

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES



FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

Study on Local Steel Profiles in Steel-Concrete
Composite Bridges

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ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

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Composite Bridges

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DECLARATION

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ABSTRACT

Building a functional transportation infrastructure is a high priority for a nation, particularly for developing country like ours. Currently, the Ethiopian government has given special attention for road construction sector and is to invest billions of dollars. Parallel to road construction, there is also a demand of construction of bridges as most of the existing ones are getting older as they are constructed, during colonization by Italians. Construction of such bridges naturally occupy longer construction time where the time required may be farther exacerbated due to shortage of budget beside having bad construction management practice which is observed in many construction undertakings. Associated with this, the normal day to day traffic flow is affected and causes discomforts to the road users. This problem is serious especially in towns where there is high traffic volume and provision of detours is not simple.

Thus, as alternate, steel-concrete composite bridge which obviously shortens the construction time and alleviates the problems with other added advantages is thought in this study. Therefore, in this thesis attempt is being made to compare the usage of locally produced steel profile in steel-concrete composite bridges with the conventional reinforced concrete deck girder bridges cost wise, beside evaluating other merits of such application.

In the thesis work, relevant literatures are reviewed, assessment of availability of local steel profiles for such application has been made to local private and governmental companies that are engaged in manufacturing, supplying and installing of such steel profiles for different engineering purposes. Kality Metal Products Factory, Mesfin Industrial Engineering PLC and Maru Metal Industry are among some whose local products are considered. Accordingly, it is observed that there is local capacity and experience of manufacturing I-section profiles by

welding steel metal sheets which are imported from abroad. The practices have been observed with application for buildings, steel bridges and manufacture of vehicle body parts.

Thus, the study makes use of these I-section profiles, for design of steel-concrete composite bridge superstructures having span lengths of 12m, 15m, 20m and 24m. For the purpose of comparison, the same span lengths and width of reinforced concrete deck girder bridges are also designed. Finally, based on design outputs, quantities of each item are summarized and using appropriate unit rates, cost analysis and evaluation of the two bridge types is carried out.

It is observed that relatively compact and lighter bridge cross-sections can be obtained by using steel-concrete composite structures than the reinforced concrete ones. The steel beams can also be manufactured in workshops prior to the start of the construction and this can reduce the required construction time. Nevertheless, it is concluded that with current market condition and work exposure of metal industries, cost required for steel - concrete composite bridge construction is higher than that of reinforced concrete deck girder bridges of the same span, for spans under consideration.

During the study, it is noticed that there are few metal industries that are involved in the manufacturing of steel profiles specially those for steel-concrete composite bridge constructions. So, investors should be encouraged to the sector so that designers could have free choices both in quality and shapes. It is also suggested that as markets are demand driven, consultants and designers should initiate and direct the metal industry by designing steel structures as an alternative. As final recommendation, further studies should be carried out to check feasibility of such constructions in our country in line with the change in development of the metal industry and construction skills, as the time saving by itself is a promising and vital item to be considered in bridge construction.

NOTATIONS

A_a	area of steel
A_c	area of concrete
A_v	effective shear area
b_{eff}	effective concrete slab flange width
b_1	top flange width of I section profile
b_2	bottom flange width of I section profile
c	distance from neutral axis to extreme fiber
c	half flange width less half web thickness
d	distance between flanges
d	diameter of shear studs
d_a	half height of an I section profile
E	modulus of elasticity
E_{cm}	mean secant (elastic) modulus of concrete
f_{ck}	cylindrical strength of concrete
f_u	ultimate tensile strength of steel
$F_u^{(r)}$	maximum compressive force in concrete slab
f_y	yield strength of steel
h_c	concrete slab depth
I	moment of inertia of resisting cross-section
L	span length
M	bending moment
M_{apl}	plastic moment capacity of steel section alone
$M_{plf,Rd}$	plastic bending resistance contributed by flange alone
$M_{u(r)}$	reduced ultimate moment
N	number of shear connectors
N_f	number of shear connectors for full connectors
P_{RD}	design shear resistance of a shear studs
Q	statical moment about neutral axis
S	section modulus
S_r	range of horizontal shear at slab beam interface

t_1	top flange thickness of I section
t_2	bottom flange thickness of I section
t_w	web thickness
V_l	longitudinal shear force
V_{lf}	maximum longitudinal shear force
$V_{pl,Rd}$	design plastic shear resistance
V_r	range of shear due to live load and impact
V_{sd}	design shear force
W	uniform dead load
y_a	neutral axis in the upper steel flange
y_c	concrete stress block depth
Z_r	allowable range of horizontal shear on one stud
α	constant based on number of stress cycles
ε	factor
γ_a	partial safety factor of steel
γ_c	partial safety factor of concrete
γ_v	partial safety factor of shear studs
λ_w	web slenderness
σ	stress in beam
Δ_{DL}	dead load deflection

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1. INTRODUCTION

1.1 BACKGROUND OF THE STUDY

Building a functional transportation infrastructure is a high priority for a nation, particularly for a developing country like ours, where many of the towns, small cities and villages are not networked.

It is being noted from the Ethiopian Roads Authority report that, the Ethiopian government is giving special attention for road construction sector and is to invest billions of dollars. Parallel to road construction there is also a demand of construction of bridges as most of the existing ones are constructed many years before, during colonization by Italians. Thus, the construction of bridges in most cases requires demolishing and replacing of the bridges by new one which naturally occupy longer construction time. The time required may be farther extended due to shortage of budget beside having poor construction management which is observed by many contractors undertaking such jobs. Associated with this, the normal day to day traffic flow is affected and causes discomforts to the road users. This problem is serious especially in towns where there is high traffic volume and provision of detours is not simple. The cases for instance in Addis Ababa can apparently show such prevalence.

Therefore, any other alternative which could shorten the construction time and alleviate the problems is thought in this study.

In this regard, the steel concrete composite bridges can be constructed in relatively faster time with other added advantages as compared to the conventional reinforced concrete slab deck girder bridges.

Composite construction became the norm for concrete steel bridge construction since 1950 upon the development of shear connectors which made it practicable to connect the slab to the beam so as to obtain the T-beam action that had long been used in concrete construction.

Therefore, in this thesis, attempt is being made to compare the usage of locally produced steel profile in steel concrete composite bridges with the conventional reinforced concrete deck girder bridges cost wise, beside evaluating other structural advantages. Generally, the merits and demerits based on the current market condition, construction skill, and availability of construction materials will be identified.

1.2 OBJECTIVES OF THE STUDY

General objectives:

The main objective of the thesis is to design and compare the usage of locally produced profile in steel concrete composite bridges in Ethiopia with the conventional reinforced concrete deck girder bridges and evaluate the feasibility of such construction. Different span bridges are analysed and designed, quantity for the construction computed and cost compared. Besides, the structural viability will be evaluated where the merits and demerits identified accounting the current market condition, construction skill, and availability of construction materials.

Finally conclusions and recommendations will be formulated.

Specific objectives:

The following are the specific objectives of the study:

- Assess availability of construction materials and construction skill for steel concrete composite bridges.
- Identify span of bridges for which composite bridges are economical.
- Suggest construction methodologies.

1.3 CONTENETS OF THE THESIS

The thesis is organized in chapters details of which are arranged as follows.

- Chapter one deals with the introductory section of the study where the background, objective and contents of the study are presented.
- Chapter two briefs review of historical back ground of steel concrete composite construction; type and section property of materials used and construction methodologies and design considerations.
- Chapter three discusses the availability of local steel profiles for steel concrete composite construction and steel profiles section properties to be used in steel concrete composite bridges as given in design manuals.
- Chapter four addresses the design of the conventional reinforced concrete deck slab girder bridges and steel concrete composite bridges.
- Chapter five is dedicated for cost analysis and comparison of the designed bridges.
- Chapter six presents the conclusions drawn from the study and forwards recommendations.
- In the appendix attached sample design for steel concrete composite bridges and raw material and final product technical specification of Kality Metal Products Factory.

2.0 ABRIEF REVIEW OF STEEL-CONCRETE COMPOSITE

2.1 HISTORICAL BACK GROUND

Authors, [5] from the US consulting firm have traced the very early use of composite construction in North America. The year 1894 is stated as the period in which concrete encased beams were first used in a bridge in Iowa and a building in Pittsburgh. The earliest laboratory tests on encased columns took place at Columbia University in 1908, whilst composite beams were first tested at the Dominion Bridge Works in Canada in 1922. By 1930 the New York City building code recognized some benefit of concrete encasement to steelwork by permitting higher extreme fiber stresses in the steel parts of the encased members.

Welded shear studs were first tested at the University of Illinois in 1954, leading to publication of a design formula in 1956 and first use for both bridge and building projects that same year. In 1951 a partial interaction theory was proposed, also by the team from Illinois. Metal decks first appeared in the 1950s, with the first recorded use of through deck stud welding on the Federal Court House in Brooklyn in 1960. It was not until 1978, however, that this arrangement was recognized in the AISC specification. Early usage in Japan has been recorded by Wakabayashi, who refers to the use of concrete encasement to improve both fire and earthquake resistance, dating from about 1910. Termed “steel reinforced concrete” or SRC, this form of construction quickly became popular for buildings of more than 6 storeys. Its integrity was demonstrated by the good performance of structures of this type in the great Kanto earthquake of 1923. Research on the topic in Japan did not start until the 1930s; codes came much later, with the first SRC code produced by the Architectural Institute of Japan (AIJ) appearing in 1958.

Work on a design guide for composite bridges started in 1952 but the document was not published until 1959. Beginning with the earliest beam tests in 1929 and column tests in the

same year, research studies flourished in the 1950s and 1960s, leading to a heavy concentration since then on understanding the behavior of composite construction when subjected to seismic action.

One of the earliest substantial documents devoted to composite construction was the book by Viest, Fountain & Singleton. Published in 1958, it referred to early usage of the technique in the United States by 1935 and a patent by Kahn dated 1926. The book was written to complement the American Association of State Highway Officials (AASHO) 1957 Specification that covered the use of composite beams for bridges. In addition to referring to several North American examples of composite bridges constructed in the 1940s and 1950s, it includes an Appendix outlining the use of composite beams in building construction. A full set of Rules covering the design of composite beams was provided in the 1961 American Institute of Steel Construction (AISC) Buildings Specification.

Parallel developments had been taking place in Europe—especially as part of the post-war reconstruction in Germany. Reporting on this in 1957, Godfrey refers to “research in Germany, Switzerland and elsewhere” providing the basis for their “Provisional Regulations for the Design of Girders in Composite Construction” published in July 1950. Four years later the topic was addressed more formally in DIN 1078. In a follow-up paper Sattler reported on numerous examples of the use of composite construction for both bridges and buildings in Germany. This extended to the use of concrete filled tubes as columns but for beams relied on far more complex forms of shear connection than the welded studs which were at that time already being introduced into the United States by the Nelson company. In the discussion to these papers, British researchers Chapman & Johnson referred to both research in progress and buildings under construction that had been designed compositely at Imperial College and Cambridge

University. Reports on this work appeared a few years later, papers on cased stanchions, early UK composite bridge applications and background studies for buildings have appeared in the late 1950s,[5].

2.2 MATERIALS FOR STEEL- CONCRETE COMPOSITE STRUCTURES

The term “composite ” is normally understood within the context of buildings, bridges and other civil engineering structures to imply the use of steel and concrete formed together into a component in such a way that the resulting arrangement functions as a single item.

These materials can be used in mixed structural systems, for example concrete cores encircled by steel tubes, as well as in composite structures where members consisting of steel and concrete act together compositely.

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. The aim is to achieve a higher level of performance than would have been the case had the two materials functioned separately.

Structural steel, shear connectors, concrete, reinforcement bars are therefore the different materials used in steel concrete composite bridges. Though it is not a common practice in bridge construction, profile steel sheeting is widely used in composite slab construction in buildings. As shown in Figure 2.1, concrete is cast directly on top of these metal decks and provide permanent

form work during the curing operation and act as bottom reinforcements to the slab spanning transverse between the beams.

Thus, the design must recognize inherent differences in properties of the materials and ensure that the structural system properly accommodates these. Some form of interconnection is clearly necessary.



Figure 2.1: Use of metal decking

When designing composite elements, it is usual to adopt the same properties for steel and for concrete as would be the case when designing structural steelwork or reinforced concrete. Thus, codes of practice covering composite construction, such as EC4, normally simply summarize the relevant sections from the steelwork and concrete documents —EC3 and EC2 in the case of EC4, [5].

2.2.1 SHEAR CONNECTORS

2.2.1.1 TYPE OF SHEAR CONNECTORS

There are many types of shear connectors and they are most generally divided into rigid and flexible, according to the distribution of shear forces and functional dependency between strength and deformations (See Figure, 2.2).

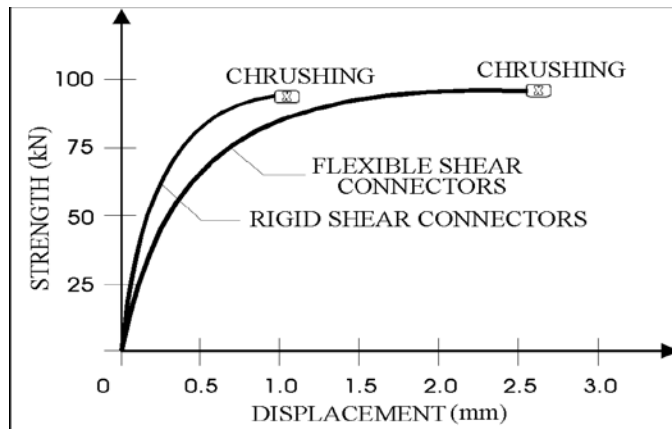


Figure 2.2: Functional dependencies between strength and displacement

a) THE RIGID SHEAR CONNECTORS

Rigid shear connectors consists of short length bars, tees, stiffened angles or channels welded on to the flange of the steel beams. This type of connectors resists shear forces through the front side by shearing, and they have insignificant deformations in the proximity of ultimate strength. They produce stronger concentrated stress in the surrounding concrete that results either in failure of concrete or in failure of weld.

In other words, rigid shear connectors can be defined as those which fracture when the ultimate load of the connector is reached without any significant slip; in this case the subsequent shear resistance falls suddenly to zero.

b) FLEXIBLE SHEAR CONNECTORS

Flexible shear connectors consist of studs, angles, channels or tees welded to the top flange of the steel beams.

Unlike the rigid shear connectors, flexible shear connectors resist shear forces by bending, tension or shearing in the root, at the connection point of steel beam, where they are subject to plastic deformations when they reach the ultimate strength values. The manner of failure of flexible shear connector is more ductile and is not prompt, [3].

2.2.1.2 THE FORCES APPLIED TO CONNECTORS

Clearly, if longitudinal shear resistance is provided by some form of connection, so that the stresses at the interface of the two materials are coincident, then the beam acts as a fully composite section.

In practice, connectors are never fully rigid, and there is always some slip between the slab and the steel section. The flexibility of the connectors allows more ductility and a variation in the distribution of longitudinal shear between slab and steel section. At ultimate load, when the plastic hinge has formed, it is likely that the end connectors will have deformed to a considerable extent and yet still be expected to carry a high longitudinal shear load. Hence, the requirement that shear connectors must have substantial ductility to perform adequately.

In determining the resistance of the beam, it is assumed that all the connectors, even when deformed, will be capable of resisting a longitudinal shear force. It is this shear resistance of the connectors that determines the resistance of the beam. If sufficient connectors are provided to withstand the longitudinal shear force generated when the full plastic resistance of the beam is developed, the beam is said to be "fully connected". It is also possible to reduce the amount of

connection so that the moment resistance of the beam is limited accordingly; this is a resistance criterion and the beam is termed "partially connected".

The slip that occurs as the connectors deform has a profound effect upon the stiffness of the beam. Very flexible but strong connectors may allow high bending resistance but because of the substantial slip there will be a loss of stiffness. The stiffness of the connection, in relation to the stiffness of the steel section and concrete slab, is often termed the interaction. Consequently, a beam where the connectors are infinitely stiff is said to have "full interaction" and one where the connection is relatively flexible is said to have "partial interaction".

It may be deduced that the strength and stiffness of both connector and concrete will affect the connection. The major force acting on the connector is one of direct shear. The shear force is generally assumed to be greatest at the level of the weld between the steel section and the connector itself. The area and shear strength of the connector and weld must, therefore, be adequate to carry the forces generated. It is unlikely that any substantial deformation will take place due to this shear.

However, relative movement between the slab and steel section does occur. The mechanism for this movement can be seen in Figure 2.3. The concrete may crush at the connector base allowing some deformation of the connector itself. However, at the head of the connector the confining concrete is not so highly stressed and this part of the connector remains in its original position. The result is bending deformation in the connector, which can be seen clearly in Figure 2.3. Long connectors are more likely to deform into this characteristic "S" shaped pattern and therefore tend to be ductile. Short stocky connectors tend to be brittle and are therefore

undesirable. Most codes of practice require stud connectors to be at least three or preferably four times longer than their diameter.

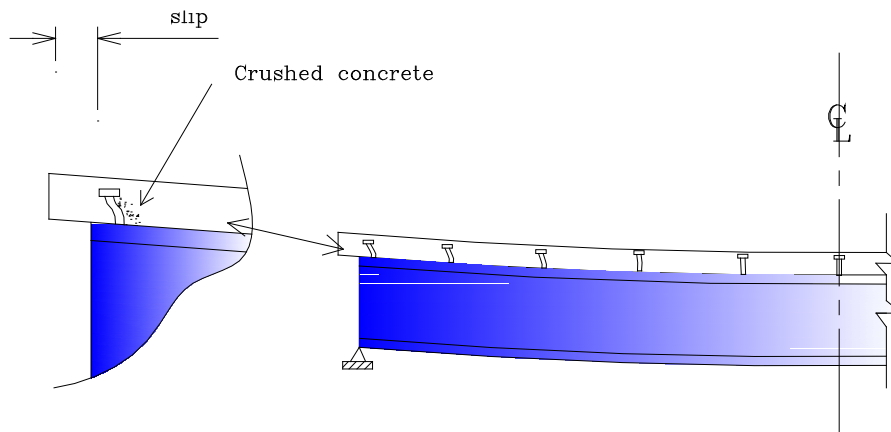


Figure 2.3: The Deformation of Flexible Connectors

The major force resisted by the concrete is one of bearing against the leading edge of the connector. It has already been mentioned that the concrete in this region is likely to crush allowing bending deformation to occur in the connector. The bearing resistance of the concrete in this region is dependent upon its volume as well as strength and stiffness. In fact, where there is sufficient concrete around the connector, the bearing stress may reach several times the unconfined crushing strength of the concrete. There is also likely to be direct tension in the connector. The different bending stiffness of the slab and the steel section, coupled with the deformed shape of the connectors, gives rise to the tendency for the slab to separate from the steel section. It is, therefore, usual for connectors to be designed to resist this tensile force.

In most composite beams the connectors are spaced along the steel section and, therefore, provide a resistance to longitudinal shear only locally to the top flange. The longitudinal shear force must, therefore, be transferred from the narrow steel section into the much wider slab. This transfer is normally achieved using bar reinforcement that runs transverse to the beam line.

These bars are normally placed below the head of the stud and extend into the slab, as shown in Figure 2.4, [7].

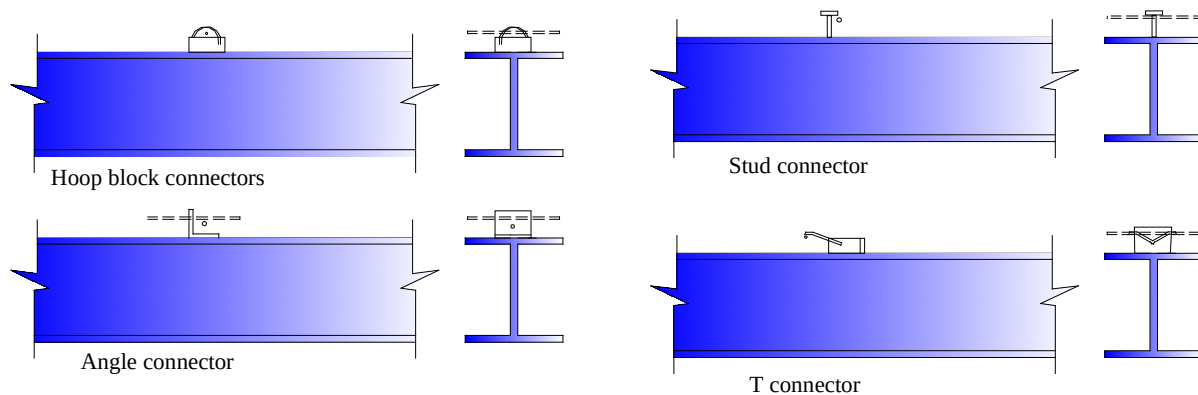


Figure 2.4: Types of connector

2.2.1.3 PROPERTIES OF SHEAR CONNECTORS

The property of a shear connector most relevant to design is the relationship between the shear force transmitted, P , and the slip at the interface, S . The load -slip curve should ideally be found from tests on composite beam, but in practice a simpler specimen is necessary. Most of the data on connectors have been obtained from various types of 'push-out' or 'push' test. The flanges of a short length of steel I-section are connected to two small concrete slabs. The details of the 'standard push test' per Euro code 4 are shown in Figure 2.7. The slabs are bedded onto the lower platen of a compression-testing machine or frame, and load is applied to the upper end of steel section (See Figures 2.5 and 2.6) .

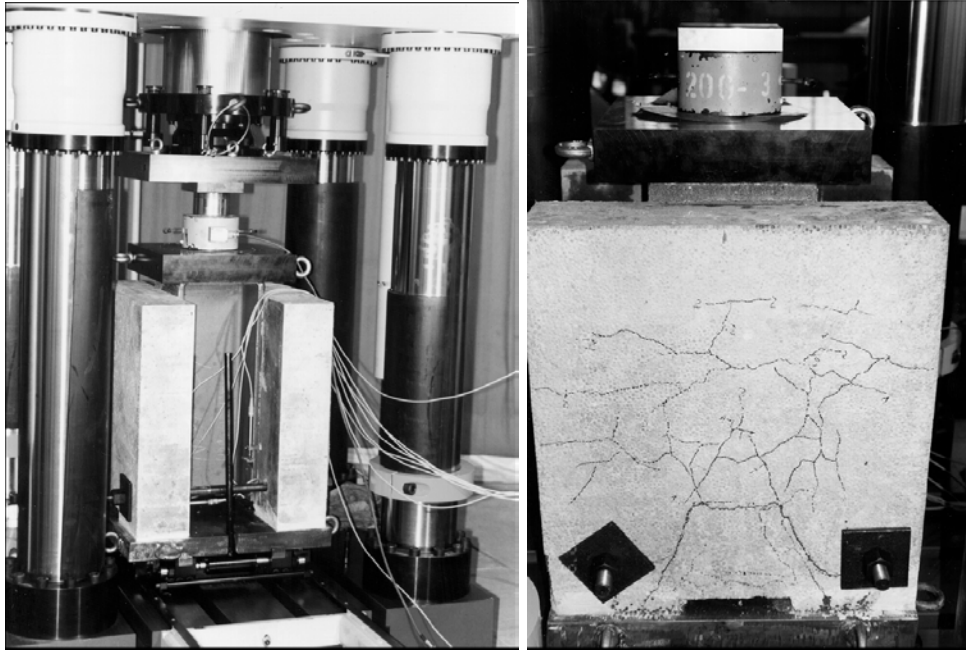


Figure 2.5: Test Specimen for Push Out



Figure 2.6: Uncovered Specimen after test

In practice, designers normally specify shear connectors for which strengths have already been established, for it is an expensive matter to carry out sufficient test to determine design strength for new type of connector. If reliable results are to be obtained, the test must be specified in detail, for the load slip relation ship is influenced by many variables, including:

- 1) Number of connectors and test specimen,
- 2) Mean longitudinal stress in the concrete slab surrounding the connectors,
- 3) Size and arrangements, and strength of slab reinforcement in the vicinity of the connectors
- 4) Thickness of concrete surrounding the connectors,
- 5) Freedom of the base of each slab to move laterally, and so to impose uplift forces on the connectors,
- 6) Bond at the steel -concrete interface,
- 7) Strength of the concrete slab, and
- 8) Degree of compaction of the concrete surrounding the base of each connector.

The details shown in Figure 2.7 include requirement relevant to items 1 to 6 listed above. The amount of reinforcement specified and the size of the slabs are greater than for the British standard test. Tests arranged as per Euro code gives results that are less influenced by splitting of the slabs, and so give better predictions of the behavior of connectors in beams.

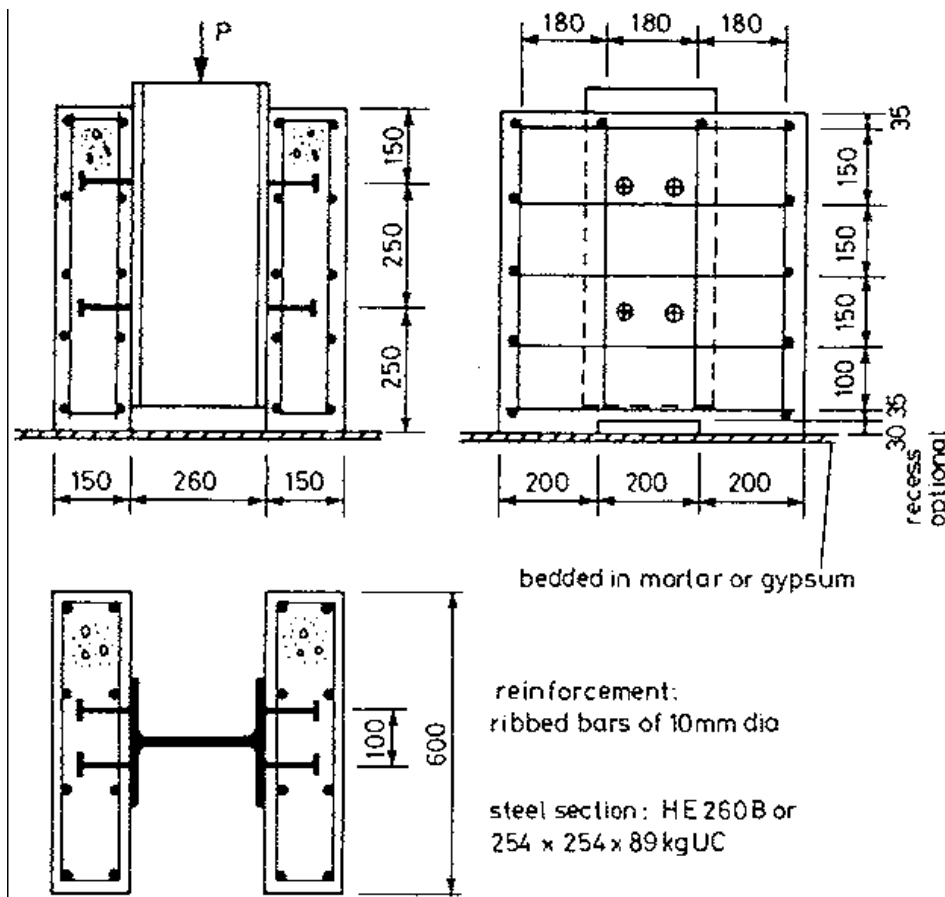


Figure 2.7: Arrangement of push out test

Slip between the steel member and the two slabs is measured at several points, and the average slip is plotted against the load per connector. A typical load-slip curve is shown in Figure 2. 8, from a test using composite slabs.

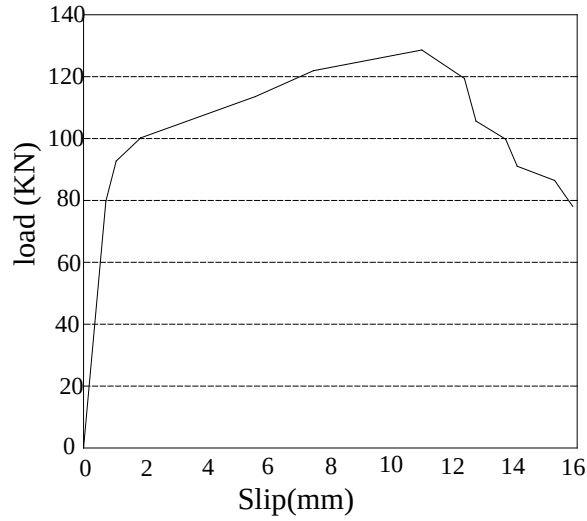


Figure 2.8: Typical load slip curve for headed stud

Tests have to be done for a range of concrete strengths because of the strength of concrete influences the mode of failure, as well as the failure load. Studs may reach their maximum load when the concrete surrounding them fails, but in strong concrete, they shear off. This is why the design shear resistance of studs with $h/d \geq 4$ is given in Euro code 4, as lesser of two values.

$$P_{RD} = \frac{0.8 f_u (\pi d^2 / 4)}{\gamma_v} \quad (2.1)$$

and

$$P_{RD} = \frac{0.29 d^2 (f_{ck} E_{cm})^{1/2}}{\gamma_v} \quad (2.2)$$

Where, f_u is the ultimate tensile strength of the steel (500 N/mm²), and f_{ck} and E_{cm} are the cylinder strength and mean secant (elastic) modulus of the concrete, respectively. Dimensions h and d are shown in Figure 2.9. The value recommended for the partial safety factor γ_v is 1.25,

based on statistical calibration studies. When $f_u = 450\text{N/mm}^2$, the first equation governs when f_{ck} exceeds about 30N/mm^2 .

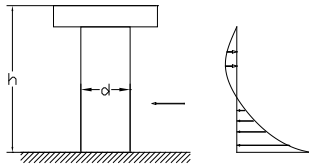


Figure 2.9: Bearing stress on shank of stud connectors

Ignoring γ_v , it is evident that equation 2-1 represents shear failure in the shank of the stud at a mean stress of $0.8f_u$. The results of push tests are likely to be influenced by the degree of compaction of the concrete, and even by the arrangement of particles of aggregate, in this small but critical region. This is thought to be the main reason for the scatter of the results obtained.

The load slip curve for a connector in a beam is influenced by the difference between the longitudinal stress in a concrete flange and that in the slabs in push test. Where the flange is in compression the load/slip ratio (the stiffness) in the elastic range exceeds the push test value, but the ultimate strength is about the same. For slabs in tension, the connection is significantly less stiff but the ultimate shear resistance is only slightly lower. This is one reason why partial shear connection is allowed such as in Euro code 4 only in regions of positive bending moment.

There are two situations in which the resistance of connector found from push tests may be too high for use in design. One, where there is a repeated loading, such as that due to the passage of traffic over a bridge. The other is where the lateral restraint to the concrete in contact with the connector is less than that provided in a push test, as in haunched beam with connectors too close to a free surface (See Figure, 2.10). For this reason, the use of standard equations for resistance of connectors is allowed in haunched beams only where the cross section of the haunch satisfies

certain conditions. In Euro code 4, these are that the concrete cover to the side of the connection may not be less than 50mm, [14].

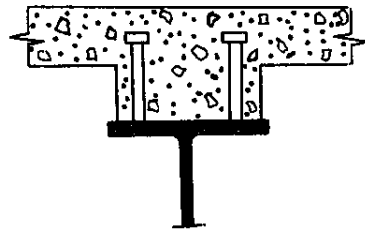


Figure2.10: Haunch with connectors too close to the free surface

2.2.1.4 SPACING OF SHEAR CONNECTORS

Shear studs can be spaced equally throughout the length of the stringer based on the shear range at support. Greater economy, however, can be realized by using variable spacing based on the horizontal shear along the length of the stringer. For a simple span bridge, this means that there will be a greater number of shear studs at the ends of the beam (i.e., a closer spacing (pitch) between rows of shear studs) than at the center of the span.

A further reason for the use of the partial factor γ_v , when determining the design resistance of connectors can be seen when the uniform spacing of connectors is examined in relation to the applied moment diagram. Figure 2.11 shows the applied bending moment for a uniform loading and the moment of resistance generated by the beam. The major component of bending resistance is provided by the composite action and, as the shear connectors are uniformly spaced, the resisting moment varies linearly from a maximum at mid span to a minimum at the support. The shaded portions of the applied bending moment diagram are, therefore, greater than the moment resistance of the beam.

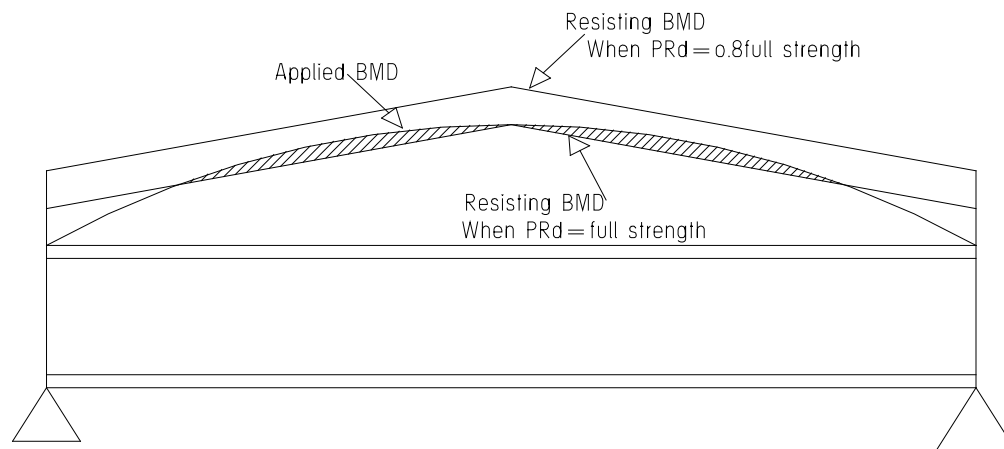


Figure 2.11: Applied and resisting bending moment diagrams for uniform loading and uniform connector spacing

If the connector resistance is assumed to be only 0.8 of its full value ($1/\gamma_v$), then the actual resistance moment generated will be increased sufficiently to ensure safety, [7].

2.3 METHODS OF STEEL-CONCRETE COMPOSITE CONSTRUCTION

2.3.1 GENERAL

Steel – concrete composite construction has become increasingly popular in advanced countries like USA and UK and is fast catching up in developing countries. Of all the steel – concrete composite construction, composite deck slab of bridges and floors of the buildings are very popular.

2.3.2 NON-COMPOSITE AND DECKS

When a deck is physically connected to and working with a superstructure load bearing member in resisting loads, it is said to be composite. If the deck is just resting on top of the superstructure and has no means of transferring longitudinal shear from primary members to the deck, then it

cannot assist in the resistance of bending moments induced by vehicle loading and is said to be non-composite.

From the above it is obvious why composite construction became the norm for concrete-steel bridge construction from the 1950's and onward.

Composite construction offers:

- o A more efficient use of materials since steel member size can be significantly reduced due to the incorporation of the deck into the resisting cross-section properties
- o Greater vertical clearance by a reduction in stringer depth
- o ability to sustain greater vehicle loading and to reduce live load deflection

Because of these significant advantages, composite construction is utilized whenever possible in an effort to maximize the performance and use of, not only different materials, but also different structural elements.

2.3.3 COMPOSITE ACTION

The use of two different materials to create a single structural member, known as composite or hybrid construction has its roots in the mid-nineteenth century. Today, most engineers tend to think of composite construction in the context of concrete and steel. In the 1840's, however, the use of cast iron and wrought iron or wood and wrought iron for truss bridges represented some of the first practical uses of composite construction in bridge design. One of the most popular types of Highway Bridge in use today is the composite steel beam bridge. A composite beam bridge utilizes a concrete slab deck which works together with steel girders in resisting loads placed on the bridge.

The new found economy of composite construction in the 1940's and 1950's led to the construction of thousands of composite steel beam bridges in the United States alone.

What is composite construction, though, and how does it offer such economy of materials? From basic strength of materials, the maximum stress in a beam subjected to pure bending is defined as

$$\sigma = \frac{Mc}{I} \quad (2.3)$$

Where σ = stress in beam

M = bending moment

c = distance from the neutral axis to extreme fiber

I = moment of inertia of resisting cross section

This is often redefined using the elastic section modulus which is dependent only on the geometry of the resisting cross section and is simply given as

$$\sigma = \frac{M}{S} \quad (2.4)$$

Where S = section modulus = I / c

From(2.4), we see that the bigger the value of S , the smaller the resulting stress. Therefore, it is in the best interest of the designer to increase the section modulus as much as possible. This is where the principal advantage of composite action comes into play. If a concrete slab simply rests on top of a steel beam, a phenomenon known as horizontal shear slippage occurs. As loads are placed on top of the slab, the top of both the slab and beam are in compression and the bottom of the slab and beam are in tension. In essence, both elements deflect like a beam, albeit independently. Since the bottom of the slab is in tension (i.e., pushing outward toward the ends of the beam) and the top of the beam is in compression (i.e., pushing inward toward the center of the beam), the resulting effect is one of the slab extending out over the ends of the beam.

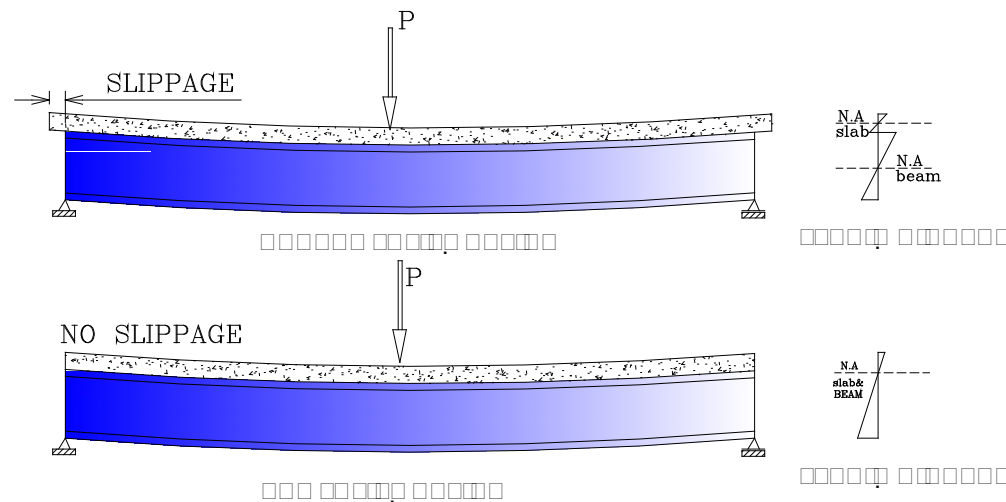


Figure 2.12: Slippage occurs at the slab-beam interface in non-composite beams.

In analyzing such a configuration, the slab and beam are treated independently, with the geometry of each element defining the neutral axis and moment of inertia of the slab and beam. Since the concrete slab alone has little strength compared with the stringers, the contribution of the strength from the deck is usually ignored for non-composite superstructures.

If the slab and beam, however, were somehow integrated, they could resist loads as a single unit as shown in Figure 2.12. In this arrangement, the neutral axis would be located somewhere in the middle of the section defined by the top of the slab and the bottom of the beam. With proper integration, the slab-beam would act as a unit with the top of the slab in compression and the bottom of the beam in tension and no slippage in between. This integration is accomplished through the incorporation of shear connectors between the slab and beam.

The shear connectors extend vertically from the top flange of the supporting beam and are embedded into the slab. Several of these connectors are placed along the length of the beam to prevent slippage which is caused by horizontal shear stress at the slab-beam interface. Figure 2.13 shows a row of shear studs attached to a steel plate girder



Figure 2.13: A shear stud installed at the top flange of a plate girder.

Shear connectors are the key ingredient to preventing slippage and creating composite action. Shear studs are the most common form of shear connector used today in composite concrete-steel beam bridges.

Early highway bridges made use of spiral shear connectors which consisted of a metal coil welded to the top flange of a steel beam. Installation of spiral connectors was an involved process generally not favored by contractors.

The automatic welding gun welds a stud to the top flange through use of an electric arc, which melts a welding material at the bottom of the stud to the steel beam (the effects of which can be seen clearly in the center stud of Figure 2.13). The shear stud's installation is facilitated by the use of an automatic welding gun as shown in Figure 2.14. This ease of installation and the relatively low cost of stud connectors have led to their popularity in composite construction.



Figure 2.14: Shear studs being welded on steel beam flange

For concrete girders, composite action is achieved by extending girder transverse stirrups into concrete deck to resist the horizontal shear stress between the deck and the girders. With shear connectors in place, the slab and beam can now be analyzed as a single unit. Coming back to Equation 2.4, this increases the size of the section modulus which in turn allows the composite beam to resist heavier loads. In essence, the I-shaped beam is replaced by a T-shaped cross section composed of the slab and stringer.

The way to calculate the bending capacity of composite concrete on steel stringers is similar to that of a reinforced concrete T section beam. Portion of the deck is in compression, and the steel stringer acts like reinforcing steel.

The economy of material realized by composite construction quickly led to its growth as a staple in highway bridge design. The decrease in size of stringers also meant that beams could now be

less deep than before. Another added benefit is that the connection between slab and stringer acts like a diaphragm between adjacent stringers to resist lateral deformation, [6].

2.3.4 ALTERNATIVE FORMS OF CONNECTION

Although welded shear studs are the major type of connector used, there are situations where alternatives are more applicable. For example, alternatives may be preferable on construction sites where only a small number of connectors are required and the hiring of expensive generators cannot be economically justified.

An alternative in these situations is the shot-fired connector shown in Figure 2.15. This connection can be fixed using modified cartridge guns which do not require any power supply. The connection itself fails when the pins pull out and is, therefore, weaker than the welded shear stud. When used in pairs these connectors can provide sufficient resistance for small beams and for beams designed with partial shear connection.

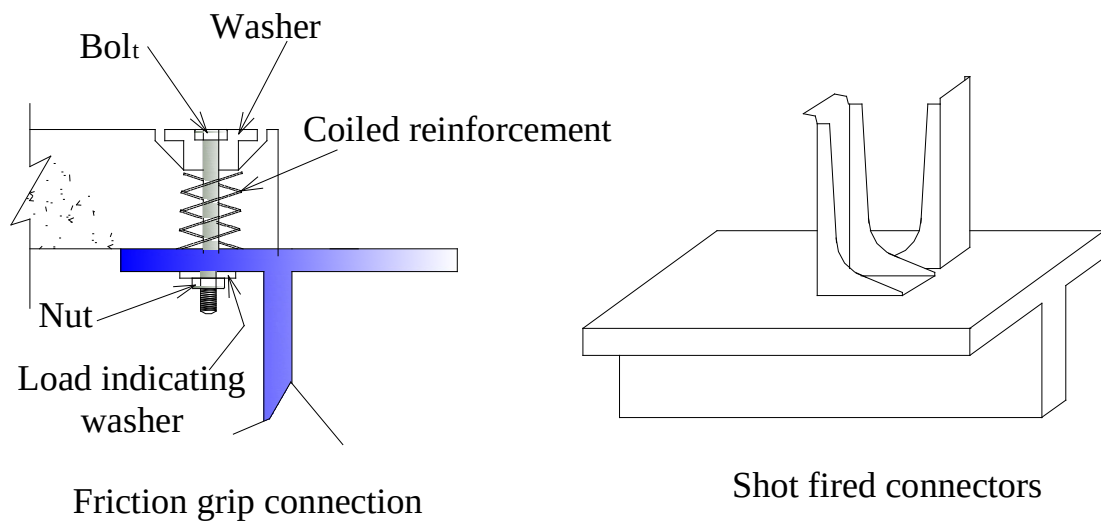


Figure 2.15: Alternative forms of connection

Another situation where welded connectors are inappropriate is the case where pre-cast concrete slabs are used compositely with the support beam. In this case high strength friction grip bolts (HSFG) have been used to fix the slabs to the steel flange, see Figure 2.15. HSFG bolts rather than simple bolts are needed so that, at the serviceability stage, no slip between the concrete slab and steel section can take place. Consequently, for serviceability design, the slip resistance of the HSFG bolt should be used and for the ultimate moment design, the bearing or shear resistance of the bolt is used,[7].

2.3.5 SHORED AND UNSHORED CONSTRUCTION

The dead loads on a slab-on-stringer bridge consist principally of the slab and beam itself. Since concrete has to cure for 28 days before it reaches full strength, the slab and beam cannot be considered to work compositely in resisting dead loads. This means that, until the concrete has hardened to a sufficient point, the steel stringer itself must resist all dead loads.

Shored construction minimizes the amount of dead load the beam has to carry by providing support at intermediate locations along the length of the span. Once the concrete has reached sufficient strength, the shores are removed and the concrete dead load is carried by the composite section.

Therefore, with shoring, smaller dead load deflection, and smaller dead load stress can be achieved. Conversely, unshored construction provides no support during casting concrete deck and expects the beam to resist all dead loads.

For most spans, the costs associated with erecting shoring to support the beam are greater than the savings in steel realized. Therefore, shored construction is not a common practice. In other instances, the provision of shoring may not be possible, [6].

2.4 DESIGN CONSIDERATIONS

2.4.1 SECTION PROPERTIES

Both elastic and plastic theories can be used for composite members. In plastic theories whole cross section of a member is assumed to be at yield, in either tension or compression.

Resistance of composite cross-section is determined using plastic analysis where ever possible, because plastic analysis is simple and leads to more economical design as compared to elastic analysis.

The basic difference of composite beams from the elastic behavior of reinforced concrete beams is that the steel section in composite beam is more than tension reinforcement, because it has a significant bending stiffness of its own. It also resists most of the vertical shear, [14].

2.4.2 CLASSIFICATION OF STEEL SECTIONS

Steel sections can be classified in to four, depending on local buckling behavior of flange and/or web in compression. Plastic design methods may be used for class1 and 2 sections; sections of class 2 are only allowed when limited rotation capacity is required. These classes are described as follows

Class 1, plastic cross-sections –can develop the full plastic capacity and demonstrate sufficient rotation so that plastic method of analysis can be used.

Class 2, compacted cross-sections –can develop the full plastic capacity however they have limited rotation capacity so that plastic analysis with limited amount of moment redistribution must be used.

Class 3, Semi compact cross-sections – could be stressed only up to the proportional limit of the extreme fibers because local buckling prevents mobilization of full plastic capacity.

Class 4, Slender cross-sections - buckle locally preventing the exploitation of even the yield limit. Their capacity is governed by local buckling.

The classification of a cross-section in one of the four classes is based on the c/t values of the flanges and the d/t values for the webs according to EBCS-3 Table 4-1. The less favorable class governs for the total cross-section.

For cross-sections under positive bending moment with plastic neutral axis within the concrete flange or top flange of the steel component are classified as class 1 irrespective of the flange width and thickness relationship.

2.4.3 Design of simply supported steel- concrete composite beam

Usually beams in bridges are designed as simply supported. Such beams have the following advantages over beams designed as continuous at supports.

- bending moments and vertical shear forces are statically determinate
- very little of steel web is in compression, and the steel top flange is restrained by the slab, so the resistance of the beam is not limited by buckling of steel
- webs are less highly stressed, so it is easier to provide holes in them for the passage of services
- there is no interaction between the behavior of adjacent spans
- Analysis are simpler, and design is quicker

2.4.3.1 ULTIMATE LIMIT STATE

A simply supported composite beam should be checked at critical sections in ultimate limit states. Considering a simply supported beam with constant cross-sections, two general terms must first be defined:

- "Critical cross-section" of the beam is the cross-section at maximum internal forces.
- "Critical length" of the beam is the length between two adjacent critical cross-sections.

For a simply supported spans for example, Length between the support and mid span is the critical length.

Referring the typical composite beam in Figure 2.16, the following are critical sections that should be considered in the ultimate limit states, [7].

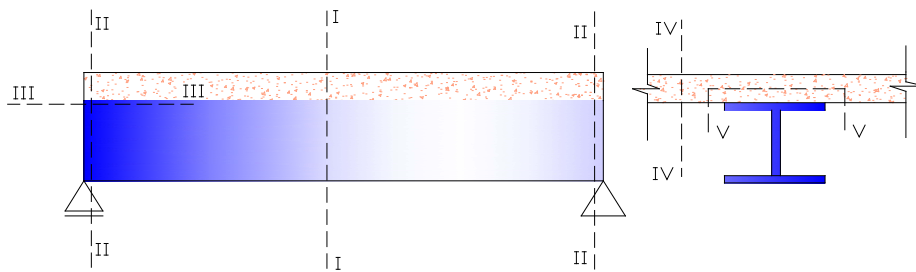


Figure 2.16: Critical sections of a simply supported composite beam

- at section I -I for moment capacity
- at section II–II for vertical shear
- at section III–III for longitudinal shear capacity at the interface of the steel and the deck slab.
- at section I and II for moment in combination with vertical shear
- at section V-V checking the surface area along the perimeter around the dowels (shear connectors)
- at section IV -IV longitudinal shear capacity of concrete flange.

2.4.3.2 FULL AND PARTIAL SHEAR CONNECTIONS

Considering any critical length of a simply supported beam, the maximum value of the total longitudinal shear force, V_{lf} , in the length can be determined by assuming that the plastic resistance moment is reached in the internal critical cross-section. Consequently, V_{lf} is equal to the lesser plastic axial resistance of either the steel member or the concrete slab in the critical cross-section concerned. The tension resistance of the steel member is equal to $A_a f_y / \gamma_a$ and the compressive resistance of the concrete slab is equal to $0.85 f_{ck} / \gamma_c$ times its area A_c (within the effective width b_{eff}). Therefore:

$$V_{lf} \leq \left[\begin{array}{l} A_a f_y / \gamma_a \\ 0.85 A_c f_{ck} / \gamma_c \end{array} \right] \quad (2.5)$$

It should be noted that the small effect of the longitudinal reinforcement in compression is usually neglected.

As each shear connector is able to transfer a force equal to its ultimate resistance P_{Rd} (assuming adequate ductility), the required number N_f of connectors to obtain full shear connection for the critical length in question is given by:

$$N_f = V_{lf} / P_{Rd} \quad (2.6)$$

If the number of shear connectors N within the critical length is less than the one required for full shear connection N_f , the critical length is in partial shear connection and its degree of connection is defined by the ratio N/N_f

$$N_{ratio} = N / N_f \quad (2.7)$$

Obviously, a beam is fully connected only if all its critical lengths are fully connected,[14]

2.4.3.3 DESIGN METHOD FOR PARTIAL SHEAR CONNCTION

For the purpose of design (assuming ductile connectors are used), it will be sufficient to prepare a reduced ultimate moment curve, without having to calculate the slip along the steel-concrete interface.

Reduced ultimate moment curve

The reduction in the ultimate moment of the internal critical cross-section, due to the reduction in the number of connectors, can be easily calculated. Assuming that each connector develops its design shear resistance, the total longitudinal shear force V_1 in each critical length is equal to the sum of the shear resistances of the connectors and also to the maximum compressive force $F_u^{(r)}$ in the slab:

$$V_1 = N P_{Rd} = F_u^{(r)} \quad (2.8)$$

Where $N < N_f$ (see Figure 2.17). The corresponding depth of the concrete stress block is equal to:

$$y_c = F_u^{(r)} / [0.85f_{ck}b_{eff} / \gamma_c] \leq h_c \quad (2.9)$$

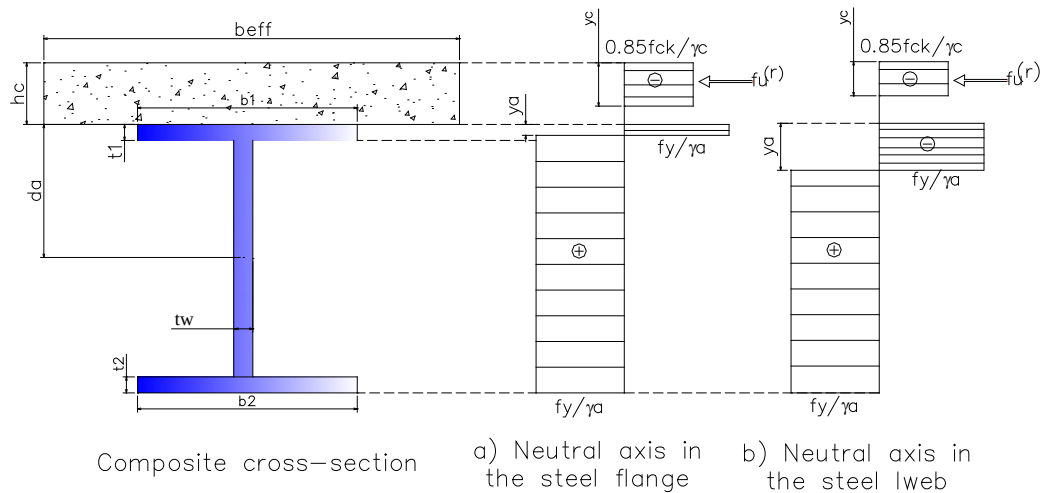


Figure 2.17: Plastic bending stress blocks with partial shear connction

The position of the neutral axis in the steel beam can be determined by equating $F_u^{(r)}$ to the tensile force in the steel beam, and the reduced ultimate moment $M_u^{(r)}$ can be calculated accordingly. The result is (see Figure 2.17)

- if the neutral axis is in the upper steel flange, its position y_a and the ultimate moment $M_u^{(r)}$ are given by the following relationships:

$$y_a = [A_s(f_y/\gamma_a) - F_u^{(r)}]/[2b_1(f_y/\gamma_a)] \text{ with: } y_a \leq t_1 \quad (2.10)$$

$$M_u^{(r)} = F_u^{(r)}(h_c + d_a - y_c/2) + 2b_1y_a(f_y/\gamma_a)(d_a - y_a/2) \quad (2.11)$$

Where d_a is half of the total height of the I profile.

- if the neutral axis is in the steel web:

$$y_a = \{(A_s + 2t_1t_w - 2b_1t_1)(f_y/\gamma_a) - F_u^{(r)}\}/[2t_w(f_y/\gamma_a)] \quad (2.12)$$

Where $t_1 \leq y_a \leq d_a$ and

$$M_u^{(r)} = F_u^{(r)}(h_c + d_a - y_c/2) + 2b_1t_1(f_y/\gamma_a)(d_a - t_1/2) + t_w(y_a - t_1)(f_y/\gamma_a)(2d_a - y_a - t_1) \quad (2.13)$$

Therefore, a well defined relationship between the flexural resistance $M_u^{(r)}$ and the number of shear connectors N per critical length, i.e. $M_u^{(r)}(N)$, can be deduced analytically. By introducing Equations (2.9) and (2.10) into (2.11) and also (2.8) and (2.12) into (2.13), it is clear that the variation of each function to obtain $M_u^{(r)}(F_u^{(r)})$, hence $M_u^{(r)}(N)$, is quadratic. Moreover, it should be noted that (see Figure 2.18):

- for $N = N_f$, the value of the moment $M_u^{(r)}$ deduced from Equation (2.11) is equal to the full plastic resistance moment M_u of the composite beam.

- for $N = 0$, the value of $M_u^{(r)}$ deduced from Equation (2.13) is equal to the plastic moment M_{apl} of the steel beam alone (in this case, $y_a = d_a$).
- for the particular value $N = N_1$ corresponding to the value $F_u^{(r)}$, when $y_a = t_1$, the continuity of the functions $M_u^{(r)}(N)$ and its first derivative is ensured.

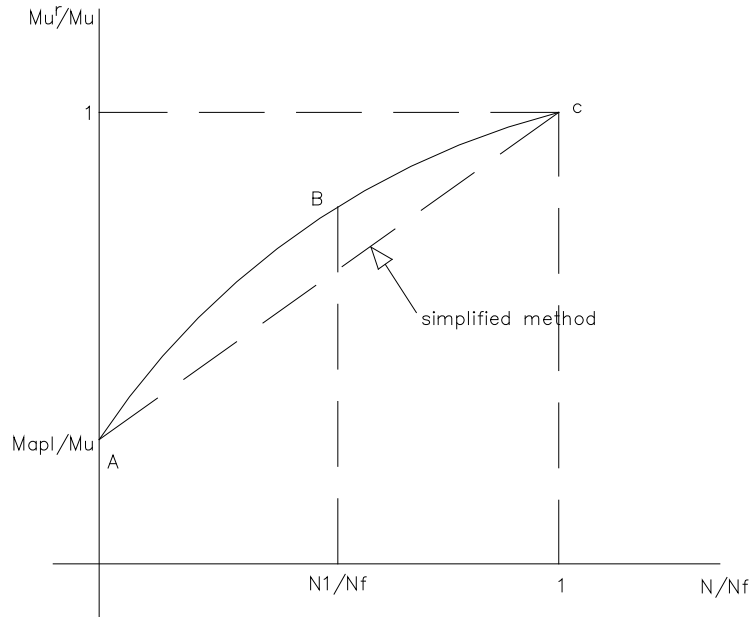


Figure 2.18: Design assumptions on the influence of degree of shear connection on moment capacity

Finally, the global relationship between $M_u^{(r)}$ and N can be represented by a convex curve ABC, as shown qualitatively in Figure 2.18. This relationship can also be considered in a non-dimensional form $(M_u^{(r)}/M_u, N/N_f)$.

The convexity of curve ABC allows an alternative simplified approach which is always safe. This approach defines the moment resistance $M_u^{(r)}$ in terms of a linear interaction with the degree of shear connection N/N_f , such that:

$$M_u^{(r)} = M_{uapl} + (N/N_f)(M_u - M_{uapl}) \quad (2.14)$$

This "linear interaction method" is represented by the dashed line AC in Figure 2.18. Generally, a significant benefit may be obtained by using the stress block method, Equations (2.10) - (2.13), in the range of $N/N_f = 0.5$ to 0.7 , [7].

2.4.3.4 VERTICAL SHEAR

Cross-section II of Figure 2.16 is only subjected to vertical shear. The contribution of the concrete slab to the resistance to vertical shear is small and difficult to determine and is, therefore, neglected. Therefore, only the web of the steel section and adjacent parts of the steel flange are taken into account. The vertical shear resistance, according to Euro code 3, is given by:

$$V_{pl,Rd} = A_v f_y / (\gamma_a \sqrt{3}) \geq V_{Sd} \quad (2.15)$$

Where A_v – is effective shear area

$$A_v = 1.04 \cdot h_a \cdot t_w \text{ -----for rolled section}$$

$$A_v = d \cdot t_w \text{ ----- for welded section}$$

h_a - total depth

d - is distance between flanges

t_w - web thickness

In addition, the shear buckling resistance of a steel web must be verified when $d/t_w > 69\epsilon$ for an unstiffened (and uncased) web,[11]. For a simply-supported beam, without intermediate transverse stiffeners, Eurocode 4 gives the following simplified rules:

$$\text{For } \bar{\lambda}_w \leq 1,5 \rightarrow V_{Rd} = V_{pl,Rd} \quad (2.16)$$

$$\text{for } 1,5 < \bar{\lambda}_w < 3,0 \rightarrow V_{Rd} = V_{pl,Rd} (3/\bar{\lambda}_w + 1/5 \bar{\lambda}_w - 1,3) \quad (2.17)$$

$$\text{for } 3,0 < \bar{\lambda}_w < 4,0 \rightarrow V_{Rd} = V_{pl,Rd} 0,9 / \bar{\lambda}_w \quad (2.18)$$

In accordance with Eurocode 3 the web slenderness $\bar{\lambda}_w$ is given by:

$$\bar{\lambda}_w = \{f_y / \sqrt{3} t_{cr}\}^{1/2} \leq 4 \quad (2.19)$$

Where t_{cr} is given by:

$$t_{cr} = \left(\frac{1000}{\frac{d}{t_w}} \right)^2 \quad (2.20)$$

2.4.3.5 INTERACTION OF BENDING MOMENT WITH SHEAR

When the design shear force V_{sd} exceeds 50% of the plastic shear strength, $V_{pl,Rd}$, then the influence of shear on the moment capacity should be taken into account. It is based on evidence from test that there is no reduction in bending resistance until $V_{sd} > 0.5 V_{pl,Rd}$ (pt.A in the Figure 2.19) and the assumption that the reduction at higher shears follows the parabola AB. At point B the remaining bending resistance $M_{plf,Rd}$ is that contributed by the flanges of the composite section including the reinforcement in the slab along the curve AB

$$M_{v,Rd} = M_{plf,Rd} + (M_{pl,Rd} - M_{plf,Rd})(1 - (2V_{sd}/V_{pl,Rd} - 1)^2) \quad (2.21)$$

Where, $M_{pl,Rd}$ is the resistance when $V_{sd} = 0$

When calculating $M_{plf,Rd}$, it is usually accurate enough to ignore the reinforcement in the slab.

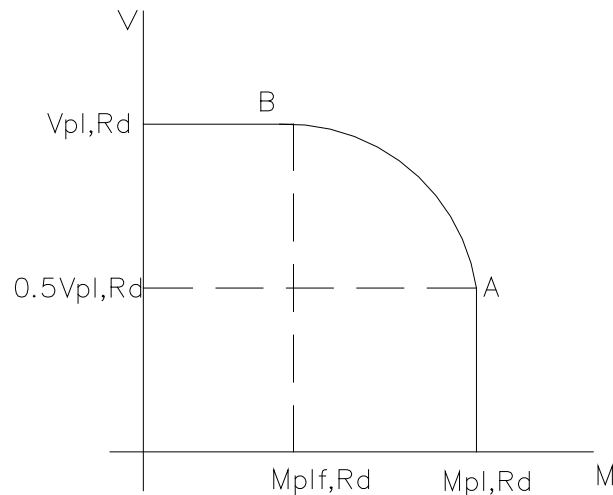


Figure 2.19: Resistance to combined bending and vertical shear

2.4.3.6 MAXIMUM LONGITUDINAL SHEAR IN THE CONCRETE SLAB

In the concrete slab, a complex (three-dimensional) force distribution occurs in the region of the connectors (as shown in Figure 2.16. critical sections IV and V). The reason for this behavior is that bending moments and vertical shear forces act parallel as well as perpendicular to the beam. It is difficult to find a physical design model for this complex stress distribution, and therefore, most design rules are empirical. However, it is possible to avoid these failure modes by providing sufficient transverse reinforcement and choosing the correct distance between the connectors. In some cases, satisfying these criteria may lead to an increase in concrete slab thickness or resistance, [7].

2.4.3.7 FATIGUE

AASHTO requires that shear connectors be designed to account for fatigue and checked for ultimate strength. Discussed below are the parameters governing fatigue criteria.

In the early days of composite construction, shear connectors were designed under a strict elastic approach. As horizontal shear varied along the stringer, shear connectors were spaced

accordingly. The composite steel-concrete beam was taken to act as a unit up to its ultimate strength with the overall goal being the elimination of slip at the slab-beam interface. This approach led to an overly conservative number of shear connectors being specified. Current practice bases the design of shear connectors on the ultimate flexural strength of a composite beam. Under such an approach, fatigue can become the governing criteria. Fatigue is caused by the repetitive loading and unloading of a structural member. Under an elastic approach for analyzing the effects of fatigue, it is the range of shear acting upon a connector rather than its magnitude of total shear which is considered. By a range of shear forces, we imply the difference between the maximum and minimum shear forces. The range of shear stress in the interface between stringer and the concrete deck is, [6]:

$$S_r = \frac{V_r Q}{I} \quad (2.22)$$

Where S_r = range of horizontal shear at slab-beam interface

V_r = range of shear due to live load plus impact

Q = first moment of the transformed slab area about neutral axis of the composite beam

I = moment of inertia of composite beam

For shear stud type connectors, this range of shear stress must be less than or equal to a corresponding allowable range of horizontal shear. This allowable range is given by

$$Z_r = \alpha d^2 \quad (2.23)$$

Where Z_r = allowable range of horizontal shear on one stud in N

α = constant based on number of stress cycles

d = diameter of stud in mm

So, the design of the composite beam's shear stud connectors should be such that

$$S_r \leq Z_r \quad (2.24)$$

2.4.3.8 Checking of the Serviceability Limit State

2.4.3.8.1 Deflections

a) Camber

The steel beams shall be fabricated with a camber so that beams in principle do not have any deflections without traffic load. The camber is calculated based on that the steel section carries the dead load before the concrete has hardened, [7]. For a simply supported beam span length L and uniform dead load W , deflection Δ_{DL} is given by

$$\Delta_{DL} = \frac{5WL^4}{384EI} \quad (2.25)$$

b) Deflection by traffic

As is the case in many standards of bridge design, the deflection due to traffic loading shall not exceed certain limiting values. For instance in accordance with ERA bridge design manual section 2.3 the live load deflection should be limited to $L/500$ of the span length. The deflection of the composite section for live load and impact shall be calculated based on effective moment of inertia.

3.0 LOCAL STEEL PROFILES FOR STEEL - CONCRETE COMPOSITE BRIDGE

3.1 AVAILABILITY OF LOCAL STEEL PROFILES

During the thesis work, assessment of availability of local steel profiles has been made to local private and governmental companies that are engaged in manufacturing, supplying and installing of steel profiles for different engineering purposes. Kality Metal Products Factory, Mesfin Industrial Engineering PLC and Maru Metal Industry are among some whose local products are considered. Accordingly, technical information including section properties, purpose, availability and cost for the purpose of the study as obtained from the respective companies are briefed below.

3.1.1 Kality Metal Products Factory

The company was established in 1961 E.C. in Addis Ababa and it manufactures standardized steel profiles including

- Square tubes of size ranging from 20x20mm up to 150x150mm with thicknesses of 1.2mm up to 5mm.
- Rectangular tubes of size ranging from 20x10 up to 160x100 with thicknesses of 1mm up to 4mm.
- Round tubes of size ranging from diameter 18mm up to 195mm with thicknesses of 1mm up to 5mm.

In each of the above steel profiles the average length produced is 6m. Besides to the standardized metal profiles, as per the need of customers, they produce C-channels by folding metal sheet mechanically. The C-channel is produced from a metal sheet of width 1m with a maximum thickness of 6mm and length as required.

The company also produces different steel profiles for door and window works and EGA sheets. They are mostly using the profiles for building construction. Table 3.1 shows technical specification of raw materials used in this company.

Table 3.1: Technical Specification of Raw Materials

Type	Weight of zinc coating	Yield (Mpa)	Tensile (Mpa)	Elongation %	Thickness and purposes
Cooled-Rolled pickled and oiled carbon steel sheet	-	220 Min.	275Min.	36Min.	From 0.8-1.5mm for furniture and general use
Hot-rolled carbon steel sheets	-	220 Min.	275Min.	31Min.	From 2.0-6.0mm for pressed products, structural hollow sections and general use
Hot-dip zinc coated galvanized coil sheet	200gm/m ²	250 Min.	410Min.	36Min.	From 0.3-2mm for pressed products, EGA sheets, Tankers

3.1.2 Mesfin Industrial Engineering PLC

Mesfin Industrial Engineering is one of the engineering companies in Ethiopia established in 1986 E.C.in Mekelle town.. It is having workshops in Mekele town and near Dukem town to serve the prevailing demand of their products. The firm provides the following technical service.

- Design, manufacture, supply and service of Low bed/High bed Dry /Liquid Trailers.
- Design, manufacture, supply and erecting of Petroleum Liquid Reservoirs.
- Manufacture and supply of steel fabricated products for industrial application.
- Manufactures a wide range of mobile and stationary crushing plants for the domestic and export market. It also manufactures concrete batching plants, mixers and other similar

construction equipments. Further more, it manufactures steel bridge and constructs steel buildings.

Regarding steel fabricated products, they produce welded I profiles having varied height and flange width by importing the raw metal sheet for rail way and bridge construction. Some of the products observed in a work shop of the company located near Dukem town are shown below. From the visit made, it is being learnt that use of automatic welding gun facilitates the welding operation and also the quality of works. In view of installation of shear studs, this is the best welding machine as discussed in section 2.3.3 of this text. There is also experience of using up to date equipments to control and check the welding quality.



Plate 3.1: Welded plate girders of I section ready to be dispatched to rail way site for steel bridge.



Plate 3.2: Metal sheet ready to manufacture I profile plate girder for steel bridge.



Plate 3.3: Raw metal sheets in store

MARU METAL INDUSTRY P.L.C

This company was established in 1967 E.C. in Addis Ababa and is engaged in the metal industry in manufacturing, marketing and trading of metal products. It also manufactures welded I section steel profiles that are used in building construction and vehicle body parts. The same manufacturing experience of these profiles is observed as in Mesfin Industrial Engineering P.L.C.

3.2 STEEL PROFILES FOR STEEL-CONCRETE COMPOSITE BRIDGES**3.2.1 DECK SLAB AND PANELS**

By far, the predominant form of deck in use is the cast –in –place reinforced concrete deck slab. In concrete - steel composite bridges the conventional slab can work with steel superstructures (primary members) creating composite action and essentially becomes an element in the superstructure component. The concrete slabs are taken continuous when supported over more than two steel stringers and the design takes in to account the continuity effect.

If properly maintained, a concrete slab will offer many years of service, even though it takes the most abuse out of any bridge component. To protect the concrete slab, a deck overlay can be used or apportion of the deck reserved to resist the wear and tear of traffic. Usually an asphalt concrete overlay rests on top of the deck that serves as a wearing surface, [6].

3.2.2 BEAMS

As mentioned above, in composite bridges, the superstructure consists of concrete slab deck which rests on a set of steel primary members connected by secondary members.

Steel primary members come in a variety of types, some of which are discussed below.

3.2.2.1. Rolled Beam.

The rolled beam is a steel girder which has been formed by hot-rolling. The most common type of rolled beam used as a primary member in highway bridges is the wide-flange variety. The wide flange differs from its I-beam cousin in that its flanges are parallel rather than tapered

3.2.2.2. Rolled Beam with Cover Plate.

To maintain an economy of material, rolled beams are sometimes equipped with a rectangular plate, or cover plate, at the bottom flange. The cover plate increases the ability of the stringer to resist flexure without having to use a larger size rolled beam or plate girder. As we will see, however, the cover plate also increases the potential for fatigue cracks by introducing welds and stress concentrations at the ends of the plate.

3.2.2.3. Plate Girder.

A plate girder, like a rolled beam, has an I-type cross section. Rather than being hot-rolled, however, the girder is constructed from steel plate elements which are connected together with welds, bolts, or rivets. For modern highway bridges, shop welding is the most predominant method. Since the designer is specifying the section properties of the stringer (i.e., flange width and thickness, web depth, etc.) a greater economy of materials results. To further reduce the amount of steel used, plate girders can be varied in depth, or hunched, to accommodate regions of low and high moment and/or shear. (See Figure 3.1.)



Figure 3.1: A haunched plate girder bridge for long spans

3.2.2.4. Box Girder.

In reality, the box girder is a form of plate girder which combines two girders into a single unit. A box girder may be designed as a four-sided box (closed box), or it may be designed as an open-box where a relatively narrow top flange is welded on top of each web. Closed box girders possess excellent torsional stiffness, so they usually do not require secondary members to provide bracing. For open boxes, bracing is usually required to provide torsional stiffness. Although the box girder provides an aesthetically pleasing structure, the amount of steel required, especially for closed box girders, can sometimes exceed that for a standard I cross-section plate girder. Box girders usually have higher fabrication costs than plate girders.

3.2.3 Secondary Members

Secondary members act as bracing for primary members. In general, secondary members are not load bearing elements but are designed to prevent cross-sectional deformation of the superstructure frame. In addition to this, secondary members provide for vertical load distribution between stringers by permitting the superstructure to work together as a unit.

Longitudinal spacing of secondary members varies depending on the type of primary member and the length of the span. Secondary members some times called diaphragms.

For steel superstructures comprised of rolled beam or plate girder primary members, a rolled beam diaphragm can be used. This rolled beam is usually of the channel or wide flange type cross-section. For rolled beam primary members, the diaphragm should be at least one-third to one-half the depth of the primary member. Diaphragms for plate girders should be one-half to three quarters the girder depth. When the primary member in a steel superstructure is exceedingly deep [approximately 48 in (1.2 m) or larger] or when curved girders are used, a cross frame style diaphragm is desirable. Cross frames are typically composed of steel angles in a cross (X) or vee (V) configuration. In steel box girder structures this type of bracing is used to increase the rigidity of the member. Diaphragms are typically connected to stringers with a connection plate. The connection plate is either bolted or welded to the primary member and the diaphragm, [6].

3.2.4 Section Property Requirements for Composite Bridges

For bridge construction, where EBCS 3 or 4 doesnot specifiy AASHTO Bridge Design Manual is referred. There fore, according to AASHTO Bridge Design Manual 2005 section 6.0, structural steel shall conform to the requirements in Table 3.2, Minimum mechanical properties of structural steel by shape, strength and thickness.

Table 3.2: Minimum Mechanical Properties of Structural Steel by Shape, Strength and Thickness.

AASHTO Designation	M 270M Grade 250	M 270M Grade 345	M 270M Grade 345S	M 270M Grade 345W	M 270M Grade HPS 345W	M 270M Grade HPS 485W	M 270M Grades 690/690W	
Equivalent ASTM Designation	A709M Grade 250	A709M Grade 345	A709M Grade 345S	A709M Grade 345W	A709M Grade HPS 345W	A709M Grade HPS 485W	A709M Grades 690/690W	
Thickness of plates, mm	Up to 100incl.	Up to 100incl.	Up to 100incl.	Up to 100incl.	Up to 100incl.	Up to 100incl.	Up to 65incl.	Up to 65incl.
Shapes	All groups	All groups	All groups	All groups	Not applicable	Not applicable	Not applicable	Not applicable
Minimum Tensile Strength, F_u , Mpa	400	450	450	485	485	585	760	690
Specified Min. Yield Point or Specified Min. Yield Strength, F_y , Mpa	250	345	345	345	345	485	690	620

3.2.4.1 Minimum thickness of steel

Structural steel including bracing, cross frames, and all type of gusset plates, except for webs of rolled shapes, closed ribs orthotropic decks, fillers, and in railings shall be not less than 8.0mm in thickness.

The web thickness of rolled beams or channels and of closed ribs in orthotropic decks shall not be less than 7.0mm. Where the metal expected to be exposed to severe corrosive influences, it shall be specially protected against corrosion or sacrificial metal thickness shall be specified.

3.2.4.2 Diaphragms and Cross-Frames

Diaphragms or cross frames may be placed at the end of the structure, across interior supports, and intermittently along the span. Diaphragms or cross frames should be as deep as practicable, but as a minimum should be at least 0.5 of the beam depth for rolled beams and 0.75 of the girder depth for plate girders.

3.2.4.3 Stud Shear Connectors

Shear connectors studs shall be made from cold-drawn bars and shall have a specified minimum yield and tensile strength of 345 Mpa and 415Mpa, respectively.

Finally, as is observed in this section where minimum strength requirements and dimensions steel profiles to be used in steel concrete composite bridges is discussed, those steel profile types as produced in Kalitty Metal Industry Factory cannot be used for such construction, while the welded plates of I section manufactured in the remaining companies can be used in steel - concrete composite bridge construction. There fore, in the design section of the study work these profiles are used.

3.3 Construction Aspect in Ethiopia.

So far, steel-concrete composite bridge construction is hardly exercised in Ethiopia. Starting from the period of Italian colonization, where modern bridges had been constructed, the common bridge types constructed were masonry and concrete arch bridges, reinforced concrete slab and concrete slab deck girder bridges, bailey bridges and steel bridges in rail ways. These bridges are briefly described as follows.

i. Arch Bridges

In our country, most of the bridges constructed in earlier times were masonry arch bridges. The masonry arch bridge over the Kebena River which is located in Addis Ababa can be taken as an example. It has been demolished and currently, construction is being carried out to replace by a new reinforced concrete deck girder bridge.

Regarding concrete arch bridges, in northern part of the country, many of these were constructed by the Italians.



Plate 3.4: Concrete arch bridge

ii. Reinforced Concrete Bridges

Now a days many of the bridges being constructed are reinforced concrete ones where commonly they are either slab or deck girder bridges.



Plate 3.5: Reinforced concrete deck girder bridge

iii. Bailey Bridges

Bailey bridges are standardized steel bridges. They can be installed in relatively shorter time once the bridge substructures are constructed. In our country, along some of the roads, these bridges are serving as a passage but recently they are being replaced by reinforced concrete ones. Usually bailey bridges are preferred for temporary passage especially where detours for bridge constructions are required.



Plate 3.6: Bailey bridge being used as temporary passage (detour)

iv. Steel Bridges

Usage of steel bridges, in our country, is common especially in railways for train passage.



Plate 3.7: Steel bridge in railway for train passage

4 DESIGN OF CONVENTIONAL CONCRETE SLAB DECK GIRDER AND STEEL-CONCRETE COMPOSITE BRIDGES

4.1 GENERAL

The design of those bridges follows the basic design principle of structural engineering that the available capacity of a structure must exceed the required capacity to support the applied loadings. The design method utilizes Load and Resistance Factor Design (LRFD) based on limit state according to ERA's Bridge Design Manual 2002 and AASHTO Standard Specifications for Highway Bridges 1998.

The different limit states that needs to be checked during the design includes

- Strength limit state where the resistance of bridge structural components to bending, shear, torsion and axial loads is checked.
- Service limit state that restricts stresses deflection and crack widths under regular service conditions.
- The fatigues limit which refers to a set of restrictions on stress ranges caused by design live loads.

4.2 DESIGN PROCEDURE

The design of superstructure of a bridge includes design of the deck slab and the supporting girders. Once the span and width of the bridge is decided, material type and qualities, number of girders and preliminary dimensions, which could be changed during the detailed design, should be specified.

In the design, the following loading conditions are considered

- Dead loads including Asphalt surfacing
- Live load HL-93 Loading as per ERA's Bridge Design Manual 2000
- Dynamic load allowances

- Seismic effects as per the Manual.

Accordingly, the factored design loads (shear force and bending moments) were calculated at different critical sections. The sections are then designed to withstand the computed design loads. Finally, dead load and live load deflections were checked against the allowable limits and camber is provided at middle of the span as per the calculated long term dead load deflection.

Like design of other structures, the design of the bridges superstructure is not one directional rather there is a possibility of going forth and back until design requirements are satisfied.

The following out line flow shows the design procedures and steps followed in the design of the bridges.

4.2.1 CONCRETE SLAB DECK GIRDER BRIDGES

The following major procedures are followed in the design of superstructure of the conventional concrete deck slab girders.

1) Design of the deck slab

a. Design Data and Specification

- Specify material properties, bridge width and spans, live loadings and design methods to be used.

b. Bridge cross-sections and preliminary dimensions

- Initial preliminary dimensions are assumed which usually bases past experiences.
- Take initial slab depths from minimum depth requirements.

c. Design of overhang slab

- Determine dead load moment, live load moment from design truck and then factored design moment.
- Provided main reinforcements for the factored design moment.

- Check whether the requirements of minimum and maximum reinforcement are full field or not, which other wise needs to revise section or material properties.

d. Design the deck slab(interior)

- Determine dead load moment, live load moment from design truck plus lane load or tandem plus lane load and then factored design moment.
- Provided main reinforcements for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which other wise needs to revise section or material properties.
- Provide distribution, temperature and shrinkage reinforcements.

2) Design of longitudinal girders (interior and exterior)

- Based on the preliminary bridge cross-sections and preliminary dimensions assumed, calculate girder dead loads and moments.
- Determine dynamic load allowances.
- For transverse load distribution, specify moment and shear distribution factors.
- Calculate live load moments and shear which is the maximum of design truck plus lane loading or tandem load plus lane loading.
- Calculate factored design moments and shear.
- Design for flexure and prepare bar cutting using moment curves.
- Provide camber for dead load deflection.

4.2.2 STEEL-CONCRETE COMPOSITE BRIDGES

The following procedure is followed in the design of superstructure of steel-concrete composite bridges.

1) Design of the deck slab

e. Design Data and specification

- Specify material properties, bridge width and spans, live loadings and design methods to be used.

f. Bridge cross-sections and preliminary dimensions

- Initial preliminary dimensions are assumed which usually bases past experiences.
- Take initial slab depths from minimum depth requirements.

g. Design of overhang slab

- Determine dead load moment, live load moment from design truck and factored design moment.
- Provided main reinforcement for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which other wise needs to revise section or material properties.

h. Design the deck slab(interior)

- Determine dead load moment, live load moment from design truck plus lane load or tandem plus lane load and factored design moment.
- Provided main reinforcement for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which other wise needs to revise section or material properties.
- Provide distribution, temperature and shrinkage reinforcements.

2) Design of longitudinal girders (interior and exterior)

- Based on the preliminary bridge cross-sections and preliminary dimensions assumed above, calculate girder dead loads and moments.
- Determine dynamic load allowances.

- For transverse load distribution specify moment and shear distribution factors.
- Calculate live load moments and shear which is the maximum of design truck plus lane loading or tandem load plus lane loading.
- Calculate factored design moments and shear.
- Check class of steel profile.
- Determine moment section capacity of beams for specified shear connector ratios and their shear strength. Then, check against design values which other wise requires revising material or section properties.
- Check interaction of bending moment with shear.
- Check deflection requirements and provide camber for dead load deflection.

5. COST ANALYSIS AND COMPARISON

GENERAL

Before the cost analysis and comparison, four span lengths are selected and design is carried out for both steel-concrete composite bridges and reinforced concrete slab deck girder bridges. The chosen spans are 12m, 15m, 20m and 24m. The same width of bridge is considered in each case; 8m carriage way width and a 1.5m walk way in both directions which is a common typical section in towns.

The following material qualities are used in the design as it can be referred to the sample designs attached in the appendix part of the thesis.

- Concrete: C-30.
- Reinforcement steel : $f_y = 420\text{Mpa}$ for diameters 20mm and above other wise 300 Mpa.
- Structural steel for girders: $f_y = 345\text{Mpa}$
- Steel for shear connectors: $f_u = 450\text{Mpa}$

Based on the design out puts, quantities of each of the items are collected and summarized.

The unit rates used in the cost analysis, especially those of the concrete, reinforcement steel and formworks are obtained from actual projects undertaking and average values are taken considering the current market conditions too. Unit rate of structural steel are obtained from the steel industries contacted and the values are those of the projects they are currently involved.

STEEL- CONCRETE COMPOSITE BRIDGES

The transverse section of the bridge considered in the design is given in Figure 5.1 and dimensions of the structural elements are summarized in Tables 5.1. The dimensions are obtained from the design calculations; in case of diaphragms, which are not primarily meant to

carry load i.e. bracing action, dimensions are provided based on minimum requirements of standards. The same is true for connection plates.

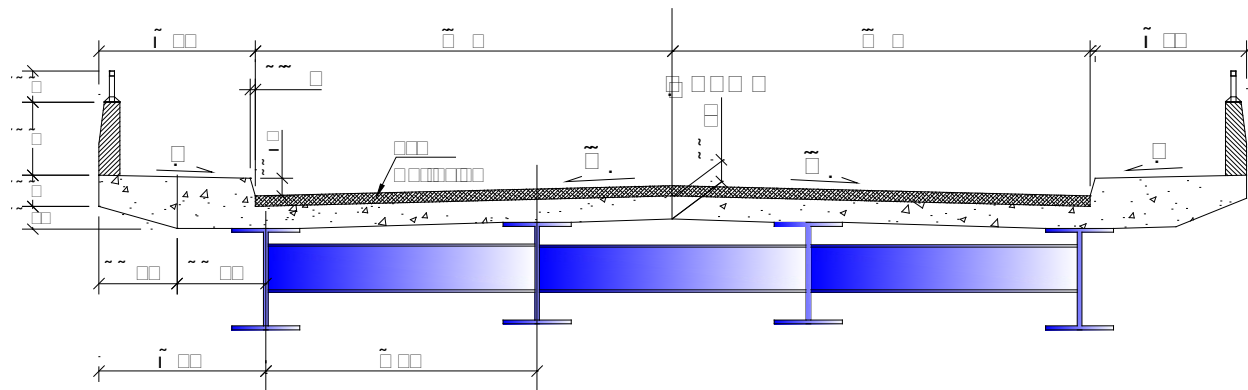


Figure 5.1 Cross-section of the Steel Composite Bridge

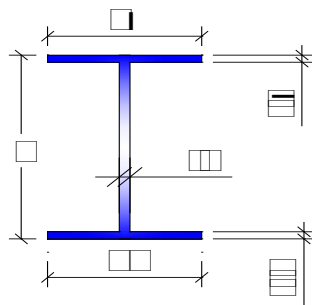


Figure 5.2 Designations for welded I profile beam section

Table 5.1: Design Dimensions of the Steel-Concrete Composite Bridges

Designations	Span 12m		Span 15m		Span 20m		Span 24m	
	Main Girder	Diaphragm	Main Girder	Diaphragm	Main Girder	Diaphragm	Main Girder	Diaphragm
b_1 (mm)	200	150	250	150	300	150	390	150
b_2 (mm)	200	150	250	150	300	150	390	150
t_{f1} (mm)	14	8	16	8	20	8	25.5	8
t_{f2} (mm)	14	8	16	8	20	8	25.5	8
t_w (mm)	12	8	13	8	16	8	16.5	8
h (mm)	700	520	740	550	890	660	970	720
Total number of connecting plates	24 L angle 140x140x8mm length =502mm		24 L angle 140x140x8mm length =534mm		24 L angle 140x140x8mm length =644mm		24 L angle 140x140x8mm length =704mm	
Total number of shear stud connectors	675 ϕ 24, length 130mm two shear studs in transverse		894 ϕ 24, length 130mm two shear studs in transverse		1033 ϕ 24, length 130mm two shear studs in transverse		1315 ϕ 24, length 130mm two shear studs in transverse	

The quantities and respective costs of the composite bridges are summarized in Tables 5.2, 5.3, 5.4 and 5.5.

Table 5.2: Cost Evaluation for 12m span Steel-Concrete Composite Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m ³)	41.71	2,950.00	123,015.00
Reinforcement Bar (Kg)	6,182.65	29.00	179,296.85
Formwork (m ²)	142.10	200.00	28,420.00
Steel (Kg)	6,970.41	40.00	278,816.40
Total			609,548.25

Table 5.3: Cost Evaluation for 15m Span Steel-Concrete Composite Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	51.82	2950.00	152,869.00
Reinforcement Bar(Kg)	7,686.17	29.00	222,898.93
Formwork (m ²)	173.40	200.00	34,680.00
Steel (Kg)	10,130.74	40.00	405,229.56
Total			815,677.49

Table 5.4: Cost Evaluation for 20m Span Steel-Concrete composite Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	68.68	2950.00	202,606.00
Reinforcement Bar(Kg)	10,073.77	29.00	292,139.33
Formwork (m ²)	225.62	200.00	45,124.00
Steel (Kg)	18,894.83	40.00	755,793.20
Total			1,295,662.53

Table 5.5: Cost Evaluation for 24m Span Steel-Concrete Composite Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	82.17	2950.00	242,401.50
Reinforcement Bar(Kg)	12084.56	29.00	350,452.24
Formwork (m ²)	261.08	200.00	52,216.00
Steel (Kg)	29526.11	40.00	1,181,044.40
Total			1,826,114.14

REINFORCED CONCRETE SLAB DECK GIRDER BRIDGES

The transverse section of the bridge considered in the design is given in Figure 5.3 and dimensions of the structural elements are summarized in Tables 5.6.

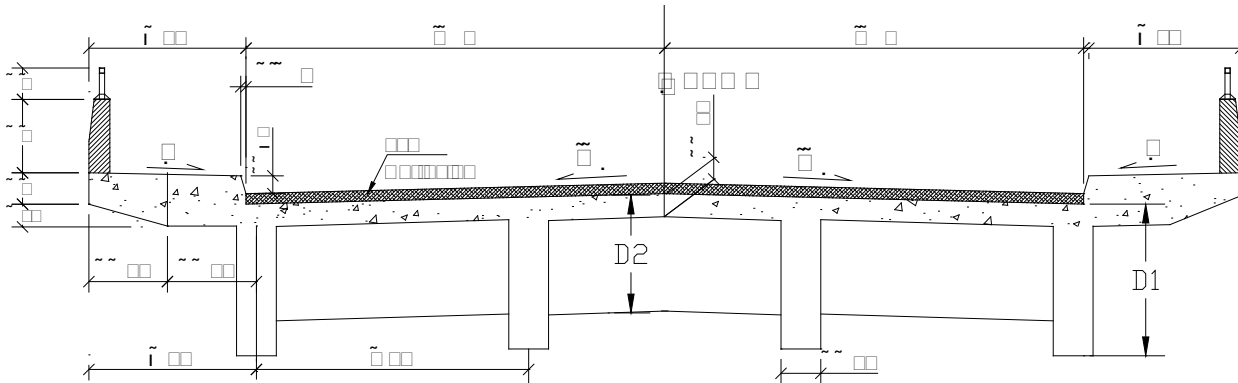


Figure 5.3 Cross-section of the Concrete Girder Bridge

Table 5.6: Design Dimensions of the Concrete Deck Girder Bridge

Designations	Span 12m	Span 15m	Span 20m	Span 24m
D1(m)	0.90	1.10	1.45	1.72
D2(m)	0.60	0.80	1.15	1.42
Diaphragm width(m)	0.25	0.25	0.25	0.25

The quantities and respective costs of the concrete girder bridges are summarized in Tables 5.2, 5.3, 5.4 and 5.5.

Table 5.7: Cost Evaluation for 12m Concrete Deck Girder Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m ³)	56.96	2,950.00	168,032.00
Reinforcement Bar(Kg)	9,921.91	29.00	287,735.39
Formwork (m ²)	275.6	200.00	55,120.00
Total			510,887.39

Table 5.8: Cost Evaluation for 15m Span Concrete Deck Girder Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	76.39	2,950.00	225,350.50
Reinforcement Bar(Kg)	13,098.73	29.00	379,863.17
Formwork (m ²)	370.30	200.00	74,060.00
Total			679,273.67

Table 5.9: Cost Evaluation for 20m Span Concrete Girder Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	113.51	2,950.00	334,854.50
Reinforcement Bar(Kg)	19,713.10	29.00	571,679.90
Formwork (m ²)	549.70	200.00	109,940.00
Total			1,016,474.40

Table 5.10: Cost Evaluation for 24m Span Concrete Girder Bridge

Description	Quantity	Rate	Cost(Birr)
Concrete Volume(m3)	144.71	2,950.00	426,894.50
Reinforcement Bar(Kg)	24,368.66	29.00	706,691.14
Formwork (m ²)	709.1	200.00	141,820.00
Total			1,275,405.64

COST COMPARISON AND DISCUSSION

COST COMPARISON

The cost comparison is summarized and given in Table 5.11. Total costs are obtained by summing up each item in each of the bridge types.

Table 5.11: Cost Summary

Span (m)	Total cost for steel-concrete composite bridges (Birr)	Total cost for concrete deck girder bridges (Birr)
12m	609,548.25	510,887.39
15m	815,677.49	679,273.67
20m	1,295,662.53	1,016,474.4
24m	1,826,114.14	1,275,405.64

DISCUSSION

In the selection of the span lengths of the bridges, it is tried to take from practical and economical span ranges of the reinforced concrete deck girder bridge which is 12m up to 25m.

The cost summary and the design of the bridges shows that

- The concrete slab deck depth is the same in both cases while the reinforcement required is a bit higher in case of the steel-concrete composite bridges.
- Depth required in reinforced concrete deck girder bridges is higher than in steel – concrete composite bridges.
- The weight of the bridge reduces in case of composite bridges and corresponding internal forces i.e. shear forces and moments are also smaller.
- For the span lengths considered, the cost for the steel - concrete composite bridges is higher as compared to the reinforced concrete deck girder bridges.
- The cost differences increases as the span increases.

- There is a 19.3% cost increment in the 12m span steel-concrete composite bridge as compared to the corresponding reinforced concrete deck girder bridge. In the same way a 20.1%, 27.5% and 43.2% increments are observed in the 15m, 20m and 24m span steel-concrete composite bridges.

Qualitative comparison of the two bridge types is discussed as follows

- The steel - concrete composite bridges are relatively lighter than reinforced concrete deck girder bridges; as a result relatively smaller area and thickness of bridge bearings can be used. In addition to this, lesser foundation structure is required for substructures.
- In reinforced concrete deck girder bridge construction, once the substructures are constructed and bearings are placed, construction of the girders requires first, set up of false works that serves as support during concrete casting second, preparation of formworks and then placing of reinforcements. As practically observed, these activities take considerable construction time. In composite construction, however, the steel beams can be manufactured in workshops prior to the commencement of the construction of the superstructure and formworks and false works are not required. Hence, significant construction time can be saved to finish the bridge as a whole and it can be opened for traffic in relatively shorter time. Subsequently, the contractor and the consultant can move their working crew and machineries to other part of their project and this is money saving. More over, this is a lot for the road users where there is discomfort in using detours during the bridge construction periods especially in towns.
- Large span bridges can also be realized by using steel- concrete composite construction especially in areas where diversion of rivers to construct intermediate pier structures is difficult and costly; it will provide an ideal solution.

- Some of disadvantages associated with steel- concrete composite construction are corrosion and fire resistance problems. In regard to fire resistance, the nature of bridges that they are not likely exposed to fire as buildings do, will lessen the fear. Due to corrosion, however, relatively frequent maintenance is required in steel concrete composite construction than in reinforced concrete ones.

6. CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSIONS

From the study the following conclusions are drawn

- Relatively compact bridge cross-sections can be obtained by using steel-concrete composite structures than in reinforced concrete deck girders.
- Steel-concrete composite results in a lighter dead load compared with the commonly used deck girder bridges which leads to
 - Smaller reactions on bearings requiring reduced size of bearings
 - Reduced reaction loads on substructure which in turn requires a lesser foundation structure.
- Local metal industries have capacity to produce varied steel profile including I-section suitable for steel-concrete composite bridge construction with considerable experience in railway, vehicle body support parts and building construction. The companies own modern equipment and apply high-tech welding operation to the steel profile products, where technicians are well trained and certified experts for the job.
- Steel beams in composite bridges can be produced in work shops prior to the commencement of construction of the superstructure; this significantly reduces the construction time required for the bridge as whole and the bridge could be open for traffic in relatively shorter time.
- With current market condition and work exposure of metal industries, cost of steel works is expensive and cost required for steel - concrete composite bridge construction is higher than that of reinforced concrete deck girder bridges of the same span. Nevertheless, on account of time saving it can be still advantageous.

6.2 RECOMMENDATIONS

From the study the followings are recommended

- Locally, there are few metal industries that are involved in the manufacturing of steel profiles specially those for steel-concrete composite bridge constructions. Hence, investors should be encouraged to the sector so that designers could have free choices both in quality and shapes. Besides, from competitions, fair and stable construction costs would be established.
- As markets are usually demand driven, consultants and designers should initiate and direct the metal industry by designing steel structures as an alternative and time saving construction.
- Further studies should be carried out to check feasibility of such constructions in our country in line with the change in development of the metal industry and construction skills, as the time saving by itself is a promising and vital item to be considered in bridge construction.

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Appendices

A. Sample Design of 20m Steel - Concrete Composite Bridge Superstructure

APPENDIX A**SIMPLE SPAN STEEL CONCRETE COMPOSITE BRIDGE (SPAN 20m)****SUPERSTRUCTURE DESIGN****1. DESIGN DATA AND SPECIFICATIONS****Subject Information:**

Superstructure type:- Simple span steel concrete composite bridge

C/C distance :	20.00	m
Clear span	19.40	
Clear Roadway	8.00	m
Sidewalk	2 *1.5	m

Material properties:**Concrete :- Grade C-30 concrete**

$f_c'(\text{cylinder}) =$	24	MPa
$f_c = 0.4 * f_c' =$	10	MPa
$E_c = 4800 \sqrt{f_c'} =$	23515	MPa
$\gamma_c =$	25	KN/m ³
partial safety factor, $\gamma_c =$	1.5	

Reinforcement steel:

Grade 420 steel: For rebars diam. 20mm and above

$f_y =$	420	MPa
$E_s =$	200000	MPa

Grade 300 steel: For rebars less than diam. 20

$f_y =$	300	MPa
$E_s =$	200000	MPa

Modular ratio, $n = E_s/E_c =$ 8.505Use $n =$ 9partial safety factor, $\gamma_s =$ 1.15**Steel for girders:**

$f_y =$	345	MPa
$E_s =$	200000	MPa
$\gamma_s =$	78	KN/m ³
partial safety factor, $\gamma_a =$	1.1	

steel for shear connectors:

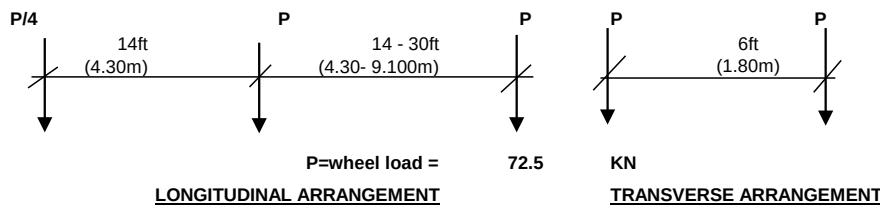
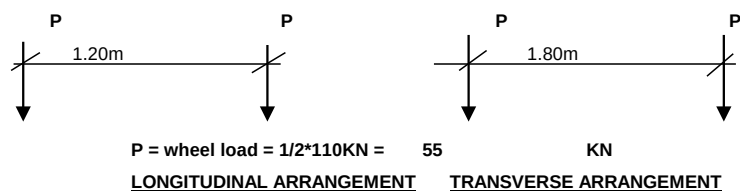
$f_u =$	450	MPa
partial safety factor, $\gamma_{vc} =$	1.25	

Design Live Loading:

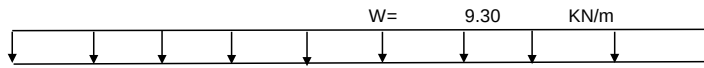
Vehicular Live loading, HL-93, which consists of a combination of

(1) Design truck or Design Tandem

(2) Design Lane Loading

Design Truck**Design Tandem**

Design Lane Loading



Note: Lane loading is uniformly distributed over a width of 3.00m.

Dynamic Load Allowance is not applied for lane loading

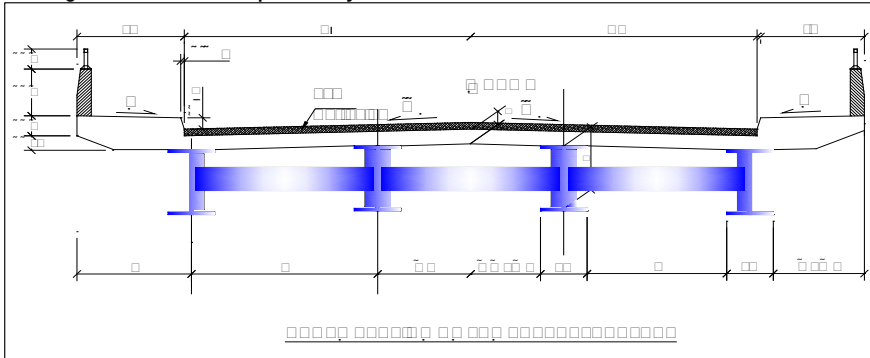
Design method: Load and Resistance Factor Design (LRFD)

Reference: ERA's Bridge Design Manual 2002

ERA's Standard Technical Specifications 2002

AASHTO LRFD Bridge Design Specifications, 1998 Edition.

2. Bridge Cross Section and preliminary dimensions

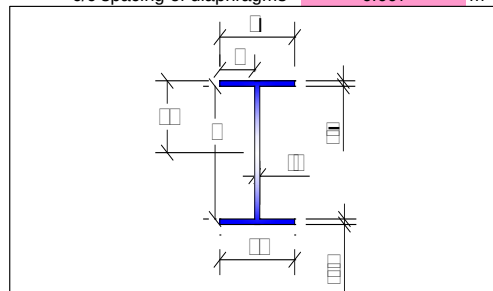


Given Data

Clear Rdwy width, W1=	4	m
Clear Rdwy width, W2=	4	m
Sidewalk width, SW=	1.5	m
Total bridge width, WT=	11	m
No. of lane=	2	
Multiple presence factor=	1.0	(Section 3.8.1)

3. PRELIMINARY DIMENSIONS

No. of girders=	4	
c/c of support=	20.00	m
Top slab thickness, T =	0.20	m
Multiple presence factor =	1.00	
c/c girder spacing, a=	2.60	m
End of slab to center of exterior girder, c=	1.60	m
Overhang slab thickness near ext. girder=	0.32	m
Total number of diaphragm=	4	
c/c spacing of diaphragms=	6.667	m



Of the steel girder

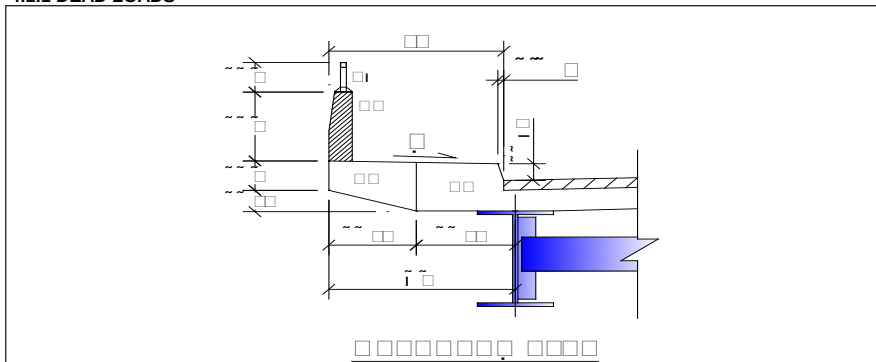
b1=	0.3000	m
b2=	0.3000	m
tf1=	0.0200	m
tf2=	0.0200	m
tw=	0.0160	m
d =	0.8500	
da=	0.445	
C =	0.1420	m
Aa =	0.0256	m ²
I =	0.003090	m ⁴

Crown and Wearing surface

Use 2.0% crown in transverse direction and assume 5cm asphalt wearing surface

Posts and Railings

Use post and railings as given and dimensioned in figure below.

4. DESIGN OF OVER HANG**4.1 LOADS****4.1.1 DEAD LOADS**

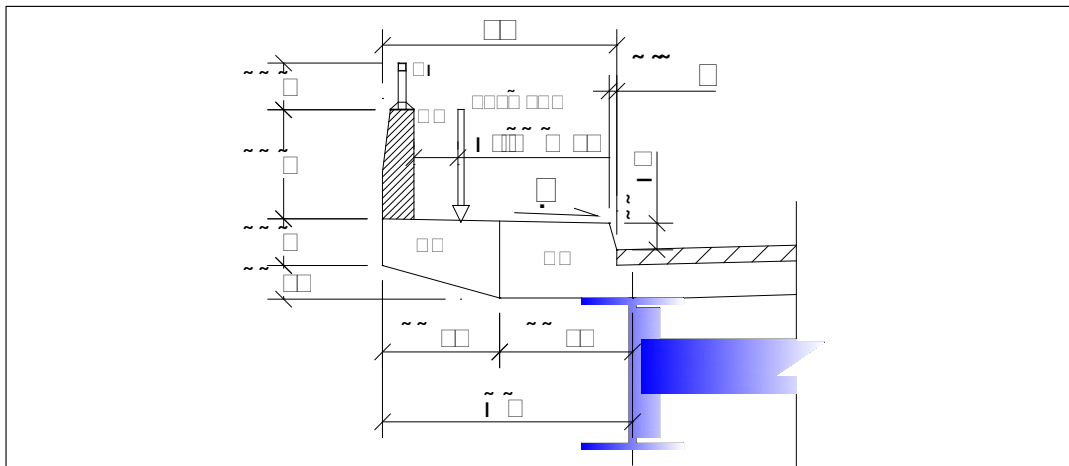
d1= 0.341

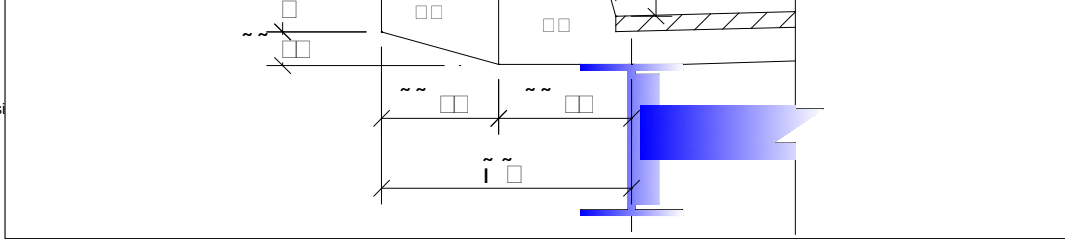
Dead loads (KN/m)		Moment arm about face of exterior girder (m)	MDL (KNm/m)
$W1=(0.30*4*0.05*0.003*78)/2.0+(4*0.05*0.003*78)$	0.054	1.400	0.075
$W2=0.20*0.70*25$	3.500	1.400	4.900
$W3=1/2*(0.30+0.30+0.22)*0.75*25$	7.875	1.001	7.887
$W4=0.37*25$	9.250	0.330	3.053
Sum W_{DL} =	20.68	M_{DL} =	15.914

i) Design Truck Load

The application of live load for the design of deck overhang is done according to AASHTO SSHB 98, Art. 3.24.2

AASHTO Art. 3.24.2.2. In designing sidewalks, slabs and supporting members, wheel load located on the sidewalk shall be 1feet from the face of the rail.





Distribution width for wheel loads (Art. 3.24.5.1.1)

$E = 0.80x + 3.75$, where x = the distance in feet from load to point of support

$E =$	6.43	ft	$=$	1.96	m
		Where	$x =$	1.020	m
			$P =$	72.5	KN

Live load moment per linear meter width of slab

$MLL = (P/E) \cdot x = 37.75 \text{ KNm/m}$

Dynamic Allowance Factor

$IM = 33\%$

Live Load plus impact:

$M_{LL+IM} = 50.21 \text{ KNm/m}$

ii) Design Tandem

The design tandem wheel load, $P = 55\text{KN}$, is smaller than the design truck load, $P=72.5\text{KN}$, and hence is considered not critical for overhang design

iii) Pedestrian Live Load

Pedestrian live load	=	85 lb/ft ²	=	4.00	KN/m ²
Moment due to pedestrian load	=			2.93	KNm/m

Total Design Moment

$$M_{TOT} = 1.25 \cdot M_{DL} + 1.75 \cdot M_{LL+IM} = 107.75 \text{ KNm/m}$$

4.2 Reinforcement

Minimum Reinforcement Requirement (Art. 8.17.1)

$$\begin{aligned} f_r &= 0.63 \cdot \text{SQRT}(f_c') = 3.09 \text{ N/mm}^2 \\ I_{cr} &= b h^3 / 12 = 2.731 \text{E}+09 \text{ mm}^4 \\ M_{cr} &= f_r \cdot I_{cr} / (y_t) = 52.67 \text{ KNm/m} \quad y_t = h/2 \\ 1.2 \cdot M_{cr} &= 63.21 \text{ KNm/m} \\ M_{design} &= 107.75 \text{ KNm/m} \end{aligned}$$

Mu =	107.75	KNm/m
Ø =	0.90	
b =	1000	mm
fy =	300	N/mm ²
fc' =	24	N/mm ²
D =	320	mm
diam =	16	mm
cover =	50	mm
d =	262	mm

Assume a = 23.40 mm

$$\begin{aligned} A_s &= M_u / (\phi f_y (d - a/2)) = 1594.43 \text{ mm}^2 \\ a &= A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) = 23.45 \text{ mm} \end{aligned}$$

Assume a = 23.45 mm

$$\begin{aligned} A_s &= M_u / (\phi f_y (d - a/2)) = 1594.59 \text{ mm}^2/\text{m} \\ a &= A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) = 23.45 \text{ mm} \end{aligned}$$

OK !!

Required A_s = 1595 mm²/m

Required spacing = 126 mm

Use diameter	16	mm	bars	c/c	125	mm
As provided	=				1608	mm ²

Checking maximum steel area

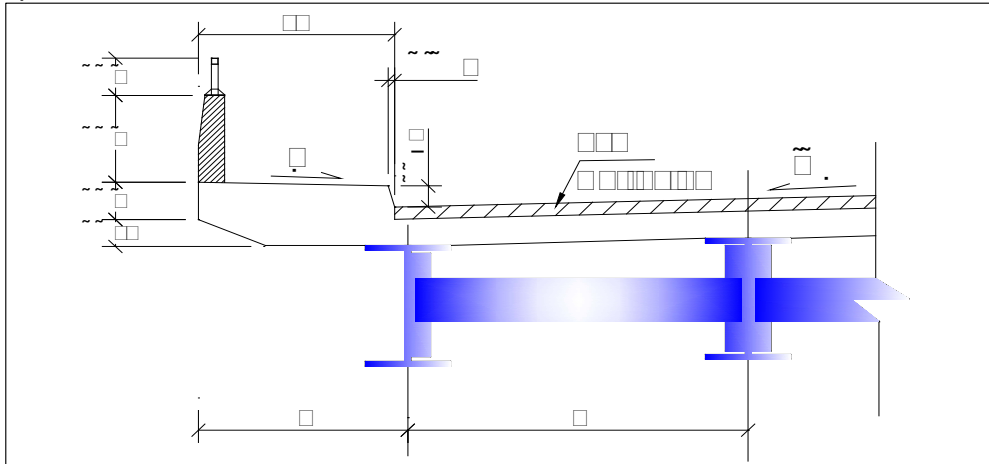
$$\begin{aligned} b_1 &= 0.85 \\ f_c' &= 24 \text{ N/mm}^2 \\ f_y &= 300 \text{ N/mm}^2 \\ r_{max} &= 0.75 \cdot r_b = 0.75 \cdot 0.85 \cdot b_1 \cdot (f_c' / f_y) \cdot 600 / (600 + f_y) \\ &= 0.0289 \\ r_{provided} &= A_s / b d = 0.0061 \end{aligned}$$

OK !!

5. DESIGN OF DECK SLAB

5.1 Loadings

a) Dead Loads



Dead loads computation

Slab 20cm thick	= 0.20 * 25 =	5.00	KN/m ²
Asphalt 5cm thickness	= 0.05 * 22.5 =	2.25	KN/m ²
w_{DL}	=	<u>7.25</u>	KN/m ²

$$M_{DL} = (w_{DL} * S^2 * 0.80) / 8 \quad \text{where 0.80 is a continuity factor}$$

Span length S = Clear span - C	=	2.46	m
$M_{DL} = 1/8 (w_{DL} * S^2 * 0.80)$	=	4.38	KNm/m

b) Live Loads

Design Truck Loading

Live Load moment for continuous slab (Art. 3.24.3.1)

$$M_{LL} = 1/32(S+2)P \times 0.80 \quad \text{where S=span length in feet (Art.3.24.1.2)}$$

S=	2.46	m
P=	72.50	KN
$M_{LL} =$	4101.07	lb-ft/ft =
$M_{LL} =$	18.24	KNm/m

Impact Factor (Art. 3.8.2.1)

IM	=	33%
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Live load plus impact

M_{LL+I}	=	24.26	KNm/m
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Design Lane Loading

Lane load per meter in transverse dir.=9.30/3.00=	3.10	KN/m
---	------	------

Where	S =	2.46	m
$M_{LL} = (W * S^2 * 0.80) / 8 =$	1.87	KNm/m	

c) Factored Design moment

$M_{TOT.} = 1.25 * M_{DL} + 1.75 * (M_{LL+IM (TRUCK)} + M_{LL (LANE)}) =$	51.21	KNm/m
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5.2 Reinforcement

$M_u =$	51.21	KNm/m
$\phi =$	0.90	
b =	1000	mm
$f_y =$	300	N/mm ²
$f_c' =$	24	N/mm ²
D =	200	mm
diam =	16	mm
cover =	50	mm

	d =	142	mm
Assume a =	21.08	mm	
As =	$M_u / (\phi f_y (d - a/2)) =$	1442.78	mm ²
a =	$A_s f_y / (0.85 * f_c' b) =$	21.22	mm
Assume a =	21.23	mm	
As =	$M_u / (\phi f_y (d - a/2)) =$	1443.60	mm ² /m
a =	$A_s f_y / (0.85 * f_c' b) =$	21.23	mm
		OK !!	
	Required As =	1444	mm ² /m
	Spacing, s =	139	mm
Use diameter	16	mm bars c/c	140 mm (top and bottom reinf.)
As provided =			1,436 mm ²

Checking maximum steel area

$$\begin{aligned}
 b1 &= 0.85 \\
 f_c' &= 300 \text{ N/mm}^2 \\
 f_y &= 1,000 \text{ N/mm}^2 \\
 r_{\max} = 0.75 \cdot r_b &= \frac{0.75 \cdot 0.85 \cdot b1 \cdot (f_c' / f_y) \cdot 600}{(600 + f_y)} \\
 &= 0.0610 \\
 r_{\text{provided}} = A_s / bd &= 0.0101 \quad \text{OK !!}
 \end{aligned}$$

Distribution Reinforcements (Art. 3.24.10.2)

For main reinforcement perpendicular to traffic, the distribution reinf. is given as percentage of the main slab reinf.

as given below

$$A_s(\text{distr.}) \% = 220 / \sqrt{S} \leq 67\%$$

Therefore, % As dist. = 77 % (S is in feet)

Where S = 2.46 m

$$\% A_s(\text{distr.}) = 67\%$$

$$A_s \text{ dist.} = 0.67 \cdot A_s \text{ provided} = 962 \text{ mm}^2/\text{m}$$

$$\text{spacing} = 208.96 \text{ mm}$$

Using diam. of bar = 16 mm

Use diam.	16	mm bars c/c	200	mm (bottom reinf. - transverse.)
As provided			1005	mm ²

Temperature and shrinkage reinforcements (Art. 8.20)

As (temp and shrink.) is not less than 1/8 square inch per feet.

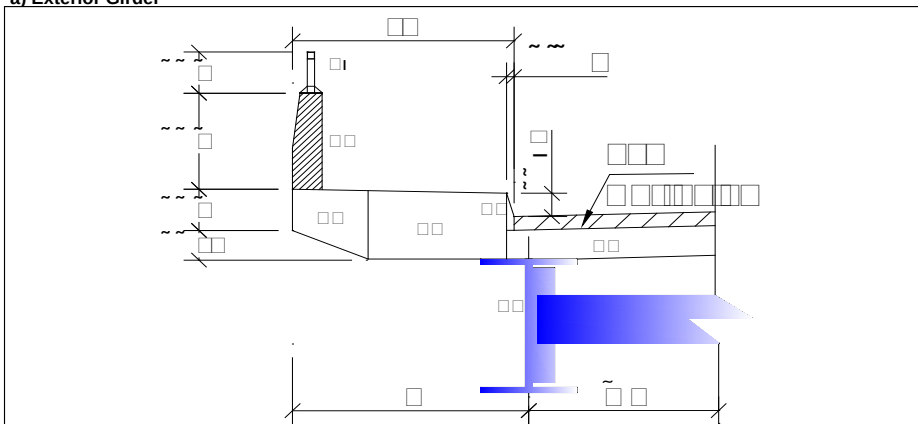
$$A_s(\text{temp and shrink.}) = 264.58 \text{ mm}^2/\text{m}$$

Using diam. 12 bar

$$\text{spacing} = 427 \text{ mm}$$

$$\text{diam. of bar} = 12 \text{ mm}$$

Use diam.	12	c/c	400 mm (top. reinf.)
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6. DESIGN OF LONGITUDINAL GIRDERS**6.1. Loads****6.1.1 Dead Loads****a) Exterior Girder**

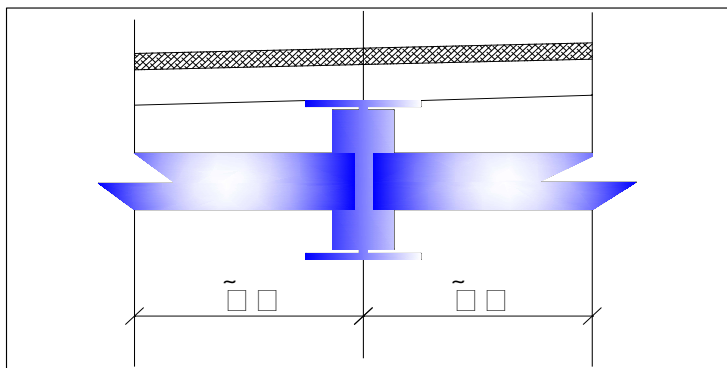
$$C = 1.60 \text{ m}$$

$$a = 2.60 \text{ m}$$

$$\text{unit weight of the steel girder} = 2.00 \text{ kN/m}$$

Uniform dead loads per linear meter span	(KN/m)
Railing and posts(W1+W2) =	3.55
Over hanging slab(W3+W4)=	17.13
slab (interior side)(w6) = (a/2+0.15)*0.2*25 =	7.25

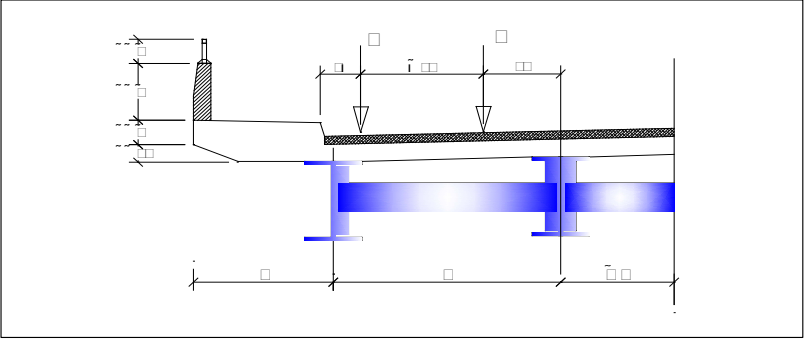
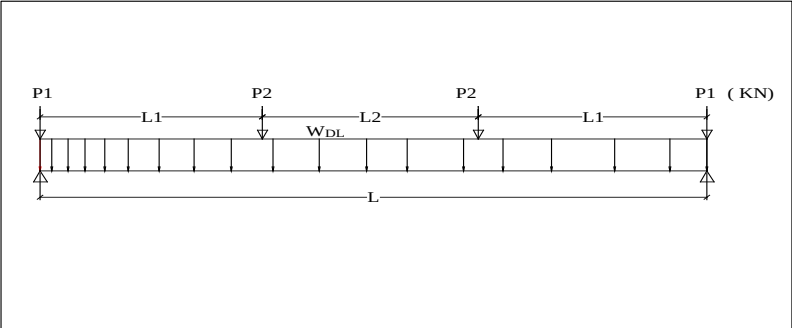
girder(W7) =	2.00
Curb(W5)=((0.34+0.05)/2)*0.05*25	0.244
Asphalt surfacing=(a/2+0.15)*0.05*22.5 =	1.63
sum	31.80

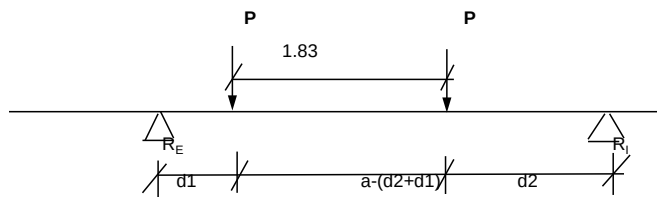
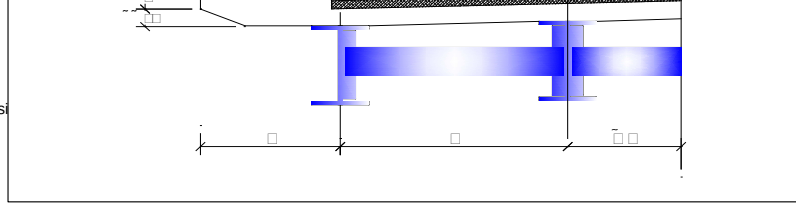


$$\underline{\text{sum WDL} = 17.92 \text{ KN/m}}$$

Diaphragms:

Conc. Load P1 = 5%(wgt. Of steel girder beam)*a	0.26	KN
Conc. Load P2 = 5%(wgt. Of steel girder beam)*a	0.26	KN





a=	2.60	m
d1=	0.21	m
d2=	0.57	m

The distribution factor for shear to the exterior girder by applying Lever Rule and including multiple presence 1.20 as per table 3-5

where Multiple presence = 1.20

$$R_{EX1}(\text{shear}) = [1/a * P * (a + d2 - d1)] * \text{Multiple presence} = 1.366 P$$

Case-2: Two or more design lanes loaded

The distribution of live load per lane for shear in exterior girder is determined according to the formulas given in Table 13-8.

where $d_e = c - bw/2 - w(\text{parapet}) = 1.392$

$$R_{EX2}(\text{shear}) = (0.60 + d_e/3000) = 1.064 \text{ per lane}$$

For two lines of wheels the distribution factor for shear in interior girder is:

where Multiple presence = 1.00

$$R_{EX2}(\text{shear}) = 2.128 P$$

There fore, $R_{EX}(\text{shear})$ in exterior girder is maximum of the above two values, R_{EX1} or R_{EX2}

$$R_{EX}(\text{shear}) = 2.128 P$$

Interior Girder:

Case-1: One Design lane loaded

The distribution of live load per lane for shear in interior girder is determined according to the formulas given in Table 13-7.

where $1100 \leq S \leq 4900$

$6000 \leq L \leq 73000$

$S = 2.60 \text{ m}$

$$R_{INT1}(\text{shear}) = 0.36 + S/7600 = 0.702 \text{ per lane}$$

For two lines of wheels the distribution factor for shear in interior girder is:

$$R_{INT1}(\text{shear}) = (2P)(0.36 + S/7600) = 1.404 P$$

Case-2: Two or more design lanes are loaded

The distribution of live load per lane for shear in interior girder

where $1100 \leq S \leq 4900$

$6000 \leq L \leq 73000$

$N_b \geq 4$

$S = a = 2.60 \text{ m}$

$$R_{INT2}(\text{shear}) = 0.20 + (S/3600) - (S/10700)^2 = 0.863$$

For two lines of wheels the distribution factor would be

$$R_{INT2}(\text{shear}) = 1.726 P$$

There fore, R_{INT} (shear), in interior girder is maximum of the above two values, R_{INT1} or R_{INT2}

$$R_{INT} \text{ (shear)} = 1.726 P$$

Distribution Factor for moment (Sec.13.4: Table 13-3 and 13-4)

Exterior Girder

Case-1: One Design lane loaded

The lever rule is applied (Table 13-4) and multiple presence is included

$$R_{EXT1} \text{ (moment)} = \text{where Multiple presence} = 1.2 \quad 1.366 P$$

Case-2: Two or more design lanes loaded

The Lever Rule is applied (Table 13-4) and the distribution factor for wheel load is

$$R_{EXT2} \text{ (moment)} = \text{where Multiple presence} = 1.2 \quad 1.366 P$$

Interior Girder

Case-1: One Design lane loaded

$$R_{INT1} \text{ (moment)} = 0.06 + [S/4300]^{0.4} [S/L]^{0.3} [Kg/L \cdot ts^3]^{0.10} \quad \text{where } 1100 \leq S \leq 4900$$

The longitudinal stiffness parameter, Kg.

$$Kg = n \cdot (I + A \cdot e_g^2)$$

I =	0.0000	m4
A =	0.0256	m2
$e_g = da + 0.5 \cdot Ts$	0.5450	m
$Kg = n \cdot (I + A \cdot e_g^2) =$	0.06843	m4

The distribution factor of live load per lane for moment in interior girder:

$$R_{INT1} \text{ (moment)} = 0.06 + [S/4300]^{0.4} [S/L]^{0.3} [Kg/L \cdot ts^3]^{0.10} = 0.467$$

where $L = 20.00 \text{ m}$
 $S \text{ (in meters)} = 2.60 \text{ m}$

Case-2: Two or more design lanes loaded

$$R_{INT2} \text{ (moment)} = 0.075 + [S/4300]^{0.6} [S/L]^{0.2} [Kg/L \cdot ts^3]^{0.10} = 0.527$$

where $S \text{ (in feet)} = 8.53 \text{ feet}$
 $S/6 = 1.42 \text{ (AASHTO 3.23.2.2)}$

The distribution factor for wheel load (i.e. two lines of wheels) is 2 times the maximum of the above two values

$$R_{INT} \text{ (moment)} = 1.422 P$$

b) Design Lane Loading

Distribution factor for Shear:

Exterior Girder:

Case-1: One design lane loaded

The Lever Rule is applied

$$\text{Dist. factor} = RE = 1/2 \cdot (RE \text{ truck}) \cdot \text{lane load} = 0.683 \text{ per lane}$$

Case-2: Two or more design lanes loaded

The distribution factor is determined according to the formula given in Table 13-8

The distribution of live load per lane for shear in exterior girder is

$$\text{Dist. factor} = (0.60 + de/3000) = 1.064 \text{ per lane}$$

where $de = c - bw/2 - w(\text{parapet}) = 1.392 \text{ m}$

Interior Girder:

Case-1: One design lane loaded

Formula in Table 13-7 is used

$$\text{Dist. factor} = 0.36 + S/7600 = 0.702 \text{ per lane}$$

Case-2: Two or more design lanes loaded

Formula in Table 13-7 is used

$$\text{Dist. Factor} = 0.20 + (S/3600) - (S/10700)^2 = 0.863 \text{ per lane}$$

Distribution factor for Moment:

Exterior Girder:

Case-1: One design lane loaded

The Lever Rule is applied

Dist. factor = $RE = 1/2*(RE \text{ truck})*\text{lane load} =$

0.683 per lane

Case-2: Two or more design lanes loaded

The Lever Rule is applied

Dist. factor = $RE = 1/2*(RE \text{ truck})*\text{lane load} =$

0.683 per lane

Interior Girder:

Case-1: One design lane loaded

Formula in Table 13-3 is used

Dist. factor =

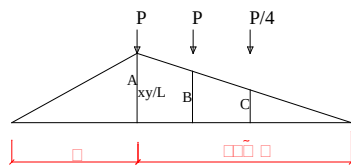
0.467 per lane

Case-2: Two or more design lanes loaded

Formula in Table 13-3 is used

Dist. Factor =

0.527 per lane



$$M_x = P (A + B + C/4)$$

Influence line for
moment at distance 'x'

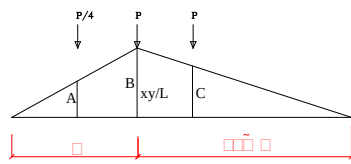
Loading 1

Coefficients

$$A = xy/L = x(L-x)/L$$

$$B = x(y-4.30)/L = x(L-x-4.30)/L$$

$$C = x(y-8.60)/L = x(L-x-8.60)/L$$



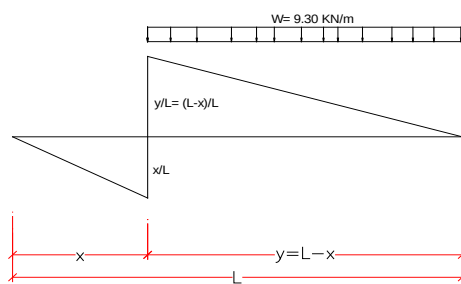
$$M_x = P (A/4 + B + C)$$

Loading 2

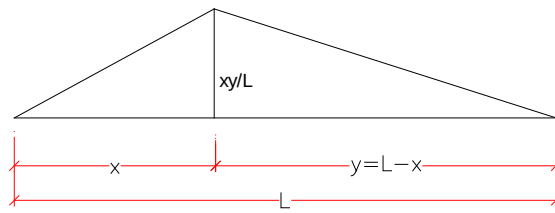
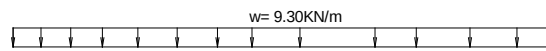
$$A = (x-4.30)y/L = (x-4.30)(L-x)/L$$

$$B = xy/L = x(L-x)/L$$

$$C = (y-4.30)x/L = (L-x-4.30)x/L$$



$$V(x) = W \left(\frac{1}{2} y \cdot \frac{x}{L} \right) = \frac{1}{2} W (L-x) \frac{(L-x)}{L}$$



$$\begin{aligned} M(x) &= W(1/2 * L * xy/L) \\ &= 1/2 * w * x * (L - x) \end{aligned}$$

a-2) Design Lane Loading

$$V_{LL} = (\text{Distribution Factor}) * W * (1/2 * (L-x)^2 / L)$$

X (m)	L-X (m)	V _{LL} (KN)
0	20.00	98.95
1	19.00	89.30
2	18.00	80.15
3	17.00	71.49
4	16.00	63.33
5	15.00	55.66
6	14.00	48.49
7	13.00	41.81
8	12.00	35.62
9	11.00	29.93
10	10.00	24.74

W = 9.30 KN/m

L = 20.00 m

Dist. Factor = 1.06 per lane

b) Interior Girder**b-1) Design Truck or Design Tandem** (maximum value will be taken)Design Truck: $V_{LL+IM} = (1.00+IM) * (\text{Dist. Factor}) * (\text{Ptruck}) * (A+B+C/4)$ Design Tandem: $V_{LL+IM} = (1.00+IM) * (\text{Dist. Factor}) * (\text{Ptandem}) * (A+B)$

IM = 33%

L = 20.00 m

Dist. Factor = 1.726

P truck= 72.50 KN

P tandem= 55.00 KN

x (m)	Live Load 1: Design Truck				Live Load Design Tandem
	Coefficients			V _{LL+IM}	Coefficients
	A	B	C	(KN)	A
0	1.000	0.785	0.570	320.86	1.000
1	0.950	0.737	0.520	302.41	0.950
2	0.900	0.687	0.470	283.68	0.900
3	0.850	0.637	0.420	264.95	0.850
4	0.800	0.587	0.370	246.22	0.800
5	0.750	0.537	0.320	227.50	0.750
6	0.700	0.487	0.270	208.77	0.700
7	0.650	0.437	0.220	190.04	0.650
8	0.600	0.387	0.170	171.32	0.600
9	0.550	0.337	0.120	152.59	0.550
10	0.500	0.287	0.070	133.86	0.500

b-2) Design Lane Loading

$$V_{LL} = (\text{Distribution Factor}) * W * (1/2 * (L-x)^2 / L)$$

X (m)	L-X (m)	V _{LL} (KN)
0	20.00	80.28
1	19.00	72.45
2	18.00	65.02
3	17.00	58.00
4	16.00	51.38
5	15.00	45.15
6	14.00	39.34
7	13.00	33.92
8	12.00	28.90

W = 9.30 KN/m

L = 20.00 m

Dist. Factor = 0.863 per lane

9	11.00	24.28
10	10.00	20.07

7.1.2.4 BENDING MOMENTS DUE TO LIVE LOAD PLUS IMPACT

a) Exterior Girder

a-1) Design Truck or Design Tandem (maximum value is taken)

Design Truck: $V_{LL+IM} = (1.00+IM) \cdot (\text{Dist. Factor}) \cdot (P_{\text{truck}}) \cdot \text{Max}(\text{Sum Coeff's})$

Design Tandem: $V_{LL} = (1.00+IM) \cdot (\text{Dist. Factor}) \cdot (P_{\text{tandem}}) \cdot (A+B)$

IM = 33%
L = 20.00 m
Dist. Factor = 1.366
P truck= 72.50 KN
P tandem= 55.00 KN

When the live load is Design Truck

x (m)	Coef. (Loading 1)			Loading 1 sum Coef.	Coef. (Loading 2)
	A	B	C	A+B+C/4	
0	-	-	-	0.000	-
1	0.95	0.74	0.13	1.815	-
2	1.80	1.37	0.24	3.405	-
3	2.55	1.91	0.32	4.770	-
4	3.20	2.34	0.37	5.910	-
5	3.75	2.68	0.40	6.825	0.13
6	4.20	2.91	0.41	7.515	0.30
7	4.55	3.05	0.39	7.980	0.44
8	4.80	3.08	0.34	8.220	0.56
9	4.95	3.02	0.27	8.235	0.65
10	5.00	2.85	0.18	8.025	0.71

When the live load is Design Tandem

x (m)	Coefficients		Sum coef.	M_{LL+IM} (KNm/m)
	A	B	(A + B)	
0	0.000	0.000	0.000	0.00
1	0.950	0.890	1.840	183.88
2	1.800	1.680	3.480	347.77
3	2.550	2.370	4.920	491.68
4	3.200	2.960	6.160	615.59
5	3.750	3.450	7.200	719.53
6	4.200	3.840	8.040	803.47
7	4.550	4.130	8.680	867.43
8	4.800	4.320	9.120	911.40
9	4.950	4.410	9.360	935.38
10	5.000	4.400	9.400	939.38

a-2) Design Lane Loading

$M_{LL} = (\text{Distribution Factor}) \cdot 1/2 \cdot W \cdot x \cdot (L-x)$

W = 9.30 KN/m

L = 20.00 m

Dist. Factor = 0.683 per lane

X (m)	L-X (m)	M_{LL} (KNm)
0	20.00	0.00
1	19.00	60.35
2	18.00	114.35
3	17.00	161.99
4	16.00	203.28
5	15.00	238.22
6	14.00	266.81
7	13.00	289.04

8	12.00	304.93
9	11.00	314.45
10	10.00	317.63

b) Interior Girder**b-1) Design Truck or Design Tandem** (maximum value is taken)

Design Truck: $V_{LL+IM} = (1.00+IM) \cdot (\text{Dist. Factor}) \cdot (P_{\text{truck}}) \cdot \text{Max}(\text{Sum Coeff's})$

Design Tandem: $V_{LL+IM} = (1.00+IM) \cdot (\text{Dist. Factor}) \cdot (P_{\text{tandem}}) \cdot (A+B)$

IM = 33%
 L = 20.00 m
 Dist. Factor = 1.422
 P truck= 72.50 KN
 P tandem= 55.00 KN

When the live load is Design Truck

x (m)	Coef. (Loading 1)			Loading 1 sum Coef.	Coef. (Loading 2)
	A	B	C	A+B+C/4	A/4
0	-	-	-	0.000	-
1	0.95	0.74	0.13	1.817	-
2	1.80	1.37	0.24	3.410	-
3	2.55	1.91	0.32	4.777	-
4	3.20	2.35	0.37	5.920	-
5	3.75	2.68	0.40	6.837	0.13
6	4.20	2.92	0.41	7.530	0.30
7	4.55	3.06	0.39	7.997	0.44
8	4.80	3.09	0.35	8.240	0.56
9	4.95	3.03	0.28	8.257	0.65
10	5.00	2.87	0.18	8.050	0.71

When the live load is Design Tandem

x (m)	Coefficients		Sum coef.	M_{LL+IM}
	A	B	(A + B)	(KNm/m)
0	0.000	0.000	0.000	0.00
1	0.950	0.890	1.840	191.35
2	1.800	1.680	3.480	361.91
3	2.550	2.370	4.920	511.67
4	3.200	2.960	6.160	640.62
5	3.750	3.450	7.200	748.78
6	4.200	3.840	8.040	836.14
7	4.550	4.130	8.680	902.70
8	4.800	4.320	9.120	948.45
9	4.950	4.410	9.360	973.41
10	5.000	4.400	9.400	977.57

b-2) Design Lane Loading

$$M_{LL} = (\text{Distribution Factor}) \cdot 1/2 \cdot W \cdot x \cdot (L-x)$$

W = 9.30 KN/m
 L = 20.00 m
 Dist. Factor = 0.527 per lane

X (m)	L-X (m)	M_{LL} (KNm)
0	20.00	0.00
1	19.00	46.53
2	18.00	88.16
3	17.00	124.90
4	16.00	156.73
5	15.00	183.67
6	14.00	205.71
7	13.00	222.85

8	12.00	235.10
9	11.00	242.45
10.00	10.00	244.89

SUMMARY OF SHEAR FORCES AND BENDING MOMENTS DUE TO LIVE LOADS**SHEAR FORCES DUE TO LIVE LOADS (SERVICE LOADS)**

	Exterior Girder				I
X	(1) TRUCK V_{LL+H}	(2) TANDEM V_{LL+H}	(3) LANE V_{LL}	(4) V_{LL+H} MAX(1, 2) + (3)	(5) TRUCK V_{LL+H}
(m)	(KN)	(KN)	(KN)	(KN)	(KN)
0.00	395.51	301.99	98.95	494.46	320.9
1.00	372.42	286.42	89.30	461.73	302.4
2.00	349.34	270.85	80.15	429.49	283.7
3.00	326.26	255.29	71.49	397.75	265.0
4.00	303.17	239.72	63.33	366.50	246.2
5.00	280.09	224.16	55.66	335.75	227.5
6.00	257.00	208.59	48.49	305.49	208.8
7.00	233.92	193.02	41.81	275.73	190.0
8.00	210.84	177.46	35.62	246.46	171.3
9.00	187.75	161.89	29.93	217.68	152.6
10.00	164.67	146.32	24.74	189.40	133.9

BENDING MOMENTS DUE TO LIVE LOADS (SERVICE LOADS)

	Exterior Girder				I
X	TRUCK M_{LL+H}	TANDEM M_{LL+H}	LANE M_{LL}	(4) M_{LL+H}	TRUCK M_{LL+H}
(m)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)
0.00	0.00	0.00	0.00	-	0.00
1.00	239.09	183.88	60.35	299.44	249.15
2.00	448.55	347.77	114.35	562.89	467.46
3.00	628.36	491.68	161.99	790.35	654.92
4.00	778.53	615.59	203.28	981.82	811.54
5.00	899.07	719.53	238.22	1,137.29	937.32
6.00	989.96	803.47	266.81	1,256.77	1032.25
7.00	1,058.30	867.43	289.04	1,347.34	1101.32
8.00	1,111.15	911.40	304.93	1,416.08	1156.33
9.00	1,134.37	935.38	314.45	1,448.83	1180.49
10.00	1,127.95	939.38	317.63	1,445.58	1173.81

7.1.3 SEISMIC FORCE EFFECTS

Earthquake zones:

EBCS Zone - I

Site Coefficient:

Type I = 1

Acceleration coefficient: $0.03 < A \leq 0.05 =$

0.04

The horizontal seismic force is the product of the site coefficient, the acceleration coefficient and the permanent loads

Computation of permanent loads:

Parapets	=	$2 \times (0.20 \times 0.70 \times L) \times 25$	=	145.60	KN
Sidewalks	=	$2 \times (0.73 \times L) \times 25$	=	759.20	KN
Slab	=	$8 \times T_s \times L \times 25$	=	832.00	KN
Main Girders	=	$4 \times A \times L \times 78$	=	166.13	KN
Diaphragms	=		=	3.12	KN
Asphalt	=	$8 \times 0.050 \times L \times 22.50$	=	187.20	KN
Sum Wp	=		=	2093.25	KN

Horizontal Earthquake force $F_H = 1.00 \times 0.04 \times W_p = 83.73$ KN

This force is transferred to the substructure through the bearings located at ends of the bridge

Earthquake force transferred to one support = $1/2 \times F_H = 41.86$ KN

This force is applied horizontally to the bearings

$$\begin{aligned}\text{Total Dead Load reaction at one support} &= 1046.62 \\ \text{Assume coef. of friction between bearing and concrete seat at abut, } \mu &= 0.50 \\ \text{The frictional resistance force developed between the interface} &= \mu \cdot \text{Dead load Rxn} \\ &= 523.31 \text{ KN} \\ \text{Earthquake Effects are Negligible !!}\end{aligned}$$

7.1.4 SUMMARY OF DESIGN LOADS /FACTORED LOADS/

LOAD FACTORS AND LOAD COMBINATIONS

The load factors and load combinations are according to ERA's Bridge Design Manual 2002, section 3.3

The load combination to be used for design is Strenght - I Limit state, Table 3-1.

Factored Load = $g_b \cdot DL + 1.75 \cdot (LL + IM) = 1.25 \cdot DL + 1.75 \cdot (LL + IM)$

$g_b = 1.25$

FACTORED SHEAR FORCES

X	Exterior Girder			Interior Girder	
	V_{DL}	V_{LL+IM}	$1.25V_{DL} + 1.75V_{LL+IM}$	V_{DL}	V_{LL+IM}
	(KN)	(KN)	(KN)	(KN)	(KN)
0.00	318.14	494.46	1,262.98	179.48	401.13
1	286.34	461.73	1,165.94	161.56	374.86
2	254.53	429.49	1,069.78	143.63	348.70
3	222.73	397.75	974.48	125.71	322.95
4	190.93	366.50	880.04	107.79	297.60
5.00	159.13	335.75	786.48	89.87	272.65
6	127.33	305.49	693.77	71.95	248.11
7	95.40	275.73	601.77	53.77	223.96
8	63.60	246.46	510.80	35.84	200.22
9	31.80	217.68	420.70	17.92	176.87
10.00	0.00	189.40	331.46	0.00	153.93

FACTORED BENDING MOMENTS

X	Exterior Girder			Interior Girder	
	M_{DL}	M_{LL+IM}	$1.25M_{DL} + 1.75M_{LL+IM}$	M_{DL}	M_{LL+IM}
	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)
0.00	0.00	0.00	-	-	-
1	302.24	299.44	901.82	170.52	295.68
2	572.67	562.89	1,700.90	323.11	555.62
3	811.31	790.35	2,397.24	457.78	779.82
4	1018.14	981.82	2,990.85	574.54	968.27
5	1193.17	1137.29	3,481.72	673.37	1,120.99
6	1336.40	1256.77	3,869.86	754.27	1,237.96
7	1447.77	1347.34	4,167.56	817.13	1,324.18
8	1527.27	1416.08	4,387.23	861.93	1,391.43
9	1574.97	1448.83	4,504.16	888.82	1,422.94
10.00	1590.87	1445.58	4,518.36	897.78	1,418.70

8.0 DEIGN OF THE COMPOSITE BEAM

8.1 Material,section properties and safety facors used in the design

Material properties

concrete

$\gamma =$	25	KN/m ³
$f_{ck} =$	25	N/mm ²
$E_{cm} =$	19100	N/mm ²

steel

$\gamma =$	78	KN/m ³
$f_y =$	345	N/mm ²
$E_s =$	200	N/m ²

Shear connectors

Section properties

Concrete slab

$h_c =$	200	mm
$A_c =$	510000	mm ²

steel

$t_1 =$	20	mm
$t_2 =$	20	mm
$t_w =$	16	mm
$d_a =$	445	mm

$f_u =$ 450 KN/m²

Partial safety factors

Concrete

$\gamma_c =$ 1.5

steel

$\gamma_a =$ 1.1

shear connectors

$\gamma_{vc} =$ 1.25

$c =$ 142 mm

$b_1 =$ 300 mm

$b_2 =$ 300 mm

$A_a =$ 25600 mm²

$I =$ 3089933333 mm⁴

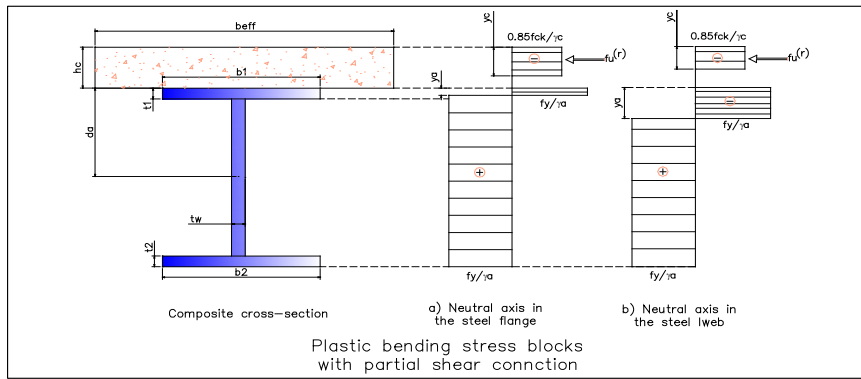
Shear connectors

$h =$ 130

$d =$ 24

Composite beam

Span = 20 m



8.2 Determination slab effective flange width

According to AASHTO Section 4.6.2.6.1 the flange width may be taken as minimum of the following

- | | | | |
|--|---|------|----|
| a) One quarter of the effective span length | = | 5000 | mm |
| b) 12 times the average depth of the slab, plus the greater of web thickness or one half the width of the top flange | = | 2550 | mm |
| c) The average spacing of the adjacent beams | = | 2600 | mm |
| b effective = | | 2550 | mm |

8.3 Check class of the steel profile of the beam(web/flange)

Class of steel beam for flanges under compression

if $c/t < 9\varepsilon$ — class 1

if $c/t < 10\varepsilon$ — class 2

if $c/t < 14\varepsilon$ — class 3

$$\text{where } \varepsilon = \sqrt{235/f_y} = 0.825$$

$$9\varepsilon = 7.428$$

$$10\varepsilon = 8.253$$

$$14\varepsilon = 11.555$$

$$c/t = 7.100$$

Therefore the flange is

class 1

Class of steel beam for web under compression

if $d/t < 72\varepsilon$ — class 1

if $d/t < 83\varepsilon$ — class 2

$$72\varepsilon = 59.423$$

$$83\varepsilon = 68.502$$

$$d/t = 55.625$$

Therefore the flange is

class 1

Therefore, the whole section is class 1

8.4 Section capacity of the the beam

8.4.1 Construction Stage

During the construction stage, the slab concrete will not contribute to the stiffness of the composite section before it hardens and attain its strength. The steel beam alone is responsible to carry its load, the fresh slab concrete and construction live load. This is possible, however, if there are metal decks (steel sheeting profiles), which are common in building floors, that serves as a stay-in-formwork and support the fresh concrete with some intermediate props provided.

In such case, the metal decks transfer the fresh concrete load to the steel girders and accordingly the steel beams capacity should be checked. Nevertheless, usage of such profiles is not a common practice in bridge constructions besides to the unavailability of such profiles locally. In our case, with the conventional formwork set up, the steel girder only carries slab concrete of its flange width—which is very small and capacity is only checked at operational stage.

8.4.2 Operational Stage

The maximum value of total longitudinal shear force, V_{lf} , in the critical length is equal to the lesser plastic resistance of either the steel member or the concrete slab

Therefore V_{lf} is lesser of

$$\text{Tension resistance of the steel member} = A_a \cdot f_y / \gamma_a = 8029.09 \text{ KN}$$

$$\begin{aligned}\text{The compressive resistance of the concrete slab} &= 0.85 \cdot A_c \cdot f_{ck} / \gamma_c = 7225.00 \text{ KN} \\ V_{lf} &= 7225 \text{ KN}\end{aligned}$$

Note that : the small effect of the longitudinal reinforcement in compression is neglected

The design shear resistance P_{RD} of stud shear connectors with $h/d = 5.42$

is given by the smaller of the following values

$$\begin{aligned}P_{RD} &= 0.8 f_u \cdot (\pi d^2 / 4) / \gamma_v = 130.305 \text{ KN} \\ P_{RD} &= 0.29 d^2 \cdot (f_{ck} E_{cm})^{1/2} / \gamma_v = 92.342 \text{ KN} \\ \text{this implies } P_{RD} &= 92.342 \text{ KN}\end{aligned}$$

For degree of shear connection = $N/N_f =$

0.7

where N is number of stud shear connectors for partial shear connection

N_f is number of stud shear connectors for full shear connection

The reduction in the ultimate moment of the internal critical cross section, due to the reduction in the number of connectors can be calculated

Assuming that each connector develops its design shear resistance, the total longitudinal force V_1 in each critical length is equal to the sum of the shear resistances of the connectors and also to the maximum compressive force $F_u^{(t)}$ in the slab

$$V_1 = N P_{RD} = F_u^{(t)}$$

$$F_u^{(t)} = V_{lf} \cdot N / N_f = 5057.50 \text{ KN}$$

Where $N < N_f$ The corresponding depth of the concrete stress block is equal to

$$y_c = F_u^{(t)} / (0.85 f_{ck} b_{eff} / \gamma_c) \leq h_c$$

The position of the neutral axis in the steel beam can be determined from equating $F_u^{(t)}$ to the tensile force in the steel beam and the reduced ultimate moment $M_u^{(t)}$ can be calculated accordingly

→ If the neutral axis is in the upper steel flange, its position y_a and the ultimate moment $M_u^{(t)}$ are given by the following relationships.

$$y_a = (A_a f_y / \gamma_c - F_u^{(t)}) / (2 b_1 f_y / \gamma_a) \text{ with } y_a \leq t_1$$

$$M_u^{(t)} = F_u^{(t)} (h_c + d_a - y_c / 2) + 2 b_1 y_a (f_y / \gamma_a) (d_a - y_a / 2)$$

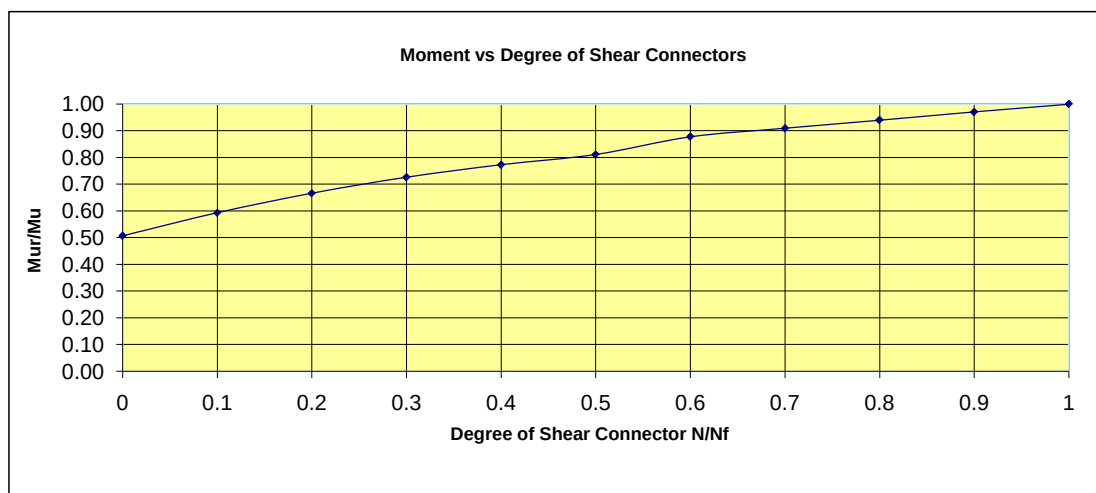
→ If the neutral axis is in the web :

$$y_a = (A_a + 2 t_1 t_w - 2 b_1 t_1 (f_y / \gamma_a) - F_u^{(t)}) / (2 t_w f_y / \gamma_a) \text{ where } t_1 \leq y_a \leq d_a$$

$$M_u^{(t)} = F_u^{(t)} (h_c + d_a - y_c / 2) + 2 b_1 y_a (f_y / \gamma_a) (d_a - t_1 / 2) + t_w (y_a - t_1) (f_y / \gamma_a) (2 d_a - y_a - t_1)$$

8.5 The table below summarizes the computation of the moment capacity of the beam corresponding to different degree of shear connectors

N/Nf	Fur(KNm)	Y _a (mm)	yaw(mm)	ya(mm)	yc(mm)
0	0.00	42.7	445.0	445.0	0.00
0.1	722.50	38.8	373.0	373.0	20.00
0.2	1445.00	35.0	301.0	301.0	40.00
0.3	2167.50	31.1	229.0	229.0	60.00
0.4	2890.00	27.3	157.0	157.0	80.00
0.5	3612.50	23.5	85.1	85.1	100.00
0.6	4335.00	19.6	13.1	19.6	120.00
0.7	5057.50	15.8	-58.9	15.8	140.00
0.8	5780.00	12.0	-130.9	12.0	160.00
0.9	6502.50	8.1	-202.9	8.1	180.00
1	7225.00	4.3	-274.9	4.3	200.00



Taking a degree of shear connector value of

0.7

The moment capacity of the composite beam corresponding to the degree of the shear connector

Mcapacity = 4561.0 KN-m from the table above
 Mdesign = 4518.4 KN-m from section 7.1.4
 Mcapacity > Mdesign **ok!**

8.6 Ultimate shear strength

The structural steel is responsible for shear

$$V_{pl,Rd} = (A_v f_y / \sqrt{3}) / \gamma_a$$

where A_v is effective shear area

$$A_v = 1.04 \cdot h_a \cdot t_w, \text{ for rolled section}$$

$$A_v = d \cdot t_w, \text{ for welded section}$$

Ours case is welded and therefore

$A_v = d \cdot t_w = 14240 \text{ mm}^2$
 $V_{pl,Rd} = 2578.6 \text{ KN}$ from section 7.1.4
 $V_{design} = 1,262.98 \text{ KN}$
 $V_{pl,Rd} > V_{design}$ **ok!**

The plastic shear capacity given above cannot be exploited if the web is slender and buckles prematurely the shear buckling resistance needs to be verified when $d/t_w > 69\epsilon$

$\epsilon = (235/f_y)^{1/2}$ where f_y is in N/mm²
 $d/t_w = 55.63$ <= $69\epsilon = 56.95$
ok! no buckling effect!

Other wise the shear buckling resistance should be checked as follows

Calculate the ideal shear buckling stress $\tau_{cr} = (1000/(d/t_w))^2$ in MPA

Determine the relative plate slenderness ratio $\lambda_{\omega} = \sqrt{f_y / (\sqrt{3} \tau_{cr})}$, <=4

Determine the shear buckling strength τ_{ba} and or τ_{ba}^*

Then for

$$\lambda_w \leq 1.5$$

$$1.5 < \lambda_w < 3.0$$

$$3 \leq \lambda_w < 4$$

$$\tau_{ba}^* = 1$$

$$\tau_{ba}^* = 3/\lambda_w + 0.2/\lambda_w - 1.3$$

$$\tau_{ba}^* = 0.9/\lambda_w$$

This implies $\tau_{ab} = \tau_{ba}^* f_{yw}/\sqrt{3}$

The design value of the ultimate resistance against shear buckling

$V_{ba,Rd} = d t w \cdot \tau_{ba}^* / \gamma_{m1}$ check this against the shear force exercised from loading condition

8.7 Interaction of bending moment with shear

When the design shear force V_{sd} exceeds 50% of the plastic shear strength, $v_{pl,Rd}$, then the influence of shear on the moment capacity should be taken into account.

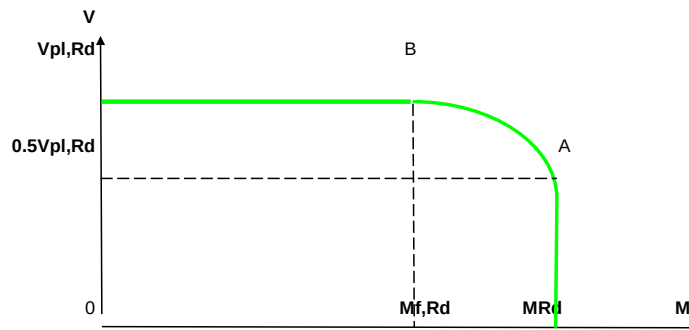
$$\begin{aligned} 50\%(V_{pl,Rd}) &= 1289.28 & \text{KN} \\ V_{sd} &= 1262.98 & \text{KN} \\ V_{sd} &< 50\%(V_{pl,Rd}) & \text{no moment capacity reduction is required} \end{aligned}$$

It is based on evidence from test that there is no reduction in bending resistance until $V_{sd} > 0.5V_{pl,Rd}$ (pt.A in the figure) and the assumption that the reduction at higher shears follows the parabolic AB. At point B the remaining bending resistance $M_{f,Rd}$ is that contributed by the flanges of the composite section, including the reinforcement in the slab. Along the curve AB,

$$M_{V,Rd} = M_{f,Rd} + (M_{Rd} - M_{f,Rd}) \left(1 - \left(2V_{sd}/V_{pl,Rd} - 1\right)^2\right)$$

where M_{Rd} is the resistance when $V_{sd}=0$

When calculating $M_{f,Rd}$, it is usually accurate enough to ignore the reinforcement in the slab. When it is included, or when the steel flanges are of unequal size, only the weaker of the two flanges will be at its design yield stress



8.8.1 Deflections

a) Camber for dead load deflection

The steel-beams shall be fabricated with a camber so the beams in principle not have any deflection without traffic load.

The camber is calculated based on that the steel section carries the deadload before the concrete has hardened.

Dead load deflection is given by $\Delta_{DL} = 5wL^4/384EI$

$$\begin{aligned} w &= 17.92 & \text{KN/m} \\ L &= 20 & \text{m} \\ E &= 200,000 & \text{Mpa} \\ I &= 0.003090 & \text{m}^4 \\ \Delta_{DL} &= 60 & \text{mm} \end{aligned}$$

Therefore, for the steel beam provide a camber of 60 mm at mid span

b) Deflection by traffic

According to ERA bridge design manual section 2.3 the live load deflection should be limited to 1/500 of the span length

$$L / 500 = 40 \text{ mm}$$

The deflection of the composite section for live load and impact should be calculated based on effective moment of inertia

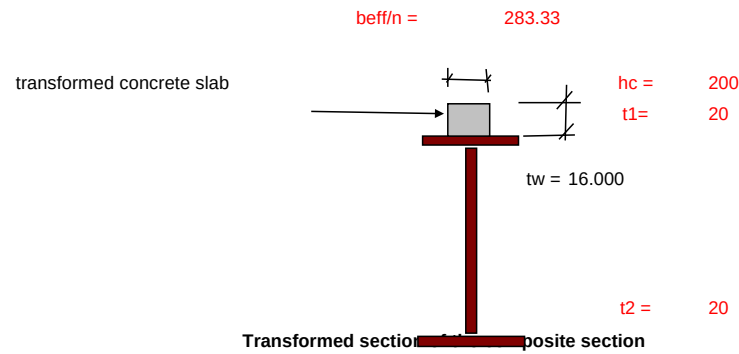
Effective moment of inertia of the composite section

Center of gravity of the composite section calculated from the bottom of the girder

Part	Area, A_i (m ²)	Centroid, y_i (m)	$A_i \cdot y_i$ (m ³)
Slab	0.057	0.99	0.056
Steel Girder	0.026	0.45	0.011
Sum	0.08		0.07

Centroid of the area

$$y_1 = \frac{\sum A_i \cdot y_i}{(\sum A_i)} = 0.82 \text{ m}$$

**Effective moment of Inertia, I_{eff} , about centroid**

Part	Area, A_i (m ²)	y_i (m)	$y_{cg} = y_i - y_1$ (m)	I_g (m ⁴)	$A_i(y_{cg})^2$ (m ⁴)
Slab	0.057	0.99	0.17	0.000189	0.002
Girder	0.026	0.45	0.38	0.003090	0.004
Sum $I_{eff} =$					

Using the above effective moment of inertia of the composite section and live loading, the max. live loading deflection is obtained from

Live load deflection from sap analysis = 14.28 mm
is less than the allowable limit calculated above **i.e OK!**

8.9 Fatigue limit state

According to AASHTO Section 6.10.10.1.2, the pitch (longitudinal spacing) of the shear connectors shall be determined to satisfy the fatigues limit state

The resulting number of shear connectors shall not be less than the number required to satisfy the strength limit state.

The pitch of shear connectors can be calculated as

$$P = n \cdot Z_r \cdot I / (V_{sr} \cdot Q)$$

Where

P = pitch in longitudinal direction in mm

n = number of shear connectors in transverse

I = moment of inertia of the composite beam

Q = first moment of the transformed slab area about the neutral axis of the composite beam.

V_{sr} = shear force range under LL+Impact

Z_r = shear fatigue resistance of an individual shear connector

Z_r can be calculated as .

$$Z_r = \alpha \cdot d^2 \geq 38 \cdot d^2$$

With d diameter of shear studs in mm

$$\alpha = 238 - 29.5 \cdot \log N$$

N number of live load cycles

In our case	n =	2	
	I =	8.516.E+09	mm ⁴
	Q =	9610372.771	mm ³
	V _{sr} =	305.06	KN - difference of Max. and Min. shear values due to live load plus impact
	d =	24	mm
	Take N =	2.80E+06	
	α =	4.78E+01	
	Z _r =	27.538	KN
	P = n·Z _r ·I/(V _{sr} ·Q) =	160	mm
	Use p =	160	mm

From strength limit state requirements as given in section 8.4

Number of shear connectors required, N, over half of the span

$$N = F_u^{(t)} / P_{RD} = 55$$

$$\text{Total} = 110$$

Considering two shear connectors in transverse total number rows of shear studs is

$$\text{Total number} / 2 = 55$$

Longitudinal spacing, P, from strength limit state requirements

$$P = \text{Bridge Span} / (\text{number} - 1) = 372 \text{ mm}$$

$$\text{Use } p = 370 \text{ mm}$$

Taking the least value of the two limit states requirement

Use shear stud connectors two in transverse with

φ	24	c/c	160
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