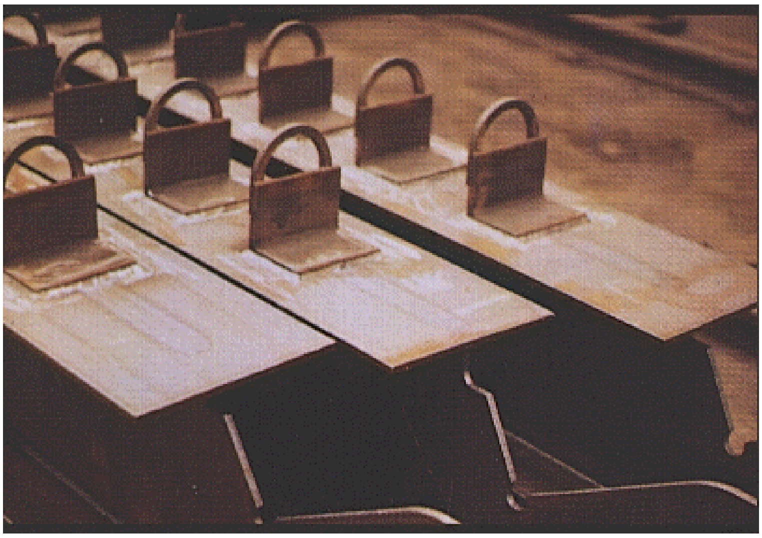
$$\mathbf{R_{sc} = 4\, L_c\, t_c^{3/4}\, f_{cu}^{2/3} \leq R_w}$$

where

$\mathbf{L_c}$ = Length of the angle connector ,cm.

$\mathbf{t_c}$ = Width of the outstanding leg of the angle connector, cm.

And $\mathbf{f_{cu}}$ in kg/cm2



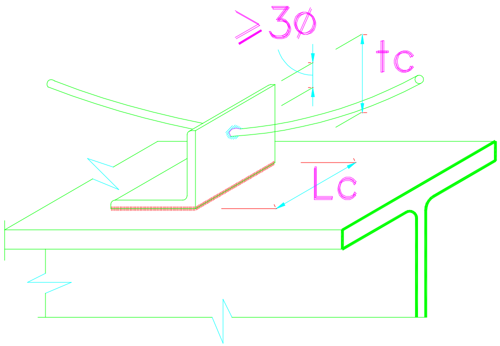


Fig. 6.19 Angle Shear Connectors

It is recommended to provide a bar welded to the angle to prevent uplift of the concrete slab, the minimum diameter of the bar shall be computed from the following:

$$\Phi \geq 0.45\ (R_{sc} / F_{ys})^{1/2}$$

Where

Φ = Diameter of the bar, cm
F_{ys} = Yield stress of the bar, kg/cm^2
R_{sc} = Allowable shear load for one angle connector.

The length of the bar on each side of the angle connector standing leg shall be computed based on the allowable bond strength of concrete following the provisions of the Egyptian Code of Practice for Design of Reinforced Concrete Structure.

6.4.4.5 Stud Connectors

Despite this wide range of connector types, the stud connector, Fig. 6.20 and 6.21, has now become the primary method of connections for composite beams. The stud can be forge welded to the steel section in one operation using a special hand held welding machine, see Fig. 6.20. These machines allow operators to weld approximately 1000 studs per day. Fig. 6.21 shows a typical shear stud before and after welding.

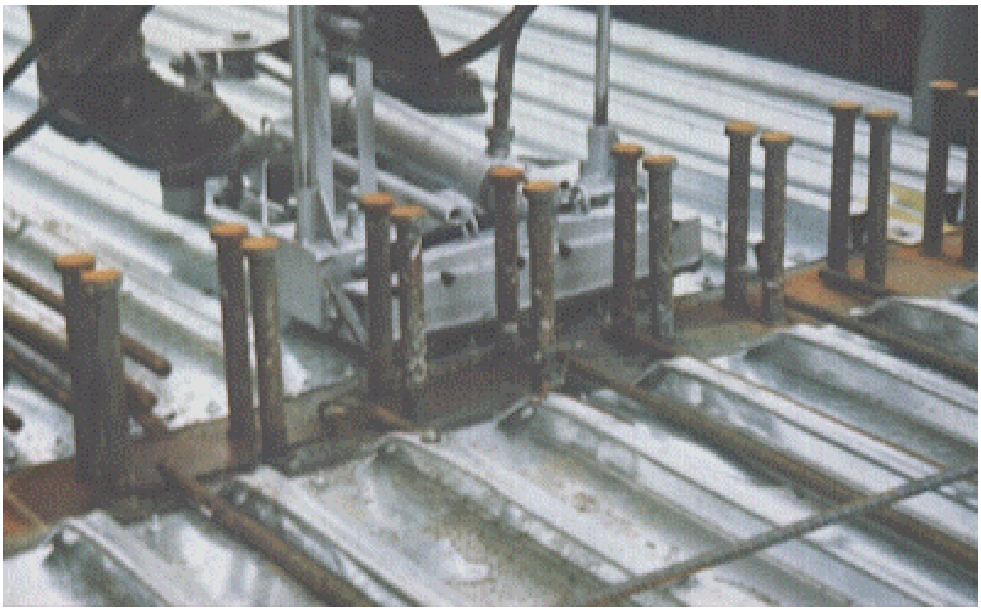
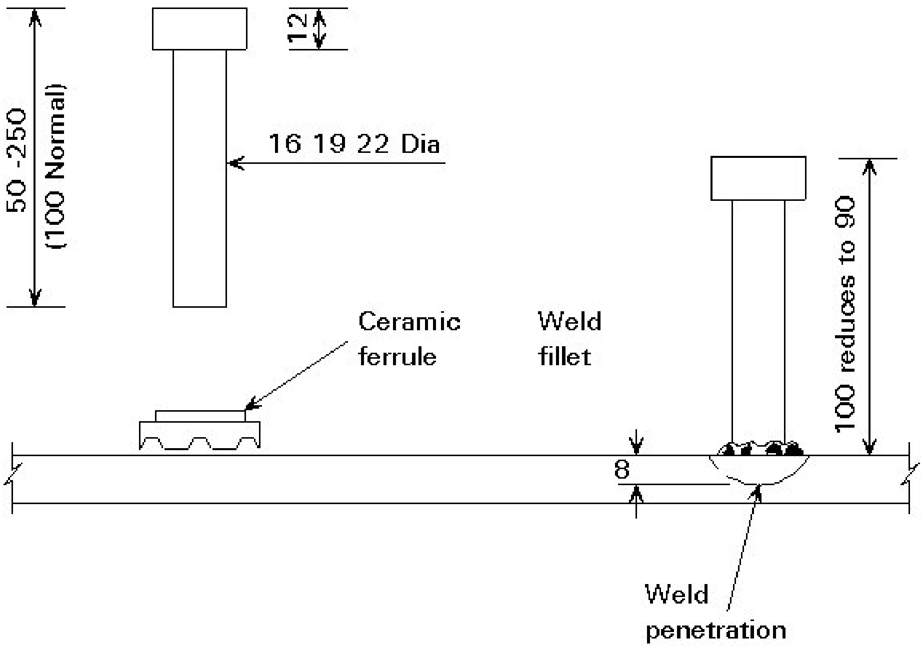


Fig. 6.20 Automatic Welding of Stud Shear Connectors

The length of the stud connector shall not be less than four times its diameter, d_s, after installation. The nominal diameter of the stud head shall not be less than one and half times the stud diameter, d_s. The value of d_s shall not exceed twice the thickness of the steel girder top flange. Except where formed steel decks are used, the minimum center-to-center spacing of studs shall be ($6d_s$) measured along the longitudinal axis of the girder ; and ($4d_s$) transverse to the longitudinal axis of the supporting composite girder , (Fig. 6.22). If stud connectors are placed in a staggered configuration, the minimum transversal spacing of stud central lines shall be $3d_s$. Within ribs of formed steel decks, the minimum permissible spacing is $4d_s$ in any direction.



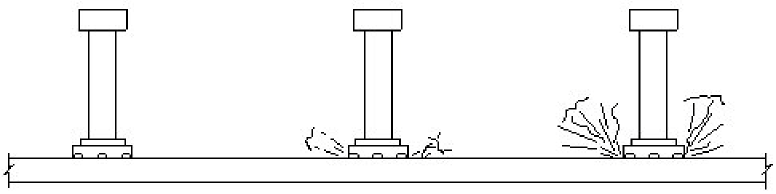


Fig. 6.21 Stud Shear Connectors

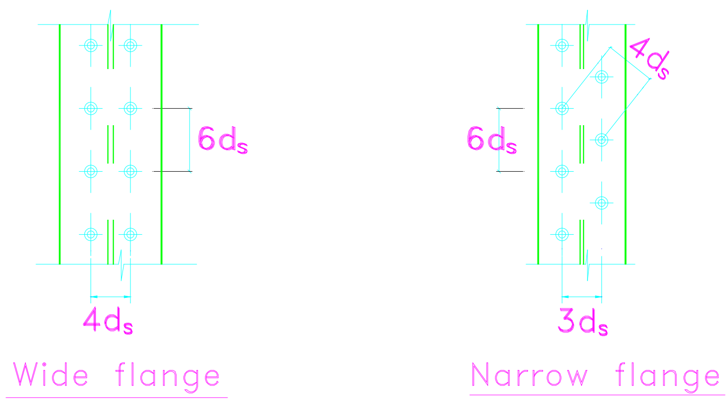


Fig. 6.22 Minimum Spacing of Stud Connectors

The allowable horizontal load, $\mathbf{R_{sc}}$, for one stud connector shall be computed from the following formula:

$$\mathbf{R_{sc} = 0.17\, A_{sc}\, (f_{cu}\, E_c)^{1/2} \leq R_w}$$
$$\mathbf{\leq 0.58\, A_{sc}\, F_y}$$

where:

$\mathbf{A_{sc}}$ = Cross sectional area of stud connector, cm^2
$\mathbf{F_y}$ = The yield stress of stud steel connectors

6.4.5 General Requirements for Shear Connectors

6.4.5.1 Connection to Steel Flange

The connection between the shear connectors and the girder flange shall be designed to resist the horizontal shear load acting on the connector; section 6.4.4.

6.4.5.2 Concrete Cover

a- In order to ensure adequate embedment of shear connectors in concrete slab, the connector shall have at least 50 mm of lateral concrete cover. On the other hand, the minimum concrete cover on top of the connector shall not be less than 20 mm.

b- Except for formed steel slab; the sides of the haunch should lie outside a line drawn at maximum of 45^o from the outside edge of the connector. The lateral concrete cover from the side of the haunch to the connector should be not less than 50 mm.

6.4.5.3 Placement and Spacing

Except for stud connectors, the minimum center-to-center spacing of shear connectors shall not be less than the total depth of the slab including haunch, $\mathbf{d_o}$. The maximum center-to-center spacing of connectors shall not exceed the least of the following:

- 60 cm
- Three times the total slab thickness (d_o)
- Four times the connector height including hoops or anchors, if any.

However, the maximum spacing of connectors may be exceeded over supports to avoid placing connectors at locations of high tensile stresses in the steel girder upper flange.

6.4.5.4 Dimensions of Steel Flange

The thickness of steel flange to which the connector is fastened shall be sufficient to allow proper welding and proper transfer of load from the connector to the web plate without local failure or excessive deformations. The distance between the edge of a connector and the edge of the girder flange to which it is welded should not be less than 25 mm .

6.4.5.5 Concrete Slab Edges

Concrete slab edges shall be provided with end closures, e.g. channels, angles, or plates, as shown in Fig. 6.23. End closures have to be fixed to the steel girders before casting the concrete slab. Besides minimizing grout loss during casting of concrete, end closures enhance the shear connectivity between concrete slab and steel girders at zones of maximum shear forces. End closures also help in resisting forces arising from shrinkage and creep.

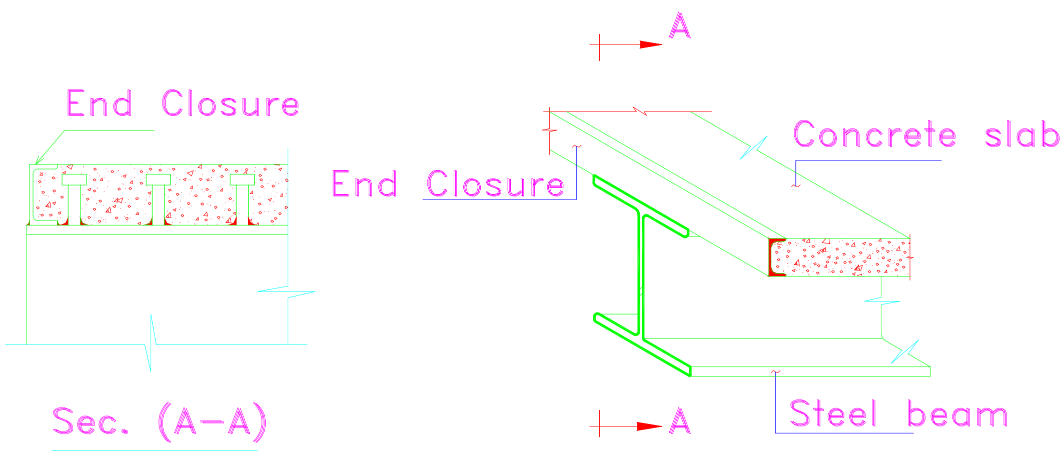


Fig. 6.23 End Closure for Concrete Slab

6.5 DESIGN EXAMPLE:

The design example presented in chapter 5 is used here to illustrate the method of design of composite plate girders. The example uses the same values of the straining actions at the middle section.

The selection of the girder cross section is essentially a trial-and-error procedure in which a trial section is assumed and used to check the resulting stresses in both steel and concrete:

The following section is assumed:

a) Web 2250×14

b) Top Flange 400×12

($b_f / 2t_f = 40 / (2 \times 1.2) = 16.667 > 21 / \sqrt{f_y} = 11$

No problem since flange is prevented from local buckling by deck slab)

c) Bottom Flange 600×32

Section properties are then computed for the following cases:

a) Steel section only:

Centroid $Y_{us} = 143.391$ cm

Intertia $I_s = 3953428\ cm^4$

Section Moduli $Z_{us} = 27571\ cm^3$

$Z_{ls} = 45965\ cm^3$

b) Effective Slab Width:

For the right girder:

b_{ER} = b* = **<u>150</u>** cm (Side Walk Slab)

b_{EL} = smaller of:

1) Span/8 = 27.5 /8 = 3.4375 m
2) Spacing /2 = 7/2 = 3.5 m
3) 6 ts = 6 * 22 = **<u>132</u>** cm governs

Total effective width $b_{ER} + b_{EL}$ = 150 + 132 = 282 cm

c) Composite section with n = 9 (F_{cu} = 300 kg / m^2)

Centroid $Y'_{us} = 57.862$ cm
Intertia $I_v = 11309956$ cm^4
Section Moduli $Z'_{us} = 195465$ cm^3
$Z'_{ls} = 65933$ cm^3
$Z_{uc} = 141619$ cm^3

d) Composite section with n = 3 × 9 = 27 (Effect of Creep)

Centroid $Y'_{us} = 98.186$ cm
Intertia $I_v = 7836139$ cm^4
Section Moduli $Z'_{us} = 79809$ cm^3
$Z'_{ls} = 59720$ cm^3
$Z_{uc} = 65200$ cm^3

Check of Bending Stresses:

a) Non-Shored Construction:

Load	Upper Steel (-) t/cm^2	Lower Steel (+) t/cm^2	Upper Concrete kg / cm^2
DL 1	$F_{us} = 385 \times 100/27571 = 1.396$	$F_{ls} = 385 \times 100/45965 = 0.838$	= 0 for non-shored construction
DL 2	$F_{us} = 115 \times 100/79809 = 0.144$	$F_{ls} = 115 \times 100/59720 = 0.193$	$F_{uc} = (115 \times 100/65200) * (1000/27) = 6.533$
LL + I	$F_{us} = 700 \times 100/195465 = 0.358$	$F_{ls} = 700 x 100/65933 = 1.062$	$F_{uc} = (700/141619) * (1000/9) = 54.92$
Total	**1.899**	**2.092**	**61.453**

Checks:
1- Compression at Upper Steel :

a) Total stress: $F_{us} = 1.899 < F_b = 2.1$ t/cm^2
(compression flange is laterally supported by deck slab)

b) Due to D.L. only Fus = **1.396** t / cm2
compression flange is laterally supported by upper bracing with
$L_u = 4.5$ m, $r_T = 8$ cm

$$L_u/r_T = 450 / 8 = 56.25$$

$$\mathbf{L_u / r_T \le 188\sqrt{\frac{C_b}{F_y}}} = 99$$

$$\mathbf{F_{ltb2} = (0.64 - \frac{(L_u / r_T)^2 F_y}{1.176x10^5 C_b})F_y \le 0.58\ F_y = \underline{1.955}}\ \text{t/cm2}$$

$$\mathbf{F_{us} < F_{LTB}\quad O.K.}$$

2. Tension at Upper Steel :

a) Total Tension: $f_{ls} = 2.092 < F_b = 2.10$ t/cm^2

b) Fatigue $f_{sr} = 0.5 \times 1.062 = 0.531 < F_{sr} = 1.02$ t/cm^2

{The allowable fatigue stress range (F_{sr}) is obtained as follows:
* From ECP Table 3.1.a: ADTT >2500, Number of cycles = 2×10^6
Detail Class = B′ (case 4.2 of Table 3.3)
Table 3.2 gives F_{sr} = <u>1.02</u> t/cm^2 > f_{sr} }

3. Compression on Upper Concrete:

Concrete $f_{uc} = 61.453 < 70$ kg/cm^2

<u>b) Shored Construction:</u>

		DL1 + DL2		LL + I	
Upper Steel	f_{us}=	50000 / 79809	+	70000 / 195465 =	
	=	0.626	+	0.358	= **<u>0.985</u>** t/cm^2
Lower Steel	f_{ls} =	50000 / 59720	+	70000 / 65933 =	
	=	0.837	+	1.062	= **<u>1.899</u>** t/cm^2

Upper Concrete: f_{uc}=50000*1000 / (65200*27) + 70000*1000 / (141619*9)
= 28.403 + 54.92 = **<u>83.323</u>** kg/cm^2

Code recommends to neglect creep in computing concrete stresses:

Upper Concrete: f_{us}=50000*1000 / (141619*9) + 70000*1000 / (141619*9)
= 39.229 + 54.92 = **<u>94.149</u>** kg/cm^2

Note that shored construction results in decrease of steel stresses and increase in concrete stresses.

Design of Shear Connectors

Assuming non-shored construction, the shear force to be carried by the connectors at the support is:

$$Q_c = 0.5 * Q_{DL1} + (Q_{DL2} + Q_{LL+I})$$
$$= 0.5 \times 62 + (18 + 100) = 149 \text{ tons}$$

The shear / unit length is $\tau = Q_c\, S_c / I_v$

Where S_c = first moment of area of the concrete slab about the neutral axis of the composite section $= A_c * y_c$

$$= (282 \times 22/9) * (57.862+11) = 47469 \text{ cm}^3$$

$$\tau = 149 \times 47469 / 11309956 = 0.625 \text{ t / cm'}$$

a) Stud Connecters:

i) Stud Capacity:

The allowable load of one stud connector is computed as:

$$R_{sc} = 0.17\, A_{sc}\, (f_{cu}\, E_c)^{1/2} \leq 0.58\, A_{SC}\, F_y$$
$$\leq R_w$$

using ϕ 24 mm studs, $A_{sc} = \pi / 4\, (2.4)^2 = 4.52 \text{ cm}^2$

$$F_{cu} = 300 \text{ kg / cm}^2 = 0.3 \text{ t/cm}^2$$
$$E_c = 240 \text{ t/cm}^2$$
$$R_{sc} = 0.17 \times 4.52\, (0.300 \times 240)^{1/2} = \underline{\mathbf{6.52}} \text{ t}$$

$$\text{OR} = 0.58\, A_{sc} \times F_y = \underline{\mathbf{6.29}} \text{ t}$$

ii) Stud Connection Capacity:

The allowable stress range in shear on the nominal area of the stud (case 26 of table 3.3) is equal to 0.4 t/cm^2 according to table 3.2; i.e.,

$$R_w = \text{Shear range / stud} = \pi / 4\, (2.4)^2 \times 0.4 = \underline{\mathbf{1.808}} \text{ t governs}$$

$$\tau_{sr} = (0.5 \times 100) \times 47469 / 11309956 = 0.210 \text{ t/cm'}$$

Using 3 studs per row:

$$\text{Spacing } e \le R/\tau_{sr} = 1.808 \times 3/\ 0.21 = 25.846 \text{ cm}$$

Use studs at 20 cm (check: $e \ge 6d = 14.4$ cm)

b) Channel Connector:

The allowable load on one channel connector is calculated from:

$$R_{sc} = 0.12\ (1 + 0.3) \times 20\ (0.3 \times 200)^{1/2} = 24 \text{ t}$$

However, the design is usually governed by the fatigue capacity of the welded connection between the channel and the top flange computed as follow:

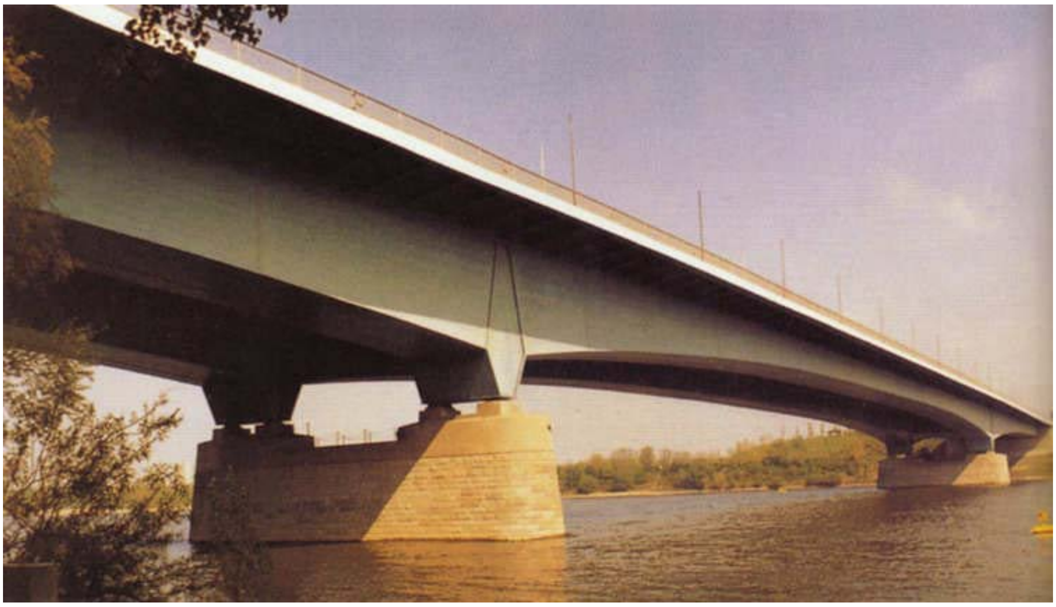
According to Case 24 of table 3.3 $\rightarrow$ Fatigue Class E′
From Table 3.2 $\rightarrow$ $\tau_{sr} = 0.41$ t/cm^2

Assuming 2 x 20 cm length of 5 mm weld, then

$$R_w = 2 \times 20 \times 0.5 \times 0.41 = 8.2 \text{ t}$$

Connector Spacing = 8.2/0.21 = 39.05 cm

Use Channel C12 spaced at 30 cm.



CHAPTER 7

BOX GIRDER BRIDGES

CHAPTER 7

BOX GIRDER BRIDGES

7.1 INTRODUCTION

Box girders have two properties which can offer substantial advantages in certain circumstances over plate girders:

1- they possess torsional stiffness, and
2- their flanges can be made much wider, thus solving the problem of providing a large steel area within a narrow width of plate.

As well as being advantageous in the completed condition of the bridge, these properties can also make box girder bridges simpler in principle to erect. The problems of lateral torsional buckling, for example, do not arise, and if a bridge is to be erected by cantilevering, very long unsupported spans can be adopted.

For medium span bridges, box girders offer an attractive form of construction. Design and construction techniques already popular and common for plate girder bridges can be utilized to produce box girder bridges of clean appearance whilst maintaining relative simplicity and speedy construction procedures. The scope of application of such designs could cover the medium span range from about 45 m to 100 m.

Until 1940 the structural possibilities for box girders were limited; most bridge girders were assembled from rolled sections, plates and riveted connections. With the development of electric welding and precision flame cutting, the structural possibilities increased enormously. It is now possible to design large welded units in a more economical way.

A bridge box girder consists of, see Fig. 7.1 :

1. a concrete deck or an orthotropic steel deck serving as the top flange.
2. a stiffened plate as a bottom flange.
3. Web plates, vertical or inclined.
4. stiff diaphragms or cross bracings at supports, and lighter diaphragms or cross bracings at distances of about 2.5 times the construction depth.

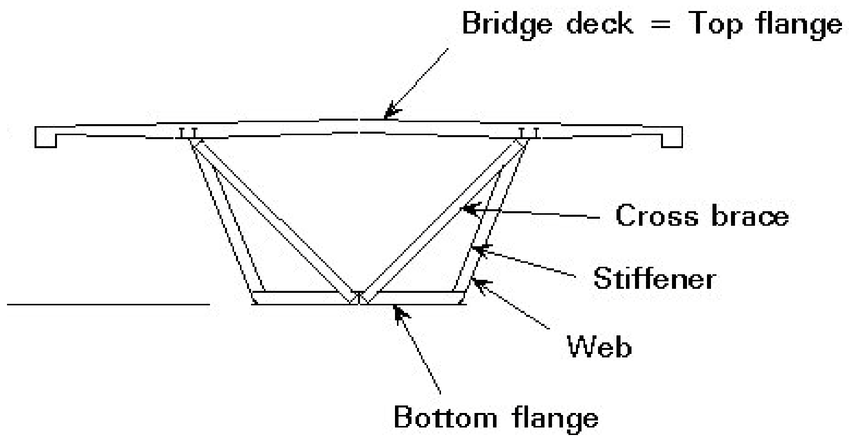


Fig. 7.1 Components of a Box Girder

Clearly, the feature which differentiates the behavior of box girder bridges from plate girder bridges is the much greater torsional stiffness of the closed section. The prime effect this has on global bending behavior is to share the vertical shear more equally between the web plates. Consequently upon this equal sharing, bending stresses in the flange plates are also more evenly shared. As a result, box girders behave more efficiently – there is less need to design for peak load effects which occur on only one plate girder at a time.

Box girders, in addition, have other advantages over plate girders which make their use attractive; such as:

i- a much neater appearance since the stiffening can remain invisible inside the box.
ii- the use of inclined webs provides an efficient aerodynamic shape which is important for long span bridges, see Fig. 7.2.

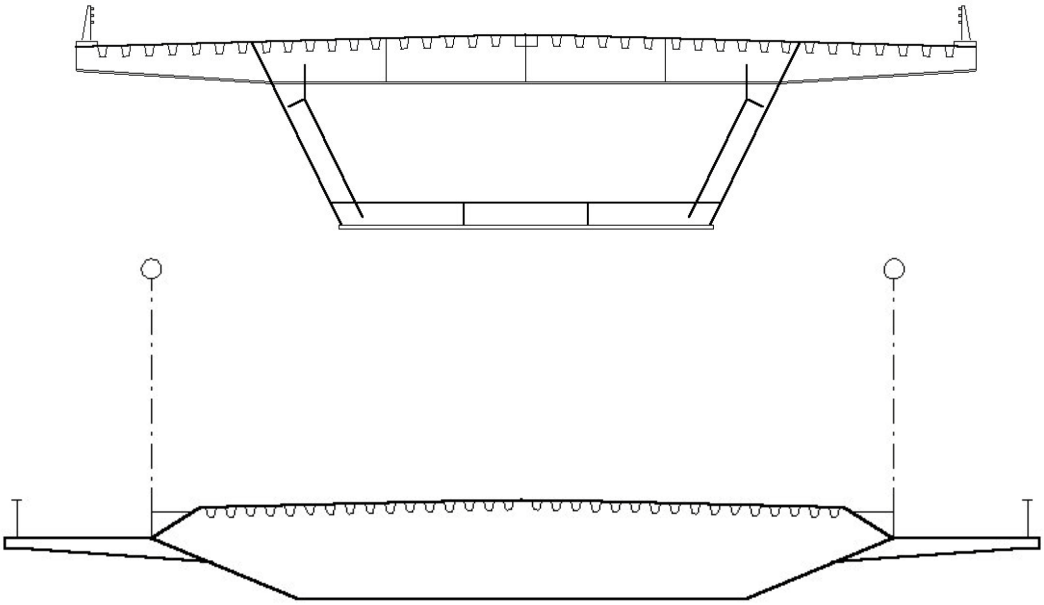


Fig. 7.2 Box Girder Section for Long Span Bridges

Usual Span Ranges: Box girders are suitable for longer span than plate girders and allow larger span-to-depth ratios. The span to depth ratio will normally be around 20 to 25 for simple girders and around 25 to 35 for continuous girders. It is possible to reduce the depth, if necessary, at the expense of additional steel. The above ratios are valid for roadway bridges. For railway bridges the ratios should be smaller, say 15 and 20. The following table gives the economic span limits for roadway bridges:

	Composite concrete deck	Orthotropic deck
Simple span	20 – 100	70 – 120
Continuous spans	30 – 140	100 – 250

The longest span so far is 300 m achieved in 1974 by costa de silva bridge in Rio de Janeiro.

7.2 CROSS SECTION ARRANGEMENTS

Box girder bridges can be constructed with single, twin or multiple box girders. Generally, ***single box*** forms, Fig. 7.3 are limited to bridges of fairly narrow width (although some very wide shallow single boxes have been used in long span suspension bridges which are outside the scope of this book).

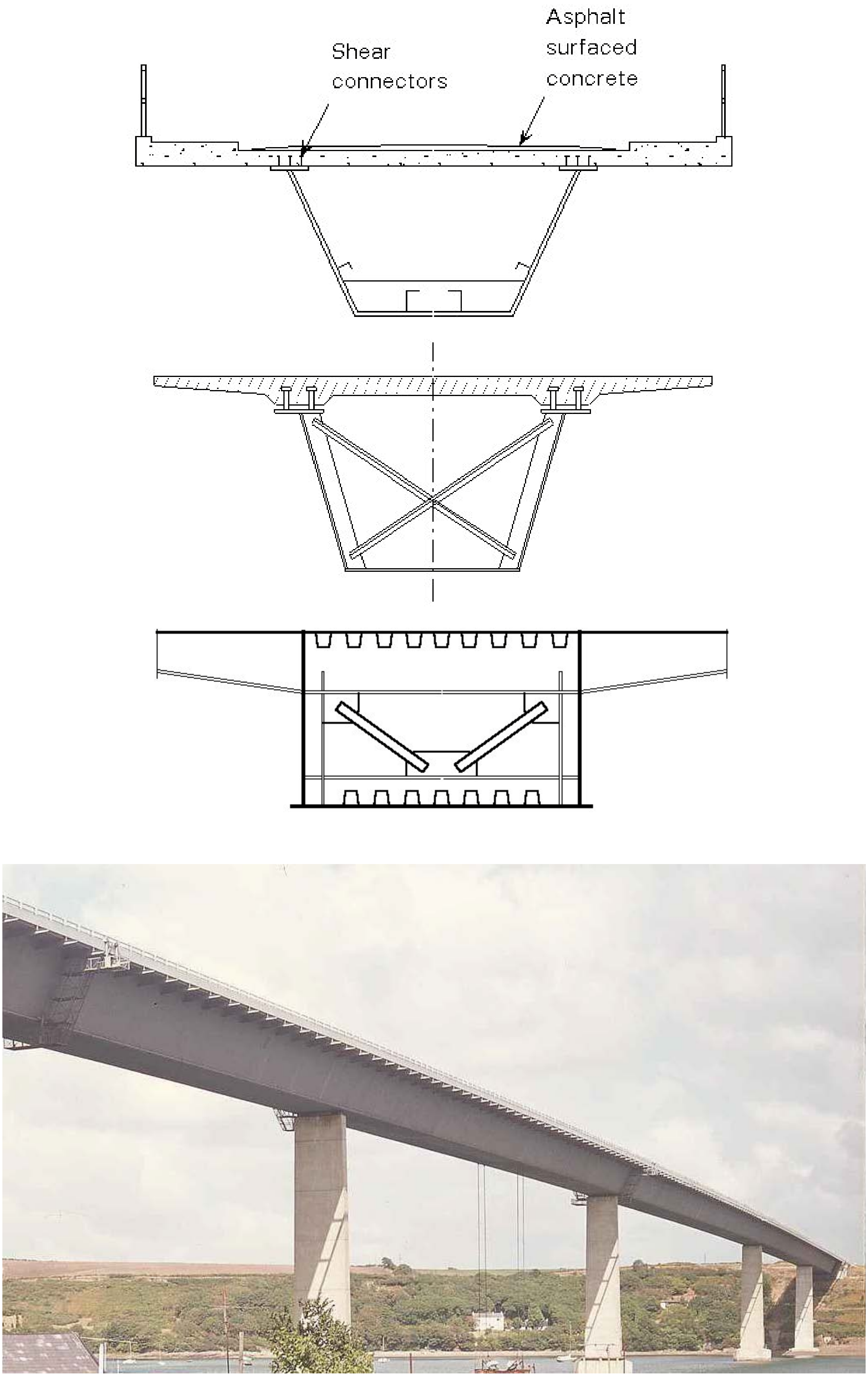


Fig. 7.3 Box Girder Bridge Sections with One Box

Typical width of the boxes themselves would be between about 2 m and 4 m although both wider and narrower boxes have been used (it is common practice for the box to be made complete in the fabricator's works in order to minimize site assembly; boxes much in excess of 4 m in width, whilst perfectly practical to design and fabricate, can cause problems in transport). Cantilever brackets would normally be made between about 3 m and 5 m long, and cross girders between 10 m and 15 m in length (again, of course, these are not absolute limits, merely common practice). Using these figures gives total bridge widths of between 8 m and 14 m for a single box with cantilevers either side, and between 20 m and 33 m for twin boxes with cross girders between them and cantilevers outside the outer webs.

The most usual layout for bridges of medium span consists of ***two longitudinal box girders***, interconnected by cross girders which are usually fabricated plate girders and having also fabricated plate cantilever brackets projecting beyond their outer webs (Fig.7.4). The deck can either be of reinforced concrete or of orthotropic stiffened steel plate. If the deck is of concrete, it will act compositely with the main box girders and also with the transverse plate girders and cantilevers; if of steel plate, it forms parts of the flanges of the boxes. As general guidance, a reinforced concrete deck would be used if the bridge span is less than about 150 m, and an orthotropic steel deck if it is over 200 m. Between these limits consideration should be given to either form of construction. It must always be remembered that special considerations may require the use of a particular type of deck outside the suggested span ranges quoted above.

An alternative solution to two lane bridges involves carrying each lane on its own individual single box girder. Such a layout has a number of advantages in addition to overcoming the problem of fitting a cross girder accurately between two longitudinal girders present in plate girder bridges. The use of separate structures for the two lanes of a dual lane bridge ensures that even if one superstructure is damaged or even destroyed the bridge can continue to be used whilst it is being repaired or replaced, by diverting two way traffic on to the remaining structure.

Multiple boxes are needed for wider roads. Alternatively, wide roads can be carried on twin box sections with cross girders, so that the deck slab works longitudinally, rather than transversally between the lines of the box webs.

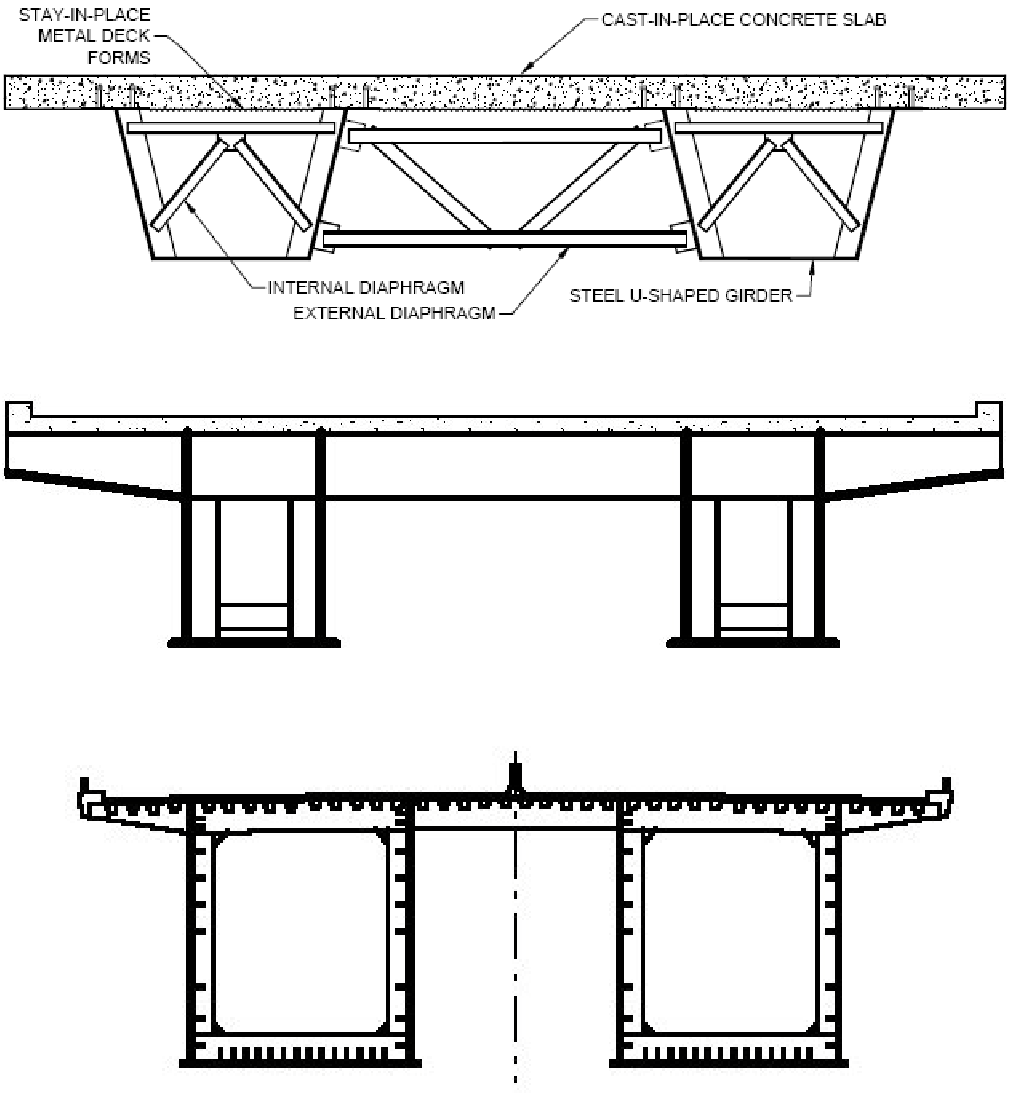


Fig. 7.4 Box Girder Bridge Sections with Two Boxes

When a reinforced concrete slab is used for the deck, the steel girders may be closed box sections or may be open sections (U – shaped) which are closed when the slab is cast. In this case separate (and relatively small) flange plates are provided at the top of each web. These flanges need to be stabilized laterally by upper horizontal bracing during construction. Shear connectors are arranged on the top of these flanges to ensure composite action.

When the bridge deck is steel, the top flange plates are stiffened orthotropically to carry traffic wheel loads as well as acting as the top flange of the box girder. This stiffening usually takes the form of longitudinal trapezoidal ribs supported at regular intervals by transverse beams.

In the descriptions above it has been implicitly assumed that the steel boxes themselves are of rectangular cross-section. Whilst this is probably the commonest cross-section, there is no reason in principle why the webs should be vertical; many boxes from the smallest to the largest have had sloping webs. In some cases of very large boxes this provides an expedient by which a two lane deck maybe carried on a single box. The loading on the deck may be transferred to the webs through deck slab action or through cross girders inside the box bearing on transverse stiffeners on the webs. If the depth of the web varies according to the bending moment requirements, the use of non-vertical webs should be avoided since this combination would give rise to extremely awkward detailing problems, and could sometimes result in ugly appearance of the bridge

7.3 BEHAVIOUR OF BOX GIRDER BRIDGES

7.3.1 Structural Analysis

A global structural analysis of the bridge is usually required in order to establish the maximum forces and moments at the critical sections of the bridge under the variety of possible loading conditions. Local analysis of the deck slab is usually carried out separately from the global analysis. For proper and efficient evaluation of bending and torsion effects, it is necessary to use computer analysis.

7.3.2 Bending, torsion and distortion

The general case of an eccentric load applied to a box girder may be resolved into two components, see Fig. 7.5:

1- A symmetrical component with both webs subjected to two equal vertical loads; and

2- An anti-symmetrical component with the two webs subjected to two equal and opposite forces forming a couple.

The symmetric component causes the box girder to be subjected to:

1- shear and bending in a vertical plan
2- distortion from bending if the girder section is open

The anti-symmetric component causes the box girder to be subjected to:

1- shear stresses from torsions
2- distortion from torsion.

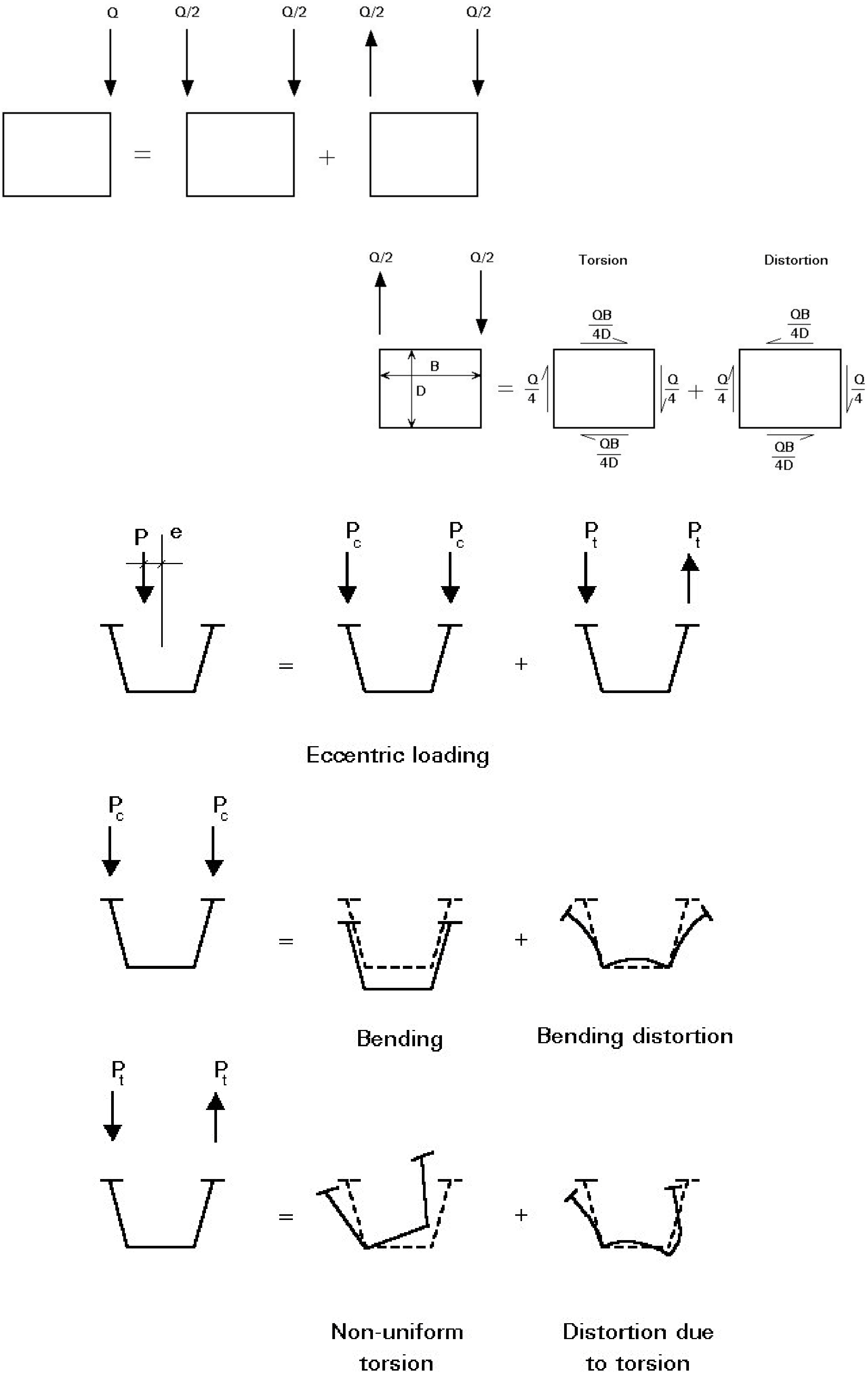


Fig. 7.5 Effect of Eccentric Loading on Box Girder Sections:

7.4 Effect Bending

Bending moments produce longitudinal normal stresses in the box girder given by:

$$\mathbf{f} = \frac{\mathbf{M_x}}{\mathbf{I_x}}\mathbf{y}$$

Since box girders contain wide flanges, the distribution of the bending stresses is non-uniform because the flange distorts in its own plane; i.e., plane sections do not remain plane. This phenomenon is known as "Shear Lag".

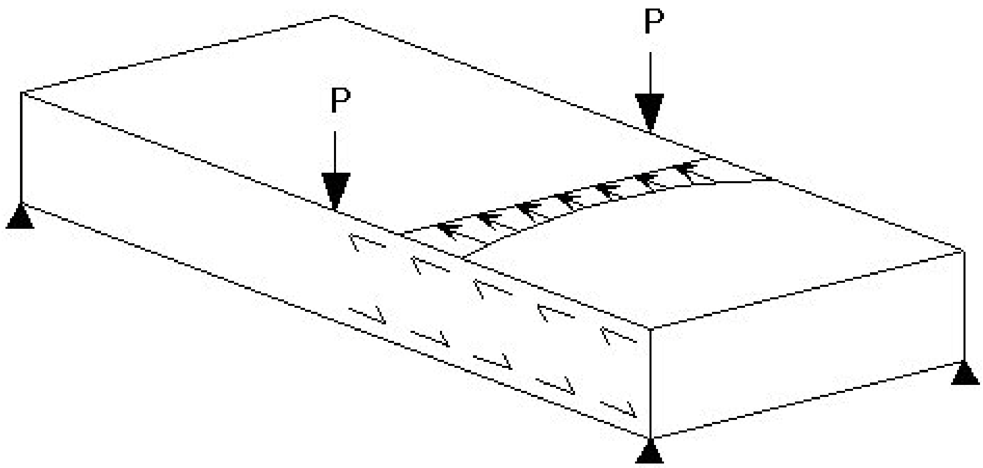


Fig. 7.6 Actual Bending Stress Distribution in a Box Girder.

7.4.1 Shear Lag Phenomenon

When the axial load is fed into a wide flange by shear from the webs, the flange distorts in its plane and plane sections do not remain plane. The resulting stress distribution in the flange is not uniform as shown in Fig. 7.6. In very wide flanges, shear lag effects have to be taken into account for the verification of stresses, especially for short spans, since it causes the longitudinal stress at a flange/web intersection to exceed the mean stress in the flange.

Shear lag can be allowed for in the elementary theory of bending, by using an effective flange breadth (less than the real breadth) such that the stress in the effective breadth equals the peak stress in the actual flange, see Fig.7.7. This effective flange breadth depends on the ratio of width to span.

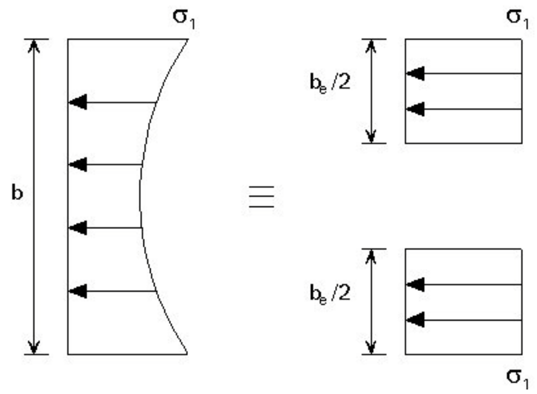


Fig. 7.7 Effective breadth for shear lag effects

The effective width is a function of the ratio of the span L to the width b of the box, the cross-sectional area of the stress carrying stiffeners, and the type and position of loading. For continuous girders, the effective widths are obtained separately for the individual equivalent simple spans between the points of inflection. Fortunately, in most situations the span/breadth ratio is not sufficiently large to cause more than 10-20% increase in peak stress, on account of shear lag.

According to British Standards **BS5400 : 3/2000:** The effective width b_e should be taken as follows:

a) $b_e = \psi b$ for portions between webs:

where

b = half the distance between centers of webs measured along the mid-plane of the flange plate;

b) $b_e = k\psi b$ for portions projecting beyond an outer web;

where

b = distance from the free edge of the projecting portion to the centre of the outer web; measured along the mid-plane of the flange plate;

k = (1 – 0.15b/L);

L = span of a beam between centers of support, or in the case of a cantilever beam, between the support and the free end;

ψ = appropriate effective breadth ratio taken from Tables 7.1 – 7.3 for uniformly distributed loads;

a = 0 if there are no stiffeners on the flange within the width b in the span direction, otherwise:

$$a = \frac{\textbf{sectional area of flange stiffeners in width b}}{\textbf{sectional area of flange plate in width b}}$$

Values of ψ for intermediate values of b/L and a and for intermediate positions in the span may be obtained by linear interpolation.

The value of ψ at an interior support should be taken as the mean of the values obtained for adjacent spans. For end spans of continuous beams the effective breadth ratios may be obtained by treating the end span as a propped cantilever of the same span.

For the purpose of calculating deflections of beams, the values of ψ given in these may be adopted for all sections in the span.

Table 7.1 – Effective breadth ratio ψ for simply supported beams

b/L	Mid-span		Quarter span		Support	
	a = 0	a= 1	a = 0	a = 1	a = 0	a = 1
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.98	0.97	0.98	0.96	0.84	0.77
0.10	0.95	0.89	0.93	0.98	0.70	0.60
0.20	0.81	0.67	0.77	0.62	0.52	0.38
0.30	0.66	0.47	0.61	0.44	0.40	0.28
0.40	0.50	0.35	0.46	0.32	0.32	0.22
0.50	0.38	0.28	0.36	0.25	0.27	0.18
0.75	0.22	0.17	0.20	0.16	0.17	0.12
1.00	0.16	0.12	0.15	0.11	0.12	0.09

Table 7.2 – Effective breadth ratio ψ for interior spans of continuous beams

b/L	Mid-span		Quarter span		Support	
	a = 0	a = 1	a = 0	a = 1	a = 0	a = 1
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.96	0.91	0.85	0.76	0.85	0.50
0.10	0.86	0.72	0.68	0.55	0.41	0.32
0.20	0.58	0.40	0.42	0.31	0.24	0.17
0.30	0.38	0.27	0.30	0.20	0.15	0.11
0.40	0.24	0.18	0.21	0.14	0.12	0.08
0.50	0.20	0.14	0.16	0.11	0.11	0.07
0.75	0.15	0.10	0.10	0.08	0.09	0.06
1.00	0.13	0.09	0.09	0.07	0.07	0.05

Table 7.3 – Effective breadth ratio ψ for cantilever beams

b/L	Fixed end		Quarter span near fixed end		Free end	
	a = 0	a = 1	a = 0	a = 1	a = 0	a = 1
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.82	0.76	1.00	1.00	0.92	0.86
0.10	0.68	0.61	1.00	1.00	0.84	0.77
0.20	0.52	0.44	1.00	1.00	0.70	0.60
0.30	0.42	0.35	0.95	0.90	0.60	0.48
0.40	0.35	0.28	0.88	0.75	0.52	0.38
0.50	0.30	0.25	0.76	0.62	0.40	0.33
0.75	0.22	0.18	0.52	0.38	0.34	0.23
1.00	0.18	0.14	0.38	0.27	0.27	0.18

7.4.2 Effect of Local Buckling

In addition, web and flange plates of a box girder must satisfy the requirements of ECP code for local plate buckling resulting from:

1- Axial compression in compression flange.
2- Bending compression in web.
3- Shear in web.

7.4.2.1 Local Buckling of Compression Flange:

The compression flange is non-compact if $\frac{b}{t} \le \frac{64}{\sqrt{F_y}}$ for stiffened elements and $\frac{b}{t} \le \frac{21}{\sqrt{F_y}}$ for unstiffened elements

If (b/t) exceeds these limits then either:

(1) provide longitudinal flange stiffeners to satisfy these requirements; or

(2) base design on effective width b_e calculated as follows :

$$b_e = \rho\, b,$$

$$\rho = \left(\bar{\lambda}_p - 0.15 - 0.05\,\psi\right) / \bar{\lambda}^2_p \qquad (\psi = 1)$$

$$\bar{\lambda}_p = \frac{b/t}{44} \sqrt{F_y / k_\sigma}$$

7.4.2.2 Web Buckling due to Bending :

Web is non-compact in pure bending ($\psi = -1$)

if: $\frac{d_w}{t_w} \le \frac{190}{\sqrt{F_y}}$

For the case $F_{all} = 0.58\, f_y$ gives $d_w/t_w = 100$ for st. 52

If $\left(\frac{d_w}{t_w}\right) \rangle \frac{190}{\sqrt{F_y}}$ Either :

1- provide longitudinal web stiffeners at d/5 from compression flange such that:

$$\left(\frac{d_w}{t_w}\right) \le \frac{320}{\sqrt{F_y}} \text{ (= 168 for st. 52) and, if need ,}$$

Use another stiffener at d/2 such that $\left(\frac{d_w}{t_w}\right) \le \frac{370}{\sqrt{F_y}}$ (= 195 for st. 52)

Or

2- base design on effective width d_e calculated as follows:

$$\mathbf{de} = \rho\, \mathbf{d_c} = \rho\ \mathbf{d/2}$$
$$\rho = \left(\bar{\lambda}_p - 0.15 - 0.05\Psi\right) / \bar{\lambda}^2{}_p \ , (\psi = -1)$$
$$\bar{\lambda}_p = \frac{d/t}{44}\sqrt{F_y / k_\sigma} \ , (k_\sigma = 23.9)$$
$$\left\{ \text{for st. 52} \qquad \bar{\lambda}_p = \frac{d/t}{44}\sqrt{\frac{3.6}{23.9}} = \frac{d/t}{113.4} \right\}$$

7.4.2.3 Web Buckling due to Shear:

Web is non-compact in shear if $\frac{dw}{tw} \le \frac{105}{\sqrt{F_y}}$

For the case $q_{all} = 0.35\ F_y$ d/t = 55 for St. 52

If $d/t > \frac{105}{\sqrt{F_y}}$ then reduce allowable shear stress to $\mathbf{q_b}$ given by :

$$q_b = \left(1.5 - 0.625\lambda_q\right) 0.35\ F_y \qquad (0.8 < \lambda_q < 1.2)$$
$$= \frac{0.9}{\lambda_q}(0.35 F_y) \qquad (\lambda_q > 1.2)$$

Where:

$$\lambda_q = \frac{d/t}{57}\sqrt{\frac{F_y}{k_q}}$$
$$k_q = 4 + 5.34/\alpha^2 \qquad (\alpha < 1)$$
$$= 5.34 + 4/\alpha^2 \qquad (\alpha > 1)$$

For vertically unstiffened webs : ($\alpha >> 1$)

$$k_q = 5.34$$
$$\lambda_q = \frac{d/t}{132.5}\sqrt{F_y}$$
$$\lambda_q = \frac{d/t}{70}$$

this gives :

$$q_b = \left(1.5 - (d/t)\sqrt{F_y}/212\right)\left(0.35\ F_y\right) \quad \text{for} \quad \frac{d}{t} \le 159/\sqrt{F_y}$$
$$= \frac{119}{(d/t)\sqrt{F_y}}\left(0.35\ F_y\right) \quad \text{for} \quad \frac{d}{t} > 159/\sqrt{F_y}$$

7.4.3 Combined Shear & Bending

In general, any cross-section of a box girder will be subjected to bending moment in addition to shear. This combination makes the stress conditions in the girder web considerably more complex. The stresses from the bending moment will combine with the shear stresses to give a lower buckling load. The interaction between shear and bending can be conveniently represented by the diagram shown in Fig. 7.8, where the allowable bending stress is plotted on the vertical axis and the allowable buckling shear stress of the girder is plotted horizontally. The interaction represents a failure envelope, with any point lying on the curve defining the co-existent values of shear and bending that the girder can just sustain. The equation representing this interaction diagram is :

$$\mathbf{F_b = \left[0.8 - 0.36\left(q_{act}/q_b\right)\right] F_y}$$

The interaction diagram can be considered in 3 regions. In region AB, the applied shear stress q_{act} is low (< 0.6 q_b) and the girder can sustain the full bending stress F_b based on the effective width b_{eff} for the compression flange. At the other extreme of the interaction diagram in region CD, the applied shear stress is high (=q_b) then the allowable bending stress is reduced to 0.44 F_y to allow for the high shear. In the intermediate region BC the allowable bending stress is reduced linearly from 0.58 F_y to 0.44 F_y.

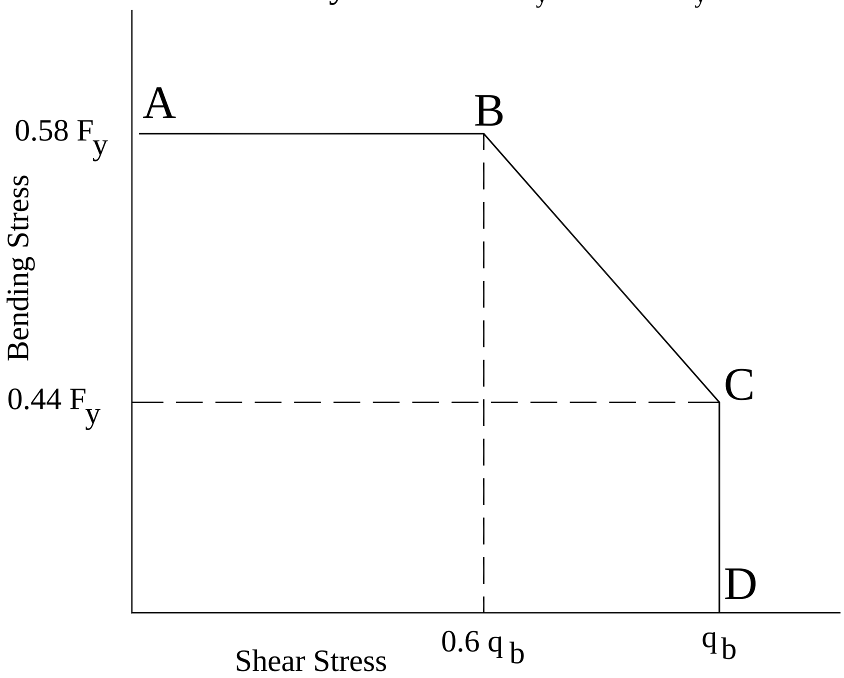


Fig. 7.8 Interaction between Shear and Bending

7.5 Effect of Torsion

The torsion component is shown in Figure 7.5 simply as a force couple. However, torsion is in fact resisted in a box section by a shear flow around the whole perimeter. The couple should therefore be separated into two parts, pure torsion and distortion, as shown in Figure 7.5. The distortion component comprises an internal set of forces, statically in equilibrium, whose effects depend on the behavior of the structure between the point of application and the nearest positions where the box section is restrained against distortion.

At supports, bearings will be provided, see Fig. 7.9. Where a pair of bearings is provided, they are usually either directly under each web or just inside the line of the webs. To resist forces reacting on the bearings as a result of the bending and torsion components, bearing support stiffeners will be required on the web. In addition, a diaphragm (or at least a stiff ring frame) will be required to resist the distortional effects consequent in transmitting the torsion from the box to a pair of bearing supports.

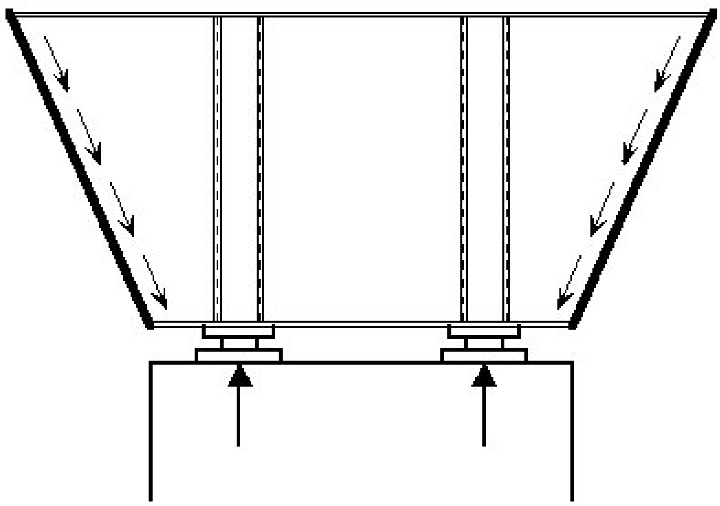


Fig. 7.9 End Bearings of a Box Girder

In some cases only a single bearing is provided; a stiffened diaphragm will be needed to resist the reaction and to distribute the force to the webs.

Between points of support, intermediate transverse web stiffeners may be provided to develop sufficient shear resistance in a thin web. Intermediate diaphragms or cross frames, see Fig. 7.10, may be provided to limit the distortional effects of eccentrically applied loads; they are particularly effective where concentrated eccentric effects are introduced, such as from a cantilever on the side of the box. Intermediate cross-frames may also be provided to facilitate construction.

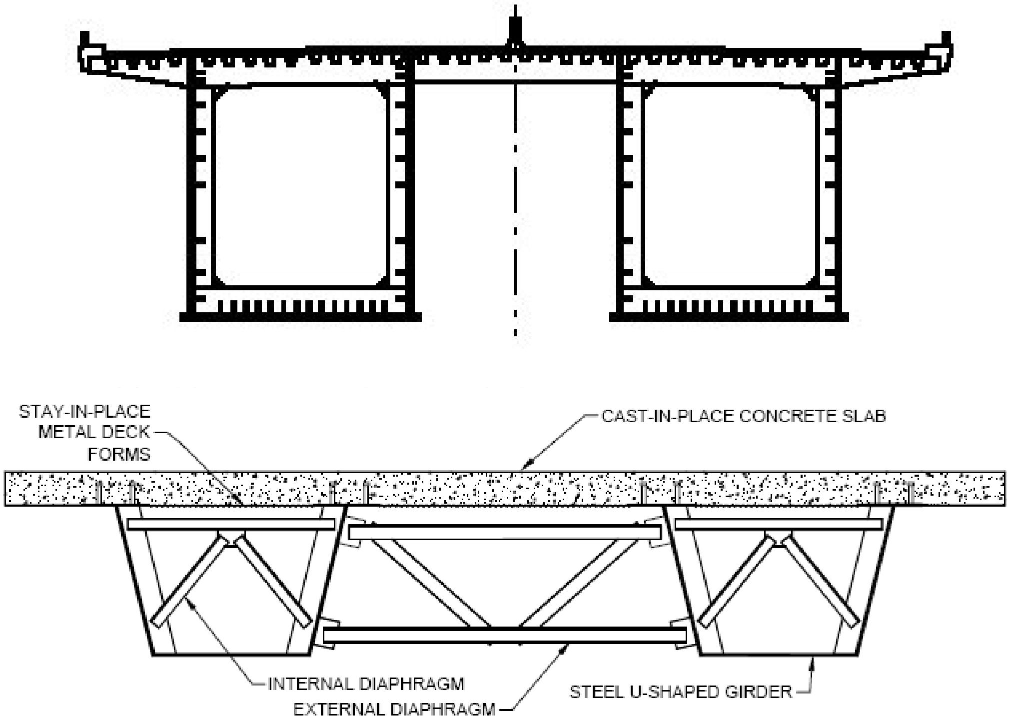


Fig. 7.10 Diaphragms and Cross Frames in Box Girders

7.5.1 Torsion and Torsional Warping

The theoretical behavior of a thin-walled box section subject to pure torsion is well known and treated in many standard texts. For a single cell box, the torque is resisted by a shear flow which acts around the walls of the box. This shear flow (force/unit length) is constant around the box and is given by $q = T/2A$, where T is the torque and A is the area enclosed by the box. (In Figure 7.5 the torque is $QB/2$ and the shear flow is $Q/4D$). The shear flow produces shear stresses and strains in the walls and gives rise to a twist per unit length, Θ which is given by the general expression :

$$\theta = \frac{T}{4A^2G}\oint\frac{ds}{t} \quad \text{or}, \quad \theta = \frac{T}{GJ}$$

where J is the torsion constant.

However, it is less well appreciated that this pure torsion of a thin walled section will also produce a warping of the cross-section, unless there is sufficient symmetry in the section as shown in Figure 7.11.

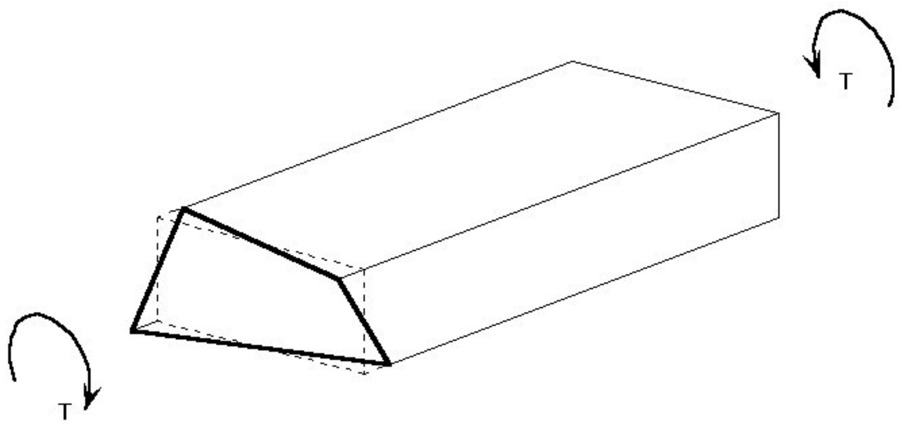


Fig. 7.11 Warping of a Rectangular Box subject to Pure Torsion.

For a simple uniform box section subject to pure torsion this warping is unrestrained and does not give rise to any secondary stresses. But if for example, a box is supported and torsionally restrained at both ends and then subjected to applied torque in the middle, warping is fully restrained in the middle by virtue of symmetry and torsional warping stresses are generated. Similar restraint occurs in continuous box sections which are torsionally restrained at supports.

This restraint of warping gives rise to longitudinal warping stresses and associated shear stresses in the same manner as bending effects in each wall of the box. The shear stresses effectively modify slightly the uniformity of the shear stress calculated by pure torsion theory, usually reducing the stress near corners and increasing it in mid-panel. Because maximum combined effects usually occur at the corners, it is conservative to ignore the warping shear stresses and use the simple uniform distribution.

The longitudinal effects are, on the other hand greatest at the corners. They need to be taken into account when considering the occurrence of yield stresses in service and the stress range under fatigue loading. But since the longitudinal stresses do not actually participate in the carrying of the torsion, the occurrence of yield at the corners and the consequent relief of some or all of these warping stresses would not reduce the torsional resistance. In simple terms, a little plastic redistribution can be accepted at the ultimate limit state and therefore there is no need to include torsional warping stresses in the ultimate limit state checks.

7.5.2 Distortion

When torsion is applied directly around the perimeter of a box section, by forces exactly equal to the shear flow in each of the sides of the box, there is no tendency for the cross section to change its shape.

If torsion is not applied in this manner, a diaphragm or stiff frame might be provided at the position where the force couple is applied to ensure that the section remains square and that torque is in fact fed into the box walls as a shear flow around the perimeter. The diaphragm or frame is then subject to a set of distortional forces as shown in Figure 7.5.

Provision of such diaphragms or frames is practical, and indeed necessary, at supports and at positions where heavy point loads are introduced. But such restraint can only be provided at discrete positions. When the load is distributed along the beam, or when point loads can occur anywhere along the beam such as concentrated axle loads from vehicles, the distortional effects must be carried by other means.

To illustrate how distortion occurs and is carried between effective restraints, consider a simply supported box which is subject to a point load over one web at mid-span. If a flexible intermediate cross-frame (a ring stiffener without any triangulated bracing in its plane) is placed at the point of application of the load, it tends to resist the distortion of the cross section by 'sway bending' of the form shown in Figure 7.12. Obviously, the stiffer the frame the less the distortion of the cross section. (Cross bracing or a plated diaphragm would be even more effective).

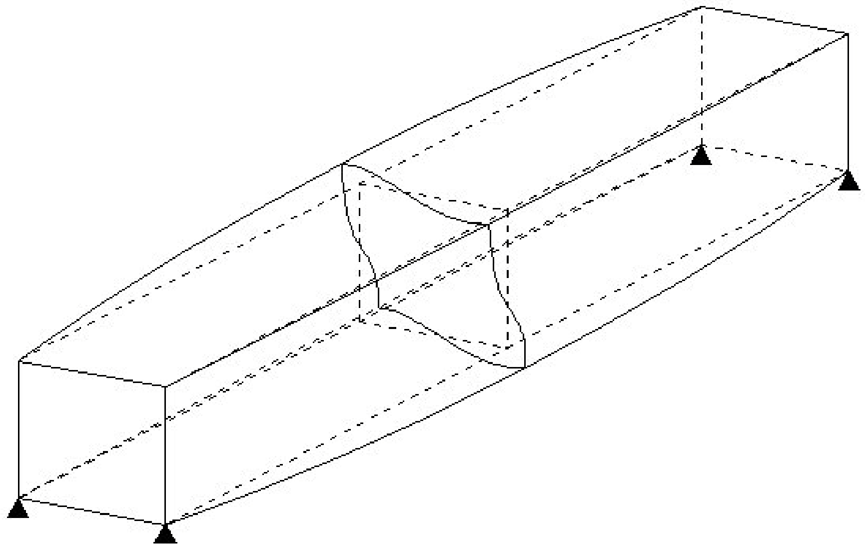


Figure 7.12 Distortion of Box Girder with Stiff Corners or Cross-Frames

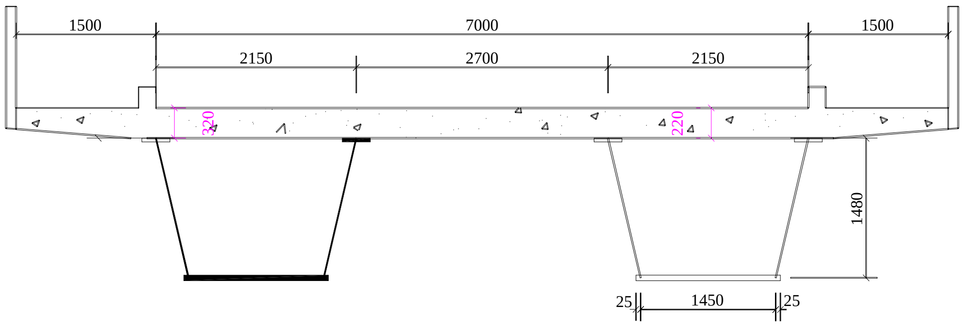
The bending of cross-frames and the walls of a box, as a result of the distortional forces, produces transverse ***distortional bending stresses*** in the box section.

In general the distortional behavior depends on interaction between the two sorts of behavior, the warping and the transverse distortional bending. The behavior has been demonstrated to be analogous to that of a beam on an elastic foundation, BEF, representing the transverse distortional bending resistance. The BEF model is used as the basis for the rules in Appendix B of BS 5400 Part 3 for calculating distortion and warping stresses in box girders. This Appendix is shown in the next section.

It must be emphasized that distortional effects are primary effects – they are an essential part of the means of carrying loads applied other than at stiff diaphragms – and they should not be ignored.

7.6 DESIGN EXAMPLE:

The design example presented in chapter 5 and chapter 6 is used here to illustrate the method of design of composite box girders. The same roadway is carried by two box girders as shown below:



The example uses the same values of the straining actions as shown next:

Action / Load Case	At Support		Mid section	
	Q (t)	M (m.t.)	Q (t)	M (m.t.)
Dead Load DL1	62	0	0	385
Add. Dead Load DL2	18	0	0	115
Live Load LL+I	100	0	25	700
Sum	180	0	25	1200

7.6.1 Web Plate Design:

Web Plate Height*:* The web plate height shall be assumed at 148 cm which corresponds to an inclined web plate length of 150 cm (standard plate width).

Web Plate Thickness: The minimum thickness for a web without transverse stiffeners is obtained from:

$$t^2 = Q / (41.65 \sqrt{F_y})$$

Since the total shear force is carried by two webs, each web carries
Q = 180/2 = 90 ton (at support). This gives:

$$t^2 = 90 / 41.65 \sqrt{3.6}) = 1.135 \quad \text{i.e., } t = 1.065 \text{ cm}$$

Use t = 12 mm (next even integer) without transverse stiffeners.

Check of web buckling due to shear:

Allowable Buckling Shear Stress = q_b = (119 / (d/t) $\sqrt{F_y}$) (0.35 F_y)

q_b = (119 / (150/1.2) $\sqrt{F_y}$) (0.35 F_y) = 0.632 t/cm^2

Actual Shear Stress:

$$\mathbf{q_{act} = \frac{90}{146 \times 1.2} = 0.514 \;\; t/cm^2 \; < \;\; q_b \; (O.K.)}$$

i.e., Web Plate is safe against buckling due to shear at support

7.6.2 Main Girder Design:

The section properties of the proposed cross section are as follows:

The following section is assumed:

a) Two Webs 1500 $\times$ 12

b) Top Flange 300 $\times$ 12
($b_f / 2t_f = 30 / (2 \times 1.2) = 12.5 > 21 / \sqrt{f_y} = 11$
(No problem since flange is prevented from local buckling by deck slab)

c) Bottom Flange 1500 $\times$ 22

<u>Section properties are then computed for the following cases:</u>

a) Steel section only:

Centroid $Y_{us} = 99.336$ cm
Intertia $I_s = 2360156$ cm^4
Section Moduli $Z_{us} = 23759$ cm^3
$Z_{ls} = 49310$ cm^3

b) Effective Slab Width:

For the exterior web:

$b_{ER} = b^* =$ **<u>150</u>** cm (Side Walk Slab)
b_{EL} = smaller of:
1) Span/8 = 27.5 /8 = 3.4375 m
2) Spacing /2 = 2.15/2 = **<u>1.075</u>** m governs
3) 6 ts = 6 * 22 = 132 cm

For the interior web:

b_{ER} = smaller of:
1) Span/8 = 27.5 /8 = 3.4375 m
2) Spacing /2 = 2.15/2 = **<u>1.075</u>** m governs
3) 6 ts = 6 * 22 = 132 cm

b_{EL} = smaller of:
1) Span/8 = 27.5 /8 = 3.4375 m
2) Spacing /2 = 2.70/2 = 1.35 m
3) 6 ts = 6 * 22 = **<u>132</u>** cm governs

Total Effective Slab Width = (150 + 107.5) + (107.5+132) = 497 cm

c) Composite section with n = 9 (F_{cu} = 300 kg / m^2)

Centroid $Y'_{us} = 31.372$ cm
Intertia $I_v = 8123595$ cm^4
Section Moduli $Z'_{us} = 258941$ cm^3
$Z'_{ls} = 70153$ cm^3
$Z_{uc} = 152206$ cm^3

d) Composite section with n = 3 × 9 = 27 (Effect of Creep)

Centroid $Y'_{us} = 60.896$ cm
Intertia $I_v = 5608461$ cm^4
Section Moduli $Z'_{us} = 92098$ cm^3
$Z'_{ls} = 64985$ cm^3
$Z_{uc} = 67656$ cm^3

Check of Bending Stresses:

<u>a) Non-Shored Construction:</u>

Load	Upper Steel (-) t/cm^2	Lower Steel (+) t/cm^2	Upper Concrete kg / cm^2
DL 1	$F_{us} = 385 \times 100/23759 = 1.620$	$F_{ls} = 385 \times 100/49310 = 0.781$	= 0 for non-shored construction
DL 2	$F_{us} = 115 \times 100/92908 = 0.125$	$F_{ls} = 115 \times 100/64985 = 0.177$	$F_{us} = (115 \times 100/67656) * (1000/27) = 2.575$
LL + I	$F_{us} = 700 \times 100/258941 = 0.270$	$F_{ls} = 700x100/70153 = 0.998$	$F_{us} = (700/152206) * (1000/9) = 51.10$
<u>Total</u>	**<u>2.016</u>**	**<u>1.956</u>**	**<u>53.676</u>**

Checks:

1- Compression in Upper Steel :

a) Total stress: F_{us} = **<u>2.016</u>** < F_b = **<u>2.1</u>** t/cm^2
(compression flange is laterally supported by deck slab)

b) Due to D.L. only Fus = **<u>1.620</u>** t / cm^2

Assume compression flange is laterally supported by upper bracing with

$L_u = 4.5$ m, $r_T = 8$ cm $\qquad L_u/r_T = 450 / 8 = 56.25$

$$\mathbf{L_u / r_T \leq 188\sqrt{\frac{C_b}{F_y}}} = 99$$

$$F_{ltb2} = (0.64 - \frac{(L_u / r_T)^2 F_y}{1.176x10^5 C_b})F_y \le 0.58 \ F_y = \underline{1.955} \ \text{t/cm2}$$

$$\mathbf{F_{us} < F_{LTB} \quad O.K.}$$

2. Tension in Upper Steel :

a) Total Tension: $f_{ls} = 1.956 < F_b = 2.10$ t/cm^2

b) Fatigue $f_{sr} = 0.5 \times 0.998 = 0.499 < F_{sr} = 1.02$ t/cm^2

{The allowable fatigue stress range (F_{sr}) is obtained as follows:
* From ECP Table 3.1.a: ADTT >2500, Number of cycles = 2×10^6
Detail Class = B′ (case 4.2 of Table 3.3)
Table 3.2 gives F_{sr} = <u>1.02</u> t/cm^2 > f_{sr} }

3. Compression in Upper Concrete:

$f_{uc} = 53.676 < 70$ kg/cm^2

7.6.3 Comparison between Different Designs:

The following table shows a comparison between the total weight of steel needed for the bridge according to the three designed presented in Chapter 5 using non-composite plate girders, in Chapter 6 using composite plate girders, and in Chapter 7 using composite box girders. It should be note that the total weight of the first two cases should include the weight of the stringers and cross girders, not present in the third case. The comparison shows that the composite box girder solution uses the least weight followed by the composite plate girder solution.

	Section		Weight of Main Girder	Weight of Floor Beams	Total Weight
	Web	Flanges			
1- Non-Composite Plate Girder	2250*14	600*36 600*36	31.665	15.596	47.261
2- Composite Plate Girder	2250*14	400*12 600*32	23.526	15.596	39.122
3- Composite Box Girder	2(1500*12)	2(300*12) 1500*22	32.302	0	32.302

CHAPTER 8

TRUSS BRIDGES

CHAPTER 8

TRUSS BRIDGES

8.1 TRUSS TYPES & CHARACTERISTICS

8.1.1 GENERAL

A truss is essentially a triangulated assembly of straight members. A planar truss may be regarded as a deep girder, where the girder flanges are replaced by the truss chords and the web plate is formed by an open system of web members. A truss may be used to replace a girder in several cases: as a simply supported or continuous girder; as an arch; or in the deck of a suspension or a cable-stayed bridge; see Fig. 8.1 and Fig. 8.2.

In a typical truss, the centroidal axes of all members are straight and concurrent at the joints. Because the truss is loaded only at the joints; applied loads are resisted primarily by axial forces induced in the truss members. Bending moments are generally small and have a minor effect on the axial forces. Ideally, all member bending moments should be close to zero, a condition that can only be achieved by using frictionless pins at the joints. In practice, however, most members are rigidly connected at the joints, resulting in small moments which are usually neglected, except in some few special cases.

A truss bridge has thus two major structural advantages:

(a) the primary member forces are axial loads,
(b) the open web system allows a greater overall depth than in an equivalent solid web girder. This increased depth gives more rigidity to the bridge and results in reduced deflections.

Fig. 8.1 Applications of Trusses in Bridges

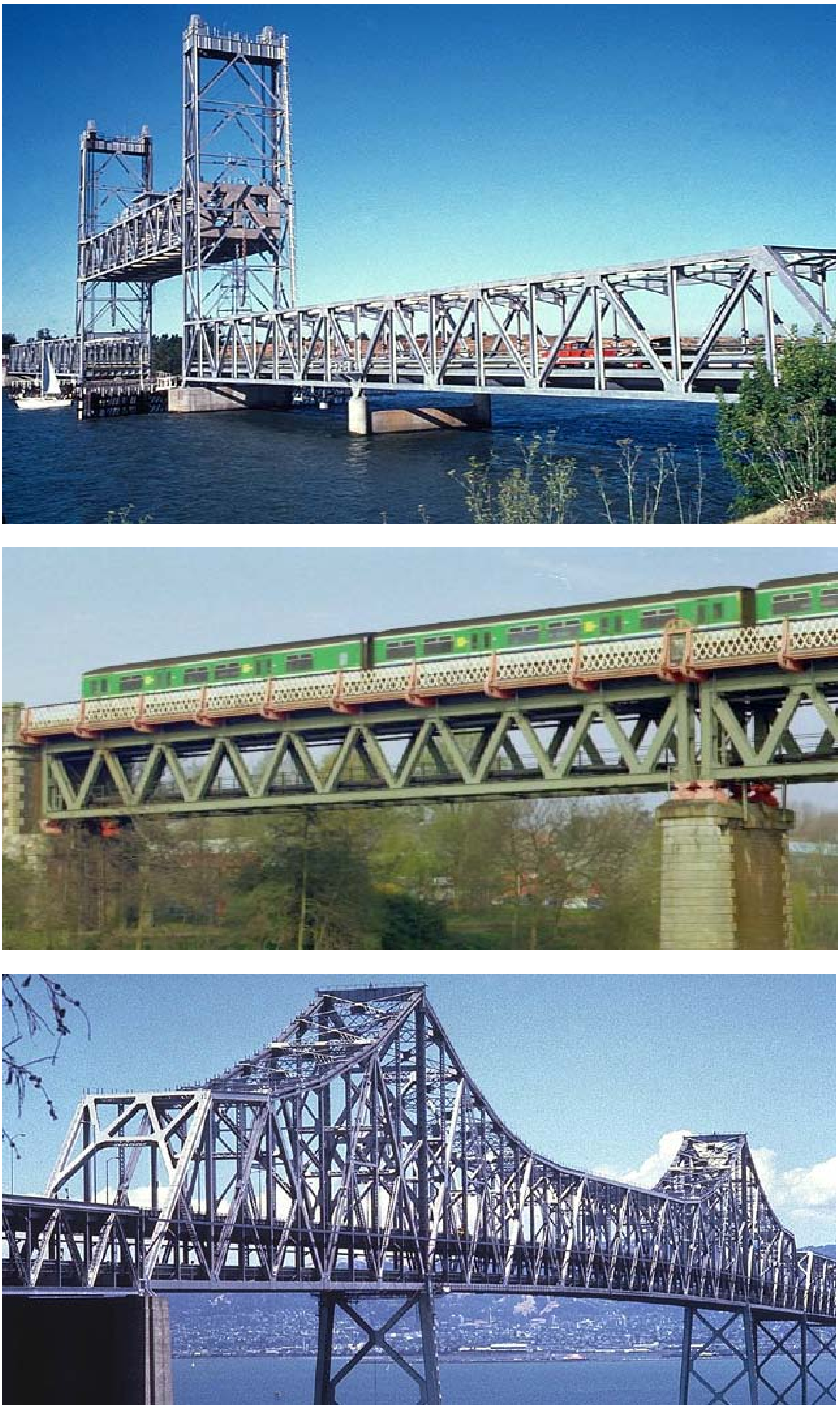


Fig. 8.2 Examples of Truss Bridges

8.1.2 TRUSS BRIDGE COMPONENTS

A truss bridge of conventional design consists of the following parts; see Fig. 8.3;

(a) a deck slab or similar structural system,
(b) longitudinal stringers directly supporting the deck slab,
(c) cross beams at truss panel points carrying the load from the longitudinal stringers,
(d) the two main truss systems,
(e) lateral bracing systems in the planes of the upper and lower chords,
(f) end sway frames transmitting the reactions of the lateral bracing systems to the bridge supports,
(g) additional intermediate sway frames distributing the transverse wind loads to the lateral systems and keeping the system stable during erection.

For through trusses, a system of upper wind bracings is always provided. This upper bracing provides rigidity, stabilizes the compression chord, and carries the main part of the wind loads to the bridge end sway frames, called portal frames. These end frames are designed as rigid frames to transmit the load from the upper bracing to the bridge supports.

Fig. 8.3 Components of a Through Truss Bridge

8.1.3 TRUSS FORMS

The most common forms of bridge trusses are:

(1) Pratt or N-Truss (Fig. 8.4 a):

In this system the diagonals are always subjected to tension while the verticals carry the shear in compression only. This case can represent an advantage since the shorter members carry the compression.

(2) Warren Truss (Fig. 8.4 b):

Where the chords carry the bending in tension and compression and the diagonals carry the shears, also in tension and compression. The vertical members carry only panel loads.

(3) Trusses with Curved chords (Fig. 8.4 c)

Truss Chords may be placed on a curved alignment to carry part of the shear and to reduce the forces in the diagonals. This alignment results in a slight increase in the fabrication cost which is offset by material savings.

(4) Subdivided Panels (Fig. 8.4 d):

The economic height-span ratio is about one-sixth to one-eighth, according to loading and span length. With increasing span lengths, truss height also increases. Thus, both the warren and Pratt trusses will result in long panel length if the diagonal inclination remains about 45^{o}. An alternative is to subdivide these trusses as shown in Fig. 8.4 d.

(5) K – Truss (Fig. 8.4 e:)

Subdivided trusses develop high secondary stresses. A better solution may be obtained by using K-trusses to keep the desired inclinations, accommodate the required truss depth, and also limit the strength span.

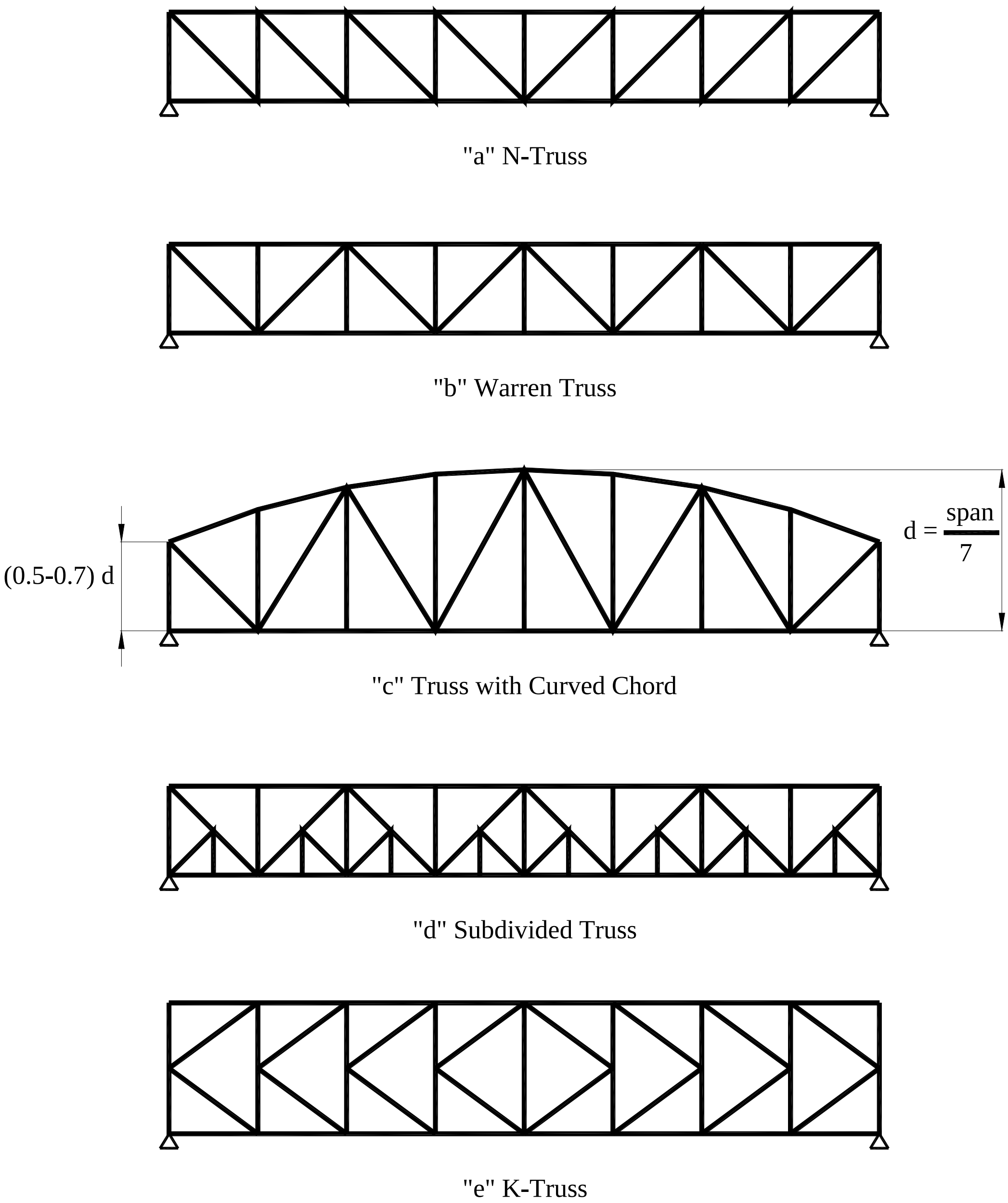


Fig. 8.4 Common Forms of Trusses used in Bridges

8.1.4 SPECIAL CHARACTERISTICS

8.1.4.1 Truss Depth

For simple span trusses, experience has shown that a depth-span ratio of 1 : 6 to 1 : 8 yields economical designs. For continuous trusses a depth-span ratio of 1 : 12 should be satisfactory. Because of the lighter live loads for roadway bridges, trusses are rarely used. If trusses are used for roadway bridges, somewhat shallower truss depths may be used.

The truss depth shall be sufficient to limit the elastic deflections due to live load without impact to L/600 for roadway bridges and L/800 for railway bridges and L/300, where L = bridge span.

8.1.4.2 Economic Truss Spans

Truss bridges are generally comparatively easy to erect because light equipment often can by used. Assembly of bolted joints in the field is relatively costly, which may offset some of the savings in steel. Consequently, trusses seldom can be economical for roadway bridges with spans less than about 130 m. Railway bridges, however, involve different factors, because of the heavier loading. Trusses generally are economical for railway bridges with spans greater than 45 m.

8.2 DESIGN OF TRUSS MEMBERS

8.2.1 Determination of Member Forces:

Structural analysis techniques may be applied to the bridge system to find the effect of applied loads and forces acting on the truss members. The following assumptions are usually made:

1- Members are connected at their ends by hinges,
2- Loads are applied at the truss joints,
3- In case of a curved member, the additional bending moment induced due to member curvature should be calculated,
4- Secondary stresses due to joint rigidity and bending moments due to own weight are neglected expect in trusses with subdivided panels, trusses with loads acting between joints, and trusses with member height more than one tenth of the member length.

Load cases that yield maximum straining actions should be considered carefully. The resulting forces in the truss members are axial compression and tension. Members are then designed using the allowable stress method. Special design considerations are outlined next.

8.2.2 *Cross Section Shapes for Truss Members:*

Members for bridge trusses generally consist of; see Fig. 8.5:

(a) Box sections made of plates or rolled sctions by welding;
(b) l-sections, either rolled or built up.

Box sections are usually used for chord members and heavy web members. I-sections are usually used for light web members. Box sections present some difficulties in their connection with gusset plates. Bolted connections with gusset plate shall require the existence of temporary erection openings in the box section to allow for bolt tightening. These openings shall be closed after the truss erection. If design permits, use of I-sections for chord members results in much easier connections.

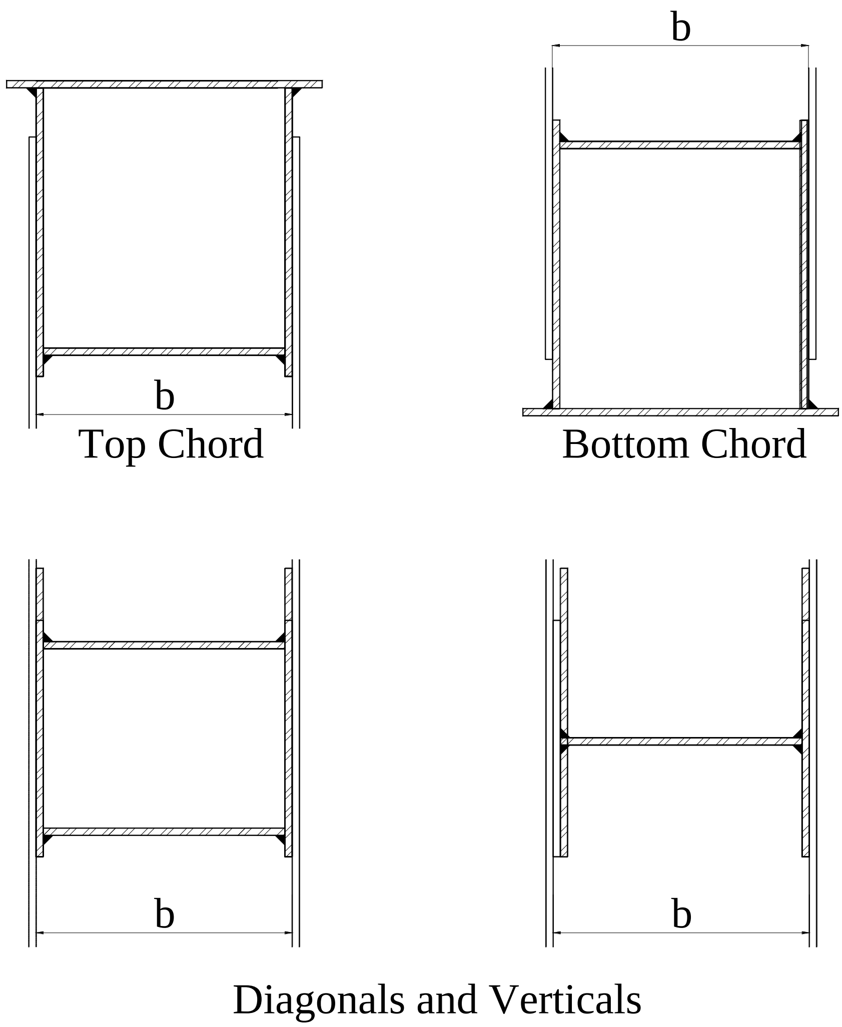


Fig. 8.5 Common Shapes of Bridge Truss Members

8.3 GENERAL DESIGN PRINCIPLES

8.3.1 Geometry

For short and medium spans, it will generally be found economic to use parallel chords to keep fabrication and erection costs down. However, for long continuous spans, a greater depth is often required at the piers. Secondary stresses should be avoided as far as possible by ensuring that the neutral axes of all intersecting members meet at a single point, in both vertical and horizontal planes. This will not always be possible, e.g. cross girders will be deeper than the bottom chord and bracing members may be attached to only one flange of the chords.

8.3.2 Compression Chord Members

These members should be kept as short as possible and consideration given to additional bracing if economical. The effective length for buckling in the plane of the truss is normally not the same as that for buckling out of the plane of the truss, depending on the arrangement of upper bracings. This effect can be further complicated in through trusses where horizontal bracing may be provided at mid panel points as well as at the main nodes. When making up the section for the compression chord, the ideal disposition of material will be one that produces a section with radii of gyration such that the ratio of effective length to radius of gyration is the same in both planes. In other words, the member is just as likely to buckle horizontally as vertically.

8.3.3 Tension Chord Members

Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions. The width out of the plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing.

It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices. This means that fracture at the net section will not govern for common steel grades.

As with compression members, box sections would be preferable for ease of maintenance but open sections may well prove cheaper.

8.3.4 Vertical and Diagonal Members

These members should be all the same width normal to the plane of the truss to permit them to fit flush with or to be slotted inside the top chord (where the top-hat section is used) and to fit flush with the bottom chord. However, the width of the diagonals in the plane of the truss should be reduced away from the supports by about 75 mm per panel. This reduction may mean that some members are understressed. It is often possible to use rolled sections, particularly for the lightly loaded members, but packs will probably be required to take up the rolling margins. This fact can make welded members more economic, particularly on the longer trusses where the packing operation might add a significant amount to the erection cost.

Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over-complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred.

8.3.5 Wind Bracings

Truss bridges should be provided with top and bottom lateral bracing systems as shown in Fig. 8.3 to carry wind and other lateral loads acting on the bridge. These lateral bracing systems are also effective in providing lateral supports to the main truss compression chords. In addition, transversal bracing should be provided at truss ends to transmit lateral loads from lateral bracing systems to the bridge supports. These transversal bracings take the form of portal frames for through bridges and cross frames for deck bridges. Similar intermediate portal or cross frames are used to provide space rigidity to the bridge and help in distributing lateral loads.

Forces to be considered in bracing design include wind, seismic loads, and centrifugal forces. The bridge truss chords act as the chords of the lateral system. In general, the design of these members is governed by slenderness ratio conditions. Because of the long unbraced lengths of these members, it is often advantageous to consider the cross bracing acting in tension only and the neglect its resistance to compression.

8.4 DESIGN OF TRUSS MEMBERS

8.4.1 Selection of Member Dimensions:

1. Member height "h" and distance between gussets "b" can be selected as follows:

$$\mathbf{h} = \frac{\textbf{Panel Length}}{\mathbf{12-15}} \leq \frac{\textbf{Panel Length}}{\mathbf{10}}$$

$$\mathbf{b} = \left(\frac{\mathbf{3}}{\mathbf{4}} - \frac{\mathbf{5}}{\mathbf{4}}\right) \mathbf{h}$$

"b" should be constant for all members.

"h" is usually the same for top and bottom chord members.

2. Top chord is symmetrical about y-axis, Bottom chord is usually symmetrical about x and y axes.

3. Start the design with the members with maximum forces.

8.4.2 Slenderness Ratios:

The maximum allowable slenderness ratios "L/i", as per the Egyptian Code of Practice are as follows:

	Railway	Roadway	Bracing	Hanger
Compression	90	110	140	---
Tension	160	180	200	300

8.4.3 Minimum Plate Thickness:

The minimum plate thickness to be used is as follows:

$$\frac{\mathbf{w}}{\mathbf{t}} \leq \frac{\mathbf{21}}{\sqrt{\mathbf{F_Y}}} \quad \textbf{(Unstiffened Element) \&} \quad \frac{\mathbf{w}}{\mathbf{t}} \leq \frac{\mathbf{64}}{\sqrt{\mathbf{F_Y}}} \quad \textbf{(Stiffened Element)}$$

where w is the plate width from the points of fixation (welds or bolts)

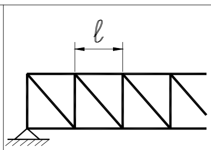
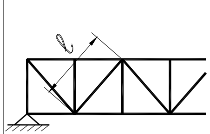
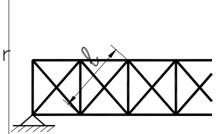
8.4.4 Allowable Stresses:

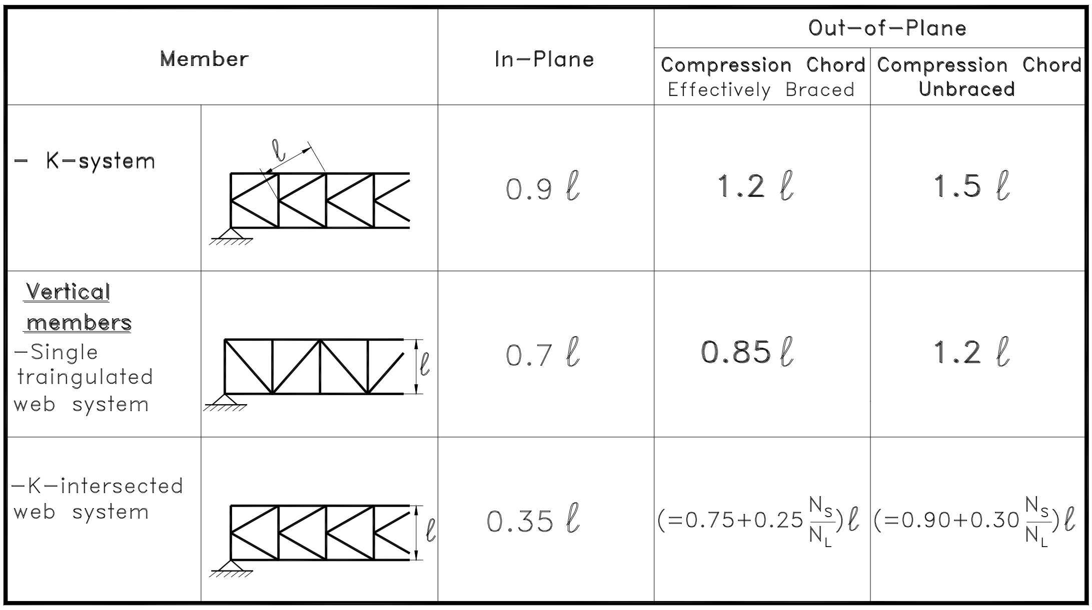
According to the Egyptian Code of Practice for the allowable stresses of Tension and Compression Members:

Grade of Steel	Allowable Stresses (t/cm^2)		
	Tension Member	Compression Member	
		$\frac{L}{i} \rangle 100$	$\frac{L}{i} \langle 100$
St. 37	1.4	$\frac{7500}{\left(\frac{L}{i}\right)^2}$	$1.4 - 0.000065\left(\frac{L}{i}\right)^2$
St. 44	1.6		$1.6 - 0.000085\left(\frac{L}{i}\right)^2$
St. 52	2.1		$2.1 - 0.000135\left(\frac{L}{i}\right)^2$

8.4.5 Buckling Length of Truss Bridge Members:

i) According to the Egyptian Code of Practice for determination of the buckling length of truss bridge members:

Member		In-Plane	Out-of-Plane	
			Compression Chord Effectively Braced	Compression Chord Unbraced
<u>Chords</u>		$0.85\,\ell$	$0.85\,\ell$	Use Equation 4.2
<u>Diagonals</u> –Single Traingulated web system		$0.70\,\ell$	$0.85\,\ell$	$1.2\,\ell$
–Multiple Intersected web rectangular system adequately connected		$0.85\,\ell/2$	$0.75\,\ell$	ℓ



Member		In–Plane	Out–of–Plane	
			Compression Chord Effectively Braced	Compression Chord Unbraced
– K–system		$0.9\,\ell$	$1.2\,\ell$	$1.5\,\ell$
Vertical members –Single traingulated web system		$0.7\,\ell$	$0.85\,\ell$	$1.2\,\ell$
–K–intersected web system		$0.35\,\ell$	$(=0.75+0.25\frac{N_S}{N_L})\ell$	$(=0.90+0.30\frac{N_S}{N_L})\ell$

N_S = Smaller value of normal force
N_L = Larger value of normal force

Buckling Length of Truss Bridge Members

Member		**Effective Buckling Length L_e**		
		In-Plane	**Out-of-Plane**	
			Compression Chord Laterally Braced	Compression Chord Unbraced
Chord Members		0.85 The Member Length	0.85 The distance between lateral bracing members	1.25 the distance between U frames or 0.75 Truss Span
Web System	Single Web System	0.7 The Member Length	0.85 The Member Length	1.0 The Member Length
	Multiple Web System	0.85 The Member Length	0.7 The distance between intersection with Main Chords	0.85 The distance between intersection with Main Chords

ii) For Pony Trusses:

For a bridge truss where the compression chord is laterally restrained by U-frames composed of the cross girders and verticals of the trusses, the effective buckling length of the compression chord (ℓ_b) is

$$\ell_b = \mathbf{2.5 \cdot \sqrt[4]{E \cdot I_y \cdot a \cdot \delta} \geq a}$$

Where,

E	=	The Young’s modulus (t/cm2).
Iy	=	The moment of inertia of the chord member about the Y-Y axis shown in Figure 4.2 (cm^4).
a	=	The distance between the U-frames (cm).
δ	=	The flexibility of the U-frame: the lateral deflection near the mid-span at the level of the considered chord’s centroid due to a unit load acting laterally at each chord connected to the U-frame. The unit load is applied only at the point at which δ is being calculated. The direction of each unit load shall produce a maximum value for δ (cm).

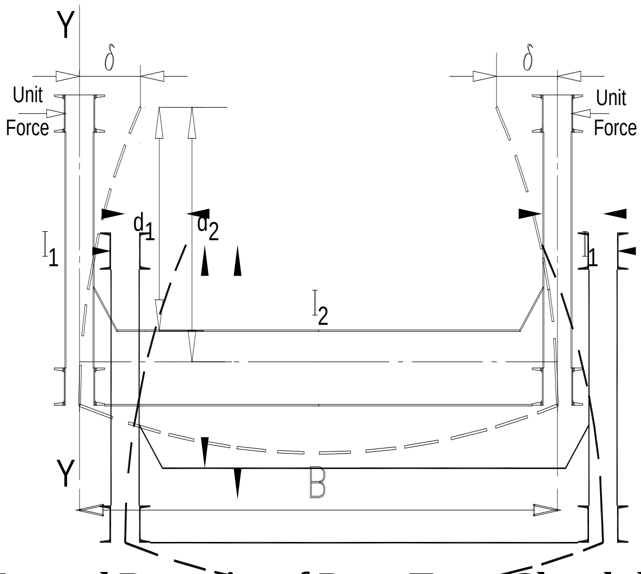


Figure 8.6 Lateral Restraint of Pony Truss Chords by U-Frame

The U-frame is considered to be free and unconnected at all points except at each point of intersection between cross girder and vertical of the truss where this joint is considered to be rigidly connected.

In case of symmetrical U-frame with constant moment of inertia for each of the cross girder and the verticals through their own length, δ may be taken from:

$$\delta = \frac{d_1^3}{3EI_1} + \frac{d_2^2 B}{2EI_2}$$

Where:

d	=	The distance from the centroid of the compression chord to the nearest face of the cross girder of the U-frame.
d	=	The distance from the centroid of the compression chord to the centroidal axis of the cross girder of the U-frame.
I_1	=	The second moment of area of the vertical member forming the arm of the U-frame about the axis of bending.
I_2	=	The second moment of area of the cross girder about the axis of bending .
B	=	The distance between centres of consecutive main girders connected by the U-frame.

The verticals of the pony truss are designed to carry a bending moment in addition to the normal forces induced due to regular loads. The bending moment is estimated as:

$$\mathbf{M = \frac{C}{100} H},$$

where C is the average compression force in the top chord members intersecting the vertical member, and H is the distance between the top chord and the top of the cross girder at the vertical member.

8.4 DESIGN EXAMPLE:

8.4.1 Design a top chord member for a roadway bridge for the following data:

Design Force = -1250 Tons (Compression)
Member Length = 1000 cm
Buckling Length $L_x = L_y = 0.85 \times 1000 = 850$ cm
Steel Grade St. 52

Selection of Member Dimensions: (Assume member stress = 1.8 t/cm^2)

$$\mathbf{h = \frac{Panel}{12-15} = 83-66 \quad \Rightarrow \quad 80-cm}$$

$$\mathbf{b = \left(\frac{3}{4} - \frac{5}{4}\right) h = 52-87 \quad \Rightarrow \quad 70-cm}$$

$$\mathbf{A_{req.} = \frac{1250}{1.8} \cong 695\ cm^2}$$

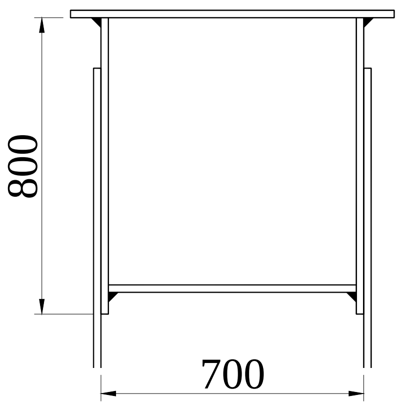
Min Thickness :

Flange : $\quad t_f \geq \dfrac{70}{33.7} = 2.075\,\text{cm, Choose}\ \ 2.2\ \ \text{cm}$

Web : $\quad t_w \geq \dfrac{75}{33.7} = 2.255\ \text{cm, Choose}\ \ 2.4\ \text{cm}$

Try the following section:

		Area (cm^2)
Top Flange	Pl. 800 × 22	176
Web	2 Pl. 800 × 24	384
Bottom Flange	Pl. 652 × 22	143.44
		703.44



Section Properties and Stress Check:

$$i_x \approx 0.4 \times h = 32\ \ \text{cm}$$

$$i_y \approx 0.4 \times a = 28\ \text{cm}$$

$$\frac{L_y}{i_y} = \frac{850}{28} = 30.4 < \quad 100$$

$$F_{buck} = 2.1 - 0.000135\,(30.4)^2 = 1.976\ t/cm^2$$

$$f_{act} = \frac{1250}{703.44} = 1.777\ t/cm^2 \ < F_{buck} \Rightarrow \quad \text{Safe}$$

8.4.2 Design a bottom chord member for a roadway bridge for the following data:

Design Force = + 1250 Tons (Tension)
Member Length = 1000 cm
Buckling Length $L_x = L_y = 0.85 \times 1000 = 850$ cm

***Use section similar to top chord*:**

$$A_{net} \approx 0.85\ \text{Agross}$$

$$f_{act} = \frac{1250}{0.85 \times 703.44} = 2.091\ \ t/cm^2 \ \langle\ 2.1\ t/cm^2$$

Fatigue Check :

$$\left.\begin{array}{l} \mathbf{F_{DL} = 300\ t} \\ \\ \mathbf{F_{LL+I} = 950\ t} \end{array}\right\} \qquad \mathbf{f_{sr} = \frac{950}{0.85 \times 703.44} = 1.588\ t/cm^2}$$

$$\left.\begin{array}{l} \mathbf{for: No.\ of\ cycles = 5 \times 10^2} \\ \\ \quad \mathbf{Detail\ Class = B} \end{array}\right\} \qquad \mathbf{Fsr = 1.80\ t/cm^2 \quad o.k.}$$

8.4.3 Design a diagonal member for a roadway bridge for the following data:

Design Force = - 180 Tons (Compression)
Member Length = 700 cm
Buckling Length L_x = 0.7 × 700 = 490 cm
L_y = 0.85 × 700 = 595 cm

Total member depth = b = 70 cm

Trial Section :

Web	660 x 20 = 132
Flange	2x300 x 20 = 120

Total Area = 252 cm^2

$$\left(\begin{array}{l} \mathbf{i_x \simeq 28} \\ \mathbf{\frac{L_x}{i_x} = \frac{495}{28} = 17.7} \end{array}\right) \qquad \mathbf{i_y \cong 0.2 \times 30 = 6\ cm}$$

$$\mathbf{\frac{L_y}{i_y} = \frac{595}{6} = 99.17 \quad \langle\ 110 \qquad o.k.}$$

$$\mathbf{f_{pb} = 2.1 - 0.000135 * 99.17^2 = 0.772\ t/cm^2}$$

$$\mathbf{f_{act} = 180/252 = 0.714\ t/cm2\ \langle\ f_{pb} \qquad o.k.}$$

8.4.4 Design a diagonal member for a roadway bridge for the following data:

Design Force = + 250 Tons (Tension)
Member Length = 700 cm
Buckling Length $L_x = 0.7 \times 700 = 490$ cm
$L_y = 0.85 \times 700 = 595$ cm

Trial Section :

Web 676 x 12 = 81.12

Flange 2x300 x 12 = 72.00

Total Area = 153.12 cm^2

$$\frac{l_y}{i_y} = 595/(0.2 \times 30) = 99.2 \quad \langle\ 180 \quad \text{o.k.}$$

$$A_{net} \approx 0.85 \times 153.12 = 130.152 \quad cm^2$$

$$f_{act} = 250/130.152 = 1.92\ t/cm2 \quad \langle\ 2.1 \quad \text{o.k.}$$

8.5 DESIGN OF TRUSS CONNECTIONS

8.5.1 Truss Joints

Members of bridge trusses are usually connected by gusset plates at the joints where members meet. Connections are usually made by bolting the members to gusset plates on both sides of the cross section as shown in Fig. 8.7.

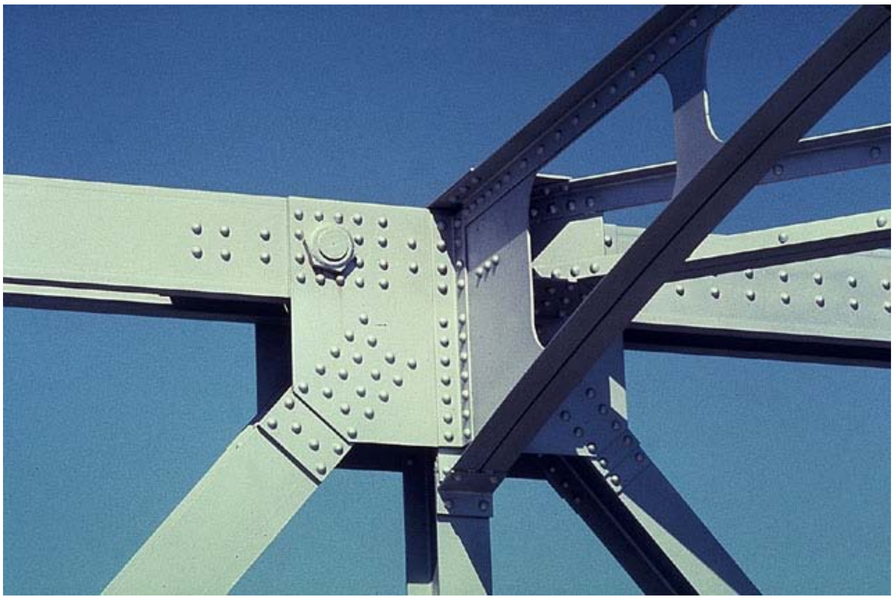


Fig. 8.7 Bolted Truss Joints

The usual gusset plate thickness is 14-20 mm. At every truss joint, working lines of the intersecting members should meet at one point to avoid eccentric loading. Force transmission through the gusset plates at a truss joint may be achieved in one of the following two ways:

(a) If the chord member runs continuous through the joint, the main portion of the force is transferred directly within the chord, and only the difference of the chord forces is carried through the gusset. This arrangement if often used to relieve the gusset plate of any excessive load. In this case, chord members are usually spliced outside the joints, see Fig. 8.8.

Fig. 8.8 Bolted Truss Joints with Splice outside Joint

(b) If the chord members are spliced at the joint, the gusset plates at this location will be subjected to heavy stresses because it transmits the entire amount of the chord forces.

At the nodes of a truss where the web members are connected to the chords, there is a change in load in the chord which necessitates a change in its cross-section area. The node is, therefore, the point at which there is a joint in the chord as well as being the connection point of the web members.

The web members are connected to the chords by vertical gusset plates. They are usually bolted to the chord webs and the web members fit between them (Figure 8.9a).

The chord joint is effected by providing cover plates. They should be so disposed, with respect to the cross-section of the member, as to transfer the load in proportion to the respective parts of the section (Figure 8.9b).

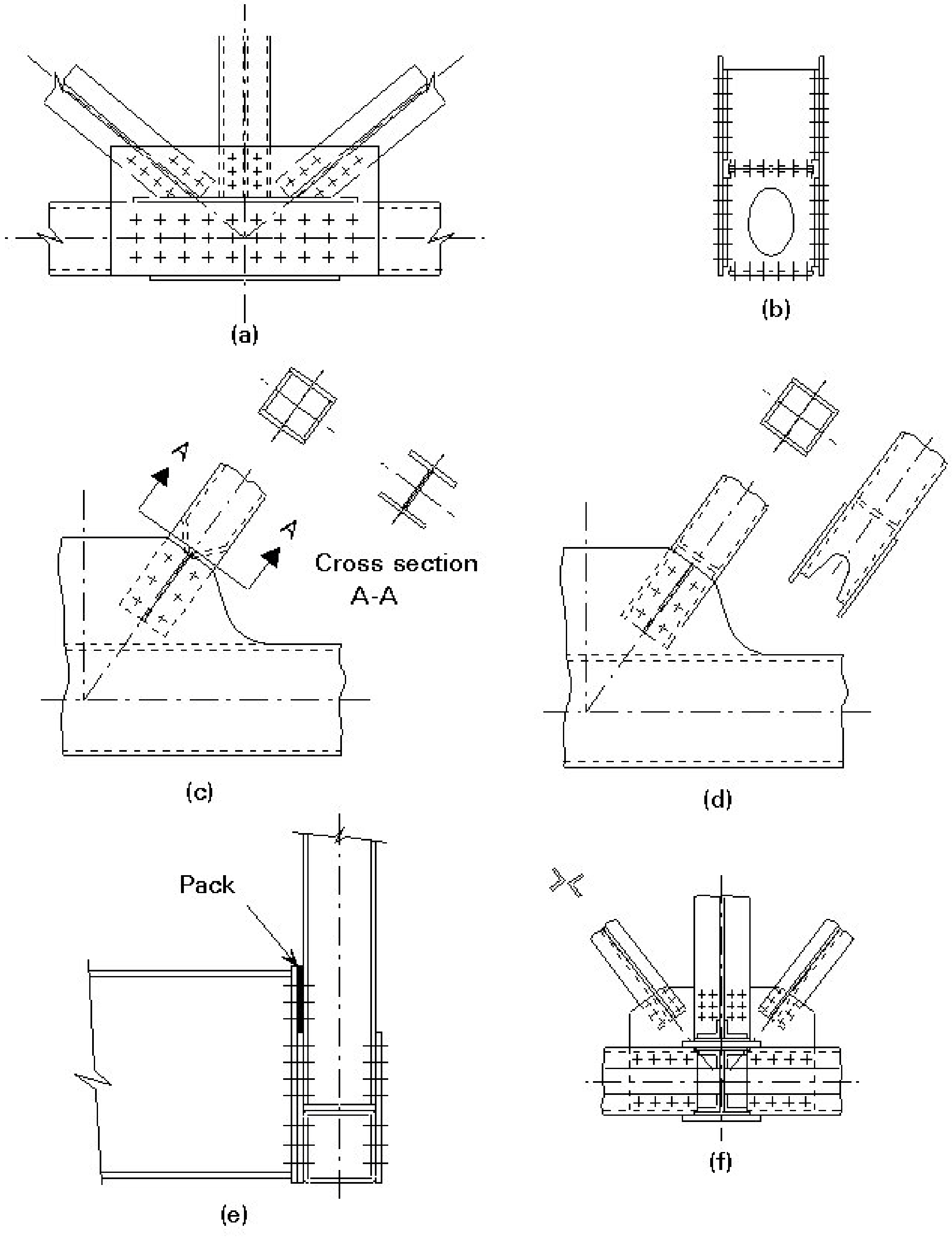


Fig. 8.9 Bolted Truss Bridge Connections

The gusset plates form the external web cover plates. Since they work in the dual capacity of cover plate and web connector, their thickness takes this into account. The joint is designed to carry the coexistent load in the lesser loaded chord plus the horizontal component of the load in the adjacent diagonal. The load from the other diagonal is transferred to the more heavily loaded chord through the gussets alone. In compression chords which have fitting abutting ends in contact, design codes allow up to 75% of the compressive load to be carried through the abutting ends.

Sometimes the gusset is formed by shop-welding a thicker shaped plate to the chord in place of the chord web. The web members are then all narrower than the chords and the chord splice is offset from the node. An advantage occurs in erection as the web connections can be made before the next chord is erected.

At the connections of all tension members and elements, care has to be taken in the arrangement of bolt holes to ensure that the critical net section area of the section is not so small that fracture will govern. If necessary remember that the critical net section is usually at the ends of the section or the centre of the cover plates, and that elsewhere some of the load has been transferred to the other parts of the joint and more bolt holes can be tolerated.

Connections of web members to gussets are quite straightforward and special treatment such as the use of lug angles is rarely required. In connecting rectangular hollow sections the method shown in Figure 8.9d is preferable to that of Figure 8.9c.

Unsupported edges of gussets should be such that the distance between connections does not exceed about 50 times the gusset plate thickness (Fig. 8.9a). If this is unavoidable, the edge should be stiffened.

8.5.2 Cross Girder Connections

They are quite straightforward. The 2 or 4 rows of bolts in the cross girder end plate are made to correspond with the equivalent central rows of bolts in the gusset. Packing plates may be required to accommodate the difference in height of gussets and cross girders (Figure 8.9e).

8.5.3 Lateral Bracing Connections

The axes of the lateral systems should be in the same planes as those of the truss chords. This requirement is met in 2 of the 3 types of lateral members and connections described below:

i. For long and medium spans, the lateral members are frequently made from two rolled channel sections connected by lacing to give an overall depth the same as the chords. They are connected to the chords by gussets bolted to the chord flanges exactly as the main web members are connected to the main joint gussets.

ii. For medium spans, laterals consisting of two rolled angles arranged toe to toe in "star" formation and with intermediate battens are often ideal. They are connected to the chords by gussets positioned at the chord axis (Figure 8.9f). Note, angles "back-to-back", but separated by a small gap should never be used because of maintenance problems.

iii. On short spans single laterals often suffice. They can be connected by a gusset to the upper or lower chord flange, as the moments due to eccentricity are small.

8.5.4 MEMBER SPLICES

Splices of bridge truss members are needed because of the limitations imposed by:

(a) the available length of plates and shapes;
(b) length limits imposed by the transportation facilities; and
(c) capacity of the erecting cranes.

Splices made in the shop are dictated by the available plate lengths. Full penetration butt welding of the V or X type is usually used for shop splices. Splices made in the field are preferably made using high strength bolts. Splices are usually designed to carry the maximum strength of the spliced parts computed from:

$$S_{max} = A_{net} \text{ x } F_t \qquad \text{(Tension)}$$

$$= A_{gross} \text{ x } F_c \qquad \text{(Comp.)}$$

Member splices made with shear plates require a complete design of load transfer from the spliced parts through splice plates. On the other hand, for compression members bearing against each other at the splice location, the bearing surfaces may be milled for full contact and direct load transfer.