ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

Analysis and Design of Precast - Cast in Situ Concrete Composite Bridges

Kitaw Zewde

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A Thesis Submitted to
School of Graduate Studies,
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In
Partial Fulfillment of the
Requirements for the Degree of
MASTER OF SCIENCE
In
CIVIL ENGINEERING

Dr.Asnake Adamu
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Abstract

The weather condition and the terrain of Ethiopia need a number of bridges to be constructed for every km length of the roads. In addition to this the bridges constructed during the occupation of Ethiopia by Italian government are aged and deteriorated. These bridges require maintenance or replacement.

Considering the nature of this need and construction of new roads, the Ethiopian government investing billions of birr for road sector development out of which more than 30% of the project cost is assigned to the construction of bridges.

In spite of the large number of bridge construction in the country, the method of construction of cast in place concrete requires long time of construction and tremendous volume of false work. Further more, un expected non-seasonal rainfall may occur at the up stream of bridge construction site and affect the constructed false work partially or damage it as a whole.

Thus, this study has attempted to analyze, design and compare the amount of time and false work saved by constructing composite concrete bridges against fully cast in place concrete bridges. The quantities of construction material like concrete and reinforcing bars were increased in construction of composite bridges. Further equipments like crane and law bed are used in composite construction. As per the analysis, design and cost comparison, the developed system offer a better economic alternative in view of overall cost of construction practiced in the country.

The intention of this thesis is therefore to develop analysis and design procedure of a precast-cast in place concrete composite bridge that can be constructed out side of the site or near the site, transported and erected at the site and completed with in less time than the time required for bridge construction practiced in the country. The developed procedure is illustrated using practical example where saving in cost is compared.

List of Symbols

A_c =contact area between the section taken

A_s = Area of reinforcement computed for composite section

A_v = Area of shear reinforcement

B = Buoyancy

b = precast panel width in meter
b_c = width of precast girder in contact with cast in situ concrete
b_e = effective flange width

b_v =Width of cross section at contact surface

b_w = web width in mm

C = Centrifugal force

CF = Centrifugal force
D = dead load
I=Impact

d = effective depth

DF = Distribution factor

E = Earth pressure

EQ = Earth Quack

f_c = concrete cylindrical strength in MPa

f_y =yield strength of reinforcement

H = depth of girder

I = Impact factor

I_c =moment of inertia of composite section

K=factor to account for the distribution shear along the member

L= portion of the span that is loaded to produce maximum stress in meter

L_c= half of the span length

LF = Longitudinal force
ℓ_{vh} = Distance from zero bending moment and maximum moment of composite section
Md = Design moment of composite section

M_{L+I} = live load plus impact moment due to live load

M_{sd} = Moment due to superimpose load

M_x = Moment at a point x distance from an end support

N= Group Number

Ø = reduction factor for shear

P=load on one rear wheel of track)

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

R= Radius of curve (m)

S= Design Speed, Kilo meter per Hour

SF = Stream flow pressure

T = Temperature

t=thickness of deck in meter

V=shear force acting on section in question

V_c= shear carrying capacity of concrete

V_n = Nominal shear strength

V_{nh}= Nominal Horizontal shear strength

V_P =punching shear force

V_s= nominal shear strength provided by reinforcement

V_u =factored shear force at the section considered

W= distributed load in kN/m

X= distance measured from end support

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1. Introduction

1.1. Background

The development of communication system has been closely related to the economic development of a nation. Invariably, the developed nations have a well developed system for roads as well as other system of communications. The construction of short and long span bridges is part of the construction of these roads. Thus, it becomes mandatory to construct bridges for development of roads and hence for the development of communication.

Bridges are constructed in order to improve and facilitate the infrastructural development of transportation in the country. The construction of bridges would take place on a river, by diverting or canalizing it. The diverted river may return back to the river course when there is a sudden rain fall around the site by over topping the constructed temporary dam and destroy the false works for the super structure. It is not uncommon to see bridge construction without unexpected rainfall occurring at the upstream of the river and causing a great damage on the project. In addition to this if the bridge is intended to be constructed on a river of high and narrow gorge, the cost of construction of false work requires tremendous material resource and time. Thus, this study is intended to address these problems and seeks one alternative solution by preparing analysis and design procedure of concrete composite bridges with least application of false work and where construction is to be carried out within the shortest possible time.

1.2. Objectives

Most construction in our country is cast in place and precast construction is mainly used in construction of buildings. The main objective of this thesis is to address the problem of long time of construction of bridges and methods of shortening the construction time. In due course the volume of false work involved in this construction shall be also reduced.

The specific objective of the thesis is to develop analysis and design procedure and illustrate the application using a practical design example with an aim to serve as a guideline for designers, and demonstrate the saving attained.

1.3. What is in the Thesis

The study of the thesis mainly focuses on literature review of books and design standards of bridge and buildings. The specific reference book used for the analysis and design of concrete composite section is not available. The reference material used for the thesis is mainly presented in books written on precast and prestressed concrete structures. In those books, precast-cast in situ concrete composite structure is presented as a subtitle. In addition to this the criteria and concept used for the development of guideline of composite section is presented in bridge design standard and manuals.

Based on the main objective of the thesis, the study has focused on developing analysis and design procedure for a bridge of concrete composite section with particulars to the practice in the country. In addition to this the developed guideline is utilized and design of a typical concrete girder bridge with precast girder and half precast and half cast in situ deck is presented. The typical section selected for the girder is rectangular for the purpose of reducing the total precast weight that will require heavy equipment used for erection and transportation. If the availability of equipment is known, other cross sections like precast T-sections for interior girders and L-sections for exterior girders can be selected as a precast part of girder. The developed guideline can also serve for the analysis and design of the mentioned sections.

Assessments of sites for comparison were done to see the practical application of the thesis concerning precast - cast in situ composite construction as part of the study. For this reason two different sites were selected. The first site is where precast-cast in situ composite bridge was being constructed and the second is where cast in situ bridge was being constructed. The site selected for construction of cast in situ concrete bridge is the Kilkile River Bridge situated at 530 km on the highway from Addis Ababa to Moyale, while the site selected for precast-cast in situ composite bridge is found on the road segment from Arbaminch to Jinka. These bridges are the first concrete composite bridge designed and constructed by local consultant and contractor. Further, the saving from this type of construction is also included compared with cast in situ system.

1.4. Application of the Study and Limitation

The result of this study can be applied to construction of concrete composite bridges in which the deck slab shall be half precast and half cast in situ with precast girders.

The precast elements can be prepared during construction of substructure in which the time of construction will be reduced. Moreover, the requirements of false work shall also be reduced. Thus, from the study, both employers (client) and constructors of these bridges would get the benefit, as the construction time is tremendously reduced.

The scope of the study has been limited to the preparation of analysis and design procedure for concrete composite bridges and comparing with cast in situ concrete bridges in many terms (time and cost). Finally, the application of analysis and design procedure is illustrated through practical design example of a sample and common span of a bridge.

2. Precast, Cast in Situ and Concrete Composite Bridges

2.1. Classification of Bridges

A bridge is a structure that affords passage over low ground, water or other obstructions and can be divided in to two main parts:

- Substructure and
- Superstructure

The function of sub structure is similar to that of foundation column and wall of building because, it supports the supper structure of the bridge. The supper structure is that part of the bridge over which the traffic moves with safety. It consists of parapet or railing, road way and girder, arch or trusses over which the road way is supported.

There are several approaches of classifying bridges which includes:

- Function as: highway, rail way, foot, aqueduct and viaduct.
- Material of construction of the superstructure.

Based on the materials of construction of the superstructure:

a) Timber Bridge

b) **Masonry Bridge**: This may be of brick or stone masonry and may further assume one of the following types of arch.

1) Three hinged arch, which have hinge at supports and mid span.
2) Two hinged arch, which have hinges only at the supports.
3) Fixed hinge, which have no hinge through out the span.

c) **Steel bridge:** can be further classification as:

1) Steel- through plate type
2) Steel- girder type
3) Steel frame type
4) Steel suspension type

A further classification can be made depending on connection as riveted, welded, pin connected and bolted bridges.

d) **Reinforced concrete bridge:** These can also be classified as:

- Slab bridge

- T-beam bridge
- Box girder bridge
- Rigid frame bridge
- Multiple span portal frame bridge
- Cantilever type bridge
- Barrel arch bridge
- Spandrel arch bridge

- Depending on the life of the bridge it can be classified as
  a) Temporary bridge: Bridge used for short time and can be demolished in future and used in other areas. Example Military bridges.
  b) Permanent bridges: Bridge that is used through out its life time.
- Depending on relative position of flooring, bridge can be classified as deck type, through and semi-through type. This classification is based on the location of flooring deck with respect to the supporting structures.

In the deck type of bridge, the deck or flooring is supported at the top of supporting structure. When the flooring is supported at the bottom, it is called Through Bridge. But, if the flooring is supported at the intermediate level of the structures, it is called Semi-through Bridge.

- Depending on the form of supper structure it is classified as arch, girder, truss, suspension bridge.

Based on the above classification, the study of the thesis will focus on the bridges grouped under reinforced concrete bridge with cross section of T–beam. Further classification on concrete T–Beam Bridge may be done based on the method of construction as a composite or non composite bridge. Therefore a reinforced concrete T-beam bridges with *composite section* is studied in this study.

2.2. Factors Affecting the Selection of Type of Bridge

Factors affecting the selection of type of bridges include:

1) Volume and nature of the traffic
2) The nature of the river and its foundation soil
3) The availability of construction material and fund

4) Construction time limit for completion of the bridge.
5) Physical feature of the river
6) Availability of skilled manpower
7) Facility available for construction and maintenance
8) Economic span length of the bridge
9) Level of high flood level and the clearance requirement
10) Climatic condition
11) Foundation condition
12) Total length of the bridge
13) Width of the bridge
14) Live load on the bridge

With the help of the above factors, every aspects of the bridge shall be considered and the type of bridges which is most economical and can give maximum service should be selected.
Out of the factors affecting the selection of type of bridge, some of the criteria's will be the main factors that affect the selection of type of bridge studied in the thesis.
These factors are:

- Availability of construction material and fund
- Construction time limit for completion of the bridge
- Availability of skilled manpower
- Facility available for construction and maintenance

2.3. Precast Concrete Bridge

Precasting involves the casting of concrete away from the site of final position. It can also be produced near the site (4). The member of precast element produced either in a permanent plant or temporary arrangement and eventually erected at the final position. Precasting permits better control in mass production. It is also economical. Cast in place concrete require more formwork and false work per unit of product but save the cost of transportation and erection, and it is better used for large and heavy members.

In between these two methods of construction, it is possible to precast part of the member, erect it, and then cast the remaining portion in place, which is called composite construction.

The precast element in a structure of composite construction can be more easily joined together than these in totally precast structure. By composite construction, it is possible to save much form work and false work as compared to fully cast in place construction. However the stability of each type must be studied with respect to particular condition of structure. Precasting technique is also used to produce prestressed concrete.

Precast concrete ranges from unreinforced concrete to conventionally reinforced with mild steel to prestressed concrete. Precasting offers the advantage of prefabrication, quality, mechanized production, speed and economy. There are two main uses of precasting; the first and the most use is the mass production of standardized elements and secondly the casting of unique and complex units in position under conditions where by the necessary quality and tolerance control can be achieved or made (2).

The specific advantages of precasting include:

- ***Control of shrinkage***: in which with low water cement ratio and steam curing, much of the shrinkage is reduced and further the major portion of it may take place before erection.
- ***Reduction of creep:*** with proper curing and aging before erection, the strength, maturity of element and its modulus of elasticity are all at higher levels at the time of loading which reduces the effect of creep.
- ***Control of dead load deflection:*** Precast concrete element may be erected and adjusted to exact position on the theoretical profile.
- ***Quality control:*** Precast concrete elements may be manufactured under the best conditions for forming, placement of reinforcement, placement and vibration of concrete and curing.
- ***Timely availability:*** Many standardized mass produced elements can be supplied to a construction site with in a very short time.
- ***Erection over existing traffic:*** Erection of precast element over existing traffic minimize false work requirement. Moreover, the time required for temporary false work is small and maximum reuse of false work is facilitated.

- ***Economy:*** Labor and equipments can be utilized with maximum efficiency, reuse of form work to a maximum number of times, erection during the most favorable seasons; all make the system more economical as compared to fully cast in situ.
- ***Suitability for composite construction:*** Precast concrete segments may be combined with cast in place concrete to act in composite action as a monolithic structure making provision for horizontal shear transfer. Thus, the precast unit is designed to serve as a formwork for cast in place concrete. Accordingly, due consideration shall be given to various stages of loading to prevent over stressing of the precast elements while the cast in place concrete is green.

The objective of the thesis is to use this later advantage of precast concrete element in the construction of short span girder bridges. By so doing, the thesis introduces this method to all parties of users to practice the system in the country. Therefore, method of analysis and design of precast girder with half precast and half cast in place deck bridge is developed in this study.

2.4. Composite Construction

2.4.1. Definition

A section is said to be composite if it is made of two different materials. In the case of concrete structure, it is said to be composite, if it is casted at different time one over the other after curing of precast element. Composite sections act monolithically if proper transfer of horizontal shear is provided.

2.4.2. Advantage of Composite Sections

a) Manufacturing of precast element is simplified as it does not require elevation of casting.

b) Standard precast girders may be grouped together or spread apart to accommodate heavy or light loads, respectively.

c) The cast in place top slab ties the structure together and gives a uniform continuous surface.

d) The precast girder and precast part of the slab support the cast in place part of the slab.

e) Combination of light weight concrete with conventional sand and graveled concrete is facilitated.

f) Shear transfer is more easily provided as compared with fully precast girder and precast deck.

Success with composite construction depends on insuring proper shear transfer at the interface. The horizontal shear transfer may be obtained by bond alone, provided the upper surface of precast member is roughened. Thus, just before pouring the top slab, the surface should be cleaned with a water jet. When the shear loads are higher, as in bridge construction, reinforcement for shear transfer has to be provided.

With precast girder, reinforcement ties in the form of mesh of bars are required. In addition to the rough surface, Shear keys resist the horizontal shear. But a number of tests show that shear keys have been found to be less effective than formerly believed (7). The shear keys do not come to action until the surface bond has failed and shear keys are generally not used in composite slabs. In using tie reinforcement, a large number of small bars give better shear transfer than few number of large diameter bars (7).

2.4.3. Disadvantages of Composite Sections

The disadvantage of composite section in bridge construction are that they

a) Require heavy machine for erection and transportation of precast element

b) Need additional reinforcement for horizontal shear

c) Require accuracy of dimension of the precast element

d) The overall depth may be increased as the precast unit is required to fulfill the deflection requirement by it self.

2.4.4. Lifting and Erection of Precast Elements

Cracking should not occur while lifting and handling precast units. Buckling should be prevented by suitable stages or supports. Particular care should be taken at points of picking to avoid concentrated detailing and additional reinforcement while lifting. It should be noted that when precast units are removed from forms, there may be additional load due to friction, and suction (4). Picking and handling should be treated as a dynamic load. Lifting loop for picking and handling must be designed with a safety factor of 6 that is, the ultimate capacity of the loop should be 6*DL, where DL is dead load (2). Embedment of the hook should be adequate to prevents pull out bond failure under the above design load. Adequate concrete section and reinforcement should be available to develop the ultimate design load.

The angle which a line makes with the lifting loop, at all positions during picking and handling, should be considered. The increased force due to angular load shall be used in design. In doing so, the member should be checked for the actual components of the loop forces to insure capacity of a section against buckling (2).

Bundled loops of stand may be employed provided that they are equalized and all strands will work together. Their end can be splayed out in the concrete. Smooth mailed steel may be used provided that adequate embedment against pull out is provided (6).

Deformed reinforcing bars should not be used for picking loops because the deformation results in stress concentrations to the shackle pin. In addition, the reinforcing bars are hard grads or re rolled rail steel with low ductility and low impact strength at low temperature.

2.4.5. Transportation

Moderate precast units can be transported by truck. Specially precast units of slab should be transported by truck. During transportation, precast members are subjected to dead load stresses, dynamic (impact) stresses and lateral instability. Ideally, a member being transported will be supported at the same points as in the final structure but, it is not practical in most cases, and thus special design studies must be made for transportation (4). Lateral instability is the course of most accidents in transportation. To overcome this problem, two or more such units are transported together by putting them side by side and tying them to give lateral stability.

2.4.6. Erection methods

The method required for the erection of precast element is determined by the span, height of structure, its location, the topography, the weight, size and configuration of precast element. In addition, the availability of erection equipment also determines the method of erection (4). One of the following methods can be used for erection.

i. Crane

In using crane lifting loops must be suitable for all angles of lift to prevent localized crushing or over-stressing. The position of crane, angle of lift and working radii must be plotted on the working drawing. When two cranes are used to erect a single member, each should have capacity to take at list 66 % of the total load (8).

ii. Direct launching

A light steel launching nose is over balanced by a counter weight or heavy rearward extension. The forward movement may be accomplished by jacking, rolling, trucked carriages or cranes. The girder must be analyzed for temporary stress condition as it is cantilevered for ward. This method is most suitable for single span in remote areas.

Other methods stated in literatures are complicated to practice in our country for example sliding of segments, launching gantry, cantilever- suspended spans are the methods of erection in some parts of the world.

3. Analysis and Design of Precast and Cast in Situ Concrete Composite Bridges

3.1. Design Load

In order to form a consistent basis for design, standard loading conditions are applied to the design model of structure. The principal loading which highway bridges are designed for is a truck loading. Even if a variety of trucks are using the bridge, a standard set of design loading will be used. In Ethiopia AASHTO standard design trucks are used for bridge. In 1944 a hypothetical trucks classes designated as H and HS class were developed by AASHO. These design vehicles were created with two and three axles, respectively.

According to the axle load survey in Ethiopia in 1998, it was found that some 20% of the heavy trucks were overloaded. Based on this, ERA developed new design manual. This manual increases the axle load of a design vehicle. In this manual two different design vehicles are selected. These are used for design of new bridges and for determination of load bearing capacity of existing bridges.

For existing bridges, the design vehicle should be HS-25 (instead of HS-20) according to the results from several studies (10). The loading configuration of HS-25 is the same as that of HS-20 but the loading of each axel are amplified by 25%. For existing bridge, before a major repair or reconstruction such as widening of a bridge starts, the actual load carrying capacity of existing bridge should be checked for HS-25 loading condition. The decision can be made whether any strengthening is necessary together with major construction work.

In the case of new bridge construction two loading conditions such as HL-93 vehicle load or tandem load shall be considered. The HL-93 load consists of a design truck and a lane load at the same place. This load can not be compared with HS-25 as the load factor method is different from the earlier empirical methods. The design vehicle live load for Ethiopia is therefore HL-93 load, which also includes a lane load of 9.3kN/m. This makes the total load some 25% greater than the previous AASHTO based HS-20 design vehicle (10).

The design tandem for Ethiopia represents exceptional loading. This tandem load shall be applied for military vehicle loading and for strategic bridges. Thus, tandem loading is selected as a design load for a bridge that can not be replaced by temporary bridges.

3.1.1. Permanent Load

Permanent loads are those loads, which remain, and act on a bridge through out its life. Permanent loads are divided into three major categories. Only loads acting on superstructure is of concern in this study.

a. **Dead Lead**: - The dead load on supper structure is the aggregate weight of all superstructure elements above the bearing. This would include deck, wearing surface stay-in-place forms, side walk, railing, parapets and girder.
b. **Superimposed Dead Load**: - These types of load exist in composite construction. These are loads placed on super structure after the deck has cured and begun to act as a load resisting part.
c. **Pressures: -** pressure due to water and soil are also considered as a permanent load if their effect is transferred to the supper structure element.

3.1.2. Temporary Loads

Temporary loads are those loads which act on a bridge for short period of time. As dead load is the principal permanent load, vehicle load represents the major temporary loading.

a) Vehicle Load Based on HS-20

The term live load means a load that moves along the length of span. Therefore a person walking along the bridge can be considered as a live load. However, high way bridges have to be designed to with stand vehicle loads which are more conservative for pedestrian loading.

According to 1944 AASHO classification, Traffic loadings are grouped in to five standard vehicles based on their gross weight.

H10-44	(20,000 lb – 90kN)
H 15- 44	(30,000 lb- 135kN)
H 20- 44	(40,000 lb – 180kN)
HS 15-44	(54,000 lb- 240kN)
HS 20-44	(72,000 lb- 325kN)

In the classification, the first number is half of the gross load in ton and the second is the year at which the standard is published. The distribution of the load is 20% for front axel, and the remaining 80% of the load to rear wheel for H truck. For HS type truck the load on every rear axel is equal to that of H truck.

The selected design truck is moved on the bridge along the span to determine the point at which maximum internal forces is produced by this load. The HS truck has a variable spacing between the two rear axles. This distance varies between 4.27m to 9.14 m. For simple span the minimum value (4.27m) produce maximum moment. In the case of continuous span the distance between the two axles would be selected so that it produces maximum negative moment. Uniformly distributed load in conjunction with a concentrated load is used to replace a train of truck loads for simple span bridges and for determining maximum positive moment in continuous spans, only one concentrated load is used with uniformly distributed load, however to determine maximum negative moment in continuous spans, two concentrated loads are used. Reduction in live load for three or more lanes is permitted. Thus, use 90% for three lanes and 75% for four or more lanes (AASHTO Art. 3.12)

b) Design Vehicle Load Based on HL-93

Vehicular live loading on a roadway of a bridge or incidental structures, designated as HL-93 shall consist of a combination of the:

- design truck or design tandem, and
- design lane load

The live load model consists of either a truck or tandem coincidental with uniformly distributed lane load. The lane load is assumed to occupy 3m transversely within the design lane.

Design Truck

The weight and the spacing of the wheel for design truck shall be as specified in Figure 3.1. Unless specified, the spacing between the two 145kN axels shall vary between 4.3 to 9m to produce extreme internal effect.

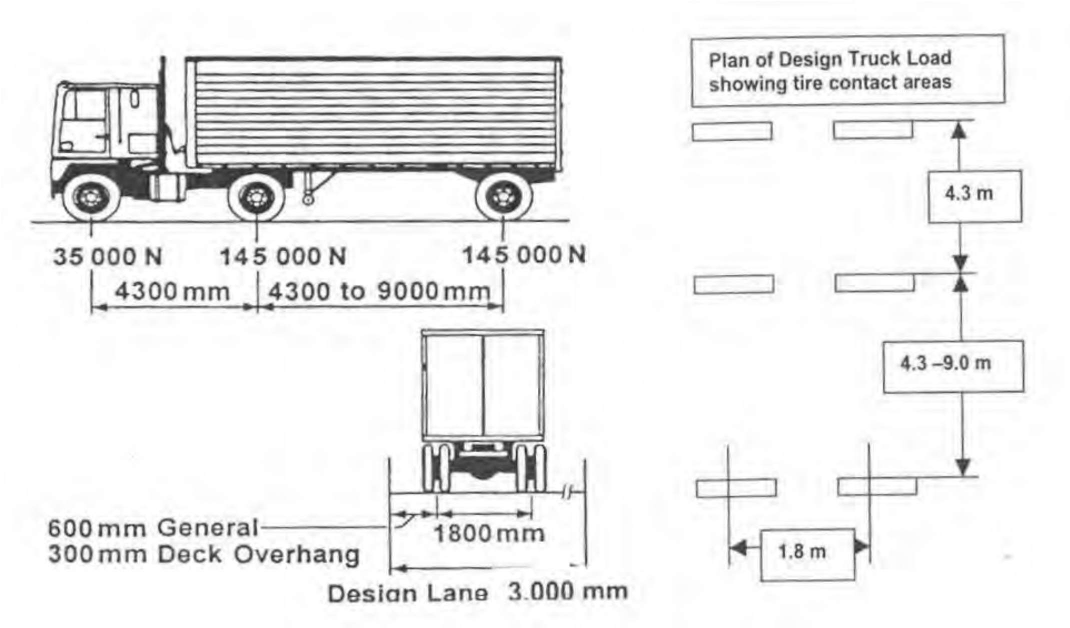


Figure 3.1 Characteristics of the Design Truck (10)

Design Tandem

The design tandem used for strategic bridge shall consist of a pair of 110kN axles spaced at 1.2m apart. The transverse spacing of the wheel shall be taken as 1.8m. The design tandem load is shown in the Figure 3.2

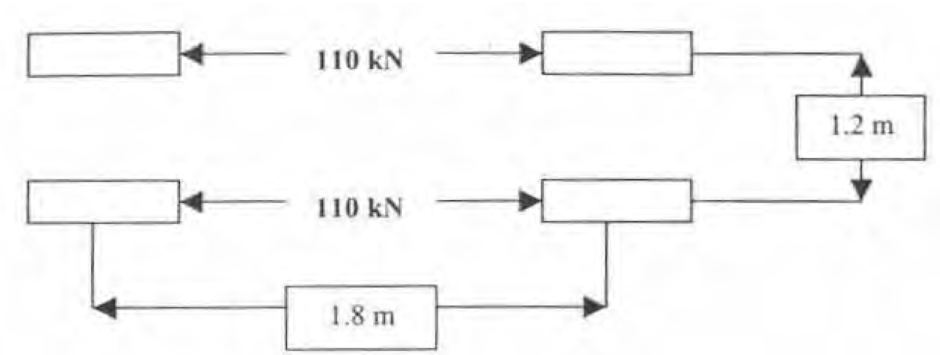


Figure 3.2 Design Tandem Loads (10)

Design Lane Load

The design lane load shall consist of a load of 9.3kN/m, uniformly distributed in longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over 3m width with in the design lane.

c) Earthquake Loading

Earth quake loading is a product of natural loading which depend on geometric location of the bridge. Seismic force primarily affects bridge substructure components. The earth quake forces can be described as a function of

-The acceleration coefficient

-Soil Type

-The fundamental period

The effect of earthquake in Ethiopia is grouped in to four different zones.

Bridge in seismic Zone 1-3 need not be analyzed for seismic loads, regardless of their importance and geometry. Seismic analysis is not required for single-span Bridge, regardless of the seismic Zone (10). Connection between the bridge superstructure and abutment shall be designed for minimum force requirements

d) Wind Load

Like earth quake loading, wind load offers a complicated set of loading condition with regard to superstructure of a bridge, wind forces are applied in transverse and longitudinal direction at the center of gravity of the exposed region. For conventional girder or beam type of bridge the wind load of 2.39 kN/m^2 with minimum total force being 4.38kN/m shall be used for base wind velocity (106.9km/hr).The wind load on structure varies in the ratio of the square of the design wind velocity to the square of the base wind velocity (AASHTO Art. 3.15).

e) Longitudinal Forces

Longitudinal forces are result from vehicles breaking or acceleration while on a bridge. To account the effect of longitudinal force 5% of the live load along with the concentrated force for a moment be used (AASHTO Art. 3.9).

This force is applied 1.8m above the top of the deck surface. The effect of longitudinal force on supper structure is not important but substructure will be highly affected by longitudinal forces. The more stiff or rigid the structure, the severe the effect of longitudinal forces will be.

f) Centrifugal Forces

For structures on horizontal curve, the effect of centrifugal force must be calculated. Centrifugal force simulates a vehicle traveling along the bridge and following a curvilinear path.

C= 0.000727 SD

Where C= Centrifugal force, percent of live load with out impact

S= Design Speed, kilometer per Hour

D= Degree of Curve.

The force is applied 1.3m above the surface of the deck. Unlike longitudinal forces, centrifugal forces are computed using truck loading rather than lone loading. One standard design vehicle is placed in each design traffic lane.

If the span is supper elevated, it has to be taken into account for selected design speed.

g) Impact

In order to account for the dynamic effect of a sudden loading of vehicle on a structure, an impact factor is used as a multiplier for certain structural elements. From basic dynamics, we know that a load that moves across a member will produce large stress than one statically placed on it. The amount of impact allowance increment is expressed as a function of live load stress, and shall be determined by the formula: (AASHTO Art. 3.8.2)

$$I = \frac{15.25}{L + 38.125}$$

I= Impact factor (maximum 30%)

L= portion of the span that is loaded to produce maximum stress meter

Live load stress is multiplied by this factor and added to the stress due to live load. As the span length increase the impact factor will be reduced. Therefore any span less than 12.7meter, the impact factor, I will be 30%. In addition to the dynamic effect of the vehicle, impact factor take the effect of vibrating and striking imperfection in the deck.

h) Construction Loads

During the erection of a structure, various members come under loading condition which is induced by self weight and dynamic effect during transportation. The designer should take into account and provide necessary detailing to withstand this load. For the effect of such picking loads, 50% to 100% of self weight is added for dynamic stresses.

3.1.3. Deformation and Response Loads

Deformation loads are those loads induced by the internal or external change in material properties or member geometry. The effects of deformation such as creep and shrinkage in concrete induce stresses on a member outside the conventional dead and live loads. Response leads are those loads caused by response of a structure to a given loading condition. Up lift and settlement are examples of response loads. During designing minimum reinforcement should be provided according to the code to take deformation and response loads.

3.1.4. Combination of Loads

Vehicle loads would be acting on a bridge when another loads like wind load, seismic loads, stream forces, etc applied to the bridge. To account for this set of load combination which are divided into various groups are formulated. These groups represent probable occurring combination of load on a structure. (AASHTO Art. 3.22)

$$GroupN = \gamma \left\{ \begin{array}{l} \beta_D D + \beta_L (L + I) + \beta_C CF + \beta_E E + \\ \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL + \beta_L LF \\ + \beta_R (R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE \end{array} \right\}$$

Where

N= Group Number

γ = Load factor

β = Coefficient

SF = Stream flow pressure

D = dead load

I=Impact

E = Earth pressure

B = Buoyancy

W = Wind load on structure

WL = wind load on live load

LF = Longitudinal force

CF = Centrifugal force

R = Rib shortening for arch or frame

S = shrinkage

T = Temperature

EQ = Earth Quack

ASSHTO present value of γ and β for working stress (Service load) and load factor (limit state) design method in AASHTO (Table 3.22.1A). From the Table γ=1.3 and β_L=1.67 β_D=1 are used for load factor design method. According to ERA bridge design manual γ=1, β_L=1.75 and β_D=1.25

3.2. Load Distribution and Types of Load Distribution

Bridge loads are transmitted from the deck to the supporting superstructure and then to the substructure element. If a truck is traveling over a girder, it is logical to say the girder carry that load. This girder, however, is connected to the adjacent girder through deck. The connectivity allows different members to act together in resisting loads. As a result of being connected with the girder of adjacent members, it is necessary to determine part of the loads that are carried by the girder and adjacent girder. The modeling of how loads are distributed is complicated but it is possible to examine the variables which influence the distribution. The influencing parameters are the function of the bridge superstructure cross section (8).

These are:

- Type of floor
- Spacing between girders
- Spacing between cross girders
- Size and position of Loading.

In order to simplify the computation of load distribution, AASHTO chooses to utilize a distribution factor based on type of flooring and stringer or girder spacing.

For the particular case, i.e. for concrete flooring with girder

$$DF = \frac{S}{1.98}$$ For single lane and S less than 1.83m.

$$DF = \frac{S}{1.83}$$ For two or more lane and S less than 3.05m.

Where ***S*** is the average girder spacing of the bridge.

For ***S*** greater or equal to 3.05 m the deck between the girders can be assumed as a simply supported and the reaction due to traffic loading or the girder can be taken as a distribution factor.

In addition to deck type and spacing of girder the orientation of the girder will affect the distribution.

The types of load distribution are:

a) Interior Longitudinal Members:-

In calculating bending moment in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. Live load bending moments are computed using one set of front and rear wheels multiplied by distribution factor (8).

b) Exterior Longitudinal Members:-

Exterior girders are subjected to dead load over and above those caused by the deck and girder. Loads such as curbs, side walks and railings are placed on the exterior girder after the deck has cured (superimposed dead load), can be distributed equally among all girders (primary members). (AASHTO Art. 3.23.2.3.1.1.)

The live load bending moment for out side road way stringer or beam shall be determined by applying to the stringer or a beam the reaction of the wheel load obtained by assuming the flooring to act as a simply supported, between stringers or beams. (AASHTO Art.3.23. 2.3.1.2.)

c) Transverse Members:-

AASHTO specifications do not allow any lateral distribution of loads for transverse members.

3.3. Guideline for Analysis of Precast and Composite Concrete Deck.

Analyses of bridge with composite deck follow different steps than non-composite one. To these effect two basic steps of analysis of precast part of the deck and analysis of the composite deck shall be considered.

3.3.1. Precast Part of the Deck

The precast deck shall be analyzed for the following loadings.

-Self Weight plus Load during transportation (50% to 100% of self weight) (8).

-Self weight plus weight of cast in situ part of the concrete plus man-power load for casting of concrete assumed to be 1.5kN/m^2 (10).

Precast part of the deck is simply supported rectangular section supported on girders in transverse directions.

3.3.2. Composite Deck Section

This part of the deck is analyzed for the loadings of superimposed dead load and vehicle loading of HL-93 associated with impact.

The composite deck is analyzed as continuous rectangular section supported over the girders in transverse directions.

3.3.3. Thickness of the Deck

Appropriate thickness need to be provided on the basis of deflection requirement. The minimum thickness of reinforced concrete depends on the support condition and the support spacing. This criterion is defined for concrete flexural members so that they are provided with adequate stiffness to resist excessive deflection. The deflection of simple or continuous slabs shall not exceed 1/800 of the span length (12). The thickness of the composite slab can be determined from.

$$t = \frac{1.2(S + 3.05)}{30} \quad \text{for simply supported slab}$$

$$t = \frac{(S + 3.05)}{30} \quad \text{for continuous slab}$$

Where, t=thickness of deck in meter.

S=effective span in meter.

This thickness has to be sub divided in to a thickness of partially precast and partially cast in situ. The precast part should satisfy the load specified in section 3.3.1.

According to EBCS-2, 1995 the minimum thickness of precast part of the deck shall be 80mm. In addition to this the minimum thickness of slab shall not be less than:

$$d = (0.4 + 0.6x\frac{f_y}{400})\frac{L_e}{\beta_a}$$

where, d = effective depth

f_y = yield strength of steel

L_e = effective span

β_a = factor in (Table 5.1 of EBCS-2.1995).

Based on this value, the thickness of cast in situ deck is determined by deducting the precast thickness from composite thickness determined above using AASHTO specification. This thickness should satisfy the minimum requirement for concrete cover a space between reinforcement and precast element.

3.3.4. Determination of Internal Actions

Unit weight of concrete =25kN/m^3

Self weight, W = $25t$ kN/m^2

a) Precast part of the slab

Moment, $M_S = \frac{WL^2}{8}$ and occur at mid span

Shear, $V = \frac{WL}{2}$ and occur at support

Where W=Self weight of pre cast + weight of cast in situ

L=Clear spacing between girders

During transportation with four point hook, span moment and diagonal shear will be reduced and no need of checking span moment, but the punching shear has to be checked. Figure3.3. shows location of hooks for transporting the precast part of the deck.

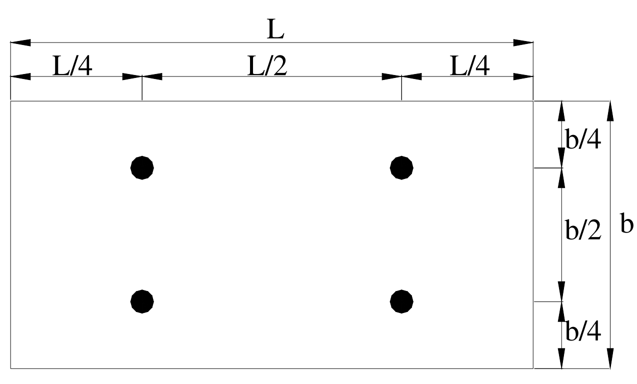


Figure 3.3 Location of hook for precast deck

$$V_P = \frac{WLbt_P}{4}$$

Where b= precast panel width in meter

t_p = thickness of precast panel

L=length of precast panel

W=25kN/m^3

V_P =punching shear force

Thus, the precast deck need be checked for the entire design criteria requirement.

b) Composite slab

$$M_1 = \frac{WL^2}{10}$$, moment due to dead load

$$M_2 = \left[\frac{S+0.61}{9.76}\right]P$$, Live load moment per unit width of slab

For slab continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formula (AASHTO Art. 3.24.3.1)

Thus, use $M_2 = 0.8\left[\frac{S+0.61}{9.76}\right]P$

S= effective span length in m

P=load on one rear wheel of truck (72.5KN)

S is span length of a member that is not built integrally with their supports or the clear span plus the depth of the member but not exceed the center to center distance of the support. In analysis of continuous and rigid frame members, distances to the geometric center of a member shall be used in the determination of moments (AASHTO Art. 8.8). But moments at the face of support may be used for design. The factored load method is used for design of the deck. Therefore, the capacity of the deck should be greater than the factored internal moment.

3.4. Guideline for Design of Precast and Composite Concrete Deck

Composite flexural members consist of precast and cast in situ concrete elements constructed in separate placement but interconnected so that all the element respond to superimposed loads and live loads as a unit (AASHTO Art. 8.14.2.1). The entire composite member shall be investigated for all critical stages of loading and shall be designed to support all loads induced prior to the full development of the design strength of composite member.

Shear reinforcements shall be provided in order to prevent separation of individual element (AASHTO Art. 8.14.2.2). As the load carrying capacity of these members depends on shear transfer for composite action, due attention must be given to horizontal shear.

3.4.1. Diagonal Shear

When the entire member is assumed to resist the diagonal shear force

$$V_u = \phi V_n$$

$$\phi = 0.85$$

Where, V_u =factored shear force at the section considered

V_n = Nominal shear strength computed by

$$V_n = Vc + V_s$$

V_c= nominal shear strength provided by concrete

V_s= nominal shear strength provided by reinforcement

V_c= $0.166\sqrt{f_C^{'}}b_W d$

$f_c^{'}$ = Cylindrical strength of concrete (MPa)

b_w=width of the beam

d= effective depth

V_s= $\frac{A_V f_Y d}{S}$

Where, A_v= area of shear reinforcement with in a distance ***S***

S= spacing of shear reinforcement

f_y= yield strength of steel

d= effective depth

When inclined stirrup is used the capacity of shear provided by reinforcement will be increased to:-

V_s= $\frac{A_V f_Y d}{S}(\sin\alpha + \cos\alpha)$

Where, α = angle of inclination of stirrups with horizontal

3.4.2. Horizontal Shear

In composite member, full transfer of horizontal shear force shall be assured at contact surface of interconnected element. The design horizontal shear force V_{nh} shall be

$V_u \leq \phi V_{nh}$

Where, V_u=factored horizontal shear force at section considered.

ϕ = strength reduction factor (0.85 for shear)

V_{nh}= Nominal Horizontal shear strength

Horizontal shear V_u may be investigated by computing the actual change in compressive or tensile force to be transferred in any segment not exceeding one-tenth of the span. Provision shall be made to transfer that force as horizontal shear between inter connected elements (8). Horizontal shear shall not exceed the permissible horizontal shear force V_{nh} (AASHTO Art. 8.15.5.5.4).

The nominal horizontal shear strength V_{nh} is the strength of the joint between the two surfaces. Based on this nominal horizontal shear strength, it is possible to express shear stresses as

$$v_{nh} = \frac{V_{nh}}{b_v \ell_{vh}}$$

Where b_v=Width of cross section at contact surface

ℓ_{vh} = Distance from zero bending moment and maximum moment in composite section and ℓ_{vh} can be determined by using Figure 3.2 for different support condition.

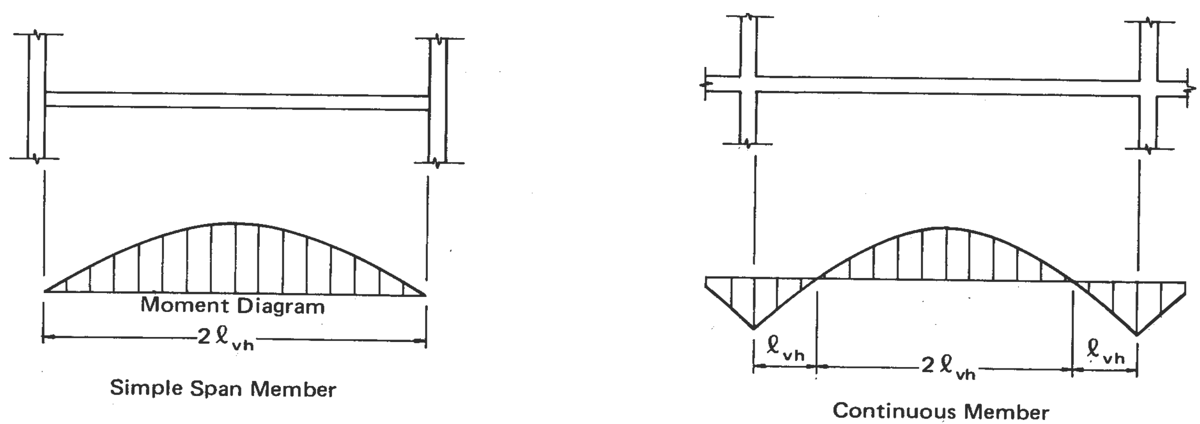


Figure 3.4 Length of horizontal shear span (ℓ_{vh}) (2)

AASHTO specifies the following allowable limit for this shear stress.

a. When contact surface is clean, free of laitance and intentionally roughened to 6.35 mm the shear stress shall not exceed 0.25MPa.

b. When minimum tie reinforcements are provided and contact surface is clean and free of laitance but not intentionally roughened, the shear stress shall not exceed 0.25MPa.
c. When minimum tie reinforcements are provided and contact surface is clean and free of laitance and intentionally roughened to 6.35mm, the shear stress shall not exceed 1.1MPa.

A newly prepared ERA 2002 bridge design manual set the capacity of the section constructed in layer as:

V_c = 0.70MPa for intentionally roughened to an amplitude of 6mm

V_c = 0.52MPa for others.

Minimum tie reinforcement for horizontal shear

The minimum area of tie reinforcement shall not be less than $A_{v\text{-}min}$

Where, $A_{v\text{-}min} = 0.345 \dfrac{b_v S}{f_Y}$ (AASHTO Art.8.16.6.5.5)

Where b_v =Width of cross section at contact surface (mm)

S= Spacing which can not exceed four times the list web width of support element or 60cm

f_y=specified yield strength of reinforcement in MPa

If the shear stress provided by concrete is not sufficient to carry the shear in the system, shear reinforcement can be provided. The area of shear reinforcement perpendicular to the axis of the member subjected to the horizontal shear can be determined by:

$$A_v = \frac{(V - V_C) b_V S}{f_y}$$

Where V= Horizontal shear in the slab

V_c= shear carrying capacity of concrete, b_v= Width of the composite element

S= Spacing of perpendicular shear reinforcement, f_y= yield strength of the reinforcement.

3.4.3. Flexure

Rectangular Section with tension reinforcement only (AASHTO 8.16.3.2.1)

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_c'}\right)$$

$$\Rightarrow M_U = \phi A_S f_y \left(d - \frac{a}{2} \right) \qquad \phi = 0.9 \text{ for flexure.}$$

Where, $a = \dfrac{A_S f_y}{0.85 f_C^{'} b}$

Exact value of "***a***" can be determined by trial and error by setting '***a =0***' at the beginning of calculation.

The balance reinforcement ratio (AASHTO Art. 8.16.2.7)

$$\rho_b = \frac{0.85 \beta_1 f_C}{f_y} \left(\frac{600.3}{600.3 + f_y} \right)$$

ρ_b =Balance reinforcement ratio

$\beta_1 = 0.85$, for $f_c^{'} \leq 27.6$MPa.

For concrete having strength greater than 27.6MPa, $\beta_1$is reduced at the rate of 0.05 for every 6.9 MPa increment. But the minimum value β_1 is 0.65

***Rectangular section with compression reinforcement*.**

The design moment strength shall be computed as (AASHTO Art. 8.16.3.4)

If $\quad \dfrac{A_S - A_S^{'}}{bd} \geq 0.81 \beta_1 \left(\dfrac{f_C d^{'}}{f_y d} \right) \left(\dfrac{600.3}{600.3 - f_y} \right)$

Then, $M_u = \phi(A_s - A_S^{'})$ fy (d-a/2) + $A_S^{'} f_y(d-d')$

Where, $a = \dfrac{(A_s - A_s^{'}) f_y}{0.85 f_C b}$

When the value of $\dfrac{(A_s - A_s^{'})}{bd}$ is less than the value required by the above equation, the effect of compression reinforcement can be neglected.

3.4.4. Distribution Reinforcement

To provide for lateral distribution of the concentrated live load, reinforcement shall be placed transverse to the main reinforcement (AASHTO Art. 3.24.10). The amount of distribution reinforcement shall be a percentage of main reinforcing steel required for positive moment.

For main reinforcement parallel to traffic

$$\text{Percentage} = \frac{55.22}{\sqrt{S}}\text{, maximum 50\%}$$

Main reinforcement perpendicular to traffic

$$\text{Percentage} = \frac{121.4}{\sqrt{S}}\ \text{maximum 67\%}$$

S= effective span of the deck in meter.

3.4.5. Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stress shall be provided near the exposed surface of slab not otherwise reinforced. The total area of reinforcement provided shall be at least 265mm^2/meter width of deck (AASHTO Art. 8.20.2)

3.4.6. Design of Overhang Deck

The slab is designed to support the load independently of the effect of any edge support along the end of the cantilever. Two types of loadings such as wheel load and railing load will act on the cantilever part of the deck. Railing load and wheel load shall not be applied at the some time.

The wheel on the element perpendicular to the traffic shall be distributed over a width: (9)

$$E=0.8X + 1.143$$

And the moment per meter width of slab shall be given as:

$$M = \left(\frac{P}{E}\right)X$$

Where X is the distance in meter from load to the point of support

The railing loads shall be applied at the center of rail. This force is acting in horizontal direction and has a magnitude of 44.8kN (AASHTO Art 2.7). The effective length of slab resisting post loading shall be:

$$E=0.8X + 1.143 \qquad \text{if no parapet is used}$$

$$E=0.8X + 1.524 \qquad \text{if parapet is used}$$

Where, X is the distance in meter from center of the post to the point under consideration.

3.5. Precast Concrete Girder

3.5.1. Analysis Guideline

Girders are the main horizontal load carrying members in bridge structure. In precast concrete structure they must at some point support self weight and weight of precast and cast in situ deck. Girder analysis falls into two distinct groups. These are internal and external girders. Internal girders are symmetric in cross section while external girders may or may not be symmetric. Precast reinforced girder may act compositely with certain types of floor slabs if a proper interface shear transfer mechanism is provided. The main advantage of using composite girder is to increase the flexural strength and stiffness and also reduce deflection of precast girder due to superimposed dead load and live load. Full interaction between the in situ concrete to precast slab is assumed.

The end of transverse bar in the precast deck hooked and projected to the cast in situ concrete so that proper transfer of horizontal shear is insured. Transverse tie steel will automatically be placed as part of the topping, but as before a minimum area ratio of 0.2 percent is recommended. Full interaction between the precast deck and the cast in situ topping is assumed. However interfaces shear calculation for shear force due to imposed load and live load is made according to the analysis and design of horizontal shear in composite deck.
Flexural analysis is carried out in two stages. This analysis is before and after the structure act compositely. To this effect, the following guidelines are used as starting point of design.

- ***Loading condition***

Precast girder is analyzed as a simply supported beam for the following loading combination

a) Self weight plus 50% to 100% of self weight due to dynamic effect of the girder during transportation. In addition to this the bending moment induced due to horizontal component of the hook during picking has to be added.

If two cranes are used, the moment due to horizontal component of the cable is ignored.

b) Self weight plus load transferred from composite deck plus $1.5kN/m^2$ of live load due to worker.

- ***Girder Depth***

Recommended minimum depth of girder is: (AASHTO Art. Table 8.9.2)

$$h=0.07S$$

Where, S is span length of girder, and h is its depth

- ***Diaphragm*** is a transverse girder that is used to stiffen the main load bearing girders.

Maximum spacing of diaphragm is 10m.Therefore; at least one intermediate diaphragm is required at maximum moment for girder bridge having a span greater than 10m in addition to diaphragm at supports.

3.5.2. Design Guideline

The precast concrete girder is designed for the first stage loading and the resulting effects. At the first stage the maximum internal action from the load combinations of self weight plus precast slab and cast in situ deck or the effects of self weight and internal action developed due to transportation. The internal action developed during transportation is 50% to 100% of self weight and moment due to horizontal component of the tension force in the cable.

The area of reinforcement determined using these loading combinations and the selected precast concrete girder cross section is A_{s1}. Using the section selected at this stage, the analysis of second stage is proceeded.

3.6. Composite Section

3.6.1. Analysis Guideline

The second stage of analysis is carried out using the cross section determined in the first stage and section of concrete deck. During this stage, the composite section is analyzed for the loading condition of

-Superimposed dead load.

-Live load and impact load due to live load

In the analysis at this stage, the internal action like bending moment at different distance from the support to the mid span can be determined. The vertical shear stress and the horizontal shear stress at the contact interface are determined. The horizontal shear stress is determined by using the method developed for composite concrete deck.

In the analysis of a composite girder for live load, internal action of the wheel load distribution shall be considered.

Computation of Design Moments

For simple span, the design dead load moment and superimposed dead load moment at any point along the length of the span can be calculated from the standard equation for a beam under uniform load.

$$M_X = \frac{WX}{2}(L - X)$$

Where, M_x = Moment at a point x distance from on end support

W= distributed load in kN/m

X= distance from end support.

L=Span length, center to center distance of bearing

The maximum moment for simple span under uniform load occurs at cater of the span and given by:-

$$M_{\max} = \frac{WL^2}{8}$$

Computation of the live load moment is slightly more involved. As the loading due to vehicle live load are two types (HL-93 and tandem load inconsistent with lane load), the HL-93 load is selected for the development of design procedure. For the moving load, the maximum bending moment occur when the centerline of the span is midway between the center of gravity of the loads and to the nearest concentrated load.

Once the maximum live load internal actions are determined, wheel load distributions factor which account for lateral distribution of the truck loads and the dynamic effect of vehicle should be applied. The factor is directly applied to the live load moment to yield live load plus impact moment. The distribution load factor is applied to wheel loads which is half of the axel loads.

Total Moment to be carried by composite section

$$M_d=1.25\ M_{sd} + 1.75\ (M_{LL+I})$$

Where M_d = Design moment of composite section

M_{sd} = Moment due to superimpose dead load

M_{LL+I} = live load plus impact moment due to live load

3.6.2. Design Guideline

a. Determination of Effective Flange Width

Even if the deck runs continuously across the supporting girder, only a portion of the deck is taken to work in a composite fashion with the girder. This portion of the deck is the part that acts as a top flange of a T-shaped girder. This section is termed as the effective flange width. The effective flange width varies depending on whether the deck forms T-shaped top flange or not. For T-shaped cross section the effective flange width is defined as (AASHTO Art. 10.39.3)

- One fourth of the span its length
- Center to center distance between girder
- Twelve times the minimum thickness of the deck.

We will use the center to center distance between the girders to compute the amount of concrete slab acting as a dead load on the girder. For overhangs where the deck is integrally reinforced and monolithically poured with the rest of the deck, the exterior deck girder configuration can be analyzed as a symmetric T-beam section. The design of composite section is carried out for the following loading combination:

-Superimposed dead load

-Live load plus impact load due to live load.

The area of reinforcing steel determined from this loading (A_{S2}) is added to the area of steel determined from the first stage (A_{S1}) to give the total reinforcement required for the girder.

$$A_s = A_{s1} + A_{s2}$$

b. Design for Flexure

Spacing limit for reinforcement

The minimum clear distance between parallel bars in a layer shall not be less than (AASHTO Art. 8.21.1)

1.5Ø

1.5 times maximum size of aggregate or 3.81 cm

When the negative or positive reinforcing bar is placed in two or more layer, the bar on the upper layer shall be placed exactly above those in the bottom layer with the clear distance between layer not less than 25.4 mm

The minimum center to center distance = 25.4 + Ø,

The clear distance between bars shall also apply to the clear distance between contact lap supplies and adjacent splices of bars. (AASHTO Art. 8.25.3)

The center to center distance is therefore, 24.5 + *3.5 Ø*

Where Ø = diameter of the bar used

Determination of reinforcement

Rectangular Section with tension reinforcement only (AASHTO Art.16.3.2.1)

$$M_U = \phi\left(A_s f_y d\right)\left(1 - \frac{0.6\rho f_y}{f_c^{'}}\right)$$

$$\Rightarrow M_U = \phi A_S f_y\left(d - \frac{a}{2}\right) \qquad \phi = 0.9 \text{ for flexure.}$$

$$\text{Where } a = \frac{A_S f_y}{0.85 f_C^{'} b_e}$$

Where b_e= effective flange width.

Exact value of a can be determined by trial and error by setting zero for “a” for the first trial.

The balanced reinforcement ratio

$$\rho_b = \frac{0.85\beta_1 f_C}{f_y}\left(\frac{600.3}{600.3 + f_y}\right)$$

ρ_b =Balance reinforcement ratio

β_1=0.85 for $f_c \leq$ 27.6MPa. (AASHTO Art. 8.16.2.7)

For concrete having strength greater than 27.6MPa, the value of β_1 reduces by 0.05 for every 6.9MPa with minimum value not less than 0.65.

When the compressive flange thickness is equal to or greater than the depth of the equivalent rectangular stress block, ***a***, the design moment strength may be computed by:

$$M_U = \phi\left((A_s - A_{sf}) f_y (d - a/2)\right) + A_{sf} f_y (d - 0.5h_f)$$

$$A_{sf} = \frac{0.85 f_c^{'} (b_e - b_w) h_f}{f_y}$$

$$a = \frac{(A_S - A_{sf}) f_y}{0.85 f_C^{'} b_e}$$

where, A_{sf} = area of reinforcement to develop compressive strength of flange.

c. Design of Web for Vertical Shear

Composite girders are not designed for vertical shear as the precast portion of the girder shall resist vertical shear.

Design of section subjected to shear shall be based on (AASHTO Art. 8.16.6.1.1)

$V_u \leq ØV_n$, V_u= Factored shear force

V_n=nominal shear strength

$V_n = V_c + V_s$

V_c= Nominal Shear strength provided by concrete alone.

V_s= nominal shear strength provided by steel reinforcement.

For members subjected to shear and flexure only,

V_c= shall be computed by

$$V_C = 0.166 \sqrt{f_C^{'}} b_w d$$

Where f_c = concrete strength in MPa

b_w= web width in mm

d = concrete depth in mm

Ø = 0.85 reduction factor for shear

Computation of shear reinforcement

When the shear reinforcement perpendicular to the axis of the member is used (AASHTO Art. 8.16.6.3.2)

$$V_S = \frac{A_V f_y d}{S}$$

Where A_v = Area of shear reinforcement

f_y =yield strength of reinforcement

d = effective depth

S = spacing of reinforcement

$V_S = V_n - V_c$

Minimum shear reinforcement (AASHTO Art. 8.19.1.2)

$$A_{V\,\min} = \frac{0.345 b_w S}{f_y}$$

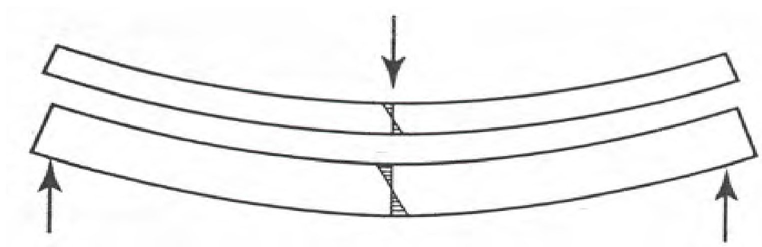
Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed (AASHTO Art. 8.19.3)

$$\frac{d}{2} \quad \text{or} \quad 609.6\text{mm}$$

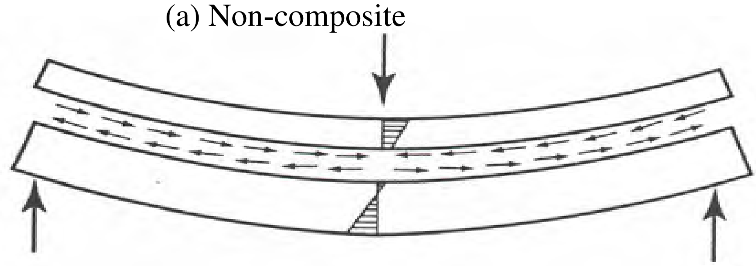
Where d = effective depth of the section under consideration.

d. Horizontal Shear in Girder

As shown in figure 3.5.a., there is no horizontal shear stress transferred from the slab to the girder. It acts as two independent members. But in figure 3.5.b. horizontal shear stress act on the interface and the girder act in composite section.



(a) Non-composite



(b) Composite

Fig. 3.5 Composite and non-composite action of a beam (8)

Different methods are developed for computation of horizontal shear stresses. From strength of materials, the horizontal shear stress on the contact surface between un cracked elastic precast beam and slab can be computed from(9)

$$V_h = \frac{VQ}{I_c b_V}$$

Where V=shear force acting on section in question

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

I_c =moment of inertia of composite section

b_v =width of the interface

This formula applies to un cracked elastic section and it is only approximation for cracked concrete beam. ACI gives two ways of computing the horizontal shear stress, but the one given by ACI section 17.5.3 is simpler to apply and it says "the horizontal shear can be computed from the change in compression or tensile forces in the slab in any segment of its length". In particular for simply supported beam, at the mid span the force is compression and all this force is assumed to act above the interface. At the support of the beam, the force in the flange is zero. Thus the horizontal shear force to be transferred across the inter face between the mid span and the support is:-

$$V_{nh}=C$$

Similar derivation could apply if the flange is subjected to tensile stresses.

From equilibrium of the section at the mid span,

$$C=T=A_s f_y =V_{nh}$$

Where A_s= Area of reinforcement computed for composite section.

The distribution of this force approximately reflects the distribution of shear force in the member (9).This implies that, the horizontal shear stress should be calculated from

$$\upsilon_{nh} = \frac{KV_{nh}}{A_c}$$

$$A_c=L_c b_c$$

Where L_c= half of the span length

b_c = width of precast girder in contact with cast in situ

A_c = contact area between the section taken

K=factor to account for the distribution shear along the member.

K is the ratio of the shear at the point to the average shear of the section. As it can be seen from Figure 3.6 the K varies from 2 at the support to zero at the mid span. For constant shear K would be 1.

In all cases the contact surface must be clean and free of laitance. The word intentionally roughened also applies to the beam to compute shear carrying capacity of the contact surface.

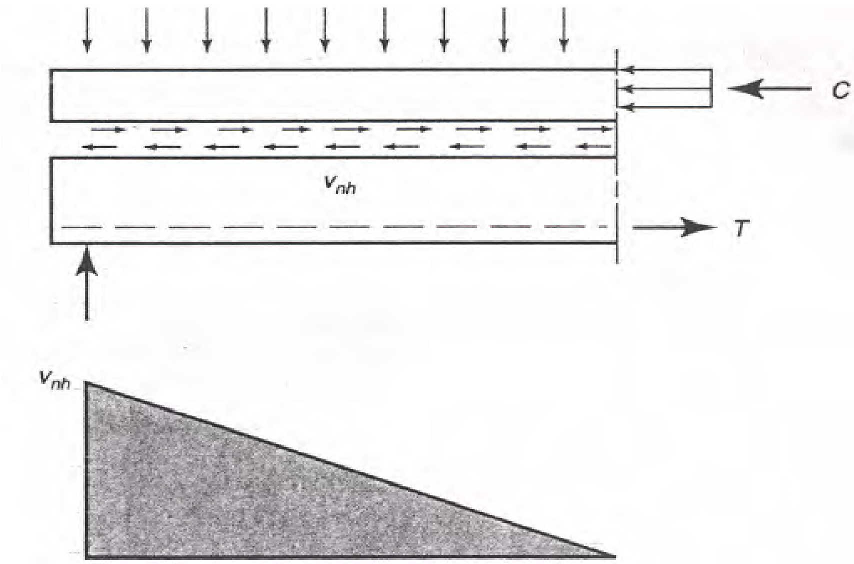


Figure.3.6 Distribution of horizontal shear stress (11)

4. Design Example

4.1. Design Data

To illustrate the application of the affirmation of design principle of precast- cast in place composite bridge of span 20m is considered.

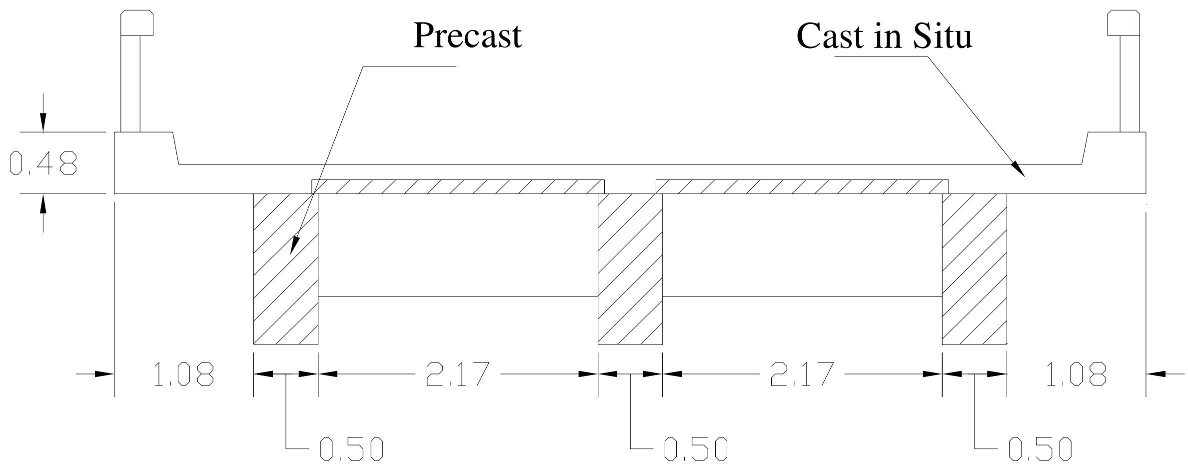


Figure 4.1Typical cross section for design example

(a) Geometric Dimensions

Before starting of analysis and design of bridge structure the following dimensions of the bridge superstructure has to be specified.

These are:

Road way width-----------------------7m

Curb width----------------------------0.5m

Total width---------------------------7+2*0.5=8m

Girder web width---------------------0.5m

Number of girders----------------------3

C/C spacing of girders----------------- $\frac{8}{3} = 2.67m$

Span length of cantilever------------ $\frac{2.67}{2} = 1.08m$

(b) Materials Used

The materials used for construction are:

Concrete

Class-A concrete

$f_c^{'} = 30MPa$

Reinforcing steel

$f_{y1} = 413MPa$ for $\phi \geq 20mm$

$f_{y2} = 300MPa$ for $\phi < 20mm$

(c) Loading

The loads considered for the analysis of the typical cross section are:

- self weight
- HL-93 with 9.3kN/m lane load
- For the purpose of cost comparison the design example with cast in situ concrete bridge constructed at kilkile river, the finishing load is ignored

(d) Design Methods

Design method –Load factor design methods

Design manual-AASHTO (Standard specification for high way bridge 1992)

ERA Bridge Design Manual 2002

4.2. Analysis and Design of Precast - Cast in Situ Composite Concrete Deck

4.2.1. Precast Section of the Deck

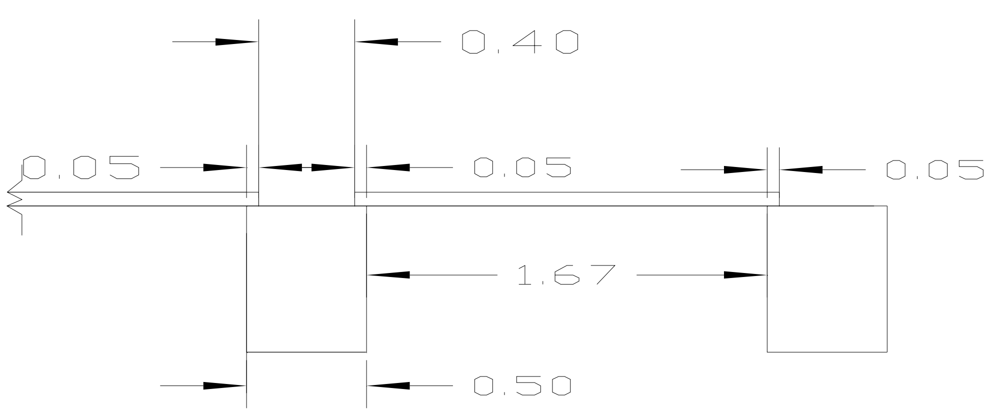


Figure4.2 Section of precast deck and girder

(a) Depth Requirement

Clear span of deck =2.65-0.5=2.17m

Clear span of cantilever deck $= \frac{2.17m}{2} = 1.08m$

The load on precast part of the deck is some what similar to slabs of building; therefore, the depth requirement for this part of the deck can be approximated by using the provisions for building codes.

From table 5.1 of EBCS-2

Depth of simply supported precast part of the deck, $t_1 = \frac{L_e}{20} = \frac{2.17}{20} = 0.108m$

Cantilever part of the precast deck, $t_2 = \frac{L_e}{10} = \frac{1.08}{10} = 0.108m$

Use 11cm depth for precast part of the deck.

(b) Depth Requirement for Composite Deck

From AASHTO Table 8.9.2

$t = \frac{(S+3.05)}{30} = \frac{(2.17+3.05)}{30} = 0.174\text{m} = 17.4\text{cm}$

Depth of cast in situ = 17.4-11 = 6.4cm

Minimum depth of cast in situ = 2.5*maximum aggregate size + Diameter of bar + top clear cover

=2.5*1.9 + 1.6+ 6 = 12cm

Therefore the thick ness of deck, d=11+12 = 23cm

(c) Loading

Precast part of the deck shall be designed for the larger of the following loading condition

- Self-weight plus 100% of self-weight for dynamic effect
- Self-weight plus load of green concrete plus 1.5kN/m^2 to account for load of laborers

Self-weight plus 100% of self-weigh

W_{DL}=2* 0.11m*25kN/m^3=5.5kN/m^2

Factored load on the deck =1.25*5.5 = 6.875kN/m^2

Self-weight plus load of green concrete

W_{DL}=0.23m*25kN/m^3=5.75kN/m^2

Factored load on the deck =1.25*5.75 +1.75*1.5 = 9.81kN/m^2

The design load should be Wd == 9.81kN/m^2

Computation of internal actions

Moment due to the dead load by taking a meter width of slab,

$$M_{DL}=\frac{W_{DL}s^2}{8}=\frac{9.81*2.17^2}{8}=5.77kN-m/m$$

Shear due to dead load

$$V_{DL}=\frac{W_{DL}s}{2}=\frac{9.81*2.17}{2}=10.64kN/m$$

(d) Vertical Shear

$V=\frac{WL}{2}=\frac{9.81*2.67}{2}=13.1kN/m$, and occur at support

where, W = Self weight of pre cast + weight of cast in situ

L = Clear spacing between girders

Span moment and diagonal shear will be reduced and no need of checking, but the punching shear developed at the hook during transportation has to be checked.

$$V_P=\frac{WLbt_P}{4}=\frac{25*2.67*1.91*.11}{4}=3.3.5kN$$

Where b= precast panel width is assume to be 1.91 meter

t_p = thickness of precast panel

L=length of precast panel

W=25kN/m^3

V_P =punching shear force

The precast shall be checked for the above loading with the cross section and reinforcement determined before.

Shear force capacity of precast concrete

$V_u = \phi V_n$

Where V_u =factored shear force at the section considered

$V_n= V_C+ V_S$

V_n = Nominal shear strength computed by

ϕ = 0.85 for shear

V_c= nominal shear strength provided by concrete

V_c= $0.166\sqrt{f_C^{'}}b_W d = 0.166 * \sqrt{30} * 1000 * 77 * 10^{-3} = 70.01 kN/m$

b_w=1000mm

d= 77mm

$\Rightarrow 70.01 kN/m > 10.64 km/m,$ Ok!

(e) Reinforcement Calculation for bending moment

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_c^{'}}\right)$$

$$\Rightarrow M_U = \phi A_S f_y\left(d - \frac{a}{2}\right) \qquad \phi = 0.9 \text{ for flexure.}$$

Where $a = \dfrac{A_S f_y}{0.85 f_C^{'} b}$ and $A_S = \dfrac{M_U}{0.9 * f_y(d - a/2)}$

$$A_S = \frac{5.77 * 10^6}{0.9 * 300 * (d - a/2)} = \frac{21370}{(d - a/2)} mm^2$$

where, the diameter of the bar is assumed to be16mm

d_b= t- c_b =110-25-8=77mm

d_b=effective depth of positive moment

c_b =cover to the centre of bottom reinforcement

(f) Determination of Reinforcement for Positive Moment

$$A_S == \frac{21370}{(77 - a/2)} mm^2$$

$$a = \frac{300 A_S}{0.85 * 30 * 1000} = 0.01176 A_S$$

The iteration yields

a=3.33

A_{s1}=284mm^2

(g) Punching Shear

$$V_P = \frac{WLbt_P}{4} = \frac{25*2*2.37*0.11}{4} = 3.26kN$$

To accommodate the dynamic effect 100% of the load is added

Thus V_{pd}=3.26*2=6.52KN

Where b= 2m (Assumed)

t_p = 110mm

L=2.37m

W=25KN/m^3

V_P =punching shear force

V_{pd} =punching shear force accompanied with dynamic effect

Place the hook below the bottom reinforcement. The failure in shear occur at distance **d** from the hook

Let the horizontal projection of the hook be *l*, then the punching area is:-

$A_p=2t_p\ell=2*110\ell=220\ell$

$v_c=0.166\sqrt{f_C^{'}} = 0.166\sqrt{30} = 0.909MPa$

$V_c= v_{c*}A_p=220\ \ell\ *0.909N=199.98\ell\ N=199.98\ell\ N$

$V_U=Øv_c$

$6.52*10^3=0.85*199.98\ell=169.98\ell\ N$

ℓ=38.36mm

4.2.2. Composite Section of the Deck

(a) Live Load

Truckload

$$M_{LL1}=\frac{(S+0.61)}{9.74}P = \frac{(2.17+0.61)}{9.74}*72.5 = 20.7KN-m/m$$

Lane load

$$M_{LL2}= \frac{WS^2}{10} = \frac{9.3*2.17^2}{10} = 4.4KN-m/m$$

$M_{LL}= M_{LL1} + M_{LL2}$

M_{LL} =20.7 +4.4 = 25.1kN-m/m

Applying a continuity factor of 0.8

M_{LL}=0.8*25.1kN-m/m = 20.08kN-m/m

(b) Impact Factor

$$I=\frac{15.04}{L+38.1}=\frac{15.04}{2.17+38.1}=0.37\geq 0.3$$

Use I=0.3

M_{LL+I}=1.3* M_{LL}=1.3*20.08=24.09 kN-m/m

M_u=1.75(M_{LL+I})

M_u=1.75*24.09) =42.15kN-m/m

(c) Reinforcement Calculation

$$M_U=\phi\left(A_s f_y d\right)\left(1-\frac{0.6\rho f_y}{f_c^{'}}\right)$$

$$\Rightarrow M_U=\phi A_S f_y\left(d-\frac{a}{2}\right)$$ ϕ = 0.9 for flexure.

Where $a=\frac{A_S f_y}{0.85 f_C^{'} b}$ and $A_S=\frac{M_U}{0.9*f_y\left(d-a/2\right)}$

$$A_S=\frac{42.15*10^6}{0.9*300*\left(d-a/2\right)}=\frac{156111}{\left(d-a/2\right)}mm^2$$

Where, the diameter of the bar is assumed to be16mm

d_t= t-c_t =230-60-8=162mm

d_t=effective depth of negative moment

d_b= t- c_b =230-25-8=197mm

d_b=effective depth of positive moment

c_t =cover to the centre of top reinforcement

c_b =cover to the centre of bottom reinforcement

(d)Determination of Reinforcement for Positive Moment

$$A_S==\frac{156111}{\left(162-a/2\right)}mm^2$$

$$a=\frac{300A_S}{0.85*30*1000}=0.01176A_S$$

The iteration yields

$a=11.8$

$A_{s2}=1000mm^2$

$A_s= A_{s1} + A_{s2}=284mm2 +1000mm2=1284mm^2$

Spacing of Ø16 bar $a_s=3.14*16^2/4=200.9$

$$S=\frac{a_S *1000}{A_s}=\frac{200.9*1000}{1284}=156mm$$

Use Ø16 C/C 150mm for bottom reinforcement.

(e) Determination of Reinforcement for Negative Moment

$$A_S == \frac{156111}{(197-a/2)} mm^2$$

$$a=\frac{300A_S}{0.85*30*1000}=0.01176A_S$$

The iteration yields

$a=9.6$

$A_s=812mm^2$

Spacing of Ø16 bar $a_s=3.14*16^2/4=200.9$

$$S=\frac{a_S *1000}{A_s}=\frac{200.9*1000}{812}=247mm$$

Use Ø16 C/C 200mm for top reinforcement

(f) Distribution Reinforcement

The amount of distribution reinforcement shall be a percentage of main reinforcing steel ($A_s=1952mm^2$) Main reinforcement perpendicular to traffic

Percentage $=\frac{121.4}{\sqrt{S}}$ maximum 67%

$S=2.67m$

Percentage $=\frac{121.4}{\sqrt{2.67}}$ =46.9 < 67%

Use 46.9 % of Positive main reinforcement

$A_{Sd}=0.469*1284mm^2=602\ mm^2$

Assume Ø12 is used, As =12*12*3.14/4 =113

Spacing, S = $\frac{113*1000}{602} = 180mm$

Use Ø12c/c 180mm

(g) Shrinkage and Temperature Reinforcement

Minimum reinforcement for shrinkage and temperature stress

$A_{S\ min}$=265mm^2/meter

Spacing of Ø10 bar a_s=3.14*10^2/4=78.5 mm^2

$$S = \frac{a_S * 1000}{A_s} = \frac{78.5 * 1000}{265} = 293mm$$

Use Ø10 C/C 280mm top distribution reinforcement

Therefore, use Ø12 C/C 180mm for bottom and Ø10 C/C280 top distribution reinforcements

(h) Horizontal Shear:

In composite member, the design horizontal shear stress V_{nh} shall be

$$V_u \leq \phi V_{nh}$$

where, V_u=factored shear force at section considered

Instead of finding the difference of compression or tensile stresses within one tenth of the span for the short span, it is more conservative to use the sum of stresses developed at mid span and at support. Figure 4.3 shows the shear stress developed between the inter face

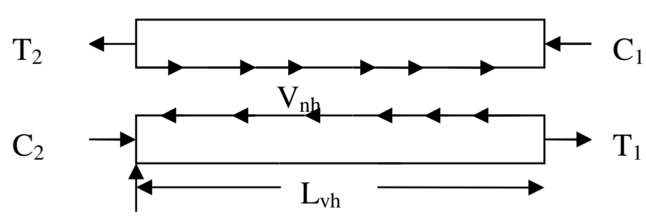


Figure.4.3 Horizontal shear stress developed at contact surface

In composite structure only the load that comes after curing of the cast in situ part of the deck produce horizontal shear.

For the design moment the area of steel per unit width is

A_{s1}=bottom reinforcement=1198 mm^2

A_{s2}=top reinforcement=1075mm^2

f_y=300MPa

The force developed for a meter width

$C_1=A_{s1}f_y=1000*300*10^{-3}=300$ kN

$T_2=A_{s2}f_{yd}=812*300*10^{-3}=243.6$kN

Total horizontal shear= $K(C_1+T_2)=2*(300+243.6)=1087$kN

Thus, the force is to be carried by the shear at the interface:

If the permissible concrete carrying capacity is insufficient to carry the computed internal force, shear reinforcement shall be provided.

Assume the concrete is intentionally roughened

$V_c = 0.7$MPa

The length of the interface,

$$L_{vh}=\frac{S}{2}=\frac{2.67}{2}=1.36m$$

Take deck width, b=1m

Total shear force carried by the concrete

$V_c = 0.7*1360*1000*10^{-3}$kN= 952kN

Therefore the section needs reinforcement for horizontal shear.

$Vnh=\phi(V_c+V_s)$

Where, ϕ = strength reduction factor (0.85 for shear)

V_c= shear stress provided by concrete with out shear reinforcement.

Vs= shear stress provided by reinforcement.

$$V_S=\frac{V_U}{\phi}-V_C=\frac{1087.2}{0.85}-952=327.1kN$$

The area of steel for reinforcement for shear

$$A_v=\frac{V_s}{f_y}=\frac{327.1*10^3}{300}=1090mm^2$$

Take the pitch distance to be coincident with the distribution bars. As it can be seen from the detailing, for every distribution bar there is single shear reinforcement crossing the section.

Number of distribution bars = $\frac{1360}{180} = 7.55$

Assume $\phi 8$ is used, a_v =8*8*3.14/4 = 50.24mm^2

Area provided by single row of reinforcement =7*50.24=351.6mm^2

Spacing, S = $\frac{a_v b}{A_v} = \frac{351.6 * 1000}{1090} = 322mm$

Let the spacing be coincident with the spacing of bottom reinforcement

S=2*150mm =300mm

The minimum area of tie reinforcement shall not be less than A_v

Where, A_v =0.345 $\frac{b_v s}{f_Y} = \frac{0.345 * 1000 * 450}{300} = 517mm^2$ Ok!

Where, b_v =Width of cross section at contact surface (mm)

S= Spacing which can not exceed four times the list web width of support or 610mm

S =300mm<4b_w

Horizontal spacing

Provide Ø8 zigzag bar having a pitch distance of 180mm at c\c spacing of 300mm

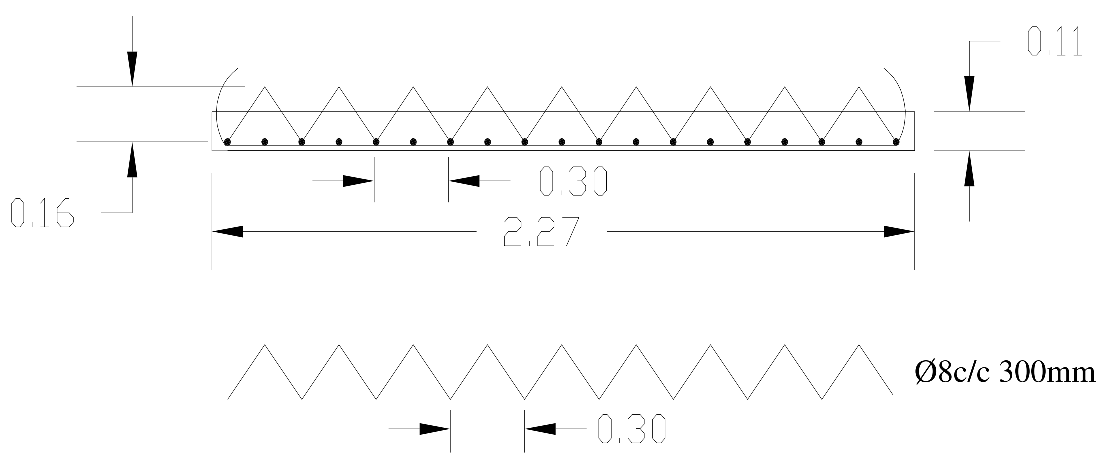


Figure 4.4 Detailing of horizontal shear reinforcement

4.2.3. Design of Overhang Deck

Distribution of loads on cantilever slab

Wheel on the element perpendicular to the traffic shall be distributed over a width

$$E = 0.8X + 1.143$$

And the bending moment per meter width of the deck shall be:

$$M = \left(\frac{P}{E}\right)X$$

Where X is the distance in meter from load to the point of support

Railing Load of 44.8 kN is applied at center of railing and the effective length of the deck resisting rail loading shall be:

$E = 0.8X + 1.143$ if no parapet is used

$E = 0.8X + 1.524$ if parapet is used

Where, X is the distance in meter from center of the post to the point under consideration. Railing load and wheel load shall not be applied at the some time.

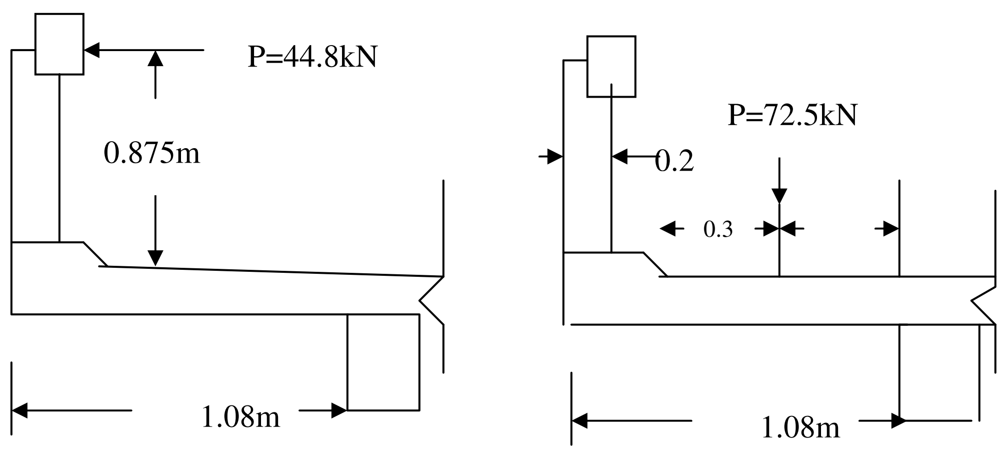


Figure 4.5 Loads on overhang deck

Loading

Assume spacing of curb = 2.5m

Post 0.2*0.3*0.8*25 / 2.5 = 0.48kN/m

Railing 0.3*0.15*25 = 1.1.25kN/m

Total =1.61kN/m

Assume uniform thick ness of deck

Curb 25**.25(.45+.5)/2 = 2.97kN/m

Deck 0.23*25 = 5.75kN/m

Dead load moment for meter width of slab at face of support

M_d = 0.1.61*0.93 + 2.97*0.605 + 5.75*1.08^2/2 = 6.65kN-m/m

Effective length of the deck assuming parapet wall

E = 0.8X+1.143 =0.8*.28+1.143 = 1.367m for wheel load

E = 0.8X+1.524 =0.8*0.93 + 1.524 = 2.268 for railing

M_{LL} = (72.5/1.367)*.28 = 14.85kN-m/m for wheel load

M_{LL} = (44.8/2.268)*0.875 = 17.28kN-m/m

$$I = \frac{15.25}{L + 38.125}$$

$$I = \frac{15.25}{0.28 + 38.125} = 39.7\%$$

$$\Rightarrow I = 0.3$$

MLL+I = 14.85*1.3 = 19.3kN-m/m

Thus the design moment is

Mu = 1.25*6.25+1.75*19.3 = 42.28kN-m/m

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_c^{'}}\right)$$

$$\Rightarrow M_U = \phi A_S f_y \left(d - \frac{a}{2}\right)$$ ϕ = 0.9 for flexure.

Where $a = \frac{A_S f_y}{0.85 f_C^{'} b}$ and $A_S = \frac{M_U}{0.9 * f_y (d - a/2)}$

$$A_S = \frac{42.28 * 10^6}{0.9 * 300 * (d - a/2)} = \frac{156592}{(d - a/2)} mm^2$$

Where, the diameter of the bar is assumed to be 16mm

d_t= t-c_t =230-60-8=162mm

d_t=effective depth of negative moment

$$A_S == \frac{156592}{(162 - a/2)} mm^2$$

$$a = \frac{300 A_S}{0.85 * 30 * 1000} = 0.01176 A_S$$

The iteration yields

a=11.7

A_{s2}=994mm^2/meter

Spacing of Ø16 bar a_s=3.14*16^2/4=200.96 mm^2

$$S = \frac{a_S * 1000}{A_s} = \frac{200.96 * 1000}{994} = 202mm$$

Use Ø16 C/C 200mm for top reinforcement

Thus provide Ø16 C/C 200mm for top and bottom reinforcement

4.3. Analysis and Design of Precast Concrete Girder

Precast girder is analyzed as a simply supported beam for the following loading combination.

(i) Self weight plus 100% of self weight due to dynamic effect of the girder during transportation. In addition to this the bending moment induced due to horizontal component of the hook force during picking has to be added. If two cranes are used, the moment due to horizontal component of the cable perpendicular to the longitudinal axis of the girder is ignored.

(ii) Self weight plus load transferred from composite deck and 1.5kN/m^2 of live load due to laborers.

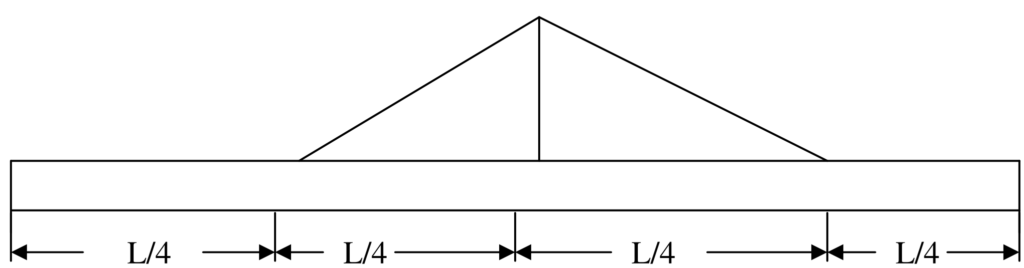


Figure 4.6 Detail showing point of hook for girder by single crane

(a) Girder Depth

S= 20m

H = 0.07S =0.07*20=1.4m

h = total depth of girder

b_w = 50cm

Depth of simply supported precast part of the deck, $t_1=\frac{S}{20}=\frac{20}{20}=1m$ (table 5.1 of EBCS-2)

Total depth $h=t_1+t_d=1+0.23=1.23<1.4m$

Use h=1.4m

$t_1=1.4-0.23=1.17m$

t_1=depth of precast girder

Self weight W1=1.17*0.5*25=14.625kN/m

$M_x=\frac{W_1x(L-x)}{2}=\frac{14.625x(20-x)}{2}$

$M_{D1}=\frac{W_1L^2}{8}=\frac{14.625*20^2}{8}=731.25kN-m$

$V_{D1}=W_1\left(\frac{L}{2}-x\right)=14.625(10-x)$

(b) Load from the Deck

Effective flange width is the lesser of:

$W_E=\frac{20}{4}=5m$ (One fourth of the span its length)

W_E= 2.67m (center to center distance between girder)

W_E= 12*0.23=2.76m (Twelve times the minimum thickness of the deck.)

Use W_E=2.67m

W_2=2.67*0.23*25=15.3525kN

$M_{D2}=\frac{W_2x(L-x)}{2}=\frac{15.3225x(L-x)}{2}$

$M_{D2}=\frac{W_2L^2}{8}=\frac{15.3225*20^2}{8}=767.625kNm$ >731.25kNm (100% of self weight)

Thus, self-weight governs during transportation.

$V_{D2}=W_2\left(\frac{L}{2}-x\right)=15.3225(10-x)$

$V_{total}=V_{D1}+V_{D2}$=14.625(10-x) +15.3525(10-x)=29.605(10-x)

Use $M_{DL}=M_{D1}+M_{D2}$ =731.25+767.625=1528.88kN-m

Live load due to labor

1.5kN/m2 is assumed to be applied on center to center distance of the flange.

Live load =1.5*2.67=4.005kN/m

Live load moment

$$M_{LL}== \frac{W_2L^2}{8} = \frac{4.005*20^2}{8} = 200.25kN-m$$

Design moment

M_U=1.25M_{DL}+1.75M_{LL}) = 1.25*1528.87+1.75*200.25) = 2262KN-m

(c) Design of Precast Concrete Girder

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_C}\right)$$

$$\Rightarrow M_U = \phi A_S f_y \left(d - \frac{a}{2}\right) \qquad \phi = 0.9 \text{ for flexure.}$$

Where $a = \frac{A_S f_y}{0.85 f_c^{'} b}$ and $A_S = \frac{M_U}{0.9 * f_y (d - a/2)}$

(d) Spacing Limit for Reinforcement

The minimum clear distance between parallel bars in a layer shall not be less than

1.5Ø =1.5*32=48mm

1.5 times maximum size of aggregate=1.5*20=30mm

38.1mm

Use clear spacing = 48mm

Assume four layer of reinforcement is used

Assume Ø32 bar be used

d_t= t-cover to the center of top reinforcement=1170-70-54-27=1019mm

d_t=effective depth of positive moment

Assume one layer of top reinforcement with Ø25 is used

d_b= t-cover to the center of top reinforcement=1170-25-12.5=1129mm

d_b=effective depth of negative moment

$f_y = \frac{413.7}{1.15} = 359.7$

(e) Reinforcement for Positive Moment

M_U= 2262kN-m

$$M_U = \phi A_S f_y\left(d - \frac{a}{2}\right)$$

$$2262*10^6 = 0.9*A_S*413\left(1019 - \frac{a}{2}\right)$$

$$A_S = \frac{6085553}{(1019 - a/2)} mm^2$$

$$a = \frac{413.7 A_S}{0.85*30*500} = 0.0324 A_S$$

The iteration yields

a=216.4

A_s=6682mm^2

(f) Reinforcement for Negative Moment

Self weight W1=1.17*0.5*25=14.625kN/m

Let the hook is placed 5m from each end

Negative moment at hook M_s is:-

$$M_s = \frac{W_1*5^2}{2} = \frac{14.625*5^2}{2} = 182.81 kN - m$$

Adding 100% of the load for dynamic effect

M_s=2*182.81=365.62kN-m

Factored moment

M_u=1.25*365.62=457kN-m

Effective depth d_s=1129mm

M_U= 457kN-m

$$M_U = \phi A_S f_y\left(d - \frac{a}{2}\right)$$

$$a = \frac{A_S f_y}{0.85 f_C^{`} b} = \frac{347.8 A_S}{0.85*30*500} = 0.02728 A_S$$

$$457*10^6 = 0.9*A_S*413\left(1043 - \frac{a}{2}\right)$$

$$A_S = \frac{1229554}{(1129 - a/2)} mm^2$$

The iteration yields

$a=30.2$

$A_s=1104m^2$

Use Ø25, $a_s=24*24*3.14/4=452mm^2$

Number of Ø25= $\frac{1104}{452}=2.44$

Use 3 Ø24 for top reinforcement for the moment developed during transportation.

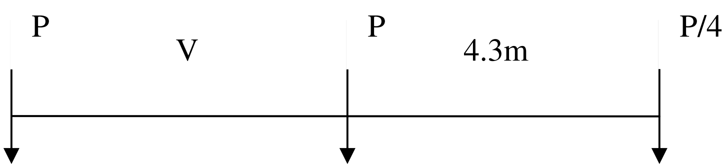
4.4. Analysis and Design of Composite Girder

In the first stage of analysis and design the dead load due to self weight of composite section is considered. The dead load left as a superimposed dead load is the load from curb and finishing dead load.

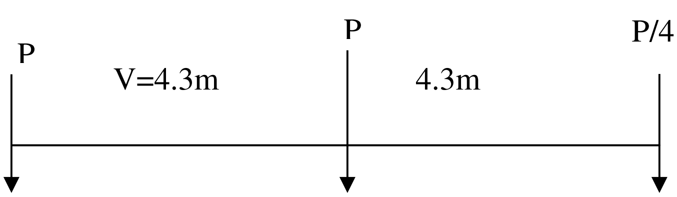
Live Load

Analysis of internal action due to live load

Track load of HL-93 is selected for design live load



For simply supported structural system the maximum internal action will be produced if the value of V is smaller, i.e. 4.3m.



Distribution of wheel load

$$DF=\frac{S}{1.38}=\frac{2.67}{1.38}=1.46 \qquad \text{for } S=2.67m$$

Impact factor

$$I=\frac{15.24}{L+38.1}=\frac{15.24}{20+38.1}=0.26<0.33$$

Use I=0.33

Part of the axle load carried by exterior girder.

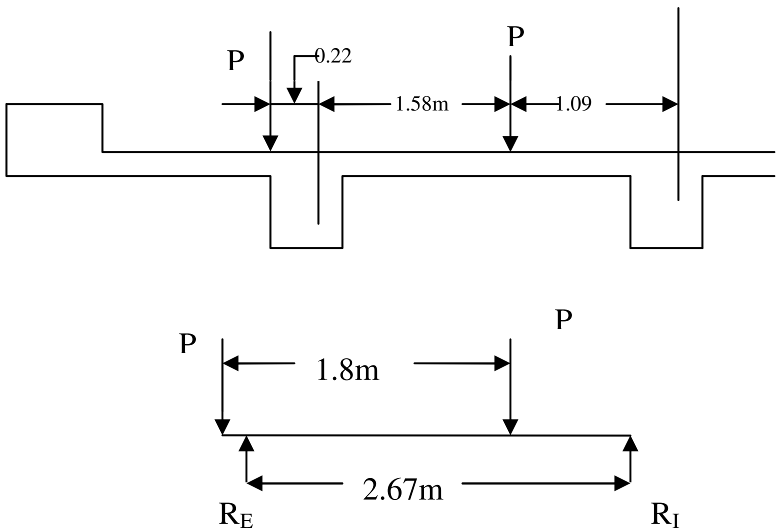


Figure 4.7 Location of wheel load for maximum exterior girder

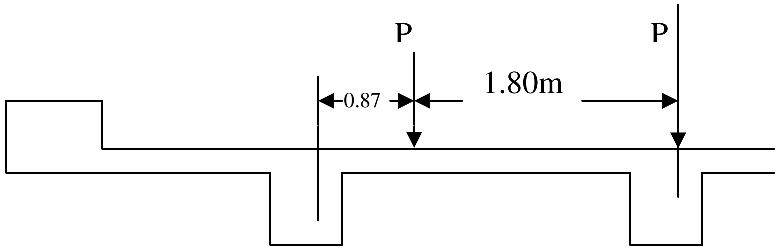
$$\sum M_I = 0$$

$$\Rightarrow \frac{(1.09+2.89)P}{2.67} = R_E$$

$$\Rightarrow R_E = 1.49P$$

$R_{E(LL+I)}$ =1.26*1.49P=1.88P

Part of the axel load carried by Interior girder



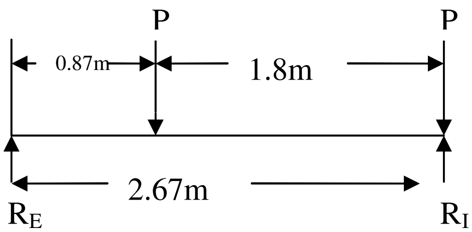


Figure 4.8 Location of wheel load for maximum internal girder

$$\sum M_E = 0$$

$$\Rightarrow R_I = \frac{(0.87 + 2.67)P}{2.67}$$

$$\Rightarrow R_I = 1.32P$$

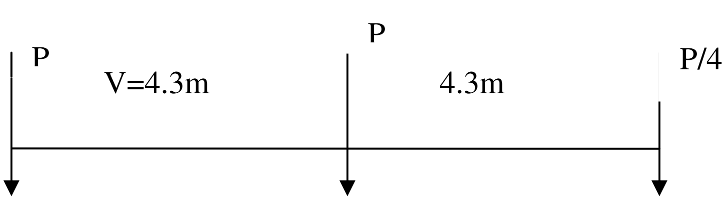
For moment and shear at every point other than shear at support

$R_{I(LL+I)}$ =DF*(1+I)P=1.32*1.26P=1.66P

For shear at support

R_I=1.31P

From the above loading condition the internal action due to live load can be determined using HL-93 loading as:-



Where P=72.5KN

For moving live load the maximum internal action can be determined by constructing influence line at that point.

Instead of constructing influence line for every point, a representative influence line as a function of distance from the support can be formulated.

Influence Line

a. Influence Line for a Moment at x Distance from the Support.

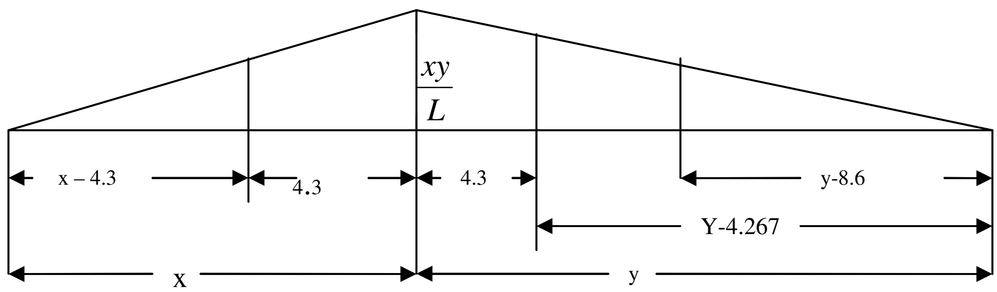


Figure 4.9 Influence line diagram for bending moment

For moving load from left to right and putting the maximum concentrated load at the required section the following expression can be obtained.

$$M_x = P\left(\frac{xy}{L}\right) + P\left(\frac{y-4.3}{L}\right)x - \frac{P}{4}\left(\frac{y-8.6}{L}\right)x$$

$$\Rightarrow M_x = \frac{Pxy}{L}\left(1 + \frac{y-4.3}{y} + \frac{y-8.6}{4y}\right)$$

For moving load from right to left and putting the maximum concentrated load at the required section the following expression can be obtained

$$M_x = P\left(\frac{xy}{L}\right) + P\left(\frac{y-4.3}{L}\right)x - \frac{P}{4}\left(\frac{x-4.3}{L}\right)y$$

$$\Rightarrow M_x = \frac{Pxy}{L}\left(1 + \frac{y-4.3}{y} + \frac{x-4.3}{4x}\right)$$

For the purpose of tabulation

Let $A = \frac{y-4.3}{y}$

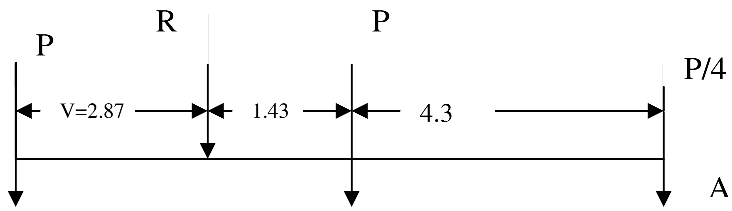
$B = \frac{y-8.6}{y}$

$C = \frac{x-4.3}{4x}$

$D = \frac{xy}{L}$

b. Absolute Maximum Moment

For moving load on a simply supported span, the absolute maximum moment occurs when the center line of the span is midway between the center of gravity of the loads and the nearest concentrated load. This moment occurs at the point under the nearest concentrated load.



R=2.25P

$$\sum M_A = 0$$

$$\Rightarrow R(x) = (2 * 4.3 + 4.3)P$$

$$\Rightarrow x = \frac{12.9P}{2.25P} = 5.73$$

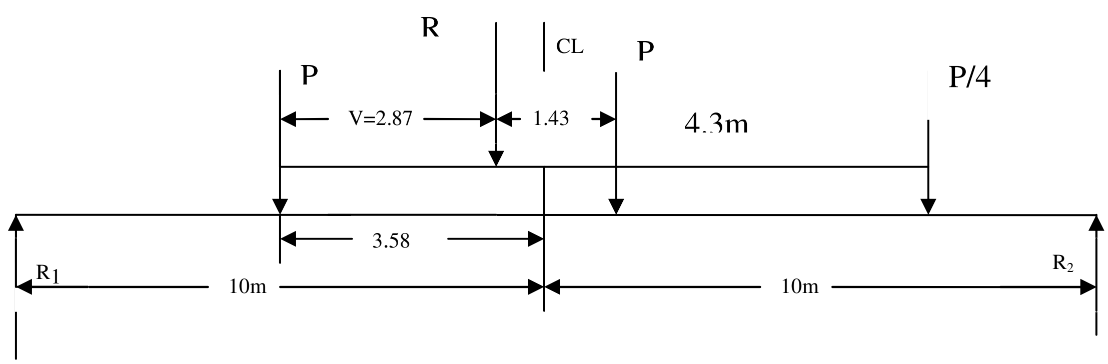


Fig. 4.10 Location of wheel load for absolute maximum moment

$$\sum M_1 = 0$$

$$\Rightarrow R_2(20) = P(10 - 3.58) + P(10 + 0.715) + \frac{P}{4}(10 + 5.02)$$

$$\Rightarrow R_2 = \frac{P(6.42 + 10.715 + 3.75)}{20} = 1.044P$$

R_1=2.25P-1.044P=1.206P

After the load is placed on the span, the live load moment at a given location on the span is determined by analyzing the girder from one end to the point in question. For simple span the point at 0.711m offset (point under the nearest concentrated load to the resultant of all loads) represent point of maximum moment.

c. Influence Line for Shear

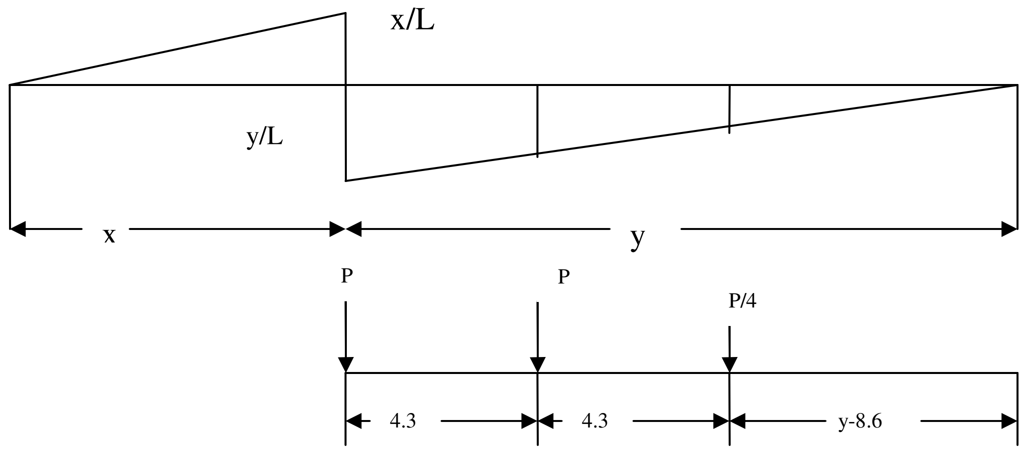


Figure 4.11Influence line diagram for shear

$$V_x = P\left(\frac{y}{L}\right) + P\left(\frac{y-4.3}{L}\right) + \frac{P}{4}\left(\frac{y-8.6}{L}\right)$$

$$\Rightarrow V_x = \frac{Py}{L}\left[1 + \left(\frac{y-4.3}{y}\right) + \left(\frac{y-8.6}{4y}\right)\right]$$

For the purpose of tabulation

Let E=$\frac{y-4.3}{y}$

F=$\frac{y-8.6}{4y}$

G=$\frac{y}{L}$

Table 4.1 Coefficient of bending moment

X	y=20-x	A=(y-3.4)/y	B=(y-8.6)/y	C=(x-4.3)4x	D=x y / L	D*(1+A+B)	D(1+A+C)
0.00	20.00	0.79	0.57	0.00	0.00	0.00	0.00
1.00	19.00	0.77	0.55	-0.83	0.95	2.21	0.90
2.00	18.00	0.76	0.52	-0.29	1.80	4.11	2.65
3.00	17.00	0.75	0.49	-0.11	2.55	5.72	4.18
4.00	16.00	0.73	0.46	-0.02	3.20	7.02	5.48
5.00	15.00	0.71	0.43	0.04	3.75	8.03	6.56
6.00	14.00	0.69	0.39	0.07	4.20	8.73	7.41
7.00	13.00	0.67	0.34	0.10	4.55	9.14	8.03
8.00	12.00	0.64	0.28	0.12	4.80	9.24	8.44
9.00	11.00	0.61	0.22	0.13	4.95	9.05	8.61
10.00	10.00	0.57	0.14	0.14	5.00	8.55	8.56

Table 4.2 Coefficient of Shear force

x	y=20-x	A=(y-4.3)/y	B=(y-8.6)/y	G=y / L	(1+A+B)	G(1+A+B)
0.00	20.00	0.79	0.57	1.00	2.36	2.36
1.00	19.00	0.77	0.55	0.95	2.32	2.32
2.00	18.00	0.76	0.52	0.90	2.28	2.28
3.00	17.00	0.75	0.49	0.85	2.24	2.24
4.00	16.00	0.73	0.46	0.80	2.19	2.19
5.00	15.00	0.71	0.43	0.75	2.14	2.14
6.00	14.00	0.69	0.39	0.70	2.08	2.08
7.00	13.00	0.67	0.34	0.65	2.01	2.01
8.00	12.00	0.64	0.28	0.60	1.93	1.93
9.00	11.00	0.61	0.22	0.55	1.83	1.83
10.00	10.00	0.57	0.14	0.50	1.71	1.71

In addition to the truck load a 9.3kN/m of lane load is distributed over a width of 3m with in the design lane. Design lane of the typical bridge is 3.5m in which the lane load will become:

$$\frac{3}{3.5}*9.3 = 7.97kN/m$$

In the case of the bridge under design the center to center width of the girder is 2.67m for internal girder and 1.87 for external girder. Thus the lane load to be considered shall be:

W_{LL2} =2.67*7.98=21.3kN/m for internal and 1.87*9.3kN/m =14.92kN/m for external

For internal	*Eternal girder*
$W_{(LL+I)2}$ = 21.3kN/m and	14.92 kN/m
$V_{x(LL+I)2}$ =21.3*(10 – x)	14.92(10-x)
$M_{x(LL+I)2} = \frac{WX}{2}(20-X) = 10.65X(20-X)$	7.5X(20-X)

d. Superimposed Dead Load

Dead load of the curb and railing

Curb loading

W_1=0.25*0.25*25=3.125kN/m

Railing load

W_2=0.2*0.15*25=0.75kN/m

Total superimposed dead load

D_{sd}=3.125+0.75=3.875kN/m

Internal moment due to superimposed dead load

$$M_D = \frac{Wx}{2}(L-x) = \frac{3.875x(20-x)}{2}$$

Summary of Design Moment and Shear Force

It is assumed that the superimposed load due to finishing material is not involved in the analysis of the bridge.

a. Internal Girder

$P_{(LL+I)}$ =1.66*P=1.66*72.5kN=120.4kN

V_{xLL2} = 28.32*(10 – x)

$$M_{xLL2} = \frac{WX}{2}(20-X) = 14.17X(20-X)$$

V_{SD}=0*(10-x)

M_{SD}=0*x (20-x)

V_{DD}= V_{D1}+ V_{D2}

V_{DD} =14.6 (10-x) +15.4(10-x)

V_{DD} =30(10-x)

M_{Total}=1.25*M_{DD}+1.75M $_{(LL+I)}$

V_{Total}=1.25*V_{DD}+1.75V $_{(LL+I)}$

Maximum live load moment

Maximum moment occur at 9.29m from support.

M $_{max}$=3137.27KN-m

V $_{max}$=1044.20KN

Table 4.3 Design moment and shear force for internal girder

X	M_{SD} (KN-m)	V_{SD} (KN)	$M_{(LL+I)1}$ (KN-m)	$M_{(LL+I)2}$ (KN-m)	$M_{(LL+I)}$ (KN-m)	$V_{(LL+I)2}$ (KN-m)	V_{DD} (KN)	$V_{(LL+I)1}$ (KN-m)	Mtotal (KN-m)	Vtotal (KN)
0	0.00	0.00	0.00	0.00	0.00	248.00	300	283.54	0.00	1044.20
1	0.00	0.00	265.48	235.60	700.19	223.20	270	279.45	700.19	982.25
2	0.00	0.00	494.84	446.40	1312.38	198.40	240	274.91	1312.38	919.50
3	0.00	0.00	688.09	632.40	1836.55	173.60	210	269.84	1836.55	855.82
4	0.00	0.00	845.21	793.60	2272.71	148.80	180	264.13	2272.71	791.02
5	0.00	0.00	966.21	930.00	2620.87	124.00	150	257.66	2620.87	724.90
6	0.00	0.00	1051.09	1041.60	2881.01	99.20	120	250.26	2881.01	657.16
7	0.00	0.00	1099.85	1128.40	3053.14	74.40	90	241.73	3053.14	587.42
8	0.00	0.00	1112.50	1190.40	3137.27	49.60	60	231.77	3137.27	515.20
9	0.00	0.00	1089.02	1227.60	3133.38	24.80	30	220.00	3133.38	439.81
10	0.00	0.00	1030.93	1240.00	3044.12	0.00	0	205.88	3044.12	360.30
9.289	0.00	0.00	1075.51	1233.73	3115.87	17.63	21.33	0.00	3115.87	38.96

b. External Girder

P $_{(LL+I)}$ =1.88P=1.88*72.5KN=136.3KN/m

$V_{x(LL+I)2}$ =19.4*(10 – x)

$$M_{x(LL+I)2} = \frac{WX}{2}\left(20 - X\right) = 9.7X(20 - X)$$

V_{SD}=3.875(10-x)

M_{SD}=1.9375x (20-x)

V_{DD}= V_{D1}+ V_{D2}

V_{DD} =14.6(10-x) +15.4(10-x)

V_{DD} =30(10-x)

M_{Total}=1.25M_{DD}+1.75M $_{(LL+I)}$

V_{Total}=1.25V_{DD}+1.75V $_{(LL+I)}$

Maximum live load moment

Maximum moment occur at 9.29m from support.

M_{max}=3168.17kN-m

V $_{max}$=1145.38kN

Table 4.4 Design moment and shear force for external girder

X	M_{SD} (KN-m)	V_{SD} (KN)	$M_{(LL+I)1}$ (KN-m)	$M_{(LL+I)2}$ (KN-m)	$M_{(LL+I)}$ (KN-m)	$V_{(LL+I)2}$ (KN-m)	V_{DD} (KN)	$V_{(LL+I)1}$ (KN-m)	M-total (KN-m)	V-total (KN)
0	0.00	38.75	0.00	0.00	0.00	169.90	300	320.99	0.00	1145.38
1	36.81	34.88	300.54	161.31	687.26	152.91	270	316.36	724.07	1193.60
2	69.75	31.00	560.19	305.64	1285.98	135.92	240	311.22	1355.73	1113.49
3	98.81	27.13	778.95	432.99	1796.16	118.93	210	305.47	1894.97	1032.33
4	124.00	23.25	956.83	543.36	2217.81	101.94	180	299.01	2341.81	949.91
5	145.31	19.38	1093.81	636.75	2550.91	84.95	150	291.68	2696.23	865.98
6	162.75	15.50	1189.90	713.16	2795.48	67.96	120	283.31	2958.23	780.22
7	176.31	11.63	1245.10	772.59	2951.52	50.97	90	273.65	3127.83	692.21
8	186.00	7.75	1259.41	815.04	3019.01	33.98	60	262.38	3205.01	601.38
9	191.81	3.88	1232.83	840.51	2997.97	16.99	30	249.06	3189.78	506.96
10	193.75	0.00	1167.07	849.00	2891.37	0.00	0	233.07	3085.12	407.88
9.289	192.77	2.76	1217.54	844.71	2975.40	12.08	21.33	0.00	3168.17	50.56

Design of Composite Girder

a. Flexure

i. Internal Girder

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_C}\right)$$

$$\Rightarrow M_U = \phi A_S f_y \left(d - \frac{a}{2}\right) \qquad \phi = 0.9 \text{ for flexure.}$$

Where $a = \frac{A_S f_y}{0.85 f_C^{'} b}$ and $A_S = \frac{M_U}{0.9 * f_y (d - a/2)}$

Spacing limit for reinforcement

The minimum clear distance between parallel bass in a layer shall not be less than

1.5Ø =1.5*32=48mm

1.5 times maximum size of aggregate=1.5*19=28.9mm

38.1mm

Use clear spacing=48mm

Assume four layer of reinforcement is used

Assume Ø of bar be 36mm

d_t= t-cover to the center of top reinforcement=1400-70-54-27=1249mm

d_t=effective depth of positive moment

f_y=413MPa

Effective flange width is the lesser of:-

$W_E = \frac{20}{4} = 5m$ (One fourth of the span its length)

W_E= 2.67m (center to center distance between girder)

W_E= 12*0.23=2.76m (Twelve times the minimum thickness of the deck.)

Use W_E=2.67m =2670mm

M_u =3137.27kN-m

Assume the neutral axis is within the flange and rectangular section with tension reinforcement only.

$$M_U = \phi A_S f_y \left(d - \frac{a}{2} \right)$$

$$3137.27 * 10^6 = 0.9 * A_S * 413 \left(1249 - \frac{a}{2} \right)$$

$$A_S = \frac{8440328}{(1249 - a/2)} mm^2$$

$$a = \frac{413 A_S}{0.85 * 30 * 2670} = 0.00607 A_S$$

$h_f = 230mm$

The iteration yields

a=41.7

a=45.1mm < h_f = 230mm

A_s=6873mm^2

ii. Exterior Girder

$$M_U = \phi(A_s f_y d)\left(1 - \frac{0.6\rho f_y}{f_C}\right)$$

$$\Rightarrow M_U = \phi A_S f_y\left(d - \frac{a}{2}\right) \qquad \phi = 0.9 \text{ for flexure.}$$

Where $a = \frac{A_S f_y}{0.85 f_C' b}$ and $A_S = \frac{M_U}{0.9 * f_y(d - a/2)}$

all dimensions are the same to interior girder section

M_U=3168.17kN-m

Assume the neutral axis is within the flange and rectangular section with tension reinforcement only.

$$M_U = \phi A_S f_y\left(d - \frac{a}{2}\right)$$

$$3168.17 * 10^6 = 0.9 * A_S * 413\left(1249 - \frac{a}{2}\right)$$

$$A_S = \frac{8523459}{(1249 - a/2)} mm^2$$

$$a = \frac{413.7 A_S}{0.85 * 30 * 2670} = 0.00607 A_S$$

$h_f = 230mm$

The iteration yields

a=41.5

a=52.3 < h_f = 230mm

A_s=6939mm^2

The area of reinforcing steel determined from this loading (A_{S2}) is added to the area of steel determined from the first stage (A_{S1}) to give the total reinforcement required for the girder.

Determination of number of bars

$$A_s = A_{s1} + A_{s2}$$

- ***Interior Girder***

A_{s1}=6682 and A_{s2}=6873

$A_s = A_{s1} + A_{s2}$=6082+6673 =12775mm^2

Area of ∅32=803.84mm^2

$$\text{Number of } \varnothing 32 = \frac{12775}{803.84} = 15.8$$

Use 16 ∅32

- ***Exterior Girder***

A_{s1} =7150 and A_{S2}=6939mm^2

$A_s = A_{s1} + A_{s2}$=5150+6939 =12089mm^2

Area of ∅32=803.84mm^2

$$\text{Number of } \varnothing 32 = \frac{12089}{803.84} = 15.04$$

Use 16 ∅32

b. Design for Vertical Shear

The entire vertical shear in the bridge is assumed to be carried by the precast girder. Thus, the analysis and design of the composite section is done in once.

From the table

Vu=1044.20kN for interior girder and

V_u=1145.38kN for exterior girder

The section of both exterior and exterior girder is the same.

Shear force caring capacity of the girder with out reinforcement

$$V_C = 0.17\sqrt{f_C^{'}}b_w d$$

Where $f_C^{'} = 30MPa$

b_w=500mm

d=1170mm

$V_C = 0.166\sqrt{30} * 500 * 1170 * 10^{-3} = 544.7kN$

$V_u \leq ØV_n$,

$V_n = V_c + V_s$

The design of shear is done for the shear at distance of effective depth from the support

$$Vu = \frac{1145.38 * (10 - 1.17)}{10} = 1011.4kN$$

Shear to be carried by shear reinforcement

$$V_s = \frac{V_u}{\phi} - V_c = \frac{1011.4}{0.85} - 544.7 = 635kN$$

f_y=300MPa

Assume ∅12 is used

A_v=2(12*12*3.14/4) =226.08mm^2

Av =shear reinforcement at every spacing

$$\text{Spacing} \quad S = \frac{A_V f_y d}{V_S} = \frac{226.08 * 300 * 1170}{635.1 * 10^3} = 125mm$$

Provide ∅12 c/c120mm

$$A_{V\min} = \frac{0.345 b_w S}{f_y}$$

$$\Rightarrow S_{\max} = \frac{A_V f_y}{0.345 b_w} = \frac{226.08 * 300}{0.345 * 500} = 393mm$$

Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed

d/2 =1170/2=585 or 609.6mm

Thus S_{max}=390mm

For the purpose of simplifying the construction two type of spacing shall be used

Shear caring capacity for spacing of 240mm

$$V_{S\max} = \frac{A_V f_y d}{S_{\max}} = \frac{226 * 300 * 1170 * 10^{-3}}{240} = 330kN$$

Total shear $=V_c + V_s$=544.7+330=874kN

V_u = □V_c=0.85*874=743kN

From the table the factored shear (□V_u) at 7m from the support has a value of 717kN which is less than 743kN.

Thus provide ∅12 @180mm for a length of 7m from each support and ∅12 @240mm in between 7m from the support.

c. Design Horizontal Shear in Girder

V_{nh}=C=T

T =V_{nh}= $A_s f_y$

A_s = A_{s2}=6873mm^2 for interior and 6939mm^2 for exterior girders

Use 6873mm^2

f_y =300MPa

V_{nh} =$A_s f_y$ =6873*300*10^{-3}=2061.9kN

$$\upsilon_{nh} = \frac{KV_{nh}}{A_c}$$

Ac=$L_c b_c$

L_c= 10m=10000mm

b_c= 0.4m=400mm

A_c =10000*400=4*10^{6}mm^2

K=2

$$\upsilon_{Uh} = \frac{KV_{nh}}{A_c} = \frac{2*2061.9*10^3}{4*10^6} = 1.03MPa$$

Concrete shear caring capacity, V_{ch} 0.7MPa

Shear to be carried by steel, $V_{sh} = \frac{V_{uh}}{\phi} - V_{ch} = \frac{1.03}{0.85} - 0.7$ =0.41MPa

Total shear force Vs=0.41*10000*400*10^{-3} = 1640kN

V_{sh} = A_{hs}*f_y

$$A_{sh} = \frac{V_{hs}}{f_y} = \frac{1640*10^3}{300} = 5467mm^2.$$

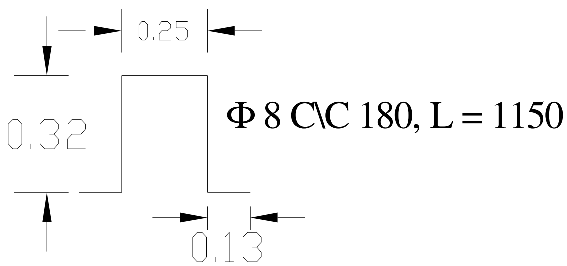
Assume ∅8 is used

a_{vh}=50.24mm^2

Number of ∅8 $= \frac{5467}{2*50.24} = 54.4$

Spacing S=$\frac{10000}{54.4} = 183mm$

Provide ∅8@180mm



Reinforcement for horizontal shear

5. Design and Construction Assessments

5.1. General

For the purpose of comparison, typical bridges under construction are selected. The selected bridges are the Kilkile Bridge and the four bridges on Arbaminch-Jinka road segments. Kilkile Bridge is a 20m span cast in situ concrete girder bridge and the four Arbaminch-Jinka bridges are a composite concrete girder bridge.

Both bridges were under construction during the period of visit. Kilkile Bridge is constructed on the road from Addis Ababa to Moyale at 530km from Addis while the bridges on Arbaminch-Jinka project were constructed on the road from Addis Ababa to Jinka on the road segment of Arbaminch-Jinka. For purpose of comparison, the cross section and span of the design example is taken to be the same with the bridge constructed on Kilkile River. This bridge is constructed by Ethiopian Roads Authority Soda District.

5.2. Comparison with Respect to Analysis Phase

Analysis of a composite section is the same as that of cast in situ section. Precast part of a composite section is analyzed for some part of the load that will be applied to the final structure and the remaining load will be analyze to be resisted by composite sections. Additional analysis that is not required for cast in situ section will be involved in composite section analysis. These are involvement of analysis of the section for punching shear in precast part of the deck and positive bending moment in girder during transportation and the horizontal shear between the contact surfaces of precast and cast in situ during the service. This aspect is viewed as follows

5.2.1. Analysis of Deck

The analysis of a composite section involves some additional analysis like determination of punching shear, bending moment due to dynamic effect during transportation and the horizontal shear developed at the interface when the section act as a composite section.

The deck of cast in situ concrete is analyzed as continuous slab supported at the girder in one step but the analysis of composite section require two stage loadings. These are analysis of precast part of the deck and composite section.

The precast part of the deck is analyzed as a simply supported one-way slab supported on the precast girder. In doing so, the thickness of slab required could be approximated using code provisions given for buildings. This thickness satisfies the depth requirement for deflection. The composite deck is analyzed as a continuous one way slab and supported at the girders points. The thickness required for composite section is the larger of:

a) The minimum thickness of deck requirement for bridge
b) The sum of thickness of the precast part of the deck and minimum thickness of toping of the precast.

5.2.2. Analysis of Girder

The cross section of precast part of the girder is selected to be rectangular because of the limitation in the availability of erection equipment. Some other sections like precast single and double T-section, for internal girder and L-section for external Girder are possible sections. The selection of the precast sections will affect the amount of horizontal shear force to be carried by the reinforcement which changes the amount of shear reinforcement. Figure 5.1 shows some of the possible precast concrete girders.

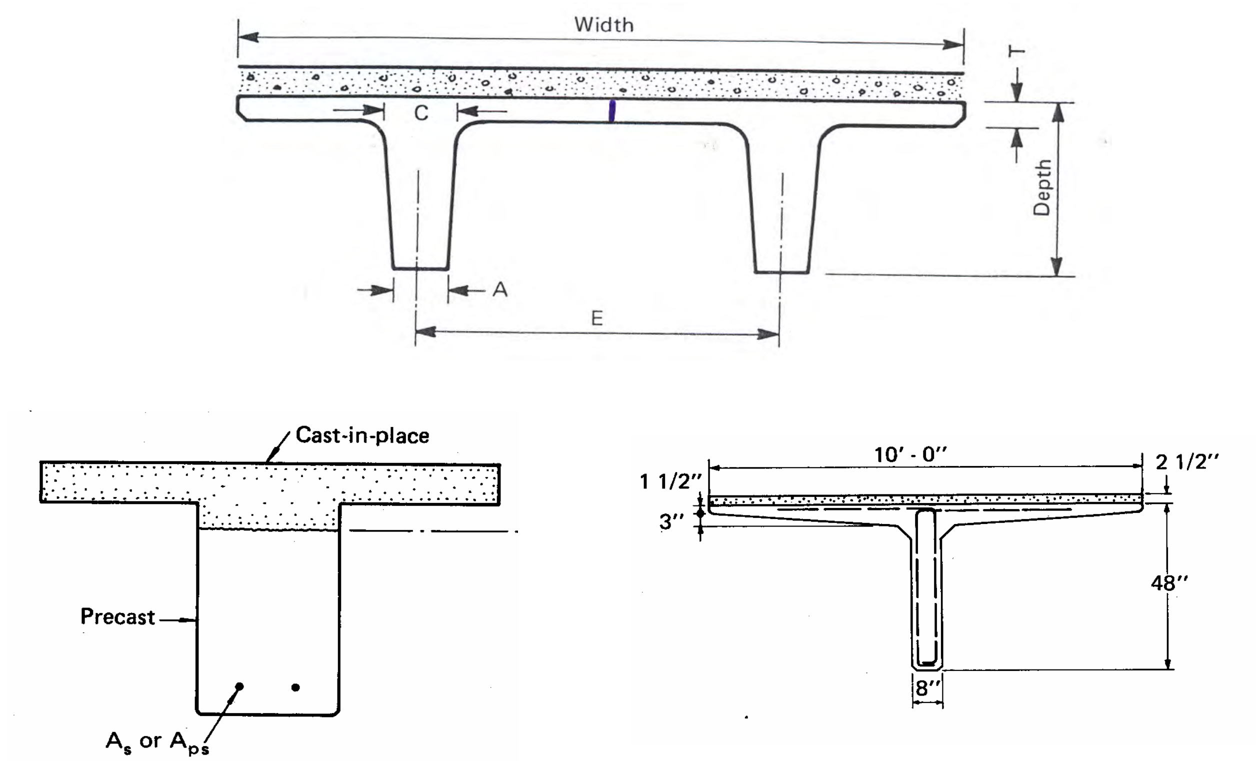


Figure 5.1 Available typical precast-cast in situ composite sections (2)

In addition to the analysis of internal action in cast in situ concrete analysis of the internal action developed like negative bending moment during transportation, the horizontal shear developed at the interface is required to be determined.

The girder of cast in situ section is analyzed as a simply supported T-section at the bearing. The analyses of the composite girder require two steps. These are:

- The precast part of the girder is analyzed as a rectangular simply supported section
- The composite section is analyzed as a simply supported T-section

The rectangular precast section should satisfy the depth requirement for deflection requirement of building codes while the depth of a composite section should be determined from the AASHO design manual or any other high way bridge design codes.

5.3. Comparison with Respect to Design phase

In design of composite section, the basic equation applied for cast in situ and composite section remains the same. The difference in design between cast in situ and composite concrete section is that designing of the sections for the additional internal actions developed during transportation and the horizontal shear developed at the interface of precast and cast in situ sections.

5.3.1. Design of the Deck

The design of cast in situ bridge deck can be done in one step for the result obtained from the analysis, but design of a composite deck is carried out in two steps.

The first step is the determination of reinforcement for:

- The loading of self weight, and dynamic effect of self weight during transportation
- Self weight plus weight of cast in situ plus live load during casting of toping concrete.

The second step is design of the composite section for:

- The loading of self weight plus superimposed dead load plus the live load (vehicle loading).

The capacity of cast in situ deck is checked against the diagonal shear at the face of support and bending moment at the girder and mid span. In addition to this, the composite section is designed for horizontal shear developed at the interface and punching shear at the hook.

5.3.2. Design of Girder

The design of cast in situ girder is carried out for the bending moment and vertical shear stress determined from analysis and the reinforcement is determined for these internal actions. The shear forces obtained from the analysis is checked against the capacity using total depth of the girder for cast in situ bridge.

In the design of a precast girder, the shear forces from the two step analysis should be resisted by the same rectangular precast section. Though different loadings are considered, with out changing in size of the composite section a two stage shear is checked against the concrete carrying capacity of precast part of the girder. In this case more shear reinforcement may be required as compared with cast in situ girder.

In designing a girder bridge, one step is sufficient for cast in situ sections but the design of the composite section requires two steps.

1) The precast rectangular section is designed for the larger of the following loading combination.

- Self weight plus the internal action developed during transportation due to dynamic effect.
- The internal action developed from self weight plus dead load from the composite deck plus live load from working laborer during casting of the cast in situ part of the deck.

2) The composite T-section is designed for live load (Traffic loading) and superimposed dead load. The sum of the area of reinforcement from the two steps is used as the design value.

5.4. Comparison with Respect to Construction phase

The design and analysis of the composite and cast in situ sections differ with steps that follow but the general formulas applied are some what the same. The construction of cast in situ section of a bridge requires construction of false and form work at the site and the construction of false work start from the river bed. The height of the false work depends on the elevation difference of river bed and the road. This shows that, large volume of material for false work is required for cast in situ bridges than the false work for the composite bridges. The construction of composite concrete bridge requires additional equipments like crane for erection and low bed for transporting the precast element. If precast element is fabricated on the site the need of low bed can be eliminated.

5.4.1. Equipments

The equipment required for cast in situ bridge construction is also needed for construction of composite section but the time at which that equipment stays on the site differs. This is because precast part of the bridge can be constructed in parallel schedule to the construction of substructure of the bridge in which the time of construction for form work and false work is reduced. Thus, the time at which the bridge is needed to be constructed is reduced nearly by the time required for construction of false work and form work. This time difference depends on the height difference of the river bed and road.

5.4.2. Materials

The construction material used for construction of a bridge like cement, aggregate, sand is all most the same for both cast in situ and composite section. The difference in the amount of material used to construct the bridge is the amount of reinforcement, form work and false work.

5.4.2.1. Reinforcements

(a) Reinforcement for bending moment

The amount of total reinforcement used in composite section is higher than those in cast in situ as additional reinforcement is required for positive moment during transportation of girder and for the effect of reduced effective depth in the first step design of precast sections. In addition to this the first step design may be governed by dynamic loading during transportation which increases the required reinforcement.

(b) Reinforcement for vertical shear

The concrete shear force carrying capacity of a composite bridge is reduced as the total depth of the composite girder is not used in the computation of the section capacity. This leads to an increase in amount of shear reinforcement used for a bridge of the same span and material classes.

(c) Reinforcement for horizontal shear

It is not essential to carry out analysis and design for the horizontal shear in cast in situ concrete construction. However, it is mandatory and important to analyze and design a composite bridge for horizontal shear. This shows concrete composite bridge requires additional reinforcement than cast in situ concrete bridge. Figures 5.1 to 5.4 are the picture taken from Kilkile River bridge project.

Figure 5.2 Reinforcement arrangements for positive moment of the deck

Figure 5.3 Reinforcement arrangements for girder shear and moment

Figure5.4 Reinforcement arrangement for girder shears and deck flexure

5.4.2.2. Form work and false work

(a) Form work

To construct cast in situ bridge the total area of form work required for a deck and girder should be supplied to the site at one time. However, the formwork for constructing a composite bridge and deck can be reused. Thus, the amount of form work required for specific site is reduced.

(b) False work

The picture below shows the false work used at Kilkile bridge construction. From the figure, a large volume of false work is utilized. But the construction of a precast concrete bridge is done on the ground in which the false work required is minimum. This leads to the reduction of deforestation of the country which reduces the environmental impact of bridge construction. In addition to this a large number of labor working days are required to construct the false work. Figure 5.5 and Figure 5.6 are the picture taken from Kilkile bridge and showing the amount of false work used for cast in situ bridge construction.

Figure 5.5 False works under construction for precast bridge at Kilkile River

Figure 5.6 Completed false work for Kilkile Bridge

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5.5. Assessments of composite design and construction

5.5.1.Design

Precast concrete is mainly produced and utilized for building construction. This is because of lack of experience in using this technology in bridge construction. In addition to this, composite section is not included as an option during preliminary design. In doing the assessment on composite design by local consultant, it is found that TCDSCo (Transport Construction Design Share Company) is the first to design composite bridge for Arbaminch-Jinka bridge project.

5.5.2.Construction

The construction of composite bridge in the country has started this year. The first composite bridge constructed is the one designed by TCDSCo for Arbaminch-Jinka bridges. These bridges are constructed by ERA (Ethiopian Roads Authority) Bridge and Structure Branch. The construction of the bridges was started in December 2004 and completed in May 2005.

From last experience of the company, much construction time is required to finish the cast in situ concrete bridge of the same span and section. This will prove the time saved from construction of bridge by composite construction system. Figure 5.7 to Figure 5.10 take from the four bridges of Arbaminch – Jinka project during the period of construction.

Figure 5.7 Placing of reinforcement for precast girder

Figure 5.8 Erection of precast girder

`Figure 5.9 Erection of precast part of the deck

Figure 5.10 False work required for cross girder and casting of toping deck

5.6. Cost Comparison

The cost comparison is made for a cost involved for construction of superstructure part of a bridge. This cost comparison is made for bridge constructed on Kilkile river and the design example.

5.6.1. **Time saved**

The time saved for construction of concrete composite bridge compared with of cast in situ concrete bridge as considered from experience of ERA at the Kilkile river bridge project and the bridges constructed on Arbaminch-Jinka project:

Additional time required for cast in situ concrete construction can be computed as:

Construction of false work takes	30 days
Demolishing of false work	20 days
Reinforcement placing for girder	6 days
Bottom reinforcement placing	2 days
Casting of girder	2 days
Total additional days	**60 days**

Additional time taken for concrete composite bridge construction

Erection of girder and deck	3 days
Construction of form work for cast in situ part of the bridge	6 days
Total additional days	**9 days**

Construction time saved = 60-9 = ***51 days***

Monthly expenses of the project

Consultant expense ranges between 45,000 to 60,000 Birr

Contractor expenses including equipments for the project ranges between 100,000 to 120,000

Total monthly expenses by taking the average value = 50,000+110,000 = 160,000Birr/month

Saved amount by Composite construction from saved time = (51/30)*160,000 = **272,000 Birr**.

5.6.2.Construction Material Used

a) Cast in situ concrete bridge

Class-A concrete		$77.6m^3$ *1976.97Birr/m^3=153412.87Birr
Reinforcement		17750.57kg * 9.2Birr/kg =163305Birr
False work	Φ12cm eucalyptus tree	940 piece * 15 Birr/Piece = 14100Birr
	Φ10cm eucalyptus tree	314 piece * 12Birr/Piece = 3768 Birr
	Φ8cm eucalyptus tree	180 pieces * 9 = 1620 Birr
	Total	**336,205.87Birr**

b) Composite bridge

Concrete		$82.7m^3$ *1976.97Birr/m^3=161518.45Birr
Reinforcement		19977.57kg * 9.2Birr/kg =183783.6Birr
False work	Φ12cm eucalyptus tree	134 piece * 15 Birr/Piece = 2010 Birr
	Φ10cm eucalyptus tree	240 piece * 12Birr/Piece = 1680 Birr
	Φ8cm eucalyptus tree	160 pieces * 9 = 1440 Birr
	Total	**350432.05Birr**

Additional expenses on material for composite construction=350432 -336205 =**19,398**

5.6.3.Equipment Utilized

Crane for erection	3 days*8haurs/day*3000Birr/haur =72,000 Birr
Law-bed for transporting crane	2days *35000Birr/day = 70,000 Birr
Total	142,000Birr

Total additional expense for equipment and material = 142000+19398.82 = 162398.82Birr

Total saved amount by composite construction = 270,000-142,000 = 128,000 Birr

Thus by constructing concrete composite bridge of 20m span, it is possible to save ***128,000*** Birr in addition to the lesser time for opening of the bridge to traffic.

6. Conclusions and Recommendations

6.1. Conclusions

The study has addressed the design and construction of concrete composite bridges. Based on the analysis and comparison made so far, the study have proved that, construction of concrete composite bridge is more economical than cast in situ concrete bridge. This is because composite construction reduces the time required for completion of a bridge and the saving the false work and form work requirements for the same geometric dimensions (span and width) of cast in situ concrete bridge. Especially for many short and intermediate girder bridges in road construction project and if the project owns the equipments required for the purpose of erection and transportation, the cost of construction of those bridges is much economical.

In addition to the fact that concrete composite bridge is economical, the contents of this study have shown that a minimum construction time is required for the bridge which is a major constraint in road construction. Furthermore, the construction of composite bridge will reduce the volume of false and form work requirements through which deforestation of the forest in the country is reduced. Especially, if the superstructure of a main highway bridge fails during the rainy season, the construction of false work and detour may be difficult. In such cases the construction of composite bridges without false work at the site will solve such problem.

The construction method developed in this study will lead to the design, fabrication and construction of prestressed concrete girder bridge which is not practiced in Ethiopia at this time by local construction companies.

Moreover, the guideline developed in this thesis can facilitate the use of composite bridge construction and used as a bases for design of other composite section bridges for tomorrow usage in the country.

6.2. Recommendations

Being an alternative solution to the method of construction in the country, the following recommendations can be made based on the study.

1. An emphasis should be made in the utilization of other construction methods of a bridge like composite, prestressed and precast constructions.
2. The road construction sector needs to be aware of this alternative construction method.
3. Design and fabrication of standard precast sections should be encouraged
4. Private companies that design and fabricate precast sections for a building should be encouraged and given supports for the development of the system
5. The clients of bridges have to lead consulting firms to use the developed method of construction as an alternative option during preliminary design.

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APPENDIX

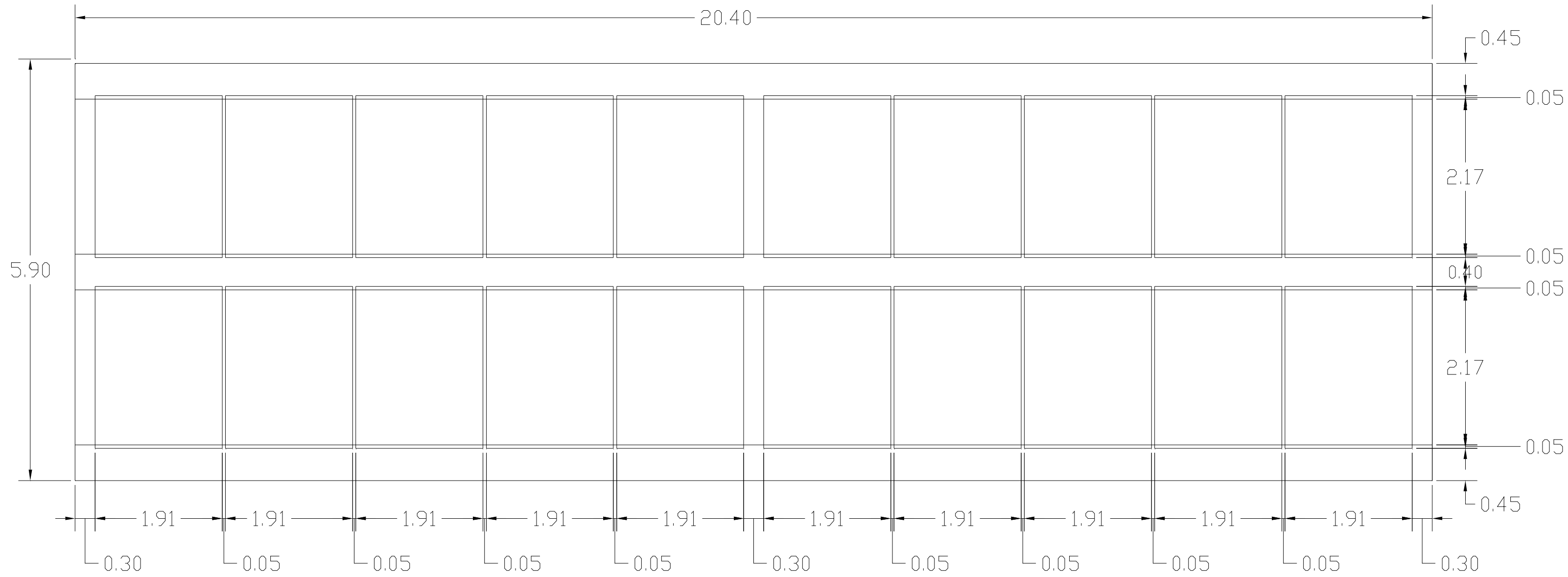
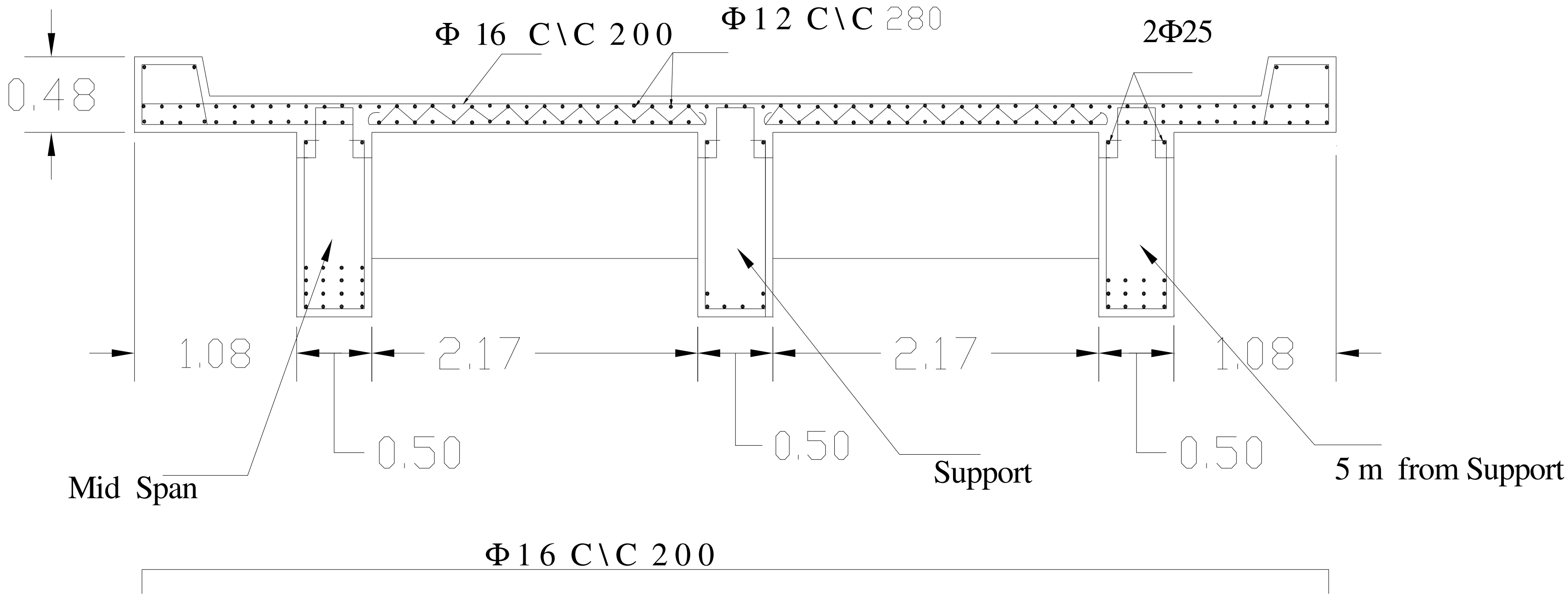
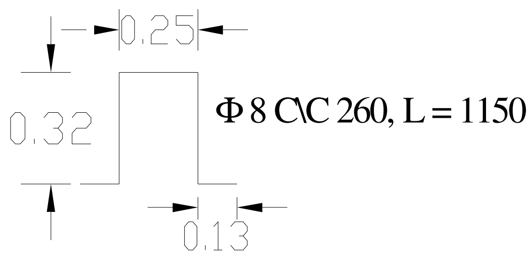


Figure A1 Plan showing arrangement of precast girder and deck after erection





Horizontal shear reinforcement

Figure A2 Cross Section of composite bridge

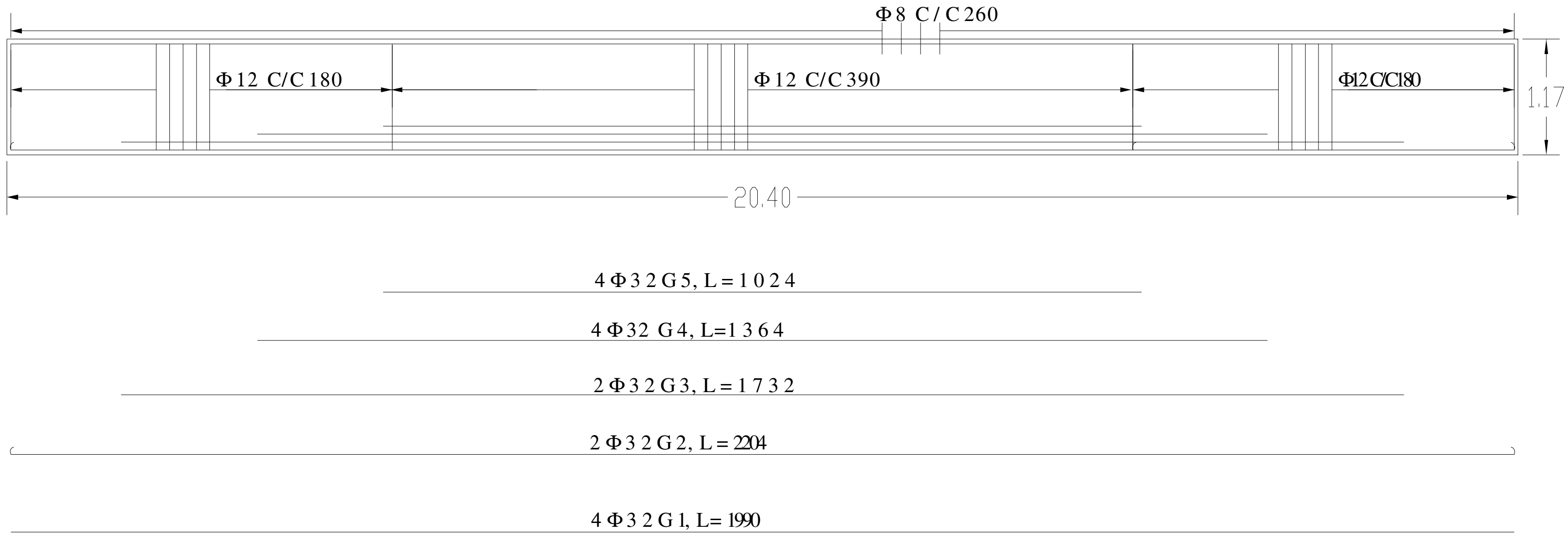
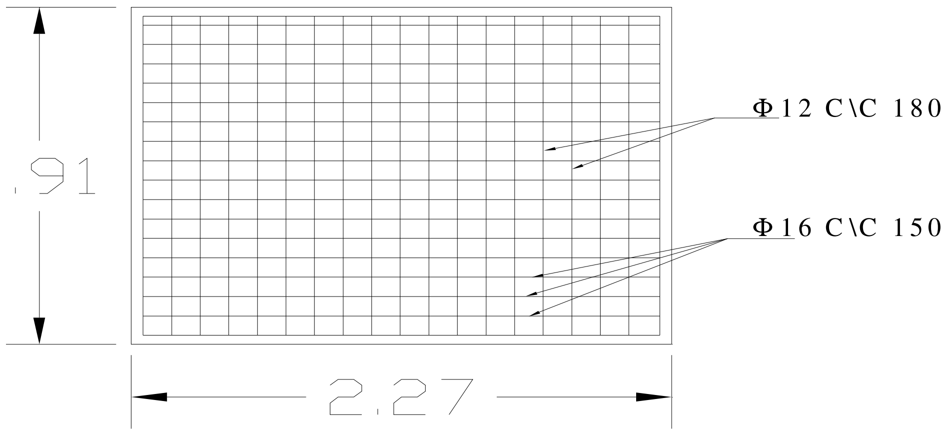


Figure A3 Typical precast girder for 20m



Plan of typical precast deck

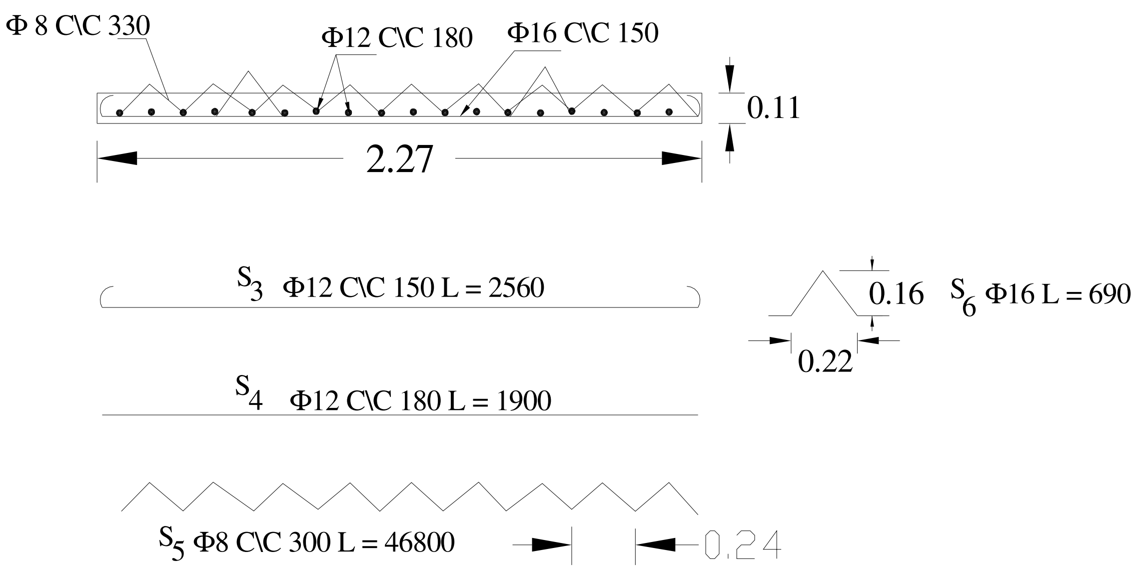


Figure A4 Plan and cross section of typical precast deck

DECLARATION

I, the undersigned, declare that this thesis is my work and all sources of materials used for the thesis have been duly acknowledged.

Name	Kitaw Zewde
Signature	____________
Place	Addis Ababa University, Faculty of Technology
Date of submission	October 2005