

**ECONOMIC DESIGN OF STEEL CONCRETE  
COMPOSITE BRIDGES**

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# ECONOMIC DESIGN OF STEEL CONCRETE COMPOSITE BRIDGES

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## **Abstract**

In the current world, one of the most common bridge construction technologies is steel concrete composite bridge. The primary advantages of this technology are: their economical nature and related to construction time. Due to these, lots of researchers have been contributing different approach. Thus, we are interested to dive into this research.

This thesis provides practical information regarding steel concrete composite bridge design and construction methods practiced currently. Presenting different steel concrete composite bridge types and materials used, their economic design consideration and international versus Ethiopia practice of composite technology are also considered. Moreover, it is presented basic design concept, section properties, connection consideration and also new steel-concrete shear connections for composite bridges as well. Finally, illustration with steel concrete composite bridge design example is made.

The choice of a particular composite bridge type depends on several factors, which may include the specific application, initial cost, life cycle cost and durability, weight or owner requirements.

Finally, we recommend this economical technology in order to expand and answer the demand of bridge in Ethiopia. In addition, it is summarized by listing some future works.

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## Abbreviations

<b>Symbols</b>	<b>Definition</b>
$A_s$	Area of reinforcement
$h_p$	height of the profile
P	Effect on the resistance
F	longitudinal shear force
FRP	Fiber-reinforced polymer
IPC	Inorganic Phosphate Cement
IM	Dynamic load allowance
$f_y$	Yield strength of steel
SLS	Serviceability Limit States
$F_u$	Tensile strength of steel
$E_s$	Modulus of elasticity
$f_c'$	compressive strength
$E_c$	concrete modulus of elasticity
$n$	modular ratio
ULS	Ultimate limit state
DC1	Component dead load1
DC2	Component dead load2
DW	Wearing surface load
$M_u$	Unfactored moment
MD1	Moments due to the factored loads applied to the steel composite section
MD2	Moments due to the factored loads applied to the long term composite section
<i>MAD</i>	Moments due to the factored loads applied to the short term composite section

$s_s$	section modulus for the non composite steel
$s_n$	section modulus for the short-term composite sections
$S_{3n}$	section modulus for the long-term composite sections
$V_{DC1}$	Shear force due to dead load 1
$V_{DC2}$	Shear force due to dead load 2
$V_{DW}$	Shear force due to wearing surface
$M_{DC1}$	Moment due to dead load 1
$M_{DC2}$	Moment due to dead load 2
$M_{DW}$	Moment due to wearing surface
$M_{LL+IM}$	Moment due to live load and impact
$V_{L+IM=}$	Shear force due to live load and impact
$b_{eff}$	effective flange width
$L_{eff}$	effective Length
$t_s$	thickness of the slab
$A$	Cross sectional area
$Y$	Location of neutral axis
$I$	moment of inertia of short-term composite section
$M_y$	Yield moment
$M_p$	Plastic moment
$D_{cp}$	Depth of the web in compression at the plastic moment
$D_p$	Distance from the top of the slab to the plastic neutral axis
$bl$	The projecting width
$R_b$	load shedding factor
$R_h$	Hybrid factor;
$bf$	Full width of steel flange
$F_{ys}$	Specified minimum yield strength of

<i>It</i>	is the moment of inertia for the transverse stiffener
<i>Dp</i>	is the web depth for webs without longitudinal stiffeners.
PNA	Plastic Neutral Axis

# 1 INTRODUCTION

## 1.1 Background

Rapidly increasing construction and maintenance costs of reinforced concrete bridges and also the heavy traffic on major roads have led to an enhanced interest in alternative technologies for bridge design and construction methods to reduce cost, traffic interferences during construction, faster assembly and shorter maintenance times [1].

In recent years, due to the aforementioned reasons this area of research has been getting especial attention to develop new and more durable design materials and methods.

Steel has long history in the tradition of bridge construction. It is a well controlled material in terms of production, fabrication, construction, durability and design methods [2]. Using these advantages of steel, a new advanced constructional technology called *composite construction* has emerged. The design of composite construction is developed with the use of cost effective construction techniques and advanced design procedures to gain economical benefit [3]. The choice of appropriate steel grade and concrete quality affects the gain of economical advantages [4].

Composites offer inherent advantages over traditional steel materials with regard to high strength-to-weight ratio, design flexibility, corrosion resistance, low maintenance, and extended service life [5].

In Ethiopia, most of the bridges are typically short span bridges, concrete box culverts and decks with girders underneath in which most parts of the structure is dominated by concrete [6]. However, steel-concrete composite bridges utilize the advantages of steel and concrete to

produce concrete superior bridge structure [7]. Thus, we are interested to dive into this research area and add value. This thesis covers the different aspects that a bridge designer can use to design a cost effective bridge structure using steel concrete composite materials. In addition, it provides design example on steel concrete composite bridges as illustration.

## 1.2 Problem Identification

In response to continuous economic development over the past few years, Ethiopia has mobilized a program of large scale road construction [6, 8]. Even though there is a high technological advancement in bridge construction, Ethiopia does not follow advanced technology of structural system, material usage, construction methods and equipments as per the expected level [6]. Most of the bridges found in the country as well as those are under construction are reinforced concrete [6].

Reinforced concrete construction especially girder bridges (T or box type) takes longer time [9]. Due to this long time inquiry, it has negative outcomes especially during flood season. These outcomes are: take of the forms and false work prior to casting and sometimes include the freshly casted concrete.

Steel concrete composite in bridge construction uses steel girder which can be either truss or steel profile such as I-section, wide flange and etc with cast in place reinforced concrete deck to form the composite section [7]. This makes the construction of the main girder faster. Furthermore, it gives high strength in the whole tension zone of the bridge, and huge amount of economical benefits [10].

Due to the aforementioned advantages, lots of researchers have been giving especial attention to advance this technology in bridge design and construction methods. Therefore, we are interested to study the current available technologies from their area of applications.

## 1.3 Scope of the Study

This study focused on steel concrete composite with emphasis being placed to assessment of the state of the art in view of introducing to the Ethiopian practice.

This thesis addresses the following key points:

- Profiles and material properties for composite construction
- Locally available profile steel sheet forms and their application
- Types and components of possible steel concrete composite bridge sections

It is then narrowed at illustrating practical design example for use to the Ethiopian practice.

#### 1.4 The Structure of the Thesis

This thesis is organized as follow:

**Chapter one:** - discusses background of composite bridges design and construction, problem identification, and scope of the thesis.

**Chapter two:** - deals with steel profiles and state of the art in composite construction, material used in composite construction, types and component of steel concrete composite bridges and new structural materials in composite construction.

**Chapter three:** - focuses on economical design and construction consideration of steel- concrete composites, methods of economical and efficient use of fabrication technology. The Ethiopian and international practices of steel concrete composite bridges is also addressed.

**Chapter four:** - discusses basic design concept including section properties, Connection Consideration with design example as illustration.

**Chapter five:** - is devoted at summarizing and highlights of the design concept using practical example.

**Chapter six:** - addresses about conclusion, recommendation on future direction(s) of such study.

## **2 STEEL PROFILE FOR COMPOSITE CONSTRUCTION AND STATE OF THE ART**

Composite deck is widely used in bridge construction industry. Its casting of concrete is carried out on corrugated steel profiles as permanent formwork which is supported by floor beams. In composite slabs the steel decking has two main structural functions, which are [11]:

1. During the time of concreting, the decking supports the weight of wet concrete and reinforcement together with the temporary loads associated with the construction process. It is normally designed to be used without temporary propping.
2. In service time, the decking acts compositely with the concrete to support the loads on the deck. Composite action is obtained by shear bond and mechanical interlock between the concrete and decking.

In addition, the decking has other important functions which are [11]; to stabilize the beams against lateral torsional buckling during construction, and the building or the bridge as a whole by acting as a diaphragm to transfer wind loads to the walls and columns. The decking together with the fabric mesh reinforcement placed in the top of the slab; helps to control cracking of the concrete caused by shrinkage effects.

### **2.1 Types of Steel Profiles and Design Classification**

Currently, there are lots of steel profile sheets and design classifications. These technologies plays great role in bridge construction [12, 13]. In the coming subsections, we will address main points about steel profile sheet and its analysis.

### 2.1.1 *Steel Sheet Profile*

There are many types of steel profile sheets used in composite construction. This classification depends on the required span length for the product, resulting resistance and stiffness requirements in the construction and composite stage [12].

The structural thickness of the profiled steel sheets to which the stress and section properties apply is the bare metal thickness of the sheets excluding any protective or decorative finish such as zinc coating or organic coating [14].

Deck profiles are usually in the range of 40 – 80mm height and 150 – 300mm trough spacing with sheet thicknesses between 0.8 and 1.5mm. Using the lower thickness is limited due to local buckling; and the upper thickness controlled due to difficulties in rolling [15]. Due to the local buckling and difficulties in rolling; the lower thickness is limited and upper thickness is controlled respectively.

There are different types of steel profiles based on their shape. The shape of the profile is controlled by a number of points; some these points are [12]:

- Requirement to increase efficiency of the cross-section in bending
- Requirement to develop sufficient composite action with the concrete by use of indentations or by the shape of the profile itself
- Well-organized transfer of shear

There are two well known types of decking profiles, the re-entrant profile and trapezoidal profile. The following Figures depict these two well known decking profiles.

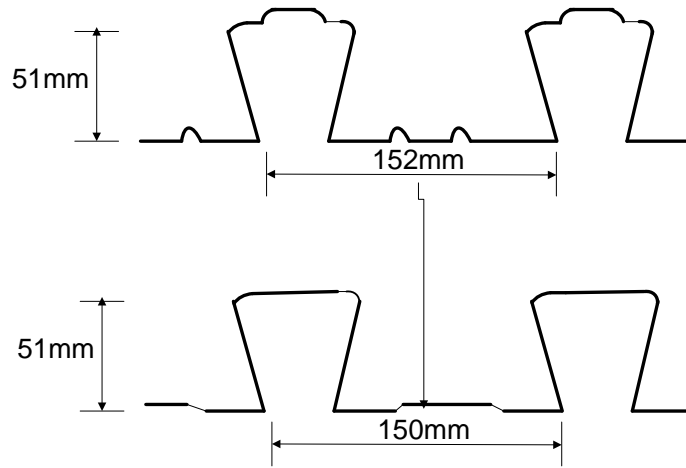


Figure 2-1 : Re-Entrant profile sheet decking [15]

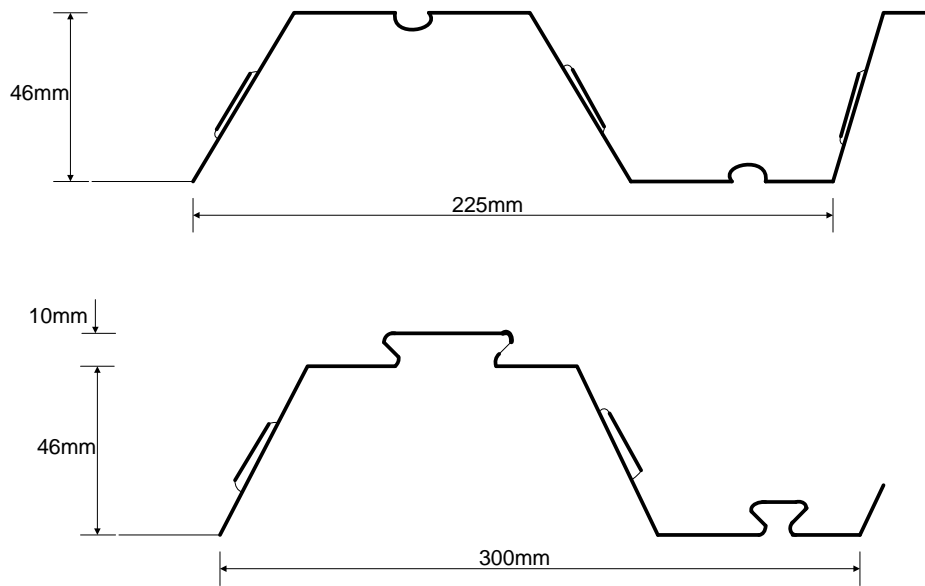


Figure 2-2 : Trapezoidal profile sheet decking [15].

### 2.1.2 Analysis for Internal Forces and Moments

It is possible to permit the moment over the internal supports to be redistributed into the span at the ultimate limit state during the analysis of unpropped profiled steel sheeting [12]. Most commonly, the amount of plastic redistribution has been using within profiled steel sheeting is about between 5 to 10 percent [12, 16]. On the other hand, greater values of redistribution can be possible for some modern steel sheets. In such great value of redistribution, the exact amount that may be assumed in design has to be evaluated from tests [12].

The use of temporary props permits much longer spans and thinner sheets to be used [12, 15]. In this situation the spanning capability of the system is often dictated by the longitudinal shear resistance of the shear connection to the composite slab. However, unlike unpropped construction does not permit plastic redistribution at the ultimate limit state when temporary supports are used [12].

## 2.2 Girder Profile

For simple spans over about  $25m$ , a construction depth of between about  $1/18$  and  $1/30$  of the span can be achieved with fabricated beams; but the most economical solution is by making the deeper end of this range [7, 17]. The cost of applying a protective coating system may increase with the increased surface area of a deeper beam in comparison with a shorter beam. For shorter spans, the depth is likely to be proportionately greater, particularly for spans under  $20m$ . This might limit the use of a welded beam due to the available depth in comparison to a fabricated beam [18].

For composite continuous spans with parallel flanges, the construction depth is typically between  $1/20$  and  $1/25$  of the major span [7, 19]. The use of curved or tapered haunches can reduce construction depth at mid-span, at the expense of increasing depth at the internal supports [18].

## 2.3 The State of Art of Steel Concrete Composites

Composite member is a structural member with components of concrete and structural or cold-formed steel. It is interconnected by shear connector to limit the longitudinal slip between the concrete and steel, and the separation of one component from the other [10].

Bridges benefit most from composite action. As the state of the art in the development of steel-concrete composite bridges, the following key points have to get huge attention [18].

- Analysis and design methods of composite bridge structures,
- Connections between composite bridge components,
- Establishment of new concrete-steel bridge systems,
- The development of alternative materials to be used in composite bridges.

### 2.3.1 *Forms of composite construction*

Most commonly steel-concrete composite structures take a simple slab and beam forms. These forms might be used for range of structures [20].

#### 2.3.1.1 Composite slabs

There is a wide variety of composite slabs considering all combinations of composite slabs with in-situ concrete and the wide range of pre-cast slab products [15]

The most common composite slabs are made from the combination of steel decking and in-situ concrete placed onto the steel decking [15, 16, 21]. When the concrete has gained strength, it acts as a composite slab with the tensile strength of the decking [21]. There are different types of in-situ composite slabs considering their dimensions and also the steel decking used [21].

In the actual building construction pre-cast concrete floors are widely used [22]. There is an increase of composite frames and slim floors construction where the pre-cast slabs are designed to interact structurally with the steel frame [23]. Like in-situ composite slabs, the composite action can be developed by welded shear connectors attached to the steel beams and by transverse reinforcement. The most common types of pre-cast slabs are [22]:

- I. Hollow core units, with continuous circular or elongated openings along their length (Fig. 2.3)
- II. Solid planks, which are designed to use with an in-situ concrete topping (Fig 2.4)

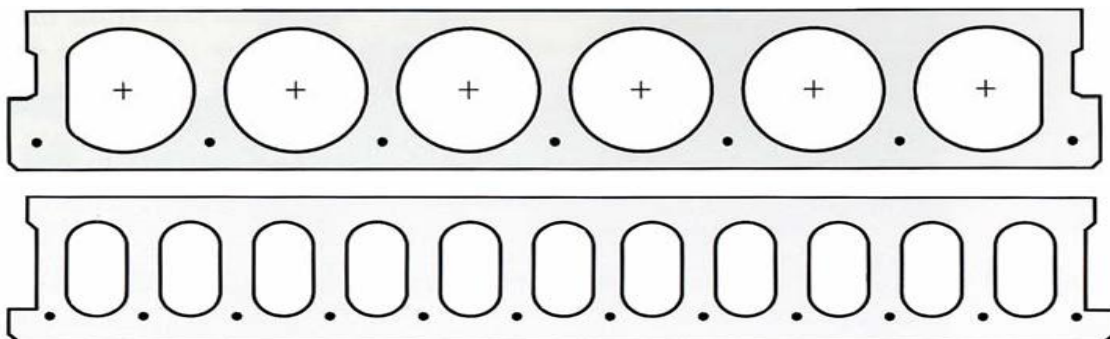


Figure 2-3: Hollow core slabs [15]

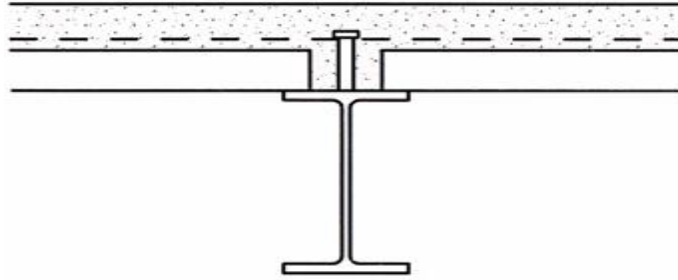


Figure 2-4: Solid plank slab[15]

### **2.3.1.2 Composite beam**

Composite beams typically consist of steel “I” sections acting structurally with a concrete slab by means of shear connectors attached to the top flange of the steel section [15]. The beams are generally designed to be simply supported, and an effective part of the slab is taken as acting as a part of the composite section on either side of the centre line of the section. It is possible for a significant saving of steel weight and structural floor depth due to the composite action of the steel beam and the concrete slab which increases the stiffness of the beam and the load capacity.

The size of steel section used in composite beam often chosen based on its serviceability consideration. There are two main groups of composite beams in composite construction [15]:

- I. Primary beams, and
- II. Secondary beams.

Primary beams have decking spanning in a direction parallel to them, so the decking do not provide laterally restrain.

Secondary beams are perpendicular to the decking spanning direction and the top flange of the steel beam is restrained laterally due to the decking and fixings. This is very important to consider at the design stage to choose the proper steel “I” section.

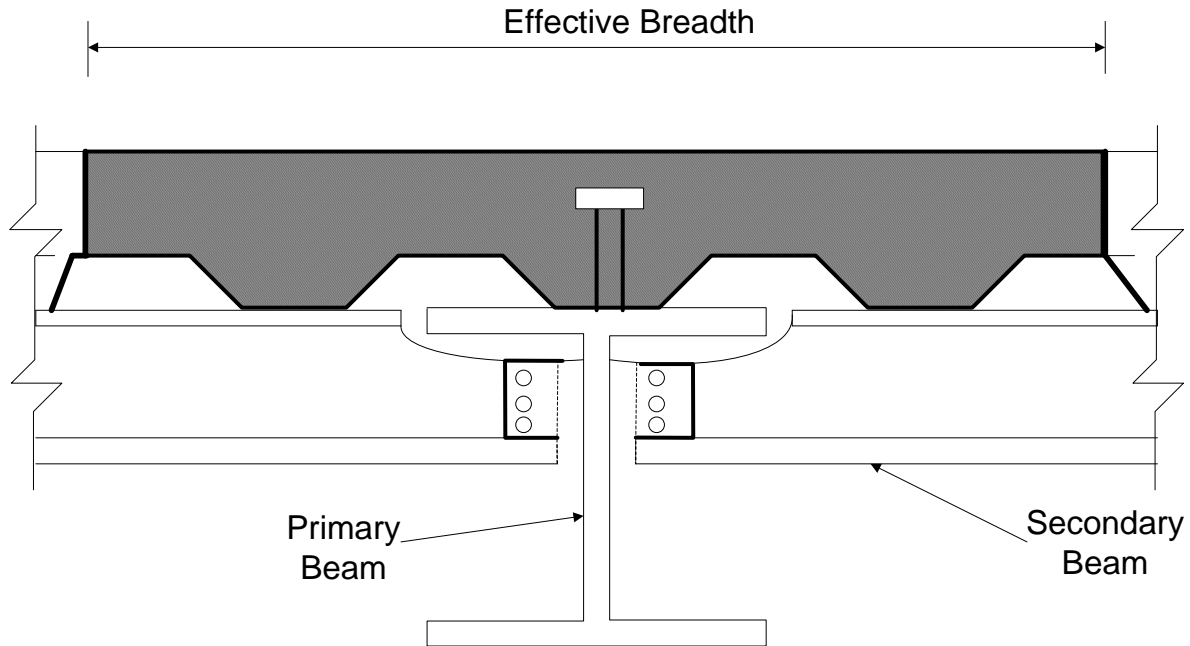


Figure 2-5: Primary and secondary beams [15]

### 2.3.2 *Material Properties in Composite Construction*

There are different materials used to implement composite construction. The following basic materials have enormous role to determine its strength and also entire structural properties [5].

#### 2.3.2.1 *Structural steel*

In composite construction design, the two main grades of steel that can be used are steel with minimum yield strength of  $275 \text{ N/mm}^2$  and  $355 \text{ N/mm}^2$ , respectively, is guaranteed [15].

#### 2.3.2.2 *Profiled steel decking*

The way in which the grades of steel for profiled steel sheeting or decking are specified is in terms of the yield strength of the steel. The common usable strength of grades for sheet is about between  $280 \text{ N/mm}^2$  and  $350 \text{ N/mm}^2$  [15, 21].

#### 2.3.2.3 *Concrete*

Concrete grade between *C25* and *C35* can be used in accordance with the design requirements [24, 25]. Light weight concrete is commonly used because of its weight which helps to save effect and insulation qualities. In case of concrete, the minimum grade for a wearing surface should be *C35*.

Generally, concrete grade in composite construction is chosen based on the following key points [16]:

- Overall structural requirements
- Exposure conditions
- Flooring to be laid on the slab

#### 2.3.2.4 Reinforcement bars

Reinforcement bar used in composite slab construction. It usually takes the form of a relatively light mesh, commonly supplemented by some bar reinforcement. The mesh reinforcement is required to perform a number of different functions [15]:

- Provide bending resistance at the supports of the slab in the fire condition.
- Control and reduce cracking at the supports.
- Distribute the effects of localized point loads and line loads along a greater area.
- Increase the strength at the edges of openings.
- Acts as transverse reinforcement to the composite beams

In shallow composite slabs, the reinforcement should be supported sufficiently above the top most of the deck to allow concrete placement around the bars. It is necessary to have a proper cover [15].

## 2.4 Types and Components of Steel-Concrete Composite Bridges

In steel-concrete composite construction, there are different types and components which are used to design different bridge structures. This subsection discusses the main types and also their components.

### 2.4.1 *Types of Steel Concrete Composite Bridges*

Most common composite bridge structures are either pre-cast, pre-stressed concrete beams with a cast concrete slab or steel girders with a concrete slab configured in different orientations [7].

The typical configurations include the following main types are:

- I. **Multi-Girder Steel-Concrete Composite Bridge**, which consists of a number of steel girders with bracing in between and a slab on top.

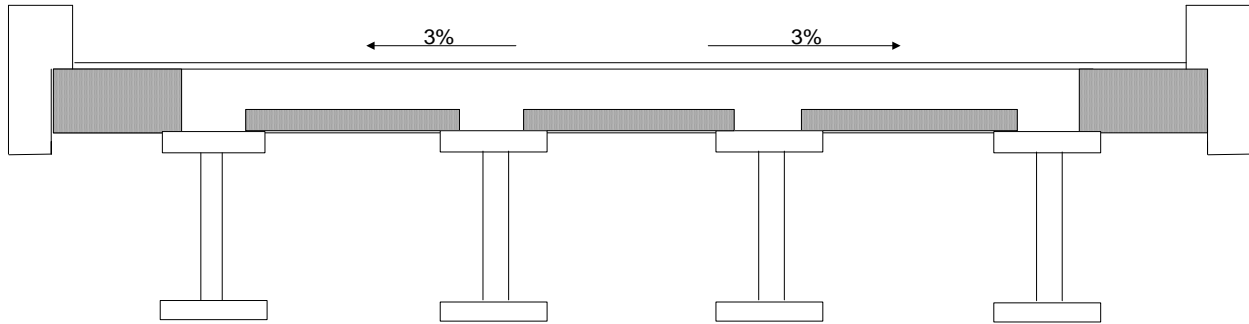


Figure 2-6: Cross-Section of a typical multi-girder bridge [7]

- II. **Ladder Deck Bridge**, which consists of two main girders with a number of secondary cross-girders in between that support and act with a deck slab.

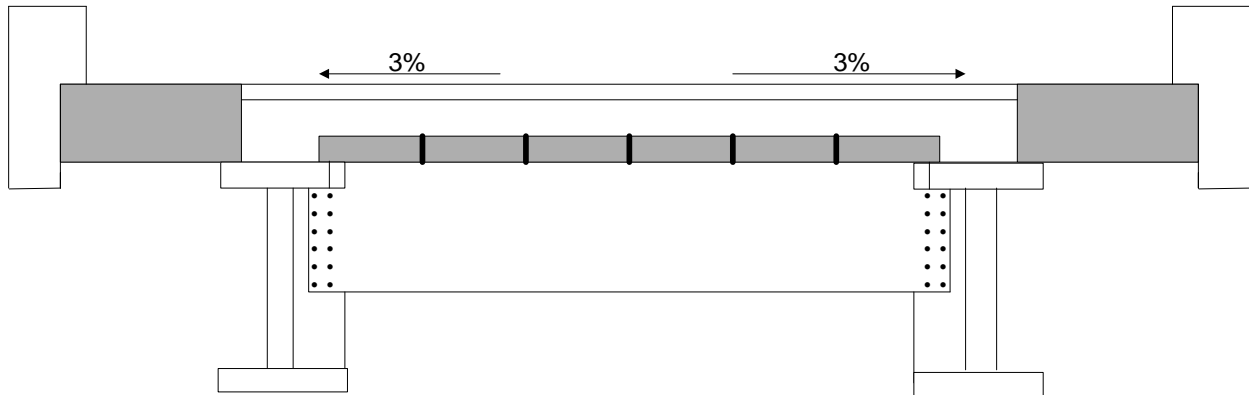


Figure 2-7: Cross-Section of a typical ladder deck bridge [7]

- III. **Box Girder bridges**, Steel-concrete composite box girders offer an attractive and economic form of construction for medium span bridges ranging between 50m and 100m. They are commonly used for spans where plate girder sizes may be excessive or where torsion, curvature or wind and seismic forces demand greater torsional stiffness. Torsional stiffness is provided by the hollow box shape of the girder that is either rectangular or trapezoidal in cross section. Basically, the box girder comprises two ‘webs’ that are connected by a single bottom flange, while each web has its own top flange for an open box girder or another single top flange for a closed box girder. Stiffeners are evenly spaced along the length of the girder and usually a single wide box girder is sufficient for a single or double lane bridge. For wider bridges, two or more box

girders can be used, with additional restraints provided, if required at supports, between them [7].

- IV. **Network arch bridges**, A network arch is a tied arch structure where the hangers supporting the tie, which also acts as the deck of the bridge, are inclined and arranged in such a way that they cross one another at least twice. Such structures are used to carry a roadway, railway or footpath.

Like any tied arch, the load on the deck is carried principally as compression in the arch and tension in the tie. Increasing the rise of the arch reduces the axial forces in both the arch and tie. The majority of the shear force is taken by the vertical component of the arch top chord force, with any variation in the shear force taken by the hangers [7].

All types provide a cost-effective solution and the choice between them depends on economic considerations and site-specific factors such as form of intermediate supports and construction access.

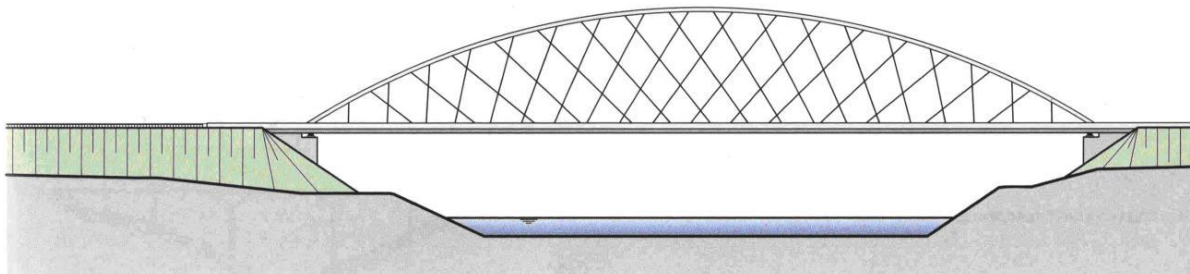


Figure 2-8: Network arch as Innovative alternative Railway Bridge [26]

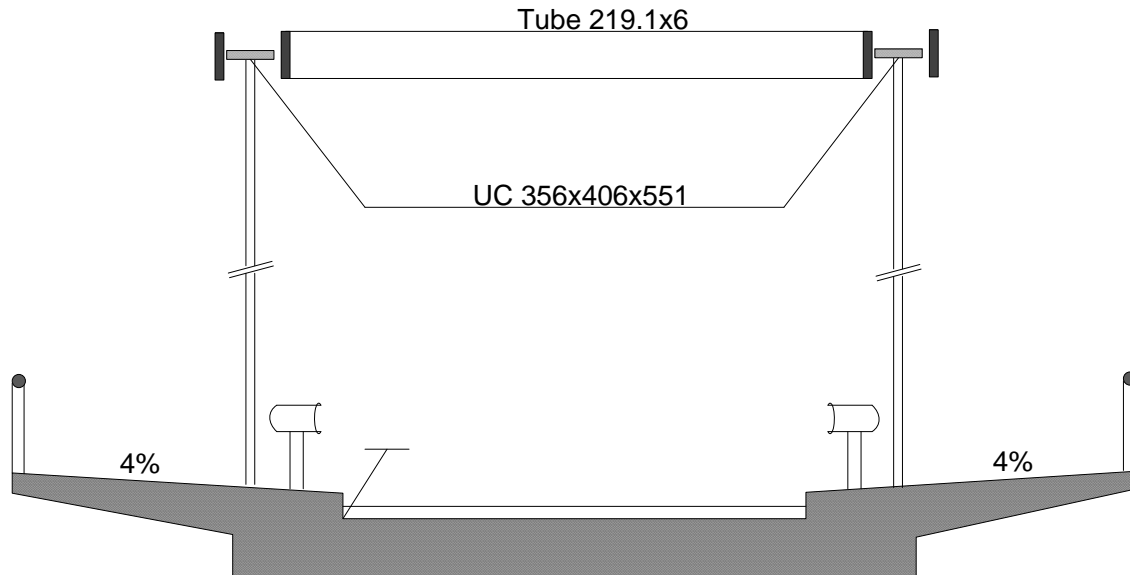


Figure 2-9: Cross-Section for Network arch Bridge [of Figure 2.8,Ref. [26]]

#### 2.4.2 Components of steel concrete composite bridges

A Typical steel concrete composite bridge consists of a concrete deck at the top and steel girders supporting the deck and also the following main parts [16, 27]:

- I. Shear studs are welded to the top flanges of the girders to achieve composite action.
- II. Stiffeners consist of longitudinal, transverse, and bearing stiffeners are used to prevent local buckling of plate elements and to distribute and transfer concentrated Loads
- III. Top lateral bracing is designed to resist shear flow and flexure forces in the section prior to curing of concrete deck.
- IV. Internal diaphragms or cross frames are usually provided at the end of a span and interior supports within the spans. Internal diaphragms provide warping restraint to the box girder and also improve distribution of live loads, depending on their axial stiffness which prevents distortion.

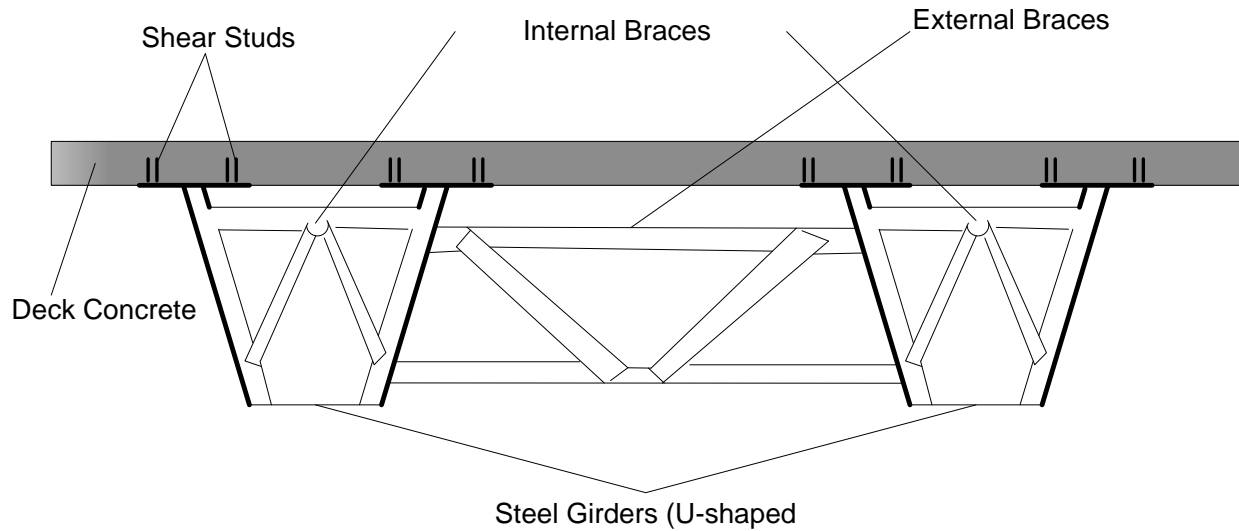


Figure 2-10: Typical steel concrete composite bridge cross-section [7]

## 2.5 New Forms and Materials Used In Composite Bridges Construction

The behavior of composite structure is heavily influenced by the properties of its component materials and forms [28]. For example, the use of a concrete slab on a steel girder uses the strength of concrete in compression and the high tensile strength of steel. Looking at the basic behavior of a composite structure there are two fundamental effects that need to be considered [29]:

- I. The differences between the materials, and
- II. The connection of the two materials.

Likewise steel more stronger and stiffer materials attract proportionally more load than materials like concrete. The materials will behave independently by omitting the positive effects if there is no connection. However, it acts as one whole structure if it is connected adequately [30].

New structural forms of steel-concrete composite bridges were also invented and suggested to be used in different studies. Such as using concrete filled pipes or rolled H-girders have high strength and ductility, the concept of voided slab in building construction and also partially encased composite I-girder bridges.

Steel-concrete composite box girders also may advantageously use for bridges with long spans, for bridges with significant horizontal curvature or simply for aesthetic reasons [18].

The boxes may be complete steel boxes with an overlay slab or an open box where the concrete slab closes the top of the box.

The open top form of box girders consist steel web and a bottom flange which has only small top flanges sufficient for stability during concreting. This form has various advantages; the common one is that accessibility of all parts of the section to make available for different facilities, for instance, welding, and the web can be inclined to allow a larger span in the transverse direction of the bridges [7].

## 2.6 Availability of Steel Profile, Practices and Application in ETHIOPIA

Even though local industries has been manufacturing steel profiles, It has been using for car bodies and mainly roofing and cladding purposes [6]. These profiles are also suitable to implement steel-concrete composite bridge construction technology. However, this technology does not practicing in Ethiopia due to different reasons. As mentioned under the above sections, one of the main advantage of this construction technology is that its economical benefit. Thus, is has to get especial attention to answer the bridge demands of the country. Furthermore, Ethiopia Roads Authority (ERA) has to introduce this technology to the local contractors by preparing different seminars to advance and standardize the bridge design and construction technology using domestic materials.

## 2.7 Locally Available Profiled Steel Sheet Forms

The Table below shows that the range of sizes of steel sheets imported by local factories for the production of construction materials such as rectangular and circular tubes, tankers, profiled steel sheets, etc. The entire locally manufactured profiled steel sheets' standard forms is the same. These forms of profile sheets i.e. EGA-300, EGA-400, EGA-500, EGA-600 & EGA-700 have been hardly used for composite slab construction for buildings [31].

Table 2-1: Type of sheet metals imported by local factories [31]

Item No.	Production Firm	Steel Sheet Type	Sheet Metal Thickness in mm	Plan Dimension
1	Kality Metal Factory	Galvanized steel sheets	0.25,0.3,0.4,0.5,0.6,0.7,0.8, & 1.0,1.5	One meter width in coils
		Non-galvanized steel sheets	0.8,1.0,1.2,1.5,1.6,1.8,2.0,2.5, & 3.0	One meter width in coils
			4.0,5.0,6.0,8.0,10.0,12.0	1.0 meter wide & 2.0 meters length
2	GATERPRO Metal Industry	Galvanized steel sheets	0.25,0.3,0.4,0.5,0.6,0.7,0.8,1.0	One meter width in coils
		Non-galvanized steel sheets	4.0,5.0,6.0,8.0,10.0,12.0	1.5 Meter Width & 6.0 meters length
3	Maru Tefera Metal Factory	Galvanized steel sheets	0.25,0.3,0.4,0.5,0.6,0.7,0.8,&1.0,1.5	One meter width in coils
		Non-galvanized steel sheets	1.5,2.0,3.0,4.0,5.0,6.0,8.0,10.0,12.0	In coils for sheets less than 4.0mm & in cut lengths for 4mm and above thickness
4	Kombelcha Steel Product Industry (KOSPI)	Galvanized steel sheets	0.25,0.3,0.4,0.5,0.6,0.7,0.8	One meter width in coils
		Non-galvanized steel sheets	--	--

Examples of cross-sections for cold-formed members and sheets are illustrated as follows



Figure 2-11: Profiled Sheets [32]

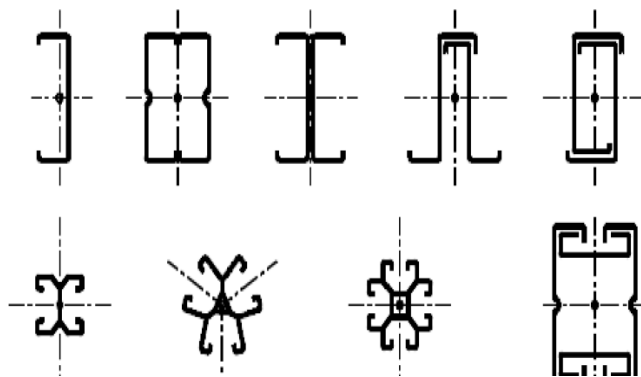


Figure 2-12: Compression and Tension Members [32]

In the current construction practice, the development of steel-concrete composite construction is developing rapidly in a wide range. However, this technology is not practiced immensely due to different reasons in Ethiopia. In Ethiopia, there are some bridges which use composite construction technology. The following two are out of them [6]:

- i. *Omo River bridge*: - 128 m central span steel truss and 36 + 36 approach RC Box girder located near Omorate, in the Southern Nations, Nationalities & People's Region. The bridge is designed to be part of the link between Addis Ababa and South Sudan and Kenya. Composite construction in which truss with concrete deck was applied for the bridge.
- ii. *Tekeze Bridge*:- Has total length of 280m. Located in the northern part of the country, between Adihageray and Tekeze. In the bridge composite construction is applied and now functioning properly.

### **3 ECONOMIC DESIGN CONSIDERATION OF STEEL CONCRETE COMPOSITE BRIDGES**

#### **3.1 Importance of Steel Concrete Composite construction**

An important and economic combination of construction materials is that of steel and concrete to build above medium buildings and also bridge constructions. Composite bridges are structures in which material utilization is effectively reduced due to different reasons. Efficient use of material as a result of composite design can reduce the size and weight of steel beams and other structural parts [28].

There are different ways of associating steel and concrete in the design and construction process of bridges. The most commonly used method is that the connection of a concrete slab to a steel structure which becomes a typical cross section [16].

#### ***Why we use composite bridge?***

Most short span bridges in most countries around the world with spans up to 30m have been built in concrete, typically comprising pre-cast concrete beams and reinforced concrete decks [7]. However, well-designed steel-concrete composite bridge comprises steel girders and reinforced concrete decks will also provide an economical and sustainable solution [18]. Such assemblies has the following advantages.

- Composite bridges have advantages with regard to foundation and settlements of supports, because the dead weight of composite bridges is much lower than concrete bridges.
- Steel structures are easily and quickly erected using incremental launching.
- Either cast in place or pre-cast panel reinforced concrete slabs are easily built when steel profiles have been launched [7].

- Concrete and structural steel are used in a way that the high tensile strength of structural steel and the high compressive strength of concrete are used in an optimal combination [5].
- In case of limiting crack width control in combination with a sufficient depth of the concrete slabs, the decks are made to have high durability and fatigue strength.
- Because of new erection methods and the use of partially prefabricated composite girders, composite bridges are often used for passing over existing rail and highways without any restriction for the traffic during construction [5].
- The steel girders can carry the weight of the formwork and the fresh concrete during casting; hence reduce the uses of false works.
- Reduction of construction time, a serious problem in reinforced concrete bridge construction
- Acts as a safe working platform, protects the workers below and supports the loads during construction and may eliminate the need for temporary propping.
- Ability to be repeatedly re-used or recycled without any degradation in the quality and mechanical properties of the material.
- Little waste material is generated during the manufacture of steel components, and most of this is recovered and recycled. On construction sites, which can often generate large volumes of waste of other material, off-site fabrication ensures that no steel is wasted, as only what is needed comes to site. Almost all of the material waste generated in the fabrication shop is recovered for re-use or recycling
- Off-site manufacture has always been a key feature of steel construction as it is for concrete with pre-cast construction such as bridge decking, which allows a composite structure to score highly on many sustainability criteria. More accurate components can be achieved with composite elements manufactured and fabricated off-site. Waste is minimized and high-quality, defect-free products are possible. In the modern fabrication workshops and pre-casting yards, where state of the art numerically controlled machinery is fully integrated with computer aided design and other software, composite elements can be easily standardized, tested and certified. Corrosion protection coatings can be applied to steel elements at the fabrication stage, reducing the overall site construction program. Off-site manufactured elements lead to more predictable construction

programmers and also benefit from just-in-time delivery, being able to hold these elements at depots.

- Since the total life time cost must be considered in civil engineering works composite bridges have relatively lower life time cost and also according to the need of maintenance requirement composite bridges mainly requires only restoration of corrosion resistance mechanisms, expansion joints and water proofing course this considerably reduce maintenance cost when compared to other structure types.
- A well designed composite bridge has potential spans to superstructure type.
- The most economical structure type may be selected as a function of span by the use of the Figure below [7].

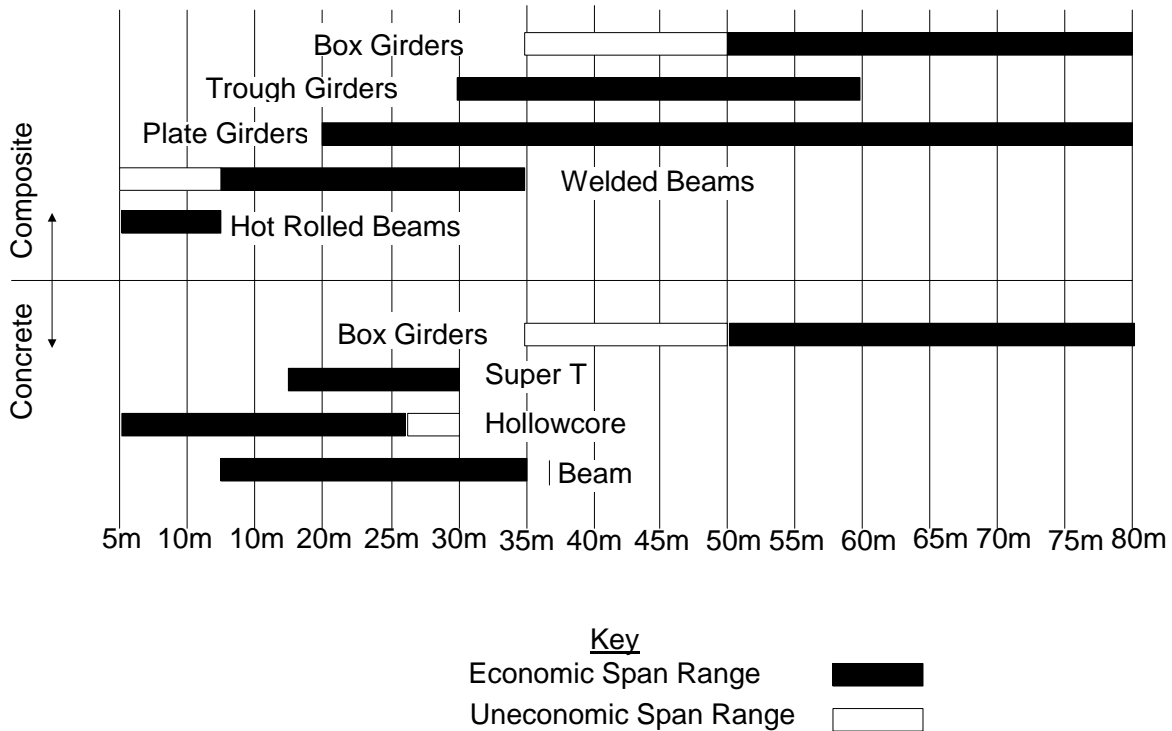


Figure 3-1: Span to super structure type for bridge construction [7]

### 3.2 Design Considerations

There are factors which have to be taking into account at the time of design. In this subsection, we discuss the common factors.

### 3.2.1 *General*

The following demands are made on bridge structures, which are all met by choosing the most economic methods of design in bridge engineering [18, 21]:

- Low production and maintenance costs
- Lower construction time
- Construction without necessarily interference of traffic under bridge
- Minimized traffic disturbance for maintenance action

These requirements are met by choosing the most economic methods of design. To satisfy the mentioned requirements in composite bridge design, the following points shall be considered [7]:

- i. Steel concrete connections,
- ii. Selection of steel profile sheeting in terms of thickness, attachment & their behavior in shear.
- iii. Identifying economic methods of design in composite construction,
- iv. Superstructure steelwork: Use the least overall main girder weight, if possible, independent of the number of girders, with minimum numbers of braces. This is found to be the best option to reduce handling and erection cost irrespective of the coating cost,
- v. Slab construction: Partial depth pre-cast solutions provide a cost-effective solution when all factors and constraints are taken into account. However, in situ concrete on steel decking as permanent formwork, with the appropriate corrosion protection system may provide additional cost savings,
- vi. Piers: Utilize the increased composite high weight to strength ratio to maximize the bridge span and to reduce the number and size of the piers,
- vii. Abutments: Replace the abutments with concrete segmental retaining walls and concrete piers or steel piles.

### 3.2.2 *Material properties*

The most important and most frequently encountered combination of construction materials is that of steel and concrete.

These essentially different materials are completely compatible and complementary to each other. They have almost the same thermal expansion, they have an ideal combination of strengths with the concrete efficient in compression and the steel in tension, concrete also gives corrosion protection and thermal insulation to the steel at elevated temperatures and additionally can restrain slender steel sections from local or lateral torsional buckling [33].

### 3.2.3 Section Properties

Section properties of main structural members have to be determined at critical points. It is required for the bare steel members at the construction stage and for composite members at service.

Critical section properties required are includes [34],

- ✚ Cross section area,  $A$
- ✚ Second moment of area about the major axis,  $I_x$
- ✚ Plastic section modulus,  $S$
- ✚ Elastic section modulus at extreme fibers,  $Z$
- ✚ Torsion stiffness,  $J$
- ✚ Out of plane stiffness,  $I_{yy}$

### 3.2.4 Loading

The actual stresses that result due to a given loading on a composite member is depend upon the manner of construction [35].

- i. *Shored construction*: - in which temporary propping is used in order to reduce the service load stress. After curing the shores are removed and the section acts compositely to resist all loads.
- ii. *Un-shored*: - in which the steel beam acts non-compositely to resist the weight of the forms, wet concrete, and its own weight. After removal of the formwork and curing of concrete the section act compositely and resist all loads.

### 3.2.5 *Analysis*

Methods of analysis used to calculate action effects [36]. The common methods are:

**Elastic Analysis:-** In elastic analysis the order of loading is importance and also the analysis to hold stress at extreme fibers should be lower than the design values and slip at the interface should be negligible.

**Plastic Analysis:-**ultimate strength of composite section is determined from its plastic capacity. Plastic capacity of section is independent on the order of loading. It is also assumed that the steel stress is at yield throughout the section and the concrete stress are at their design strength.

### 3.2.6 *Design*

The structure should conform to their use and should be both safe and functional. To meet these requirements, composite structures or structural members must be designed such that they meet the required performance in all areas such as safety, serviceability, durability and environmental compatibility under the various actions that affect them during their construction and service periods [16].

## 3.3 Construction Consideration

As discussed under the above subsection, there are factors we have to consider at the time of design. Even though the design considers all the listed aspects, it should have to be practical for construction. This subsection talks about applicability of design before going to the implementation phase.

### 3.3.1 *Design for construction*

While minimizing cost may be the most obvious consideration when embarking on the design of a highway bridge, the health and safety of all those concerned in the bridge construction and in its maintenance throughout its life time is the responsibility of all decision makers related to the procurement of the bridge. As well as aiming for a structurally efficient solution, the hazards associated with the construction process must be fully appreciated from the outset.

From the designer's point of view, composite structure is complicated, because the majority problems for both steel and concrete construction appear during the design phase [37]. Moreover there are also a lot of issues which are typical for composite construction as well.

In structural engineering, composite construction exists when two different materials are bound together so strongly that they act together as a single unit from a structural point of view [37, 38]. To achieve this requirement great attention must be given not to disturb the intended load transfer between the combined materials. One common example involves steel beams supporting concrete floor slab. If the beam is not connected firmly to the slab, then the slab transfers all of its weight to the beam and the slab contributes nothing to the load carrying capability of the beam. On the other hand, if the slab is connected positively to the beam with studs, then a portion of the slab can be assumed to act compositely with the beam. In effect, this composite creates a larger and stronger beam than would be provided by the steel beam alone.

### *3.3.2 Economic Selection of Steel-Concrete Composite Construction*

New methods for economic construction of composite bridges with different span length have now been developed. This method unites the benefits of conventional composite and prefabricated construction methods and consists of an integral system of prefabricated assembly units.

Among different factors to consider at the time of construction under economic selection of steel-concrete composite construction, steelwork fabrication is the main one [7]. In the overall appearance and minimum use of complex details are most likely to lead to an economic and efficient bridge structure, though external constraints often compromise selection of the best structural solution. With the widespread use of computers in design and in control of fabrication shop machines, geometrical variations, such as curved soffits, varying super-elevation, plan curvature and precambering, can be readily achieved with reasonably minor cost penalties.

Much of the total cost of fabrication is incurred in [7]:

- The addition of stiffeners
- The fabrication of bracing members
- Butt welding

- The attachment of ancillary items such as stud welding
- Local detailing that leads to a significant manual input to the process

Hence, fabrication advice should be obtained first directly from fabricators to assist in the choice of details at an early stage in the design which make easier the process of construction.

### 3.4 Some Economical Methods of Design in Composite

The design plays enormous role to get economical benefits in the construction of steel-concrete composite structures. If the design does not consider all the factors carefully, we will not get its advantage(s). In the coming subsection, we explain some of the economical methods.

#### 3.4.1 *The prefabricated composite construction method*

Nowadays, the prefabricated construction method becomes more and more popular in different countries. Due to its efficient use of steel and concrete in the cross section, girders prefabrication and fast assembly technology open new possibilities in composite construction [7].

The main focus of the pre fabricated composite construction method is applying prefabricated composite girders with 10-12cm thickness active concrete flange, manufactured in a factory. This flange simultaneously makes up formwork of the deck and actively co-operates with beam to contribute to the dead and useful load carrying capability [7].

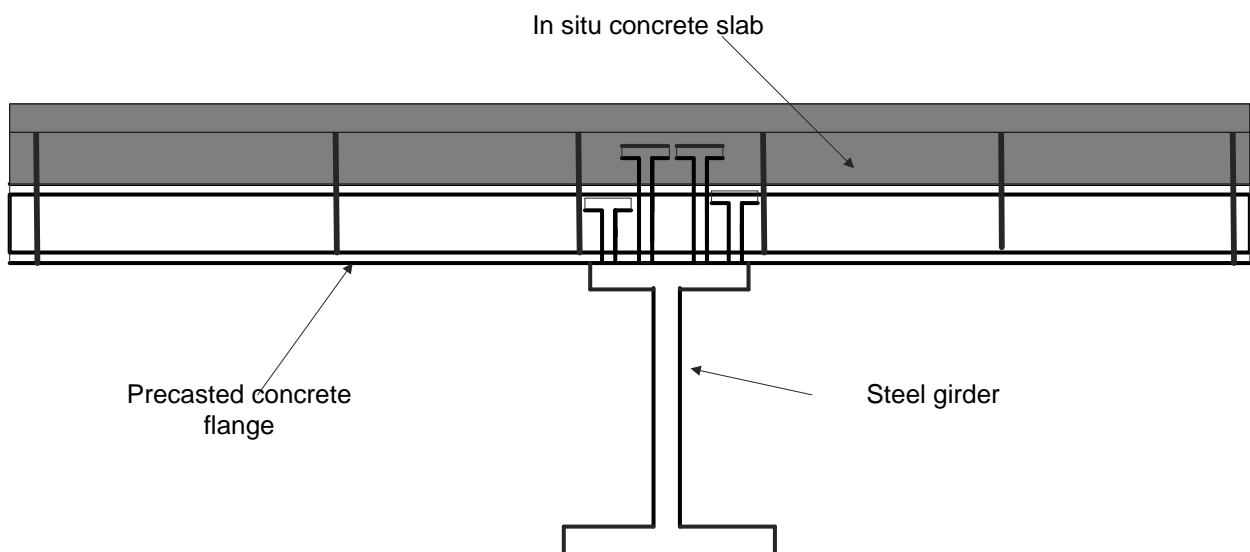


Figure 3-2: Pre-fabricated composite girder [38]

Prefabricated composite construction method has many advantages compared with other construction methods. Prefabricated concrete flange stabilizes the girder during transport and construction phase, while the in-situ concrete slab is casted. The limits involved in handling a construction unit, such as weight and rigidity, are extended. This means that larger spans can be bridged with more slender constructions which are not possible where pre-stressed concrete girders are used. Because the weight of the individual construction unit is reduced while the same rigidity is maintained, the handling limits are extended [22].

Prefabricated units are placed next to each other on previously prepared supports and jointed together at assembly time. Then, the deck's slab and supported cross-bars are carried out. In this way, a monolithic span comes into being although the bigger part of the manufacturing process takes place out of the construction site [22].

#### 3.4.2 *Pre-cast Deck Systems for Steel Concrete Composite Bridges*

Deck units may be either full-depth or partial-depth (requiring to be made composite with an in situ concrete topping), and may be connected at transverse joints using reinforced concrete, dry jointed details or they may be post-tensioned longitudinally to provide continuity .

Depending on where the bridge is being designed and built, the deck units may be reinforced concrete, pre-stressed concrete or un-reinforced concrete, in the case of steel-free decks [22].

In partial-depth construction, which effectively relies on the units to act as participating formwork, the units form an integral composite element in the completed deck. This form of construction requires a partial-depth unit to be placed on the steel superstructure which is then made composite with an in situ concrete topping. The partial-depth units provide one solution to the provision of pre-cast decks but suffer from several inherent disadvantages owing to their thin form.

Full-width deck units span over the main beams, from parapet to parapet. Units that span between adjacent beams only or over a series of beams in a multi-beam bridge are referred to as modular [22].

With full-depth units, the site concreting is considerably reduced in comparison with the partial-depth units as it is limited to the pockets or box-outs and transverse in situ stitches only, where the deck is not excessively wide. There is also a significant saving in deck reinforcement for the temporary lift-in condition as the full depth is provided from the outset, and there is no issue with flexibility of the units as the stiffness is largely unchanged from the erection condition to the in-service condition. In addition, construction traffic can be accommodated on the bridge deck at an earlier stage leading to a shorter construction period.

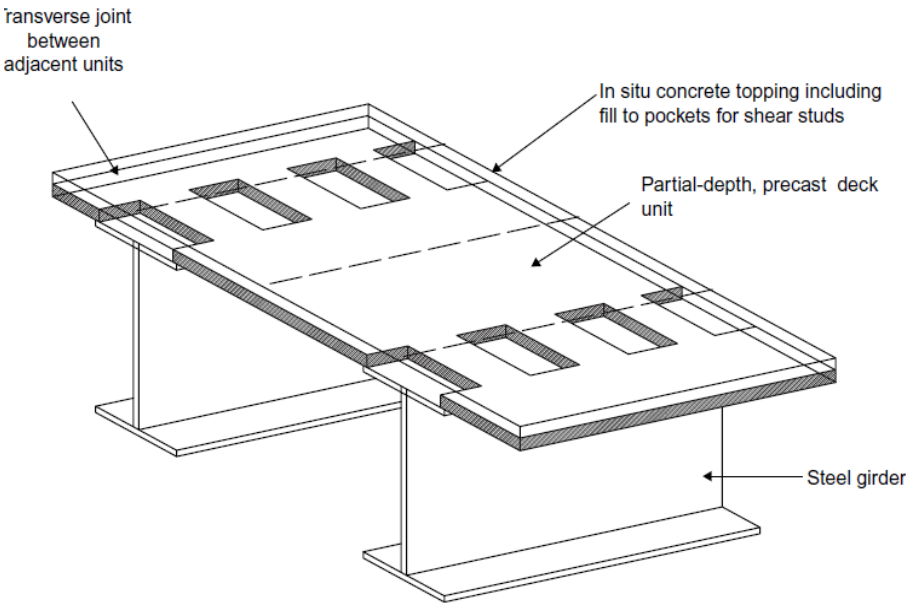


Figure 3-3: Partial depth deck unit construction on steel work [22]

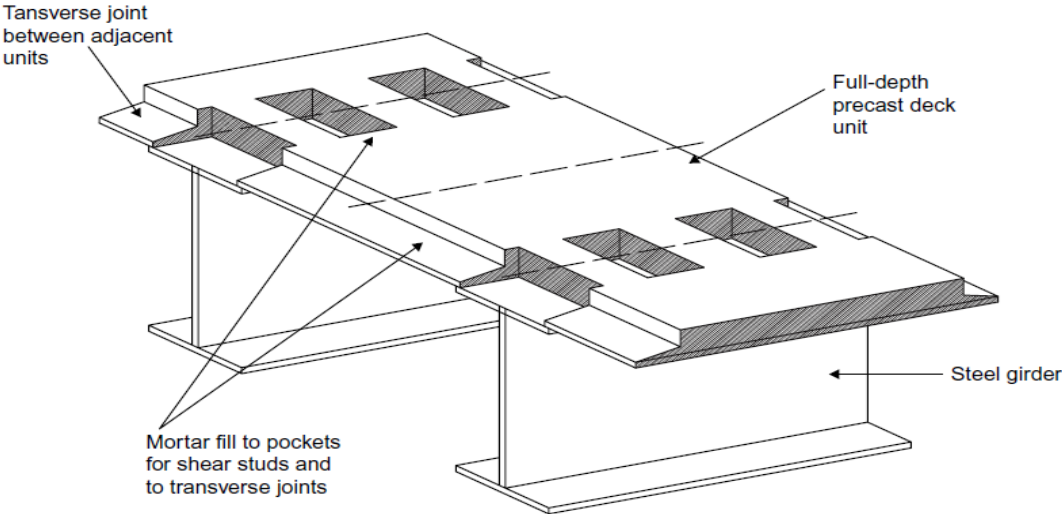


Figure 3-4: Full depth deck unit constructions on steel work [22]

Cost comparisons for the different types of bridges studied showed that the form of construction was not as significant in the overall cost as the cost of delays and disruption to traffic calculated using Highways Agency [39, 40]. These associated traffic costs could easily approach the cost of constructing the replacement bridge. The important issue then is to adopt a form of construction that reduces construction times and avoids the associated traffic delays [22].

#### *3.4.2.1 Advantages of full depth deck unit construction on steel work*

This is used in bridges where access in erecting formwork is difficult. Similarly, their use in bridges to be constructed over existing roads and railways, when the time for occupation of the site is limited would be advantageous, as there is no requirement for extensive false-work. Further, advantages of full-depth construction are the speed of construction and the reduced amount of site concreting required, leading to significant time-saving, and compared to partial-depth units, the inherent stiffness of the units. In addition, provided that they are constructed under factory conditions they should also prove to be durable products and should also have a uniform high quality finish. One possible reason for the reluctance to use such systems may have been the lack of suitable jointing details, particularly in negative moment regions in continuous bridges. However, test programmes and design studies have been undertaken to investigate such details [22].

#### *3.4.2.2 Field of Applications*

This method is mostly used for single or multi-span bridges with spans in range from 15 to 70m. The main scope of application is the execution of viaducts above active road and railway track [22]. In relation to another technology, they are achieving big economic benefits resulting from quick assembly and reduction and simplification of finishing works. Individual design of every prefabricated unit allows readjustment to particular structural and architectural demands. Obtaining very low construction depth also creates the impression that spans are more slender.

#### *3.4.3 Integral Abutment Bridges*

The cost of maintenance is an ever-growing problem for road administrations around the world, and bridges are no exception to the rule. One way to reduce the need for future maintenance, as well as the investment cost, is to make bridges without transition joints [41].

An integral abutment is defined as one that is built integrally with the end of the bridge superstructure and with the supporting piles. The abutment, therefore, forms the end diaphragm of the superstructure and the retaining wall for the approach filling. The supporting piles are restrained against rotation relative to the superstructure, but are free to conform to superstructure length changes by pile flexure [42].

Bridges with integral abutments are becoming increasingly popular around the world. However details of integral bridges differ [43].

Bridges with integral abutments have the potential to better traditional bridges with transition joints. However, an increase of the share of integral abutment bridges not only holds the advantage of reduced construction costs today but also reduces maintenance costs in the future. Moreover since the concept of integral abutment bridges supports the choice of composite instead of concrete bridges, the amount of steel in construction will be notably raised. Furthermore as the majority of bridges are concrete bridges, the use of composite bridges with integral abutments is an efficient way to strengthen the position of structural steel also for bridges with small and medium spans [41].

### **Its Application**

Integral abutment bridges are built without joints where they span from one abutment to other, and sometimes over intermediate piers without any joint in the deck. The absence of joint and bearing results in saving initial costs and reduce maintenance efforts. Apart from economical aspect, it needs periodic maintenance. This results to high trafficking, and environmental and social impacts. The integral abutment type of bridge is used more importantly in areas where construction of piers is difficult.

A comparison between the integral composite bridge and the concrete bridge with expansion joints showed that due to the minimization of maintenance operations, the integral bridge is the most economical solution in terms of the following factors [41]:

- Costs for the agency and
- Costs for the users.

From the environmental perspective the composite bridge with integral abutment had advantage compared to the concrete solution, mainly due to the recycling of steel.

The construction stage has a major influence on the final result of the analysis. The end-of-life stage was important, particularly in the case of recyclable materials [41].

“In principle, bridges with lengths not exceeding  $60m$  and skews not exceeding  $30^\circ$  shall in addition be designed as integral bridges, with abutments connected directly to the bridge deck without movement joints for expansion or contraction of the deck,” [41].

Generally, integral and semi-integral abutments are acceptable for bridges if the following factors are fulfilled [42]:

- a) Length over abutments not exceeding:
  - With concrete superstructure-  $70m$  and
  - With steel superstructure main members-  $55m$

These values may be doubled for a length of superstructure that contains an intermediate temperature movement deck joint.

- b) The abutment piles, and surrounding soil, shall possess adequate flexibility to enable superstructure length changes to occur without structural distress.
- c) An approach settlement slab, at least  $2m$  long, shall be attached to the back face of the abutment, sloped to divert surface water from flowing down the abutment or soil interface. The slab shall be deep enough below the road surface at the end remote from the bridge to distribute soil strains due to length changes without significant surface cracking.

Integral and semi-integral abutments are acceptable for longer bridges provided rational analysis is applied to evaluate the effect of the superstructure length change on the supporting piles. Adequate measures shall also be taken to ensure the bridge approach remains serviceable [42].

Thus, the basic advantages of integral abutment bridges includes [44]:-

- *Construction costs:* - It is often more economical to construct integral abutment bridges instead of bridges with joints and bearings. The construction time can often be reduced, since fewer piles are needed, and the time consuming installation of expansion joints and bearings are eliminated.
- *Maintenance costs:* - Leaking expansion joints is one of the most common reasons to corrosion problems. Expansion joints and bearings need to be maintained, repaired and replaced. Integral bridges have no expansion joints or bearings and are therefore less expensive to maintain.
- *Modification costs:*-It is easier and cheaper to modify an integral bridge, for instance widening.
- *Riding quality:* - No expansion joints, means no bump when a vehicle enter or leave a bridge. This gives a smoother ride for the passengers and the noise level is reduced.

#### 3.4.4 *Composite Bridges with Corrugated Steel Webs*

This type of composite bridge has also many advantages in simplification and cost reduction in composite bridge design and construction tasks. Among the advantages of corrugated steel web bridges [45].

- High buckling resistance reduce requirement of stiffeners
- High transverse bending stiffness which results reduction of cross frames.

#### Connections in Corrugated Steel Webs

In hybrid structures, the connections between the concrete and steel greatly affect on the structural performance and cost. Initially, studs or angle shear connectors were used to connect the concrete slabs and corrugated steel webs. However, unique connections were developed in order to reduce the cost of connections and improve the structural performance. The following are some common types of connections:

i. Embedded Connection

In an embedded connection, the corrugated steel webs are directly embedded in the concrete slabs. Since the steel flange plates fitted with, shear connectors are not required and this is the most economical connection method. According to its characteristics, fatigue durability is high and construction tolerances are easy to absorb, [12].

ii. Perfobond Strip Connection

A perfobond strip connection is a connection using a plate with holes as shear connector. Compared with stud connectors, the stiffness of shear connection is higher. This connection is comparatively economical, because, welding of the shear connector is simpler. Nevertheless, the combination between plate and studs is frequently applied since the plate cannot solely resist transverse bending moments, [12].

iii. Twin-Perfobond Strip Connection

Different from perfobond strip connection, a twin-perfobond strip connection has two rows of perforated steel plates; therefore, the use of studs is not required since the transverse bending moments can be resisted with two rows of perforated steel plates. The load resistance and fatigue durability of twin-perfobond strip connection was confirmed by many experiments. [12]

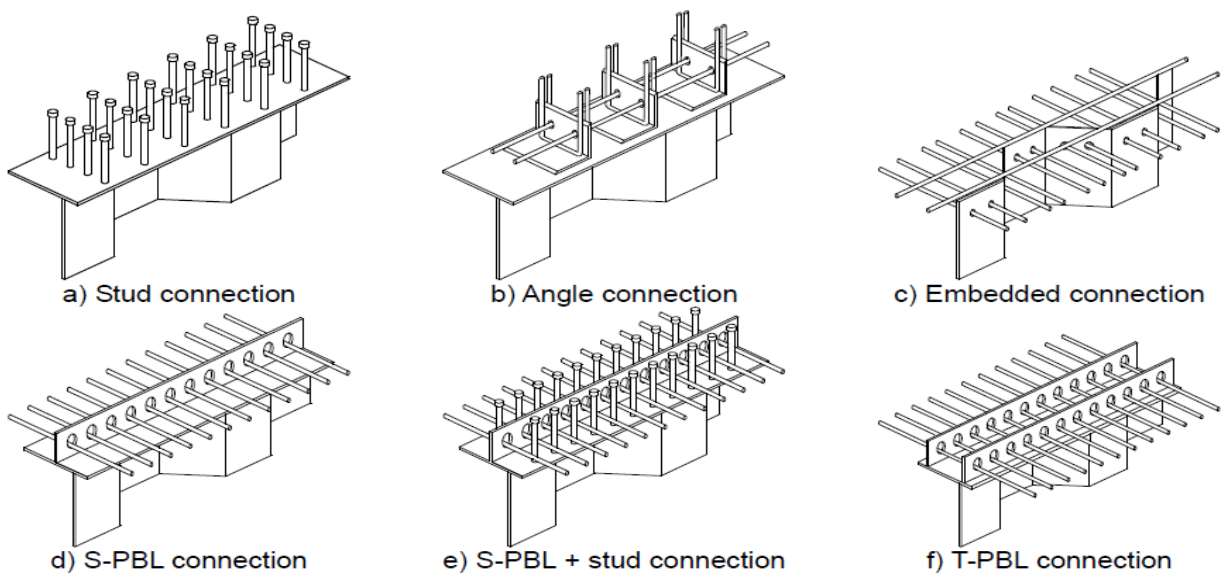


Figure 3-5: Methods of connecting slab and web [46]

### 3.5 Construction Practices and Economic Consideration

This subsection gives details about factors on economical construction practices, and international and Ethiopia construction experiences.

#### 3.5.1 Influence of the Construction Method

Composite sections respond to load differently under different construction method. Two construction methods are usually applied [47]:

- i. Steel beams are supported at close intervals during the pouring of slab and the shores remain in place until the concrete has attained a reasonable amount of its 28-day strength. Then, the composite section is available to carry all loads upon removal of the shore. However, the hardened concrete will tend to creep under the sustained action of long-term loads. The stresses due to these loads are thereby increased over their nominal value.
- ii. Forms carrying wet concrete are supported directly by steel beam. The steel section alone must carry the loads imposed at this moment. The composite cross-section is only available to carry all subsequently imposed loads after the concrete has fully hardened. The ultimate load that can be carried by a given cross-section is independent of the method of construction, regardless of differences in the two construction methods that have previously been discussed. Therefore, total load is assumed to be steel section under the dead load of the wet concrete in addition to formwork must be checked.

In order to prevent the bottom flange of an un-shored steel beam during construction from yielding under the specified loads, which is a condition that will influence deflections, the stresses in the tension flange of the steel section are not to exceed the yield strength of steel  $f_y$ , prior to hardening of the concrete. The loads to be considered are those applied both prior to hardening of the concrete and those applied after. This is a serviceability problem; therefore, specified loads are considered in analysis. The following equation expressed the above relationship [47]:

$$\frac{M1}{S_s} + \frac{M2}{S_t} \dots\dots\dots 3.1$$

Where

$M_1$  = Moment caused by the specified loads that act on the member prior to attainment of 75 % of the required concrete strength.

$M_2$  = Moment caused by the specified loads that act on the member subsequent to attainment of 75% of the required concrete strength.

$S_s$  = Elastic section modulus, referred to the bottom flange, of the steel section alone.

$S_t$  = Elastic section modulus, referred to the bottom flange, of the composite steel-concrete section.

To calculate the section modulus of the composite section, the area of the concrete slab must be transformed into an equivalent area of steel in the ratio of the modules of elasticity of the two materials. This modular ratio, [47].

$$\frac{E_s}{E_c} \dots\dots\dots 3.2$$

is usually prescribed in the applicable building code. It is customary to apply the reduction to the slab width, rather than to its thickness or to some proportion of each.

### 3.5.2 *The International Practice*

New materials, new design concepts and better understanding of the structural reliability and life-cycle costs make the next an exciting time for bridge technology.

Different construction practices for composite bridges which are proved to be extremely economical, easily constructible and more durable are being used throughout the world. And also there are many innovative ideas practiced which are leading bridge engineering to the highest peak. The design manuals are also addressing that these economical issues are valid for bridges of steel, concrete and composite structures as well.

There are different construction practices to meet the advantages of steel concrete composite technology. Construction sequence is key points which need special consideration during construction.

The sequence that most commonly needs to be evaluated for a composite bridge are completion of the substructures, up to bearing level, erection of the structural steelwork, provision of

formwork and casting the deck slab, and finally completion of the surfacing and fixtures such as barriers and drainage [7]. These points are explained as follow:

### I. Girder erection

Girder lengths are usually chosen to suit transportation. Strength verification at this stage is unlikely to require detailed evaluation but stability and buckling resistance do require careful consideration, particularly before bracing or cross girders are fully installed.

### II. Bracing

Bracing of the steelwork in the bare steel and partly complete stages is a key to the effective performance of the main girders. Several bracing schemes may need to be evaluated.

### III. Slab construction

Although deck slabs have traditionally been cast on temporary timber false work, the use of permanent formwork notably partial depth pre-cast decking that forms part of the final slab is now a day very common. Timber false work is often supported off the bottom flanges of the girders; pre-cast permanent formwork sits on the top flanges and thus needs to be considered as a destabilizing load. Whichever type of formwork, the weight at the wet concrete stage imposes quite high stresses in the top flanges of the girders. Their strength and stability at this stage require a detailed evaluation of the progressive changes in structural behavior as load is added.

The weight of the concrete cantilevers needs particular attention, because of the moment (about the longitudinal axis) that is imposed on the outer girder.

In the composite construction practice, countries have different construction approaches and have been developing innovative methods to take advantage of it especially for bridge construction. The following subsection explains practices of some countries.

#### *3.5.2.1 Composite construction in Germany*

The number of composite bridges has significantly increased in Germany within the last 15 years. The main reason for this development is the high durability and robustness of this type of

bridge structures and the development of new types of cross-sections and erection methods for bridges with small and medium spans as well as for superstructures [5].

The following two types are typical examples for new and innovative types of composite bridges in Germany:

*a) Composite bridges for freeways*

For freeways with two or three traffic lanes in each direction, they can be moved on one superstructure only if the other superstructure needs to be closed for repair and maintenance reasons[5].

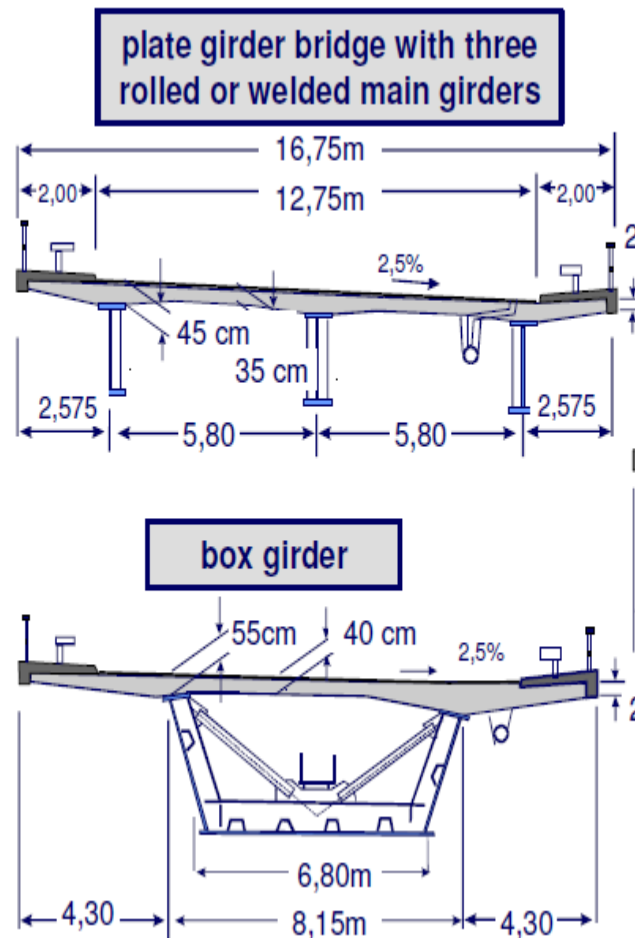


Figure 3-6: Common Cross-Sections for Typical Composite Bridges for Freeway Bridges [5]

### *b) Composite box girders with wide concrete decks*

Regarding environmental requirements and aesthetics, In Germany Composite box girders with wide concrete decks are used instead of traditional separate bridge decks and piers for each traffic direction. A typical cross-section is shown below [5].

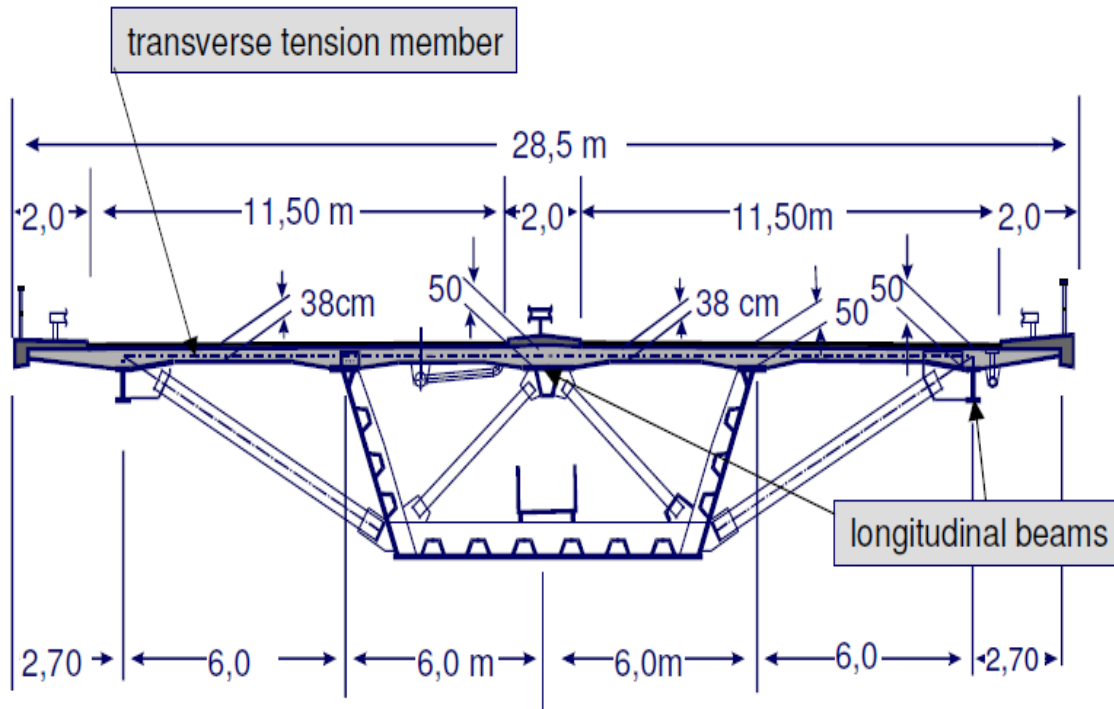


Figure 3-7: Typical cross-section of a box girder with wide concrete decks [5]

#### *3.5.2.2 Composite Construction in China*

The technology is over growing in china. Rather than the adapted technology of composite construction they use more advanced new techniques.

Steel-concrete composite structure with overlap slab is a new-type composite structure in china. Its concrete flange is combined with pre-cast slab and in-situ concrete. The concrete and steel beams are tied together by using shear connectors. In this kind of structure, the advantages of steel and concrete material can be exerted respectively [48].

Besides the characteristics of composite structure with in-situ concrete, it also has some other advantages such as saving form erecting processes and mould plates, quickening construction speed, decreasing of in-situ concrete and good comprehensive benefits [48].

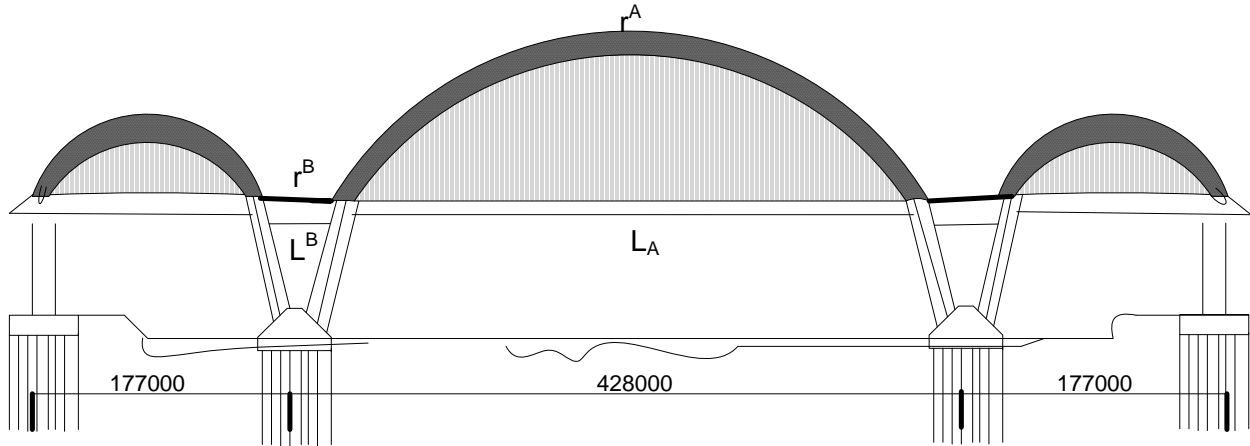


Figure 3-8: Overall layout of Overlap Slab [48]

There are lots of composite construction methods in China. Here, we explain two of them:

***Construction method 1:***

The construction procedure is as follows.

The pre-cast slabs are first supported on the steel floor beam, and the construction joints are filled with concrete. The structure system is changed from simply support to continuous support after the concrete in construction joints reaches design strength, then in-situ concrete is cast. After the in-situ concrete reaches its design strength, the deck pavement will be done and the bridge can be used [48].

**Construction method 2:**

For quickening construction plan, the top in-situ concrete and construction joint are cast at the same time. The pre-cast slabs are first supported on the steel floor beam, then the in-situ concrete and construction joints are cast at the same time. After the in-situ concrete and construction joint reaches their design strength, the deck pavement is done and the bridge can be used [48].

### 3.5.2.3 Composite Construction in Japan

A new steel-concrete composite bridge for Railway is developed in Japan which is *Pipe girders filled with concrete*.

This Railway Bridge system has been developed using steel pipes as the main girders. Concrete is filled inside the pipe girders to increase the bending strength by preventing local buckling of steel plates in compression. The concrete filled steel pipe girder is also expected to reduce noise and vibration levels induced by trains, which are the main limitations of steel girders for railway steel bridges.

The pipe girders are filled with different concrete materials depending on the span wise positions. The pipe girder near the intermediate supports is filled with concrete using light aggregates, where as it is filled with air mortar around the span-center to reduce the self-weight. Air mortar is the light mortar mixed with air bubbles in it and the density is as low as about 1.0 [49].

The girder is expected to work as a composite beam in the positive bending moment zone of the span center. The concrete slabs are supported on the pipe girders and connected with studs welded on top of the pipe girders. In the negative bending moment zone around the intermediate supports, the concrete slab becomes tensile and only the reinforcing bars resist tensile forces, [49].

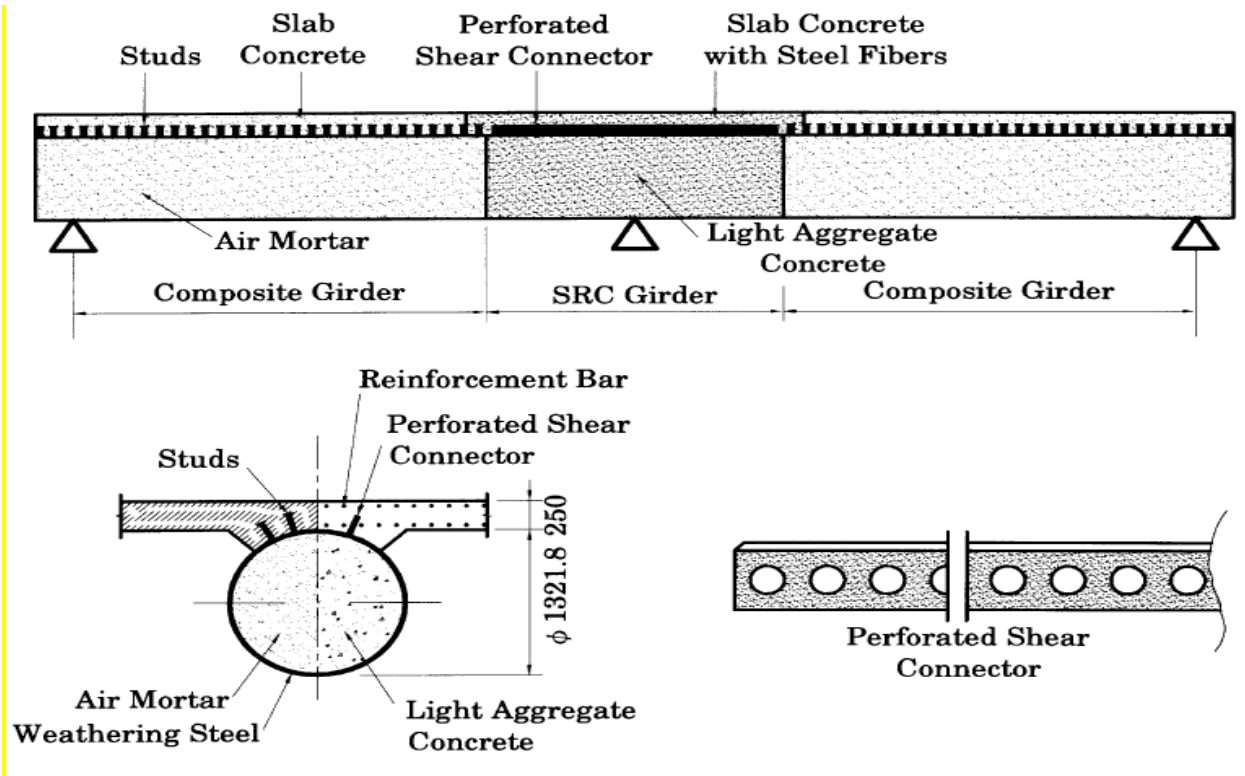


Figure 3-9: concept of composite girder [49]

### 3.5.3 The Ethiopian Practice

Steel-concrete composite construction has gained wide acceptance world wide as an alternative to reinforced concrete construction. However, this system is a relatively new concept for the construction industry in Ethiopia.

Coverage of composite bridges in the country is rather poor. Most of the bridges across the country are mainly dominated by reinforced concrete deck bridges.

In order to use the advantage of composite construction an emphasis should be given in design and construction methods of practices for composite construction and also attention must be given about the information and advancement of the technology.

## 4 DETAIL DESIGN CONSIDERATIONS

### 4.1 Basic Design Concept

Composite construction has been rapidly developing and adopting in countries possessing the necessary standards and design guides. The general philosophy adopted from Eurocodes is to consider the loads and forces applied to a structure using factors. The values of these load factors depend on the nature, and variation with time for a particular type of load. Each member within a structure, and the structure as a whole, must be checked for all potential combinations of loads. This leads to applicable bridge types which are feasible and can be compared in terms of cost aesthetics and durability.

In composite bridge basic element design focuses mainly on [50]

- i. Verification of cross-section
- ii. Justification of the Serviceability Limit States (SLS)
- iii. Stresses control at SLS
- iv. Control of cracking and
- v. Connection at the steel–concrete interface

The following factors should be also considered by the designer during the design of a bridge [27, 51].

- ***Superstructure Steelwork:*** - Use the least overall main girder weight, if possible, independent of the number of girders, with minimum numbers of braces. This is found to be the best option to reduce handling and erection cost irrespective of the coating cost. Ladder deck bridges also provide further cost savings in the substructure.

- **Slab Construction:** - Partial depth pre-cast solutions provide a cost-effective solution when all factors and constraints are taken into account. However, in situ concrete on steel decking as permanent formwork, with the appropriate corrosion protection system may provide additional cost savings.
- **Piers:-** The number of piers and size of pier, cross head girders can equate to a significant percentage of the total cost of the structure. Utilize the increased composite high weight to strength ratio to maximize the bridge span to reduce the number and size of the piers.
- **Abutments:-** Replace the abutments with concrete segmental retaining walls and concrete piers or steel piles [7].

There are factors which needs especial attention at the time of bridge design. Some of these factors are explained in the coming subsection [7].

#### 4.1.1 *Load-Carrying Capacity of Composite Steel Concrete Bridges*

It is an important factor that affects the overall bridge behaviors. Stiffness and capacity of steel-concrete composite beams is also the main point to be checked through analytical means. The effects of secondary elements such as barriers, sidewalks and diaphragms increase the load-carrying capacity of girder bridges and also basic points to get reliable design with an aim to evaluate the potential benefit of secondary elements in the system reliability of composite bridges.

#### 4.1.2 *Composite-Action*

Composite action is developed when two load carrying structural members are integrally connected and deflect as a single unit.

Steel-concrete composite bridges utilize the tensile strength of steel in the main girder and welded compressive strength of concrete in the slab to provide a cost-effective solution over a wide range of spans. The steel and concrete elements of a composite bridge are connected via shear connectors that are on the top of flange steel girder and embedded in the concrete deck. The composite action is achieved through the longitudinal shear force transferred by the shear connectors which increase the bending resistance significantly compared to the non-composite

beam. The bending resistance of the combined materials increases very much and make possible of larger spans.

The extent to which composite action is developed depends on the provision made to ensure a single linear strain from the top of the concrete slab to the bottom of the steel section. The interaction between steel girder and concrete deck slab was investigated considering the effect of partial and full interaction, developed from the horizontal shear force at the interface between the steel beam and concrete slab, on the composite bridge behavior.

#### 4.1.3 *Effect of Concrete Flange Width on Composite Bridge Behavior*

In steel-concrete composite bridges, different types of steel can be used in girders, such as carbon steel, high strength low-alloy steel, heat-treated low-alloy steel and high-strength heat-treated alloy steel [10].

The designs is mainly based on steel properties such as shape of the girder, thickness, yield stress of steel ( $F_y$ ), tensile strength of steel ( $F_u$ ), and modulus of elasticity ( $E_s$ ). The reinforced concrete bridge deck compressive strength ( $f_c'$ ) is also affect the design in terms of reinforcement properties in addition to concrete modulus of elasticity  $E_c$ , [10]. The transformed area of concrete is usually used to calculate the composite section properties using the modular ratio ( $n = E_s/E_c$ ). The concrete modulus of elasticity  $E_c$  can also be calculated according to the code requirements or standard hand books. The width of top flanges comprised the concrete slab and top steel beam flange known as effective width depends on the equivalent area carrying the compression force. In practice, simplifications of the effective width for design purposes are given by several codes and can also be performed through experimental tests.

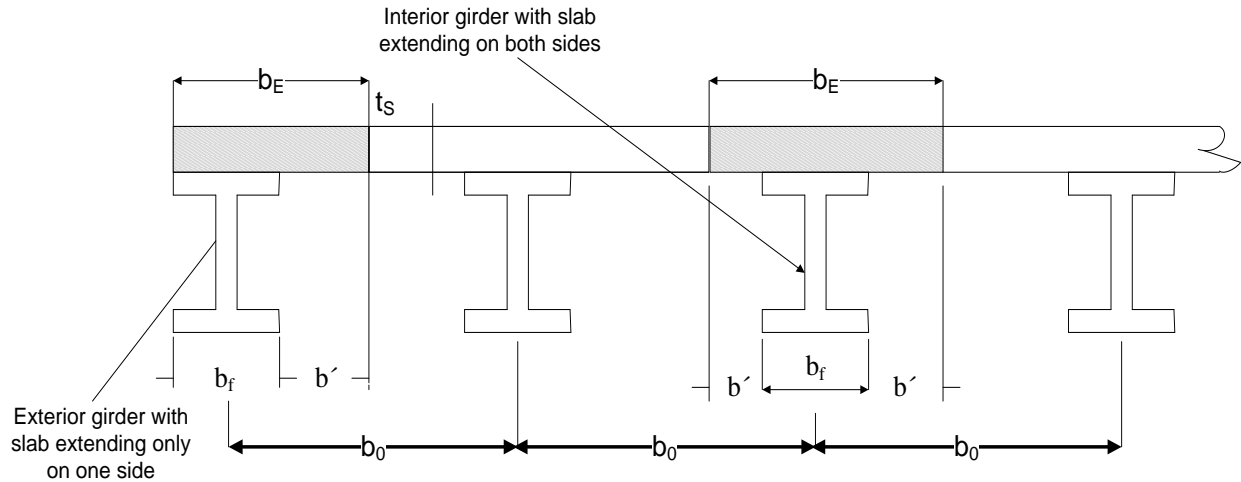


Figure 4-1 :Effective width of steel concrete composite beam [10]

## 4.2 Section Properties

Cross-section classification has been introduced into several design codes identifying the conditions for global analysis of structures. To determine the effects of actions and criteria to be used for ultimate limit state checks of cross-sections and members. Global analysis of structures involves either elastic global analysis, or elastic global analysis with specified limits of moment redistribution, or plastic global analysis. Moment resistance of a critical cross-section is then determined on either a plastic stress-block or elastic basis.

In EBCS-4 four classes of cross-sections are defined.

### ✚ Class 1.

Plastic cross-sections are those which can form a plastic hinge with the rotation capacity required for plastic analysis.

### ✚ Class 2.

Compact cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity.

➤ **Class 3.**

Semi-compact cross-sections are those in which the calculated stress in the extreme compression fiber of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.

➤ **Class 4.**

Slender cross-sections are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression resistance.

Main points to give attention in composite section are:

- I. Buckling behavior of rolled shapes in composite sections in negative moment regions (local buckling in steel webs; global instability with lateral torsional buckling),
- II. Application of high strength steel in bridge building,
- III. Structural behavior of the deck when using prefabricated concrete elements as composite formwork,
- IV. Demonstration of construction principles

#### 4.2.1 *Elastic Analysis*

In composite section analysis, the theory of the transformed sections can be used, i.e., the composite section is replaced by an equivalent steel section, the flange has a breadth of equal to  $b_{eff} = n$ . The translational equilibrium of the section requires the centroidal axis to be coincident with the neutral axis. Therefore, the position of the neutral axis can be determined by imposing that the first moment of the effective area of the cross-section is equal to zero.

In the case of a solid concrete slab, and if the elastic neutral axis lies in the slab

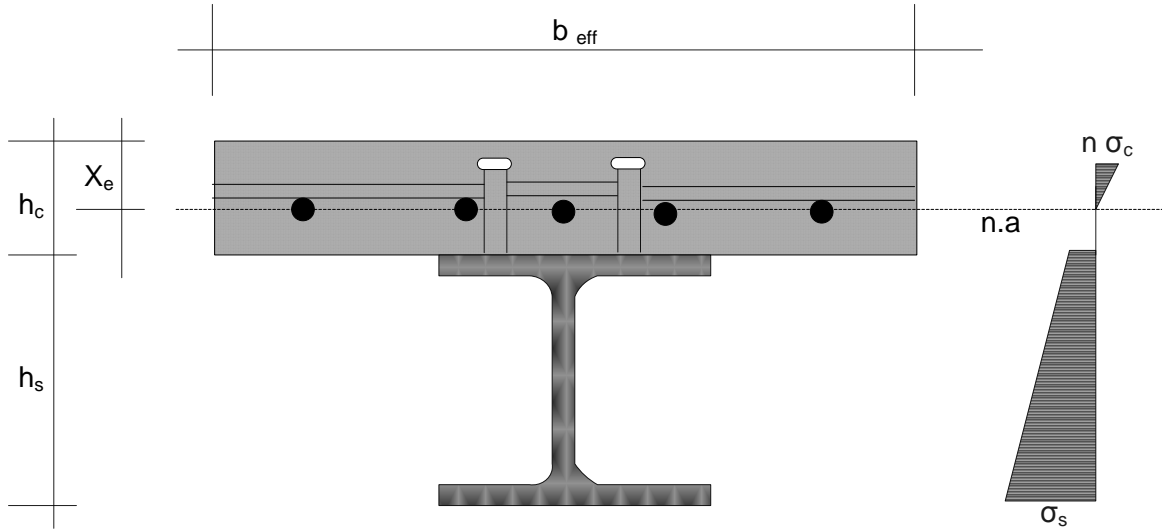


Figure 4-2: Elastic stress distribution with neutral axis in slab [52]

$$S = \frac{1}{n} \frac{b_{eff} \times x_e^2}{2} - A_s \left( \frac{h_s}{2} + h_c - x_e \right) = 0$$

Where:-

$x_e$  : - is the distance of elastic neutral axis to the top fiber of the concrete slab is calculated, the second moment of area of the transformed cross-section can be evaluated.

$$I = \frac{1}{n} \frac{b_{eff} * x_e^3}{3} + I_s - A_s \left( \frac{h_s}{2} + h_c - x_e \right)^2$$

If the elastic neutral axis lies in the steel profile

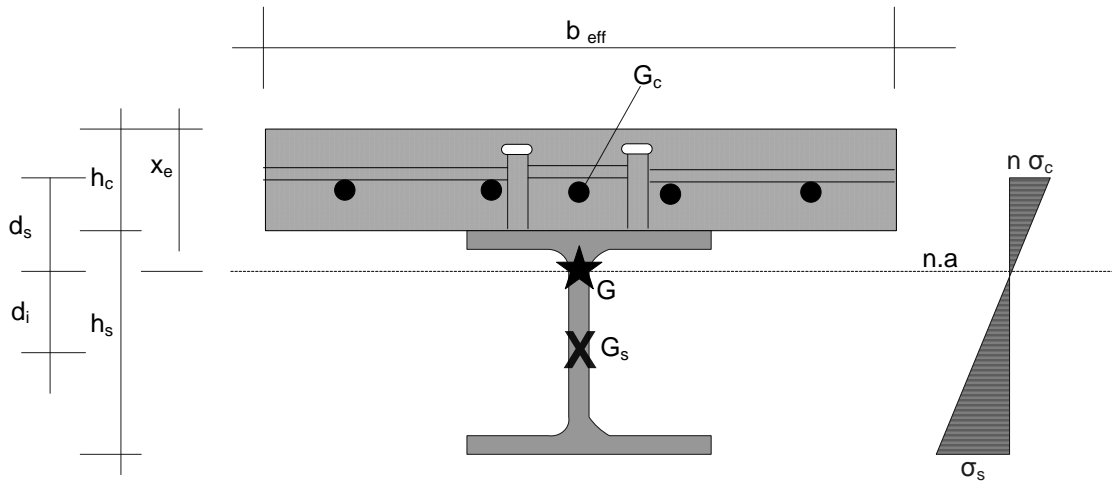


Figure 4-3: Elastic stress distribution with neutral axis in steel beam [52]

$$x_e = d_s + \frac{h_c}{3}$$

Where:-

$$d_s = \frac{A_s}{A_s + b_{eff} * h_c/2} * \frac{h_s + h_c}{2}$$

Where:-

$d_s$  :- is the distance between the centroid of the slab and the centroid of the transformed section;

$$I = I_s + \frac{1}{n} \frac{b_{eff} * h^3}{12} + A * \left( \frac{h_s + h_c}{4} \right)^2$$

$$A = \frac{A_s * b_{eff} * h_c/n}{A_s + b_{eff} * h_c/n}$$

When the neutral axis depth and the second moment of area of the composite section are known, the maximum stress of concrete in compression and structural steel in tension and bending moment  $M$  are evaluated by:

$$\sigma_c = \frac{1}{n} \frac{M}{I} * x_e$$

$$\sigma_c = \frac{M}{I} (h_s + h_c - x_e)$$

The elastic limit moment can be determined as the lowest of the moments associated with the attainment of the elastic limit condition, and obtained by making the maximum stress equal to the design limit stress.

$$M_{el} = \text{Minimum of } \left\{ \begin{array}{l} f_{cd} * \frac{n_l}{x_e} \\ f_{yd} * \frac{I}{h_s + h_c} \end{array} \right\}$$

$$M_{el} = \text{Minimum of } \left\{ \begin{array}{l} f_{cd} * \frac{n_l}{x_e} \\ f_{yd} * \frac{I}{h_s + h_c} \end{array} \right\}$$

The stress check is then indirectly satisfied if it results:-  $M \leq M_{el}$

#### 4.2.2 Plastic Analysis

Refined non-linear analysis of the composite beam can be carried out accounting for yielding of the steel section and inelasticity of the concrete slab. However, the stress state typical of composite beams under sagging moments usually permits the plastic moment of the composite section to be achieved.

In most cases the plastic neutral axis lies in the slab and whole steel section is in tension. Therefore, the plastic method of analysis is applicable to most simple supported composite beams. This tool has more advantages on practical scenario in which it is the non-linear design method for these members.

The plastic moment can be computed by application of the rectangular stress block theory. Moreover, the concrete may be assumed in composite beams to be stressed uniformly over the full depth  $x_{pl}$  of the compression side of the plastic neutral axis, while for the reinforced concrete sections usually the stress block depth is limited to  $0.8 x_{pl}$ . The evaluation of the plastic moment requires calculation of the following quantities:

If  $F_c \max > F_s \max$  then

$$F_c = F_s = A_s * f_{ys}$$

$$0.85b_{eff} * x_{pl}f_{cd} = A_s * f_{yd}$$

$$x_{pl} = \frac{A_s f_{yd}}{0.85 b_{eff} f_{cd}}$$

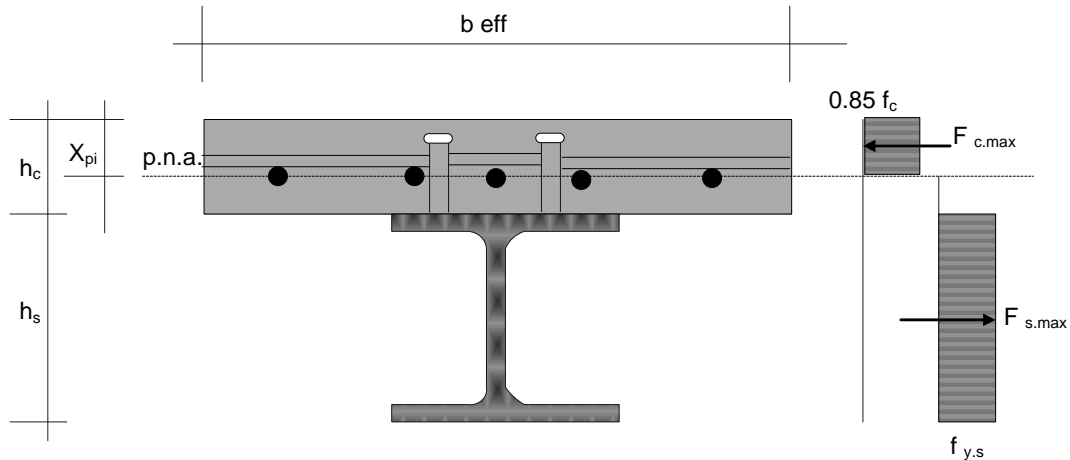


Figure 4-4: Plastic stress distribution with neutral axis in slab [52]

The internal bending moment lever arm is then evaluated by the following expression:

$$h^* = \frac{h_s}{12} + h_c - \frac{x_{pl}}{2}$$

The plastic moment can then be determined as:

$$M_{pl} = A_s f_{yd} \cdot h^*$$

If  $F_s \max > F_c \max$ , the neutral axis lies in the steel profile, then

$$F_c \max < F_s \max \quad \text{then} \quad F_c = F_s = 0.85 b_{eff} h_c f_c$$

Two different cases can take place;

***In the first case:***

$$F_c = F_w = d * t_w * f_{ys}$$

Where:

$t_w$  = is the web thickness

$d$  = is the clear distance between the flanges

$$M_{pl} = F_s max * \frac{h_s}{2} + F_c \frac{h_c}{2}$$

**In the second case:**

$$F_c < F_w = d * t_w * f_{ys}$$

$$M_{pl} = F_s max * \frac{h_s}{2} + F_c \frac{h_c}{2}$$

$$M_{pl} = M_{pls} + F_c * \frac{h_s + h_c}{2} - \frac{F_c^2}{4 * t_w * f_{ys}}$$

Where:

$M_{pls}$  = plastic moment of the steel profile

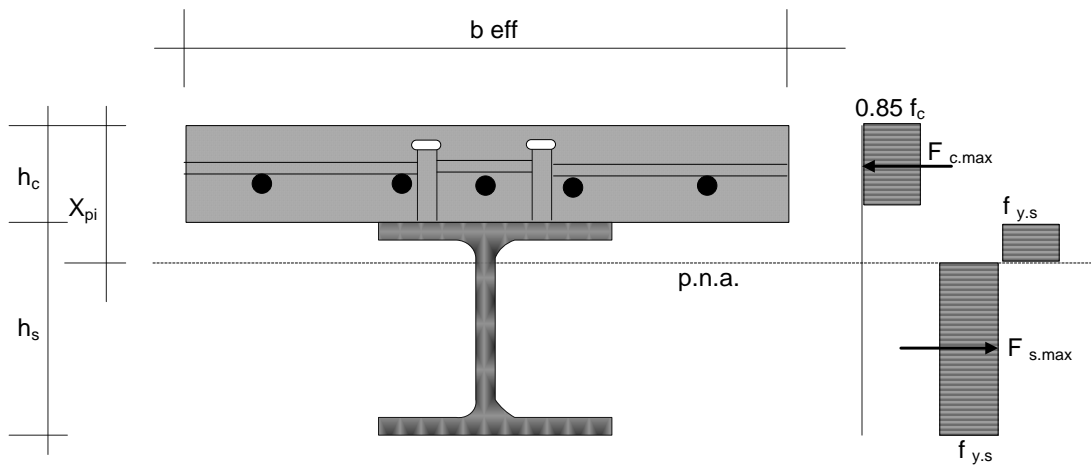


Figure 4-5: Plastic stress distribution with neutral axis in steel beam [52]

### 4.3 Connection Consideration

To ensure a full composite action, shear connectors must be provided at the interface between the concrete slab and structural steel to resist interface shear to enhance the composite performance of steel-concrete composite bridges. It is basic concern to investigate types of connectors used in composite construction. Generally, these connectors are either rigid or flexible depends on the distribution of shear forces and functions between strength and deformations.

The strength of shear connectors can be established from tests to ensure transmission of the forces at the slab-steel section interface. In most commonly encountered situations, values based on test results are given in Standards.

To determine the required number of shear connectors, the force to be transferred and the aforesaid resistance per connector are evaluated. Connectors can be uniformly distributed between the point of maximum moment and an adjacent zero moment location. The flexibility of the shear connectors permits the usage of uniform spacing. Certain restrictions are applied on the number of shear connectors when concentrated loads are present, [10].

#### 4.3.1 *Strength Calculation*

Deflection of a member acting under working loads; shear and flexural capacities are calculated to evaluate the strength of a beam which is used to make only one material. These requirements also apply to composite section, and the principles of strength calculation are based on a steel beam and solid concrete slab. In addition, a designer must ensure that these two components act as a single unit. In the case of steel beams, it is assumed that the beam web carried all of the vertical shear force and same approach is applied to composite beams.

The flexural capacity of a composite beam is evaluated based on the concrete which does not resist tension.

Two cases will be considered [52]:

- i. Neutral axis falls within the concrete slab and
- ii. Neutral axis falls within the steel section.

This is determined by trial to determine which case is applied.

#### **i. Neutral axis falls within the concrete slab**

The stress conditions for a cross-section in which the neutral axis lies in the slab are shown in the Figure below

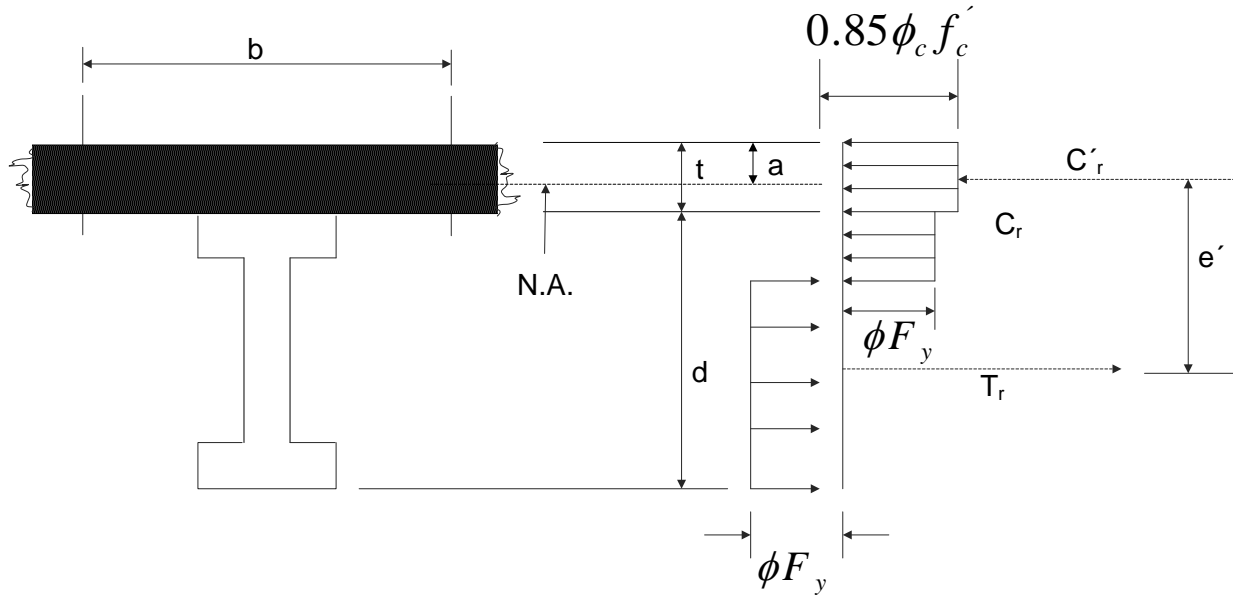


Figure 4-6: Composite Cross Section with Neutral Axis in Slab [52]

## ii. Neutral Axis in the Steel Section

If the neutral axis lies in the steel section, the full depth of the concrete slab is in compression and the steel section is fully yielded in compression above the neutral axis and fully yielded in tension below the neutral axis. This condition is shown in the Figure below.

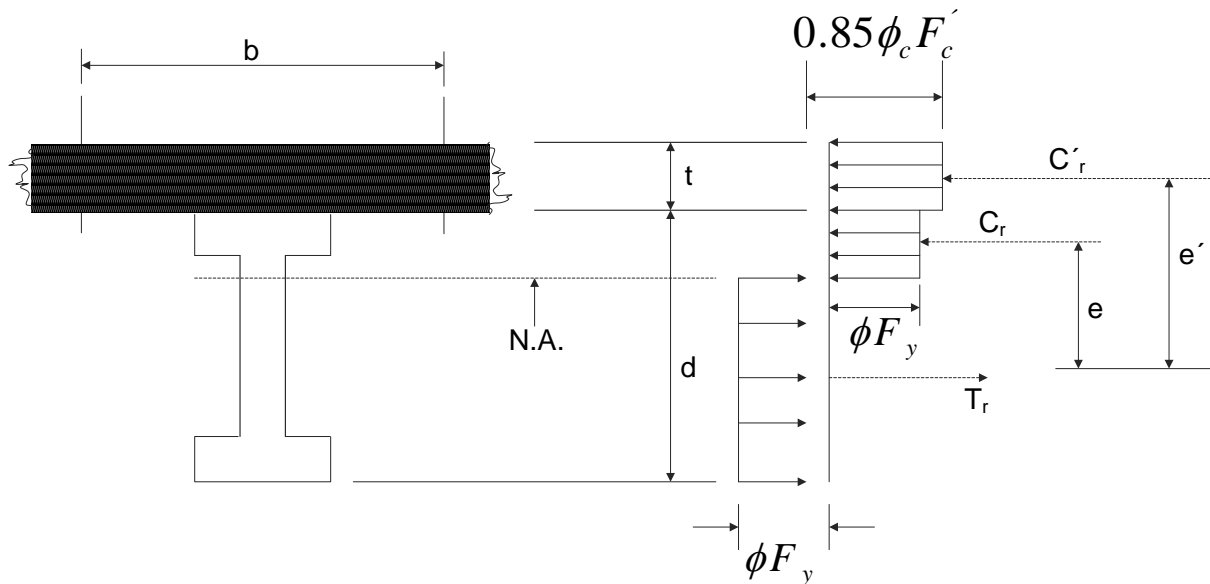


Figure 4-7: Composite Cross Sections with Neutral Axis in the Steel Section [52]

#### 4.3.2 *New Steel-Concrete Shear Connection for Composite Bridges*

Nowadays, the steel-concrete connections used in composite bridges with pre-cast slabs groups of headed studs connected to the slab when concreting the pockets in the slab on site are not well adapted with a short construction duration. Indeed, this kind of connection tends to slow down the assembly work because of the numerous small quantities of concrete that need to be poured on site to fill the pockets. Moreover, cracks may develop in the corners of the pockets, which tend to increase the risk of degradation by corrosion of the slab reinforcement [53].

The duration of on-site work has a significant influence when building new bridge structures, or widening or replacing existing bridges, not only on the costs, but also on the potentially harmful effects of the construction work such as noise, pollution, traffic jams, and deviations. Thus, it is of interest to design structures in such a way as to minimize the construction time. Steel-concrete composite bridges are ideal for this purpose as the steel beams may be welded and the concrete slab pre-cast in the factory, leaving only the erection and assembly work to be performed on site. Consequently, on site, the connection between the pre-cast slabs and the steel structures must be realized as fast and effective way as possible, this requirement leads to the need for developing new types of connection. Connections by adherence, whose resistance is due to friction between the various interfaces, constitute a very promising solution to this problem.

Figure 4.8, illustrates an example of a connection by adherence. An embossed steel plate is first welded longitudinally onto the upper flange of the steel beam. The upper flange may then be coated with a bonding layer which consists of an epoxy resin and coarse sand. The concrete deck elements are pre-cast in the factory with a longitudinal rib in the lower face. The concrete surface of this U-shaped rib is roughened by the use of a mechanical (water-jetting, sandblasting) or chemical (retarder) technique. On site, the pre-cast deck elements are laid on the steel girders. The transverse joints of the slab are glued together and the deck is then longitudinally pre-stressed. The gap between the steel girder and the concrete deck elements is finally injected with a cement paste from one end of the bridge, in a manner similar to that of a post-tensioning duct. When hardening the cement paste makes the static link between the slab (rough concrete) and the beam (embossed steel plate, bonding layer) [53].

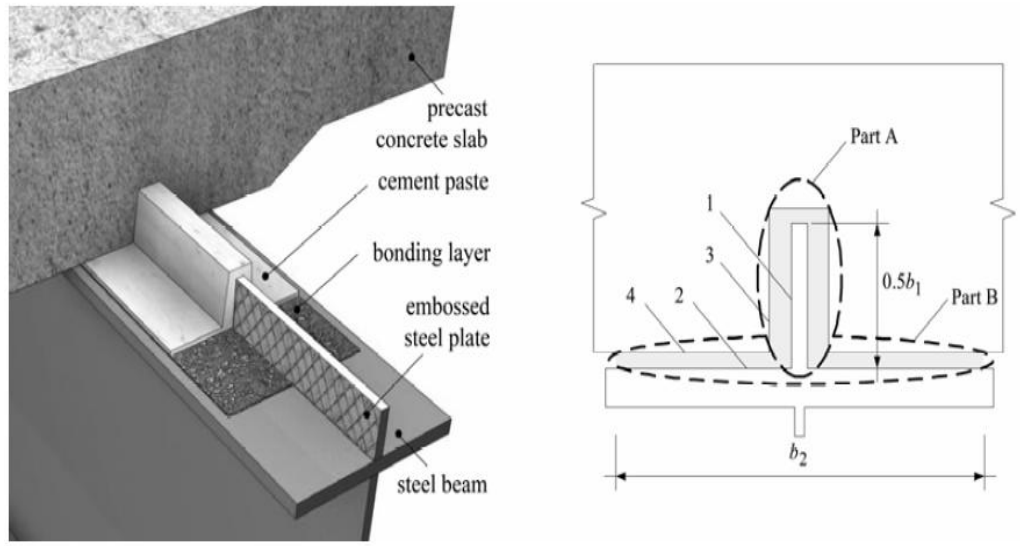


Figure 4-8: Composite cross Sections with Neutral Axis in the Steel Section [53]

### **Simplified calculation method for adherence connection types**

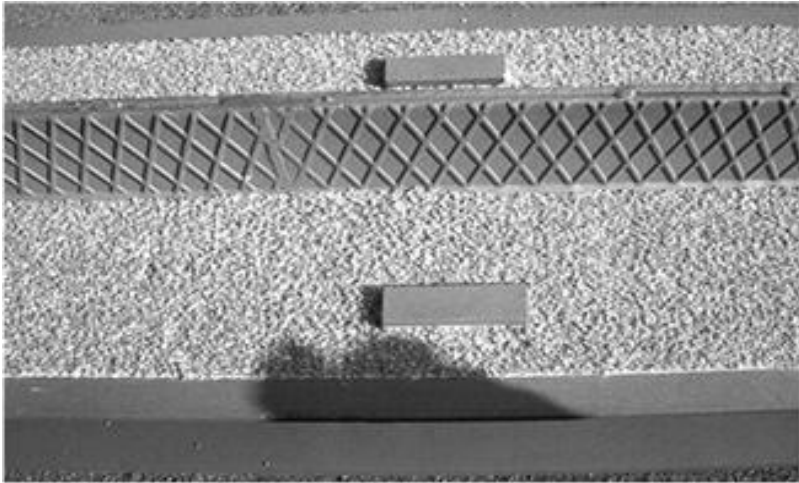
The design respect certain hypotheses of the mechanical model

- (1) In case where a bonding layer is used, its width,  $b_2$  must be larger than  $b_1$  (double height of the connector).
- (2) A transverse reinforcement must be placed as low as possible in the concrete slab, i.e. as close as possible from the top of the connector, i.e. from the top of the U-shaped rib in the slab. This has a very favorable effect on the stiffness of the slab.
- (3) Stirrups must be placed around the rib in the concrete slab. This prevents the failure surface to propagate inside the concrete slab instead of following the interfaces. The stirrups can be placed directly against the formwork without a concrete cover.
- (4) The design of possible transverse pre-stressing cables and the amount of pre-stressing should be chosen so that the lower layer of the concrete slab remains compressed under long-term loads which are applied after the connection has been injected [53].

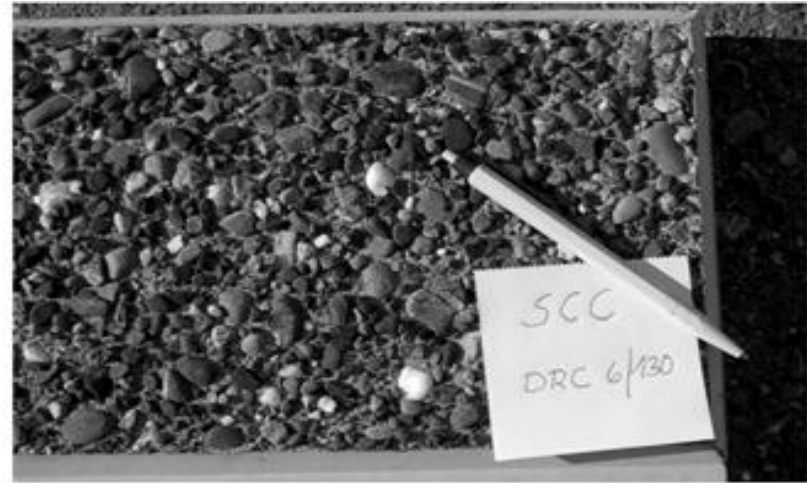
**The materials must meet the following requirements.**

- The cement paste used for the injection must have high compressive strength,
- The connector must be made of two embossed steel plates BRI 8/10 S235 welded back to back and continuously welded on the upper flange of the steel beam.

- The embossments should be oriented as illustrated on Figure 4.9 a, i.e. perpendicular to the shear force.
  - The concrete surface in contact with the cement paste must be roughened;
  - The depth of the roughness must be of at least 6 mm. Where a bonding layer is used,
  - The steel flange must be sandblasted.
  - No other corrosion protection should be applied to the upper face of the flange.
  - The sand must be of diameter 2 to 3.2 mm and applied on the fresh epoxy resin.
- Finally,
- The injection of the cement paste must be controlled, for example by using control pipes and manometer.



(a) Embossed plate and bonding layer



(b) Rough concrete

Figure 4-9: Surfaces prior to injection [53]

#### 4.4 Design Flow chart

Once the final bridge configuration has been chosen and the preliminary design completed the following chart outline the overall design procedure that should be undertaken when designing a steel-concrete composite bridge.

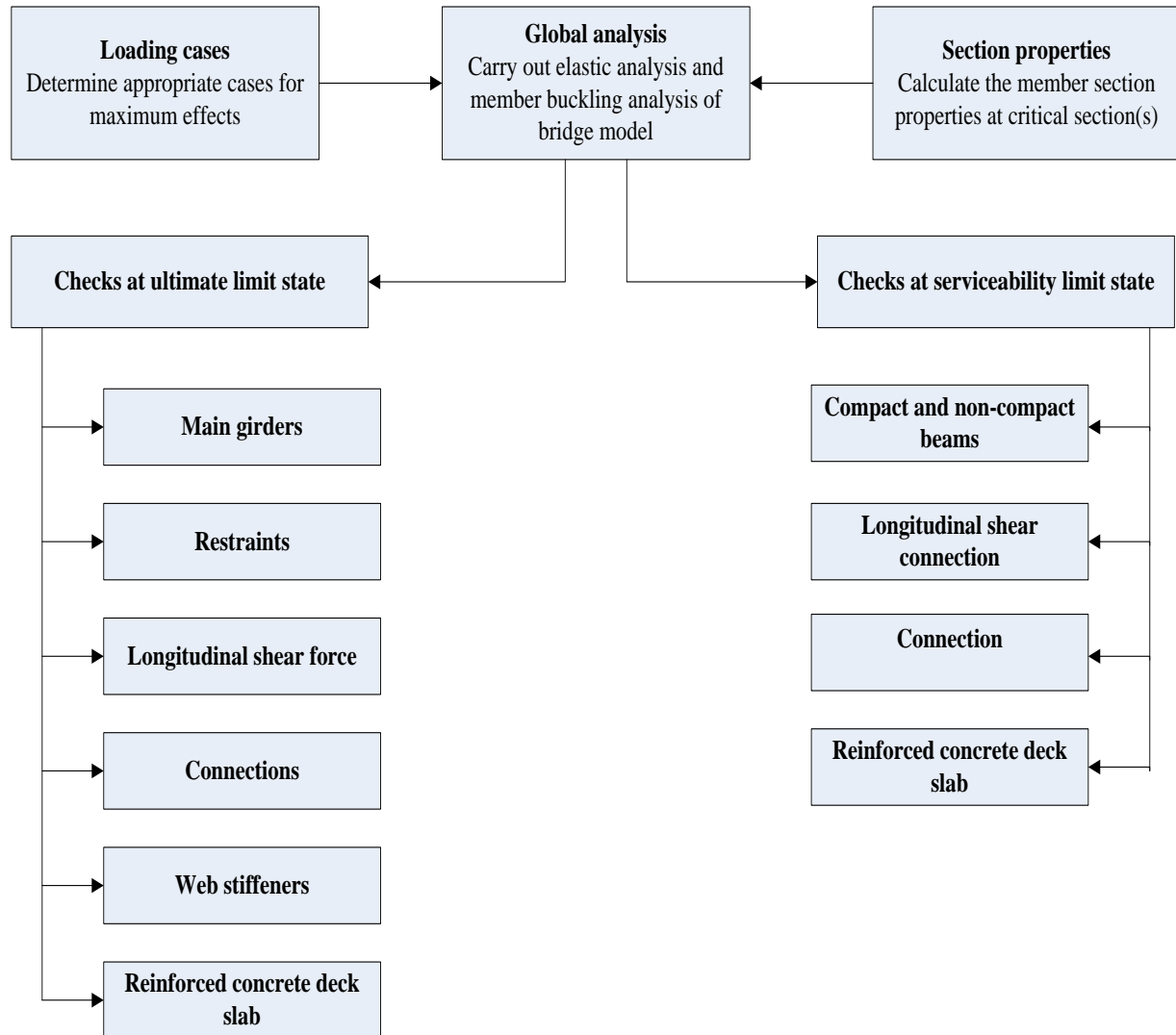


Figure 4-10: Overall design flow chart for the design of steel concrete composite bridge

## 5 DESIGN EXAMPLE

Composite box-girder bridge that has equal spans of  $30m$  and superstructure  $12.9m$  wide. Elevation and cross section are shown in the following figure.

### Subject information

Super structure type	Two Span Box Girder
Clear span	$42m$
Clear road way	$13.05m$
Side walk	$1.4m$

### Materials

Structural steel	$f_y = 300MPa$
Concrete	$f'_c = 30.0MPa$ $E_c = 22,400MPa$
Modular ration	$n = 8$

### Loading

Dead load	Girder + Concrete deck + Railing + Wearing surface
Live load	AASHTO Design Vehicular Load + dynamic load allowance
Concrete slabs deck with thickness of $230\text{ mm}$	
Density of Concrete	$24\text{ kN}/m^3$
Density of Surfacing	$22\text{ kN}/m^3$

### Specification

AASHTO-LRFD  
ERA-bridge design Manual

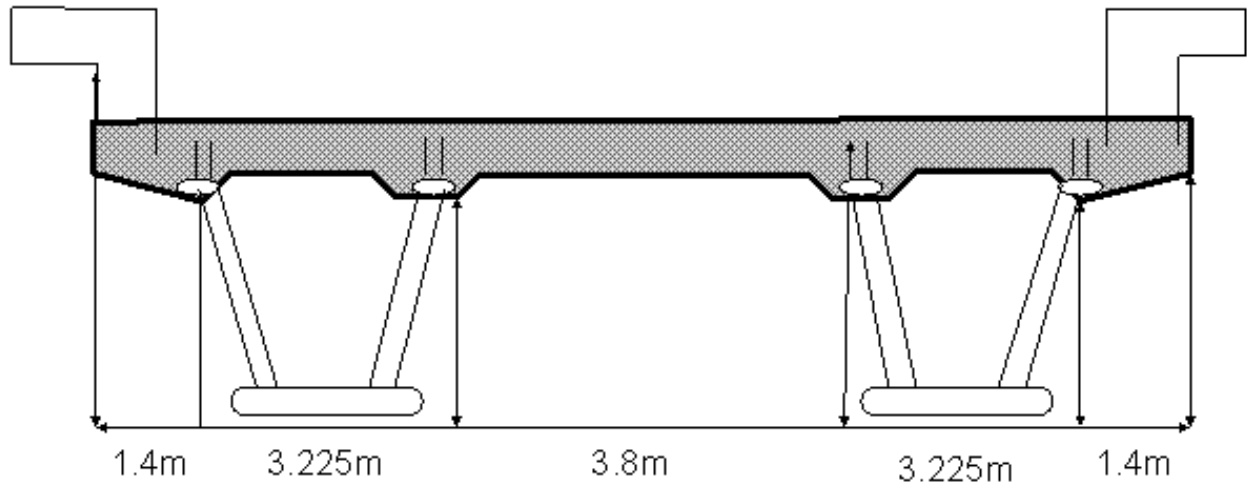


Figure 5-1: Typical Section

## 5.1 Span Load Calculation

### a) Component Dead load for a box girder

Include all structural dead loads; assume that all dead load is distributed equally to each girder by the tributary area. The tributary width for the box girder is 6.525 m.

#### ✚ Component dead load 1

Acting on non composite section

Concrete slab	$= (6.525 * 0.23 * 2400 * 9.81)$	$= 35.33 \text{ kN/m}^3$
Haunch		$= 3.5 \text{ kN/m}^3$
Girder, cross frame, diaphragm, and stiffener		$= 9.8 \text{ kN/m}^3$
<b>Total</b>		<b><math>48.63 \text{ kN/m}^3</math></b>

#### ✚ Component dead load 2

Acting on the long term composite section

Weight of each barrier rail  $= 5.7 \text{ kN/m}^3$

### b) Wearing surface of 100 mm is assumed to be distributed equally to each girder

DW: Acting on the long-term composite section  $= 11.9 \text{ kN/m}^3$

Live load distribution factors for strength limit state one [AASHTO]

$$= 0.05 + 0.85 \frac{NL}{NB} + \frac{0.425}{NL} = 0.05 + 0.85 * \frac{3}{2} + \frac{0.425}{3} = 1.5$$

## 5.2 Moments and Shear Demand

Table 5-1: Moment Envelopes for Strength Limit State I

(x)	$M_{DC1}$ (kN - m)	$M_{DC2}$ (kN - m)	$M_{DW}$ (kN - m)	$M_{LL} + I_M$ (kN - m)	$M_u$ (kN - m)
0	0	0	0	0	0
2.1	2037.354	238.8015	262.9119	498.5505	8602.031
4.2	3860.249	452.466	498.1489	944.622	15484.84
6.6	5680.957	665.874	705.7109	1390.158	21245.61
8.8	7103.87	832.656	885.598	1738.352	24572.04
11	8291.415	971.85	1037.81	2028.95	26330.84
12.2	8839.961	1036.146	1162.347	2163.182	26055.98
14.4	9663.754	1132.704	1259.21	2364.768	24709.65
16.6	10252.18	1201.674	1328.397	2508.758	22347.22
18.8	10605.23	1243.056	1369.909	2595.152	18602.94
21	10722.92	1256.85	1383.747	2623.95	20665.43

Where:-

$$M_u = 0.95[1.25(M_{DC1} + M_{DC2}) + 1.5 M_{DW} + 1.75 M_{LL} + I_M]$$

Table 5-2: Shear Envelopes for Strength Limit State I

(x)	$V_{DC1}$ (kN - m)	$V_{DC2}$ (kN - m)	$V_{DW}$ (kN - m)	$V_{LL} + I_M$ (kN - m)	$V_u$ (kN - m)
0	1021.23	119.7	249.9	179.63	3157.337
2.1	919.107	107.73	224.91	169.72	2818.661
4.2	816.984	95.76	199.92	159.82	2531.239
6.6	700.272	82.08	171.36	149.91	2141.471
8.8	593.286	69.54	145.18	140.01	1764.19
11	486.3	57	119	130.10	1355.638
12.2	427.944	50.16	104.72	120.20	1197.952
14.4	320.958	37.62	78.54	110.29	889.1668
16.6	213.972	25.08	52.36	100.39	595.1608
18.8	106.986	12.54	26.18	90.48	270.6479
21	0	0	0	80.58	35.39463

Where:-

$$V_u = 0.95[1.25(V_{DC1} + V_{DC2}) + 1.5 V_{DW} + 1.75 V_{LL} + I_M]$$

## 5.3 Composite Section Properties

Effective flange width for positive flexure region [AASHTO],

- I. For an interior web, the effective flange width is

$$b_{eff} \leq \text{minimum} \left\{ \begin{array}{l} \frac{L_{eff}}{4} = 31500/4 = 7875mm \\ 12t_s + \frac{b_f}{2} = 12 * 230 + \frac{450}{2} = 2985mm \\ S = 3800 \end{array} \right\}$$

2985mm govern the design

II. For an exterior web, the effective flange width is

$$b_{eff} = \text{minimum} \left\{ \begin{array}{l} \frac{L_{eff}}{48} = 31500/48 = 3937mm \\ 6t_s + \frac{b_f}{4} = 6 * 230 + \frac{450}{4} = 1492.5mm \\ \text{the width of the overhang} = 1400mm \end{array} \right\}$$

1400mm govern the design

Total effective flange width for the box girder =  $1400 + \frac{2985}{4} + 2985 = 5131mm$

### ✚ Elastic composite section properties for positive flexure region:

Elastic section properties are calculated for:

- ✓ Non composite,
- ✓ The short-term composite ( $n = 8$ ), and
- ✓ The long-term composite ( $3n = 24$ )

Table 5-3: Non composite section property for Positive Flexure Region

	$A(mm^2)$	$Y(mm)$	$AY(mm^3)$	$Y_i - Y_{sb}(mm)$	$A_i(y_i - Y_{sb})^2$	$I_0(mm^4)$
2 top flange $450 \times 20$	18,000	1574.2	$28.341 \times 10^6$	885	$141 \times 10^9$	$0.6 \times 10^6$
2 web $1600 \times 13$	41,600	788.1	$32.79 \times 10^6$	99	$0.41 \times 10^9$	$8.35 \times 10^6$
Bottom flange $2450 \times 12$	29,400	6.0	$0.17 \times 10^6$	-683	$13.70 \times 10^9$	$0.35 \times 10^6$
Sum	89,000	-	$61.30 \times 10^6$	-	$28.29 \times 10^9$	$8.35 \times 10^6$

$$Y_{sb} = \frac{\sum A_i * y_i}{\sum A_i} = 688.7mm$$

$$Y_{st} = (12 + 1552.5 + 20) - 688.7 = 895.5mm$$

$$I_{gir} = \sum I_0 + \sum A_i (Y_i - Y_{sb})^2 = 63.58 \times 10^9 mm^4$$

$$S_{sb} = \frac{I_{gir}}{Y_{sb}} = 53.11 \times 10^6 mm^3$$

$$S_{st} = \frac{I_{gir}}{Y_{st}} = 40.85 \times 10^6 mm^3$$

Using the same procedure the total effective flange width for the negative flexure region is 5828 mm.

Table 5-4: Short-Term Composite Section Properties (n= 8)

	$A(mm^2)$	$Y(mm)$	$AY(mm^3)$	$Y_i - Y_{sb}(mm)$	$A_i(y_i - Y_{sb})^2$	$I_0(mm^4)$
Steel section	89,000	688.4	$61.3 \times 10^6$	-611	$33.24 \times 10^9$	$36.58 \times 10^6$
Concrete slab 5131/8 × 230	147,516	1744.2	$257 \times 10^6$	414	$25 \times 10^9$	$0.43 \times 10^6$
Sum	236,516	-	$318.3 \times 10^6$	-	$55.77 \times 10^9$	$37.02 \times 10^6$

$$Y_{sb} = \frac{\sum A_i * y_i}{\sum A_i} = 1345.78 mm$$

$$Y_{st} = (12 + 1552.5 + 20) - 1299.8 = 284.4 mm$$

$$I_{com} = \sum I_0 + \sum A_i (Y_i - Y_{sb})^2 = 92.79 \times 10^9 mm^4$$

$$S_{sb} = \frac{I_{com}}{Y_{sb}} = 68.9 \times 10^6 mm^3$$

$$S_{st} = \frac{I_{com}}{Y_{st}} = 326.30 \times 10^6 mm^3$$

Table 5-5: Long-Term Composite Section Properties (3n = 24)

	$A(mm^2)$	$Y(mm)$	$AY(mm^3)$	$Y_i - Y_{sb}(mm)$	$A_i(y_i - Y_{sb})^2$	$I_0(mm^4)$
Steel section	89,000	688.4	$61.3 \times 10^6$	-338	$7.7 \times 10^9$	$36.5 \times 10^6$
Concrete slab 5131/24 × 230	49,172	1744.2	$85 \times 10^6$	698	$20.7 \times 10^9$	$5.4 \times 10^6$
Sum	138,127	-	$146 \times 10^6$	-	$28.4 \times 10^9$	$41.9 \times 10^6$

$$Y_{sb} = \frac{\sum A_i * y_i}{\sum A_i} = 984 mm$$

$$Y_{st} = (12 + 1552.5 + 20) - 984 = 796.8 mm$$

$$I_{com} = \sum I_0 + \sum A_i (Y_i - Y_{sb})^2 = 70.3 \times 10^9 mm^4$$

$$S_{sb} = \frac{I_{com}}{Y_{sb}} = 65.68 \times 10^6 mm^3$$

$$S_{st} = \frac{I_{com}}{Y_{st}} = 121 \times 10^6 mm^3$$

Elastic composite section properties for negative flexure region

As per AASHTO requirement for any continuous span the total cross-sectional area of longitudinal reinforcement must not be less than 1% of the total cross-sectional area of the slab. The required reinforcement must be placed in two layers uniformly distributed across the slab width and two thirds must be placed in the top layer. The spacing of the individual bar should not exceed 150 mm in each row.

$$A_{S_{req}} = 0.01(230) = 2.30 mm^2/mm$$

$$A_{s \text{ top layer}} = 2/3 (0.01 * 230) * 1.33 mm^2/mm$$

$$\emptyset 20 \text{ at } 200mm = 1.53 mm^2/mm$$

$$A_{s \text{ top layer}} = 1/3 (0.01 * 230) = 0.67 mm^2/mm$$

$$\emptyset 14 \text{ at } 150mm = 1.02 mm^2/mm$$

Table 5-6: Non-composite Section Properties for Negative Flexure Region

	$A(mm^2)$	$Y(mm)$	$AY(mm^3)$	$Y_i - Y_{sb}(mm)$	$A_i(y_i - Y_{sb})^2$	$I_0(mm^4)$
2 Top flange 650 × 40	52,000	1602	$83.32 \times 10^6$	911	$43.2 \times 10^9$	$6.93 \times 10^6$
2 Web 1600 × 13	41,600	806	$33.53 \times 10^6$	115	$0.55 \times 10^9$	$8.35 \times 10^6$
Stiffener WT	5,400	224.3	$1.21 \times 10^6$	-466	$1.18 \times 10^9$	$32.63 \times 10^6$
Bottom flange 2450 × 30	73,500	15	$1.1 \times 10^6$	-676	$33.57 \times 10^9$	$5.51 \times 10^6$
Sum	172,500	-	119.2		$78.49 \times 10^9$	$8.4 \times 10^6$

$$Y_{sb} = \frac{\sum A_i * y_i}{\sum A_i} = 690.8 mm$$

$$Y_{st} = (30 + 1552.5 + 40) - 690.8 = 931.4 mm$$

$$I_{gir} = \sum I_0 + \sum A_i (Y_i - Y_{sb})^2 = 86.90 \times 10^9 mm^4$$

$$S_{sb} = \frac{I_{gir}}{Y_{sb}} = 125.8 \times 10^6 mm^3$$

$$S_{st} = \frac{I_{gir}}{Y_{st}} = 93.29 \times 10^6 mm^3$$

Table 5-7: Composite Section Properties for Negative Flexure Region

	$A(mm^2)$	$Y(mm)$	$AY(mm^3)$	$Y_i - Y_{sb}(mm)$	$A_i(y_i - Y_{sb})^2$	$I_0(mm^4)$
Steel section	172,500	690.8	$119.2 \times 10^6$	73.2	$0.92 \times 10^9$	$86.90 \times 10^6$
Top reinforcement	8,665	1762.2	$15.27 \times 10^6$	998.2	$8.63 \times 10^9$	
Bottom reinforcement	4360	1677.2	$7.31 \times 10^6$	913.2	$3.64 \times 10^9$	
Sum	185,525	-	141.7		$13.19 \times 10^9$	$86.90 \times 10^6$

$$Y_{sb} = \frac{\sum A_i * y_i}{\sum A_i} = 764 \text{ mm}$$

$$y_{st} = (30 + 1552.5 + 40) - 764 = 858.2 \text{ mm}$$

$$I_{com} = \sum I_0 + \sum A_i (Y_i - Y_{sb})^2 = 131 \times 10^9 \text{ mm}^4$$

$$S_{sb} = \frac{I_{com}}{Y_{sb}} = 125.8 \times 10^6 \text{ mm}^3$$

$$S_{st} = \frac{I_{com}}{Y_{st}} = 116.63 \times 10^6 \text{ mm}^3$$

#### 5.4 Calculation of Yield Moment $M_y$ and Plastic Moment $M_p$

##### a) Yield moment $M_y$

The yield moment  $M_y$  corresponds to the first yielding of either steel flange. It is obtained as

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

Where:-

$M_{D1}$ ,  $M_{D2}$  and  $M_{AD}$  are moments due to the factored loads applied to the steel, long-term, and short-term composite section, respectively.

$M_{AD}$  can be obtained by solving the equation:

$$F_y = \frac{M_{D1}}{S_s} + \frac{M_{D2}}{S_{3n}} + \frac{M_{AD}}{S_n}$$

$$M_{AD} = S_n \left( F_y - M_{D1} - \frac{M_{D2}}{S_{3n}} \right)$$

Where:-

$S_s$ ,  $S_n$  and  $S_{3n}$  are the section modulus for the non composite steel, the short-term, and the long-term composite sections, respectively.

$$M_{D1} = 0.95 * 1.25 * M_{DC1} = 0.95 * 1.25 * 10722 = 12,732.34kN - m$$

$$M_{D2} = 0.95(1.25M_{DC2} + 1.5M_{DW}) = 0.95(1.25 * 1256 + 1.5 * 1383.747) = 3,463.3kN - m$$

For the top flange:

$$M_{AD} = 329.3 * 10^{-3} \left( 300 * 10^3 - \left( \frac{12.732}{40.85 * 10^{-3}} - \frac{3.46}{120 * 10^{-3}} \right) \right) = 98.69 \times 10^3 kN - m$$

For the bottom flange:

$$M_{AD} = 71.39 * 10^{-3} \left( 300 * 10^3 - \left( \frac{12.73}{53.11 * 10^{-3}} - \frac{3.46}{64.68 * 10^{-3}} \right) \right) = 21.34 \times 10^3 kN - m$$

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

$$M_y = 12,732.34 + 3,463.3 + 21340 = 37,535.64kN - m$$

b) *Plastic moment  $M_p$*

The plastic moment  $M_p$  is determined using equilibrium equations. The reinforcement in the concrete slab is neglected for this part of the design.

Determine the location of the plastic neutral axis

$$P_s = 0.85 * f'_c * b_{eff} * t_s = 0.85 * 30 * 5131 * 230 = 30,093kN$$

$$P_c = A_{fc} * F_{yc} = 2 * 450 * 20 * 345 = 6,210kN$$

$$P_w = A_w * f_{yw} = 2 * 1600 * 13 * 345 = 14,352kN$$

$$P_t = A_{ft} * f_{yt} = 2450 * 12 * 345 = 10,143kN$$

$$P_t + P_w + P_c = 10,143 + 14,352 + 6,210 = 30,705kN > 30,093kN$$

Accordingly, The PNA is located within the top flange of steel girder and the distance from the top of Compression flange to the PNA, is:

$$Y = \frac{t_{fc}}{2} \left( \frac{P_w + P_t - P_s}{P_c} + 1 \right) = 9mm$$

Then Calculate  $M_p$ :

Summing all forces about the PNA:

$$M_p = \sum M_{PNA} = \frac{P_c}{12t_c} (y^2 + (t_c - y)^2) + p_s d_s + p_w d_w + p_t d_t$$

$$d_s = \frac{230}{2} + 50 - 20 + 9 = 154mm$$

$$d_w = \frac{1552.5}{2} + 20 - 9 = 787.25mm$$

$$d_t = \frac{12}{2} + 1552.5 + 20 - 9 = 1569.5mm$$

$$M_p = \frac{6210}{12 * 20} (9^2 + (20 - 9)^2 + 30,093 * 154 + 14,352 * 787.25 + 10,143 * 1569.5)$$

$$M_p = 31,857.5KNM$$

## 5.5 Flexural Strength Design

In multiple box-girder design, primary consideration should be given to flexure. According to AASHTO-LRFD Specifications for design of a multiple box section the following flange distance limitations shown should be satisfied

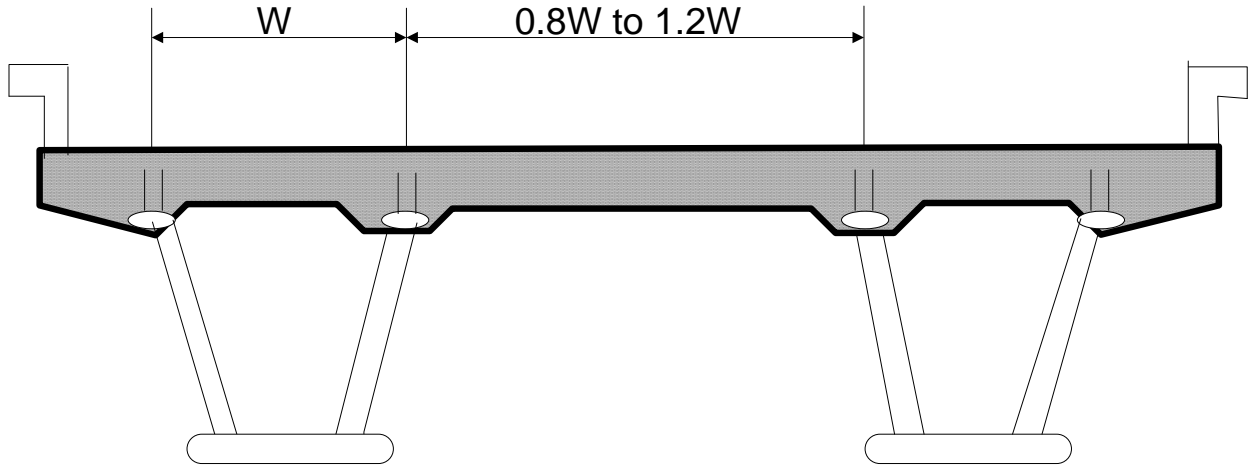


Figure 5-2: Flange distance limitation

a) *Positive flexure region:*

✚ Compactness of steel box girder

Compactness of a multiple steel boxes is controlled only by web slenderness. The purpose of the ductility requirement is to prevent permanent crushing of the concrete slab when the composite section approaches its plastic moment capacity.

The equation  $\frac{2D_{cp}}{t_w} \leq \sqrt{\frac{E}{F_c}}$  must be satisfied to check the slenderness requirement since  $D_{cp} = 0$ , the web slenderness requirement is satisfied:

$$D_p = 200 + 50 - 20 + 2.311 = 232.311mm$$

$$D' = \beta \left[ \frac{d + t_s + t_h}{7.5} \right] \text{ where: } \beta = 0.7$$

$$= 0.7 \left[ \frac{1552.5 + 12 + 200 + 50}{7.5} \right] = 169.3mm$$

Required section ductility must be less than 5: i.e.  $\left( \frac{D_p}{D'} \right) < 5$

$$= \frac{232.11}{169.3} = 1.37 < 5, \text{ it is OK}$$

Calculate Nominal flexure resistance,  $M_n$

For simple spans and continuous spans with compact interior support section:

If  $D' < D_p < 5D'$  which is  $1.37 < 5$

$$M_n = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4} \left(\frac{D_p}{D'}\right) \text{ Or } M_n = 1.3 * R_h * M_y$$

$$M_n = \frac{5*31,857.5 - 0.85*37,535.6}{4} + \frac{0.85*37,535.6 - 31,857.5}{4} (1.37) = 31,829KNm \quad \text{OR}$$

$$M_n = 1.3 * 1 * 37,535.6 = 48,796KNm$$

$$\text{Then } M_n = 31,829KNm$$

The maximum factored positive moment is  $26,330KNm$

$26,330KNm < 31,829KNm$  , it is OK

b) *Negative flexure region:*

✚ Stiffener requirement

Use one longitudinal stiffener try:  $W_T = 10.5 * 28.5$

The projecting width, of the stiffener should satisfy:

$$b_l \leq 0.48t_p \sqrt{\frac{E}{F_{yc}}}$$

Where:-

$t_p$  = the thickness of stiffener

$b_l$  = the projected width =  $\frac{267}{2} = 133.5mm$

$I_s = 33.4 \times 10^6 + 4748(190.1)^2 = 20.5 \times 10^6mm$

$$= 133.5 \leq 0.48(16.5) \sqrt{\frac{200000}{300}} = 204.5$$

Buckling coefficient,  $K$

For  $n = 1$

$$K = \left[ \frac{8I_s}{Wt^3} \right]^{1/3} < 4 = \left[ \frac{8 * 20.5 * 10^6}{1225 * 24^3} \right] = 2.13 < 4 \text{ OK}$$

✚ Calculate nominal flange stress

The nominal flexural resistance of compression flange is controlled by inelastic Buckling

$$\text{For } 0.5 \sqrt{\frac{KE}{f_{yc}}} < \frac{w}{t} \leq 1.23 \sqrt{\frac{KE}{f_{yc}}}$$

$$F_{nc} = 0.592 * R_b * R_h * f_{yc} \left[ 1 + 0.687 \frac{\sin C\pi}{2} \right]$$

$$\text{Where: } C = 1.23 - w/t \sqrt{\frac{f_{yc}}{KE}} = 1.23 - 40.8 \sqrt{\frac{300}{3.9*2*10^5}} = 0.65$$

$$R_b = 1 \text{ and also } R_h = 1$$

$$F_{nc} = 305.6 \text{MPa}$$

For tension flange

$$F_{nt} = R_b * R_h * R_{yt} = 1.0 * 1.0 * 300 = 300 \text{MPa}$$

✚  $M_{AD}$  at Interior Support

$$M_{D1} = 0.95 * 1.25 * M_{DC1} = 0.95 * 1.25 * 10,722.92 = 12,733.5 \text{KNm}$$

$$M_{D2} = 0.95 * 1.25 * M_{DC2} + 1.5(M_{DW}) = (0.95 * 1.25 * 1256.9) + 1.5(1383.7) \\ = 3,568 \text{KNm}$$

$$M_{AD} = S_n \left[ F_n - \frac{M_{D1}}{S_s} + \frac{M_{D2}}{S_n} \right]$$

$$M_{AD.com} = 0.131 \left[ 305.6 \times 10^3 - \frac{12,733.5}{0.1258} - \frac{3,568}{0.131} \right] = 22,765 \text{KNm}$$

$$M_{AD.ten} = 0.1166 \left[ 300 \times 10^3 - \frac{12,733.5}{0.0933} - \frac{3568}{0.116} \right] = 15,480 \text{KNm control}$$

Nominal flexural strength  $M_n$

$$M_n = 12,733.5 \text{KNm} + 3,568 \text{KNm} + 15,480 \text{KNm} = 31,781.5 \text{KNm}$$

Maximum factored moment is 26,330KNm

$$26,330 \text{KNm} \leq 31,781.5 \text{KNm} \quad \text{OK}$$

## 5.6 Shear Strength Design

### a. End bearing of Span 1

✚ Nominal shear resistance  $V_n$

Due to factored loads taken as [AASHTO], for inclined webs each web shall be designed for shear,  $V_{ui}$

$$V_{ui} = \frac{V_u}{2\cos\theta} = \frac{3157.33}{2\cos 14} = 1,627 \text{KN per web}$$

Where  $\theta$  is the angle of the web to the vertical accordingly.

$$\frac{D}{t_w} = \frac{1600}{13} = 123.1 > 3.07 \sqrt{\frac{E}{F_{yw}}} = 3.07 \sqrt{\frac{2 * 10^5}{300}} = 79.26$$

$$V_n = \frac{4.55 * t_w^3 * E}{D} = \frac{4.55 * 13^3 * 2 * 10^5}{1600} = 1249.5 \text{KN}$$

$$V_{ui} > \phi V_n \text{ Where: } \phi = 1$$

1,627 > 1 \* 1249.5 As a result stiffeners are required

$V_n$  for end stiffened web panel

$$V_n = CV_p$$

$$K = 5 + \frac{5}{(d_o/D)^2} \quad d_o = \text{spacing of transverse stiffeners}$$

For  $d_o = 2400$   $K=7.22$

$$\frac{D}{t_w} = 123.1 > 1.38 \sqrt{\frac{EK}{F_{yw}}} = 89.3$$

$$C = \frac{152}{123.1^2} \sqrt{\frac{EK}{F_{yw}}} = 0.65$$

$$V_p = 0.58F_{yw}Dt_w = 4162$$

$$V_n = CV_p = 0.65 * 4162 = 2705 \text{kN} > V_{ui} = 1627 \text{kN} \quad \text{Ok}$$

b. Intermediate transverse stiffener design

To prevent local buckling of the transverse stiffeners, the width of each projecting stiffener shall satisfy the following requirements.

$$\left( \begin{array}{c} 50 + \frac{d}{30} \\ 0.25b_f \end{array} \right) \leq b_t \leq \left( \begin{array}{c} 0.48t_p \sqrt{\frac{E}{F_{ys}}} \\ 16t_p \end{array} \right)$$

Try stiffener width,  $b_t = 160.0$  mm.

$$160 > 50 + \frac{1600}{30} = 103.3\text{mm}$$

$$160 > 0.25 * 450 = 112.5 \quad OK$$

Try  $t_p = 14\text{mm}$

$$b_t < \left( \begin{array}{c} 0.48t_p \sqrt{\frac{E}{F_{ys}}} \\ 16t_p \end{array} \right)$$

$$160 < 0.48 * 16 \sqrt{\frac{2 * 10^5}{300}} = 198.3\text{mm}$$

$$160 < 16 * 14 = 224 \quad OK$$

Use  $160\text{mm} \times 14\text{mm}$  transverse stiffener plates.

Moment of inertia Requirement

$$I_t \geq d_0 * t_w^2 * J$$

$$J = 2.5(D_p/d_0)^2 - 2 \geq 0.5$$

$$J = 2.5 * (1600/2400)^2 - 2 = -0.89 < 0.5 \quad \text{Take } J = 0.5$$

$$I_t = \frac{14 * 160^3}{3} = 19.11\text{mm}^4 > d_0 * t_w^2 * J = 2400 * 16^3 * 0.5 = 4.9 * 10^6\text{mm}^4 \quad OK$$

Area Requirement

To ensure that transverse stiffeners have sufficient area to resist the vertical component of the tension field.

$$A_s \geq A_{smin} = \left( 0.15BDt_w(1 - C) \frac{V_u}{\phi_v V_n} - 18t_w \right) \frac{F_{yw}}{F_{ys}}$$

Where:-

$$B = 1.0$$

$$C = 0.65$$

$$C = 0.65$$

$$F_{yw} = 300MPa \quad F_{ys} = 300MPa$$

$$V_u = 732.49kN(\text{per Web}) \quad \phi_f V_n = 300MPa \quad t_w = 13mm$$

$$A_s = 160 * 14 = 2240mm^2$$

$$= (0.15BDt_w(1 - C) \frac{V_u}{\phi_v V_n} - 18t_w) \frac{F_{yw}}{F_{ys}}$$

$$= (0.15 * 24)(1600)(13)(1 - 0.65) \frac{732.49}{1249.5} - 18 * 13^2 \frac{300}{300} = -1505mm^2$$

The web has sufficient area to resist the vertical component of the tension field.

## 5.7 Fatigue design

Fatigue requirement for web in positive flexure region

$$D_c = \frac{\frac{M_{DC1} + M_{DC2} + M_{Dw}}{S_{st}} + \frac{2(M_{LL+IM})_u}{S_{st-3n}}}{\frac{M_{DC1} + M_{DC2} + M_{Dw}}{I_{gir}} + \frac{2(M_{LL+Im})_u}{I_{com-3n}}} - t_{fc} = 169mm$$

$$\frac{2D_c}{t_w} = \frac{2 * 169}{13(\cos 14)} = 125 < 5.76 \sqrt{\frac{E}{F_{yc}}} = 148$$

$$f_{cf} < f_{yw}$$

$$f_{cf} = \frac{M_{DC1}}{S_{st}} + \frac{M_{DC2} + M_{Dw}}{S_{st-3n}} + \frac{2(M_{LL+IM})}{S_{st-n}} = 300Mpa < f_{yw} \quad \text{OK}$$

## 5.8 Shear connector design

Continuous composite bridges should normally be provided with shear connectors throughout the entire length of the bridge. Both stud and channel shear connectors are permitted. Stud shear connectors will be utilized in this example.

### I. Stud Size

Stud height should penetrate at least 50 mm into the deck and clear cover depth of concrete cover over the top of the shear stud should not be less than 50 mm.

$$\text{Try } H_s = 160 \text{ mm} > 50 + (50 - 25) = 75 \text{ mm} \quad \text{OK}$$

$$\text{Stud diameter } d_s = 25 \text{ mm} < H_s/4 = 40 \text{ mm}$$

### II. Pitch of Shear Stud, $p$ , for Fatigue Limit State

a) Fatigue resistance  $Z_r$

$$Z_r = 19d_s^2 = 19(25)^2 = 11,875 \text{ N}$$

b) First moment  $Q$  and moment of inertia  $I$

$$Q = \left( \frac{b_{eff} t_s}{8} \right) \left( y_{st} + t_h + \frac{t_s}{2} \right) = \left( \frac{5131 * 230}{8} \right) \left( 284.4 + 25 + \frac{230}{2} \right) = 62.6 * 10^6 \text{ mm}^3$$

$$I_{com} = 92.79 * 10^9 \text{ mm}^4$$

C) Required pitch for fatigue limit state

Assume that shear studs are spaced at 200 mm transversely across the top flange of steel section and using  $n_s = 5$

Calculate  $p_{rd}$

$$p_{rd} = \frac{n_s Z_r I}{V_u Q} = \frac{5 * 11.87 * 92.79 * 10^9}{V_u * 62.6 * 10^6} = \frac{87972}{V_u}$$

### III. Strength limit state check

a) Nominal horizontal shear force

$$V_h = \min \left\{ \begin{array}{l} F_{yw}Dt_w + F_{yt}b_{tf}t_{tf} + F_{yb}b_{bf}t_{bf} \\ 0.85f'_c b_{eff} t_s \end{array} \right\}$$

$$V_{h\text{concrete}} = 0.85f'_c b_{eff} t_s = 0.85 * 30 * 5131 * 230 = 30 * 10^3 \text{ kN}$$

$$\begin{aligned} V_{h\text{steel}} &= F_{yw}Dt_w + F_{yt}b_{tf}t_{tf} + F_{yb}b_{bf}t_{bf} \\ &= 300(1600 * 13 + 450 * 20 + 2450 * 12) = 17.8 * 10^3 \text{ kN} \end{aligned}$$

$$V_h = 17.8 * 10^3 \text{ kN}$$

b) Nominal shear resistance

Use minimum tensile strength  $F_u = 420 \text{ Mpa}$  for stud shear connector

$$Q_n = 0.5A_{sc}\sqrt{f'_c}E_c = 0.5 * \frac{\pi * 25^2}{4} \sqrt{30 * 22400} = 201 \text{ kN}$$

c) Number of shear stud connectors

$$n_{total} = \frac{V_h}{\phi Q_n} = \frac{17800}{0.85 * 201} = 105$$

### 5.9 Lateral torsional buckling resistance

First Compute local buckling resistance, Slenderness ratio for the top flange

$$\lambda = \frac{b_{ft}}{2t_{fc}} = \frac{450}{2 * 20} = 11.25$$

$$\lambda_{pf} = 0.57 \sqrt{\frac{E}{F_{yc}}} = 14.71$$

Since  $\lambda < \lambda_{pf}$

$$F_{nc} = R_b R_h F_{yc} \quad \text{Since } R_b \text{ is taken as 1 for constructability}$$

$$F_{nc} = 1 * 1 * F_{yc} =$$

Limiting unbraced length

$$Y_{st} = 895.5 \text{ mm}$$

$$Y_c = (Y_{st} - t_f) \sqrt{\frac{s^2 + 1}{s^2}} = (895.5 - 20) \sqrt{\frac{4^2 + 1}{4^2}} = 902 \text{ mm}$$

$$L_p = r_t \sqrt{\frac{E}{F_{yc}}} = 108 * 25.8 = 108 * 25.8 = 2788 \text{ mm}$$

$$r_t = \frac{b_{fc}}{\sqrt{12 \left( 1 + \frac{1}{3} \frac{D_c t_w}{b_{fc} t_{fc}} \right)}} = \frac{450}{\sqrt{12 \left( 1 + \frac{1}{3} \frac{902 * 13}{450 * 20} \right)}} = 108 \text{ mm}$$

$$L_b = 1.76 r_t \sqrt{\frac{E}{F_{yc}}} = 1.76 * 2788 = 4906 \text{ mm}$$

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = 3.14 * 2788 = 8754 \text{ mm}$$

Since  $L_p = 2788 \text{ mm} < L_b = 4906 \text{ mm} < L_r = 8754 \text{ mm}$

$F_{nc}$  is computed by

$$C_b R_b R_h F_{yc} \left[ 1.33 - 0.18 \left( \frac{L_b}{r_t} \right) \sqrt{\frac{F_{yc}}{E}} \right] \leq R_b R_h F_{yc}$$

Take  $C_b = R_b = R_h = 1$

$$F_{nc} = C_b R_b R_h F_{yc} = R_b R_h F_{yc} = 0.995 * F_{yc} < F_{yc}$$

$F_{nc}$  is governed by lateral torsional buckling resistance which is lesser than the local buckling resistance

Since the Equation satisfied

$$f_{bu} + \frac{1}{3}f_l < \phi_f F_{nc}$$

$$28 + \frac{1}{3} * 41.4 < \phi_f F_{nc} \dots\dots\dots \text{OK}$$

### 5.10 Cost Comparison for Concrete and Steel Concrete Composite bridges

The comparison is made for construction of superstructure only focused on the time required for both methods.

Time is the critical element in construction industry and affects the overall cost.

The time saved due to construction of composite bridge compared with that of cast in situ concrete bridge has significantly wider difference which results in increased cost of concrete bridges.

From experience of concrete bridge under construction in Dire Dawa named as **Taiwan Box Girder bridge project** which has a span length 110m the required time for super structure work compared with equivalent composite bridge can be summarized as follows.

Table 5-8: Time required for composite and cast in situ concrete construction

No.	Main Tasks	Duration(day)	
		Concrete	Composite
1	Form work construction	90	-
2	Placing of reinforcement bar	49	
3	Girder casting	15	-
4	Girder erection	-	30
5	Deck casting	10	5
6	Concrete curing time for girders	28	-
7	False work removal	10	5
8	Flood disturbance	45	-
	Total	257	40

Time difference =  $257-40=217$ days

Monthly expenses for the project

Consultancy expense **250,000** Birr per month.

Contractor expenses including Labor, equipments and other miscellaneous activities of the project various according to the activity type but on a minimum it reaches about **1,000,000** birr per month.

Total monthly expenses = **250,000+1,000,000=1,250,000** birr per month.

Time value of money results from composite construction =  $(217/30)*1,250,000 = 9,041,666.67$  Birr.

Even though the above example does not cover the detail calculations of each item, it shows the significant economical advantage of composite construction with respect to time significantly. Therefore, this technology is highly recommended.



Figure 5-3: Time consuming activities in concrete construction

## **6 CONCLUSIONS**

### **6.1 Conclusion**

In the current construction technology, a wide range of possibilities of steel concrete composite bridge design and construction methods are available around the world. Fiber reinforced polymer as lighter and more durable alternatives has been developed in recent years and also inorganic phosphate cement are used in the construction of composite bridges as well.

The design of composite bridge might be much more complicated than concrete bridge. Thus, it is very important to understand the design principles and also applicability of composite materials for structural use in order to attain economical benefits. In addition, it has fundamental advantage by saving construction and maintenance time.

Due to the aforementioned benefits, it has been practicing widely for bridge and building construction, and getting great demand throughout the world. Therefore, this study strongly recommends this technology in Ethiopia to get all the mentioned advantages throughout this presentation.

Technically, the type of shear connectors it uses has great significant importance. Moreover, its light steel sections have so thin steel elements, and make possible to design innovative shear connectors to improve performance and design resistance.

Due to fast economical growth in Ethiopia, there is enormous demand of infrastructures including bridges. However, it is very tough to fulfill such demand due to construction constraint and economical reason. Composite construction plays immense role to answer this demand. As a result, this study strongly recommends composite construction technology.

## 6.2 Recommendations and Future Directions

Generally, to achieve the objectives of constructions in economical way, composite bridge design and construction technology plays enormous role. Moreover, it has some technical advantages which are explained in this thesis. Especially developing countries has to give especial attention to get its benefit. Specifically, Ethiopia is one of the developing countries, and its economy is developing rapidly. Thus, this construction technology contributes a lot in the current context of the country to answer the demand of bridge economically. Due to this and other reasons, we are interested to jump into this topic and add value by summarizing literatures, studying international versus Ethiopia experience, and give design example to illustrate the concept behind.

In this thesis, we address the profiles and material properties for composite construction, locally available profile steel sheet forms and their application, and types and components of possible steel concrete composite bridge sections. In addition, we give details of composite construction experience of four different countries including Ethiopia. Finally, we illustrate with practical example in Ethiopia context.

Even though local industries have been manufacturing steel structure, these products have been practicing for different applications like car bodies, and roofing and cladding. Therefore, government bodies like Ethiopia Roads Authority (ERA) have to introduce this technology by preparing seminars and other means of communication. As a result, the country might save lots of foreign currency by using local industries products to flourish country's economy, and makes these industries to be competitive in the global market. Lastly, to gain all the stated benefits throughout this thesis, we strongly recommend this technology.

Based on the aforementioned advantages, it would be interesting to continue and contribute in this area of research. Hence, we recommend to extend this research by designing benchmark based on different countries construction experience, propose multi-criteria decision support systems, and to deal the economical benefits with real scenarios.

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