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DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

**DESIGN OF PRECAST PRE-STRESSED CONCRETE GIRDER BRIDGES
AS AN ECONOMICAL SOLUTION FOR CONGESTED URBAN AREAS
& WATERWAYS.**

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for
the Degree of Master of Science in Civil Engineering
(Structures)

By
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Declaration

I, the undersigned, declare that the thesis is my original work, has not been presented for a degree in any other University and that all sources of materials used for the thesis have been duly acknowledged.

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Abstract

Ethiopia is going through in a fast development and transformation process. Among which large construction activities are undergoing in road sector as part of these development program. Consequently faster completion with high quality of these projects is required. Therefore adoption of up-to-date design and construction technologies is the only way out.

However, the current bridge design and construction practice in many local contractors focuses on conventional type of bridge design & construction. This drift has affected the construction activity in such a way that by lingering the construction speeds, compromising construction quality and serviceability. The main reason is several design and construction professional believe that reinforced concrete bridges are economical and easy for construction. Likely this might be factual for limited span range. However, for long span bridges, this argument doesn't always hold true.

In another issue, bridge construction located in big cities must be safe and environmental friendly. This is to imply that the construction activity shall not affect the traffic flow and the day-to-day activities of inhabitants nearby. Since Environmental and Social impact is sensitive issue for bridge construction in urban areas, contractors and owners must use precast prestressed concrete girder bridges as a means of mitigation measure.

This research has designed precast-prestressed concrete girder bridges in parallel with reinforced concrete counterparts' in order to compare the design and construction aspects of each method. Consequently, the findings were interpreted in terms of construction cost, construction time and environmental friendliness. In view of that, the advantages of light weight Vs. long span capability of prestressed concrete girder bridges demand less number of steel reinforcements. Moreover as the precast units are repeatedly manufactured, the cost effectiveness also increases. With this regard, the research has clearly put a span range in such a way that a precast-prestressed concrete girder bridges are considered to be economical for the bridge design and construction projects.

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1. Introduction

1.1 Background

Precast/pre-stressed concrete girder bridges systems provide effective and economical design solutions for new bridge construction as well as for the rehabilitation of existing bridges. Significant traffic and congestion across urban areas, as well as waterways, creates a demand for long-span bridges. The construction of these longer spans plays a critical role in the development of modern infrastructure due to safety, environmental, and economic reasons. A variety of bridge construction practices have been observed over the years. Planning, design and construction techniques are revised and refined to satisfy several parameters including feasibility, ease of construction, safety, maintainability, and economy.

For over 60 years, precast, pre-stressed concrete girders have been used effectively in different parts of the world because of their durability, low life-cycle cost, and modularity, among other advantages. These girders are most commonly used for full length, simply supported bridges.

This study tries to disclose the benefit behind the use of precast-pre-stress concrete girder bridges for long span bridges. In addition, suitable standard precast/pre-stressed concrete girder bridges are designed, where the detail results may be used as a standard references for projects that might need the use of precast/pre-stressed girder bridges.

1.2 Objectives & Scope of the Study

The research aims at the necessity of considering precast pre-stressed concrete bridge design during different bridge type competitions. This is because the performance assessments of precast pre-stressed girder sections are much greater than Reinforced concrete bridge counterparts.

For comparison in this study, precast pre-stressed concrete girder bridges are designed based on AASHTO standard girder sections using AASHTO LRFD method. After the design is complete the design process and outputs are compared with reinforced concrete girder bridges.

However there were few setbacks during the research preparation. Among which, for reference purpose visiting local contractors whose engaged in current construction projects in Railways, highways and express-way was necessary. However these contractors use precast post tensioning system for bridge superstructure construction. Therefore, the research could not refer any of either ongoing or finished bridges. But useful references were made on the hauling, transportation and erection methods.

1.3 Thesis organization

This thesis is organized as follows.

- Chapter 1 contains the introduction, objectives and addresses the thesis organization
- Chapter 2 contains literature review on precast prestressed systems where the design and production of these elements are used.
- Chapter 3 summarizes the output obtained from the analysis and design of precast-prestressed girder bridges and reinforced concrete girder bridges from span 12m to 35m.
- Chapter 4 is about discussions regarding the design output, aptness for construction, environment & social aspects
- Chapter 5 contains conclusions based on the discussions and recommendation remarks.

2. Literature Review

2.1 Precast Prestressed Concrete Bridge System

2.1.1 General

Precast and Pre-stressed concrete structures, using high-strength materials to improve serviceability and durability, are an attractive alternative for long-span bridges, and have been used worldwide since the 1950s. The ability to quickly erect precast concrete members on all types of weather with little disruption of traffic adds to the economy of the job. For short spans, use of box sections and double T sections has proved economical. However, the most common product for short –to medium-spans is the I Girder and Bulb Ts. Spliced girders allow spans as much as 90m. Even longer spans (90m to 120m) can be achieved using precast box girder segments which are then post-tensioned in the field. Using cable stays, the spanning capability of precast and pre-stressed concrete has been increased to over 300m.

A recent innovation in bridge construction has been the use of precast concrete in horizontally curved bridges. Another growing application of precast and pre-stressed concrete in bridge construction includes the use of precast deck panels. Used as stay-in-place forms, the panels reduce field placement of reinforcing steel and concrete resulting in considerable savings. The panels become composite with the field-placed concrete for live loads.

2.1.2 Systems and elements for rapid construction

The use of lightweight prefabricated elements could prove important in the development and advancement of new innovative bridge systems. Optimizing the weight of these bridge systems through the design and/or the use of new lighter and durable materials will allow the transportation and erection of larger and longer bridges. High-performance materials that are lightweight and durable are most suited for prefabrication in large sizes.

➤ ***Prefabricated superstructures***

Since the construction of cast insitu superstructures takes a great deal of time, if part or whole is made of a prefabricated element the construction time and traffic disruption will be greatly reduced.

Precast concrete deck

Prefabricated decks offer advantages for deck construction because bridge components can be prefabricated off-site and assembled in place. Other advantages include removing deck placement from the critical path of bridge construction schedules, cost savings, and increased quality as a result of controlled factory conditions.

Partial-depth prefabricated deck panels act as stay-in place (SIP) forms and not only allow more controlled fabrication than fully cast-in-place decks, but also could increase the strength of the finished bridge deck owing to the use of prestressed panels.

Total Superstructure Systems

Increasingly, innovative bridge designers and builders are finding ways to prefabricate entire segments of the superstructure. Pre-constructed composite units may include steel or concrete girders prefabricated with a composite deck, cast off the project site and then lifted into place in one operation. Truss spans can also be prefabricated. Prefabrication of this scale offers potential advantages in terms of constructability, on-site construction time, and adherence to requirements of equipment on the construction site.

➤ ***Prefabricated Substructures***

Prefabricated substructure design provides an opportunity to apply advanced technologies and new materials to bridge systems. Specifically, the prefabricated substructure system consisting of segmental piers and bents offers an alternative that combines prefabricating and high-performance materials, resulting in rapid construction, durable performance, and an attractive appearance.

Bent Caps and Columns

The term “bent cap refers to the horizontal member at the top of the columns that supports the superstructure. Cast-in-place bent caps require extensive formwork and curing times. If these caps are fabricated off-site, curing times are not a factor. As a result, bridge owners and contractors are increasingly using prefabricated bent caps. For over water bridges, the bent caps reduce the amount of time that workers need to operate over water. Also, the use of prefabricated bent caps for bridges over existing roadways minimizes the required formwork and reduces disruption to traffic on the lower roadway. For bridges with job-site constraints, such as power lines that affect work zone safety, the use of prefabricated bent caps can limit the amount of time that workers are at risk.

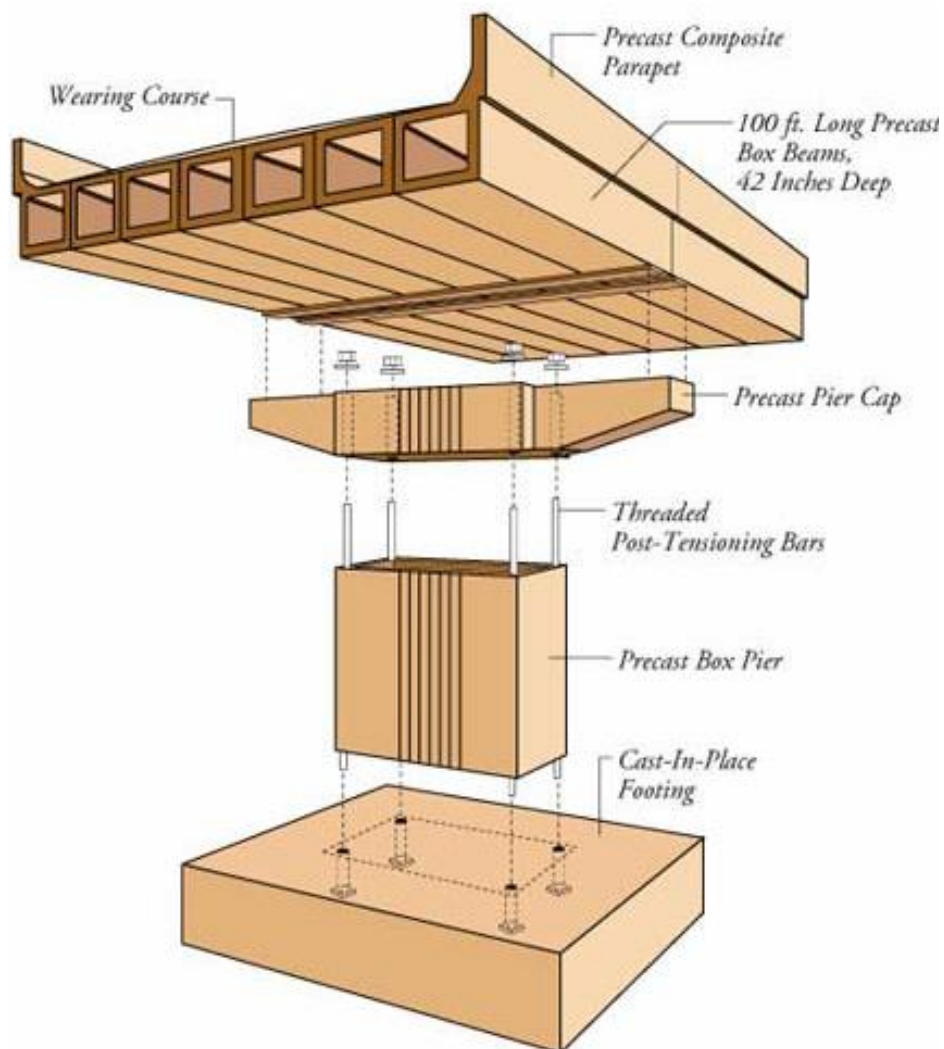


Fig. 1 - Prefabricated bridge elements

(Source: - NCHRP SYNTHESIS 324, *Prefabricated Bridge Elements and Systems to Limit Traffic Disruption during Construction*)

Use of precast pre-stressed bridges in new bridge construction

The use of precast prestressed bridge in new bridge construction mainly concerns with minimizing construction delay, minimizing lane closure time and to facilitate other construction progress. Other reasons were to improve quality and durability, to increase safety, and to minimize environmental impact and costs.

This system has also less environmental impact. This is due to the fact that, it requires no false work during construction. Using this technique in congested urban area provides less traffic closure, as a result there is less fuel cost in idle time and less gas emission.

2.1.3 Materials and facilities

I. Concrete

A 28-day cylinder compressive strength f_c' of concrete 28 to 56 MPa is used most. A higher early strength is often needed, however, either for the fast precast method used in the production plant or for the fast removal of formwork in the cast-in-place method. The modulus of elasticity of concrete with density between 1440 and 2500 kg/m³ may be taken as [1]

$$E_c = 0.043w_c \sqrt{f_c'} \dots\dots\dots 1$$

Where

w_c is the density of concrete (kg/m³). Poisson's ratio range from 0.11 to 0.27, but 0.2 is often assumed.

The modulus of rupture of concrete may be taken as [2]

$$f_r = \begin{cases} 0.63 \sqrt{f_c'} & \text{for normal weight concrete – flexural} \\ 0.52 \sqrt{f_c'} & \text{for sand – light concrete – flexural} \dots\dots\dots 2 \\ 0.44 \sqrt{f_c'} & \text{for all light weight concrete – flexural} \\ 0.1 f_c' & \text{for direct tension} \end{cases}$$

Concrete shrinkage is a time-dependent material behavior and mainly depends on the mixture of concrete, moisture conditions, and the curing method. Total shrinkage strains range from 0.0004 to 0.0008 over the life of concrete and about 80% of this occurs in the first year.

For moist-cured concrete devoid of shrinkage-prone aggregates, the strain due to shrinkage ϵ_{sh} may be estimated by [3]

$$\epsilon_{sh} = -k_s k_h \left(\frac{t}{35 + t} \right) 0.51 * 10^{-3} \dots\dots\dots 3$$

$$K_s = \left[\frac{\frac{t}{26e^{0.0142(\frac{V}{S})} + t}}{\frac{t}{45 + t}} \right] \left[\frac{1064 - 3.7(\frac{V}{S})}{923} \right] \dots\dots\dots 4$$

where

t is drying time (days); k_s is size factor and k_h is humidity factors may be approximated by

$$K_h = (140-H)/70 \text{ for } H < 80\%;$$

$K_h = 3(100-H)/70$ for $H \geq 80\%$; and V/S is volume to surface area ratio. If the moist-cured concrete is exposed to drying before 5 days of curing, the shrinkage determined by Eq. (3) should be increased by 20%.

For stem-cured concrete devoid of shrinkage-prone aggregates [5]:

$$\epsilon_{sh} = -k_s k_h \left(\frac{t}{55 + t} \right) 0.56 * 10^{-3} \dots\dots\dots 5$$

Creep of concrete is a time-dependent inelastic deformation under sustained load and depends primarily on the maturity of the concrete at the time of loading. Total creep strain generally ranges from about 1.5 to 4 times that of the “instantaneous” deformation. The creep coefficient may be estimated as [6]

$$\psi(t, t_1) = 3.5K_c K_f \left(1.58 - \frac{H}{120} \right) t_1^{-0.118} \frac{(t - t_1)^{0.6}}{10 + (t - t_1)^{0.6}} \dots\dots\dots 6$$

$$K_f = \frac{62}{42 + f_c} \dots\dots\dots 7$$

$$K_s = \left[\frac{\frac{t}{26e^{0.0142(\frac{V}{S})} + t}}{\frac{t}{45 + t}} \right] \left[\frac{1.8 + 1.77e^{-0.0213(\frac{V}{S})}}{2.587} \right] \dots\dots\dots 8$$

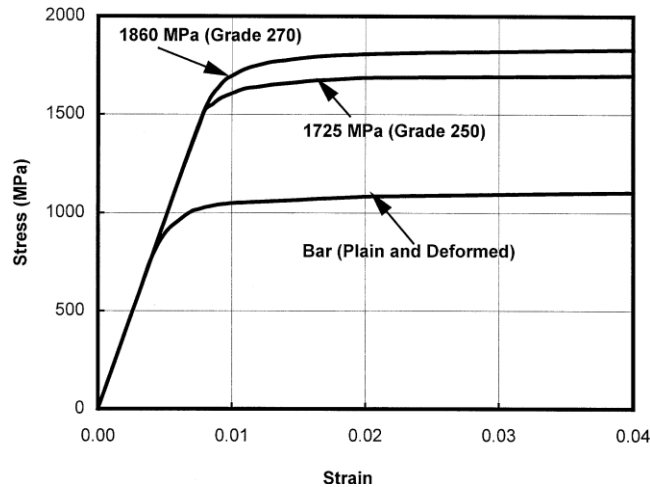


Fig. 2 Typical stress–strain curves for pre-stressing steel.

Where

H is relative humidity (%);

t is maturity of concrete (days);

t_i is age of concrete when load is initially applied (days);

K_c is the effect factor of the volume-to-surface ratio; and

K_f is the effect factor of concrete strength.

Creep, shrinkage, and modulus of elasticity may also be estimated in accordance with AASHTO LRFD Bridge design specifications.

II. Steel for Pre-stressing

Uncoated, seven-wire stress-relieved strands (AASHTO M203 or ASTM A416), or low-relaxation seven-wire strands and uncoated high-strength bars (AASHTO M275 or ASTM A722) are commonly used in pre-stressed concrete bridges. Pre-stressing reinforcement either wires, strands, or bars, are also called tendons. The properties for pre-stressing steel are shown in Table 1.1.

TABLE 1.1 Properties of Pre-stressing Strand and Bars

Material	Grade and Type	Diameter (mm)	Tensile Strength f_{pu} (MPa)	Yield Strength f_{py} (MPa)	Modulus of Elasticity E_p (MPa)
Strand	1725 MPa (Grade 250)	6.35–15.24	1725	80% of f_{pu} except 90% of f_{pu} for low relaxation strand	197,000
	1860 MPa (Grade 270)	10.53–15.24	1860		
Bar	Type 1, Plain	19 to 25	1035	85% of f_{pu}	207,000
	Type 2, Deformed	15 to 36	1035	80% of f_{pu}	

Typical stress–strain curves for pre-stressing steel are shown in Figure 2. These curves can be approximated by the following equations[9],[10]&[11]:

For Grade 250

$$f_{ps} = \begin{cases} 197,000\varepsilon_{ps} & \text{for } \varepsilon_{ps} \leq 0.008 \\ 1710 - \frac{0.40}{\varepsilon_{ps} - 0.006} < 0.98f_{pu} & \text{for } \varepsilon_{ps} > 0.008 \end{cases} \dots\dots\dots 9$$

For Grade 270

$$f_{ps} = \begin{cases} 197,000\varepsilon_{ps} & \text{for } \varepsilon_{ps} \leq 0.008 \\ 1848 - \frac{0.517}{\varepsilon_{ps} - 0.0065} < 0.98f_{pu} & \text{for } \varepsilon_{ps} > 0.008 \end{cases} \dots\dots\dots 10$$

For Bars:

$$f_{ps} = \begin{cases} 207,000\varepsilon_{ps} & \text{for } \varepsilon_{ps} \leq 0.004 \\ 1020 - \frac{0.192}{\varepsilon_{ps} - 0.003} < 0.98f_{pu} & \text{for } \varepsilon_{ps} > 0.004 \end{cases} \dots\dots\dots 11$$

III. Advanced Composites for Pre-stressing

Advanced composites–fiber-reinforced plastics (FPR) with their high tensile strength and good corrosion resistance work well in pre-stressed concrete structures. Application of advanced composites to pre-stressing have been investigated since the 1950s. Extensive research has also been conducted in Germany and Japan. The Ulenbergstrasse Bridge, a two-span (21.3 and 25.6 m) solid slab using 59 fiberglass tendons, was built in 1986 in Germany. It was the first pre-stressed concrete bridge to use advanced composite tendons in the world.

FPR cables and rods made of Aramid, glass, and carbon fibers embedded in a synthetic resin have an ultimate tensile strength of 1500 to 2000 MPa, with the modulus of elasticity ranging from 62,055 MPa to 165,480 MPa . The main advantages of FPR are

- (1) a high specific strength (ratio of strength to mass density) of about 10 to 15 times greater than steel;
- (2) a low modulus of elasticity making the pre-stress loss small;
- (3) good performance in fatigue; tests show that for CFRP, at least three times the higher stress amplitudes and higher mean stresses than steel are achieved without damage to the cable over 2 million cycles.

Although much effort has been given to exploring the use of advanced composites in civil engineering structures and the cost of advanced composites has come down significantly, the design and construction specifications have not yet been developed. Time is still needed for engineers and bridge owners to realize the cost-effectiveness and extended life expectancy gained by using advanced composites in civil engineering structures.

IV. Pre-stressing Systems

There are two types of pre-stressing systems: pre-tensioning and post-tensioning systems. Pretensioning systems are methods in which the strands are tensioned before the concrete is placed. This method is generally used for mass production of pretensioned members. Post-tensioning systems are methods in which the tendons are tensioned after concrete has reached a specified strength. This technique is often used in projects with very large elements. The main advantage of post-tensioning is its ability to post-tension both precast and cast-in-place members. Mechanical pre-stressing–jacking is the most common method used in bridge structures.

V. Basic principle and procedure for Pretensioning systems

The basic principle of pretensioning involves the tensioning of the tendons to a predetermined level, after which the concrete is placed (see Fig. 3-1a). The resulting elongation of the tendons is maintained at a constant level while the concrete hardens. After the concrete has developed sufficient strength the tendons are released and, because they are now bonded to the concrete, their shortening is resisted by the concrete. In this way the concrete is prestressed by the action of bond when the tendons are released. It is important to ensure that the elongation of the tendons is maintained at a constant level while the concrete is allowed to harden, and this can be achieved by each of the following two methods

- Pretensioning with individual moulds: According to this method the tendons are anchored directly to the individual steel moulds in which the concrete is cast. In this case, the moulds must be designed and constructed to withstand the additional forces induced by the tendons.
- Pretensioning on stressing beds: When pretensioning on a stressing bed, the tendons are tensioned between and subsequently anchored to the rigid vertical steel anchor columns, called uprights, placed at each end of the bed (see Fig. 3-1a). In this manner the tension is maintained in the tendons while the concrete is placed and cured. The stressing bed also serves as a casting and curing bed.

Pretensioning on stressing beds is by far the most common method used today, and a typical arrangement

is shown in Fig. 3-1a. This method, often referred to as the long-line or Hoyer method, lends itself to efficient mass production because a number of similar elements can be manufactured in a single tensioning operation if the bed is made long enough. The length of stressing beds varies between 25 m and 200 m, and long beds can be provided with removable intermediate uprights (see Fig. 3a) so that shorter tendons can also be tensioned.

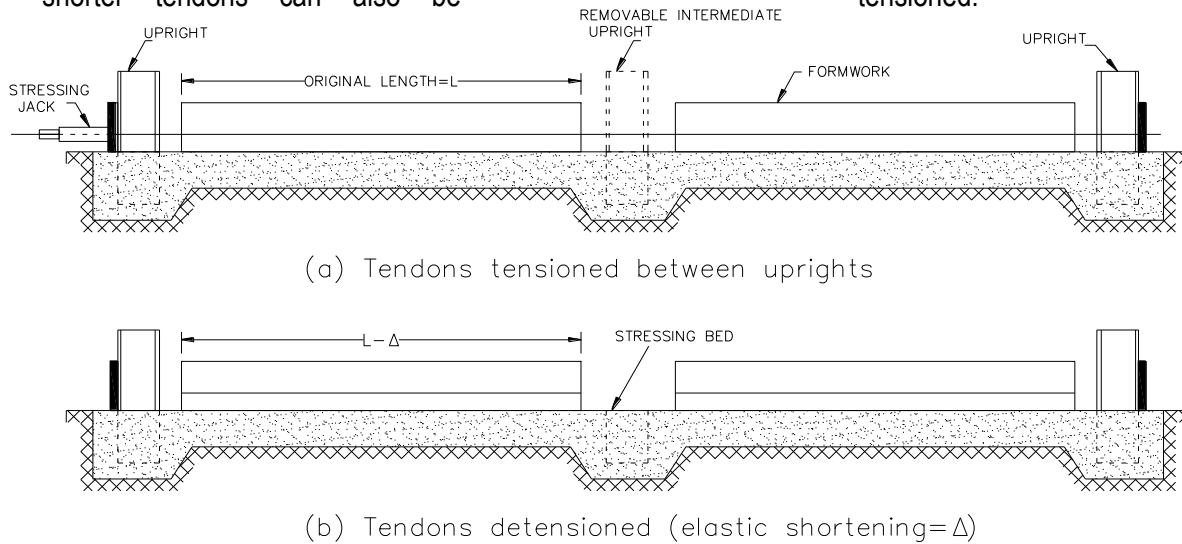


Fig. 3 Pre-tensioning on a stressing bed

Tendons are tensioned by means of hydraulic jacks, and can either be stressed individually or simultaneously from one end of the stressing bed. Special jacks with a ram stroke of at least 750 to 1200 mm must be used if the strands are to be tensioned in a one-step operation. After being tensioned, wires and strand are usually anchored by means of frictional split-cone wedges.

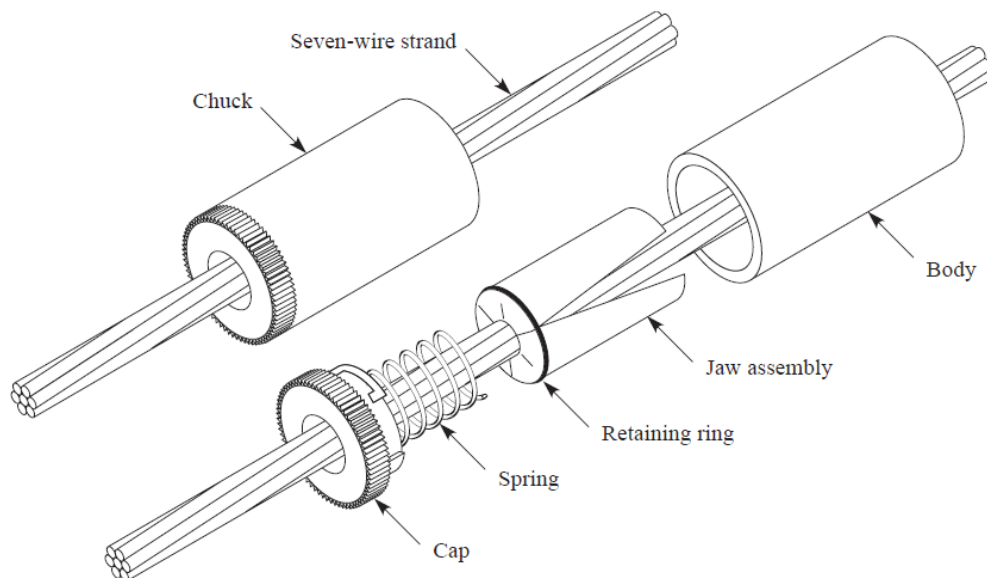


Fig. 4 Typical quick release grip

A stressing bed must allow a daily production cycle so that members can be produced in large numbers. Under such conditions the use of steel forms or moulds is preferable for the following reasons: Steel forms are durable and perform well under repeated use; they can be manufactured to a high degree of precision; they are easy to handle when being erected or stripped; they can be made adjustable to easily accommodate variations in member shape; and they can easily be made strong enough to allow form vibration. Figure 3 shows a steel mould with removable side forms and a form vibrator for a bridge girder.

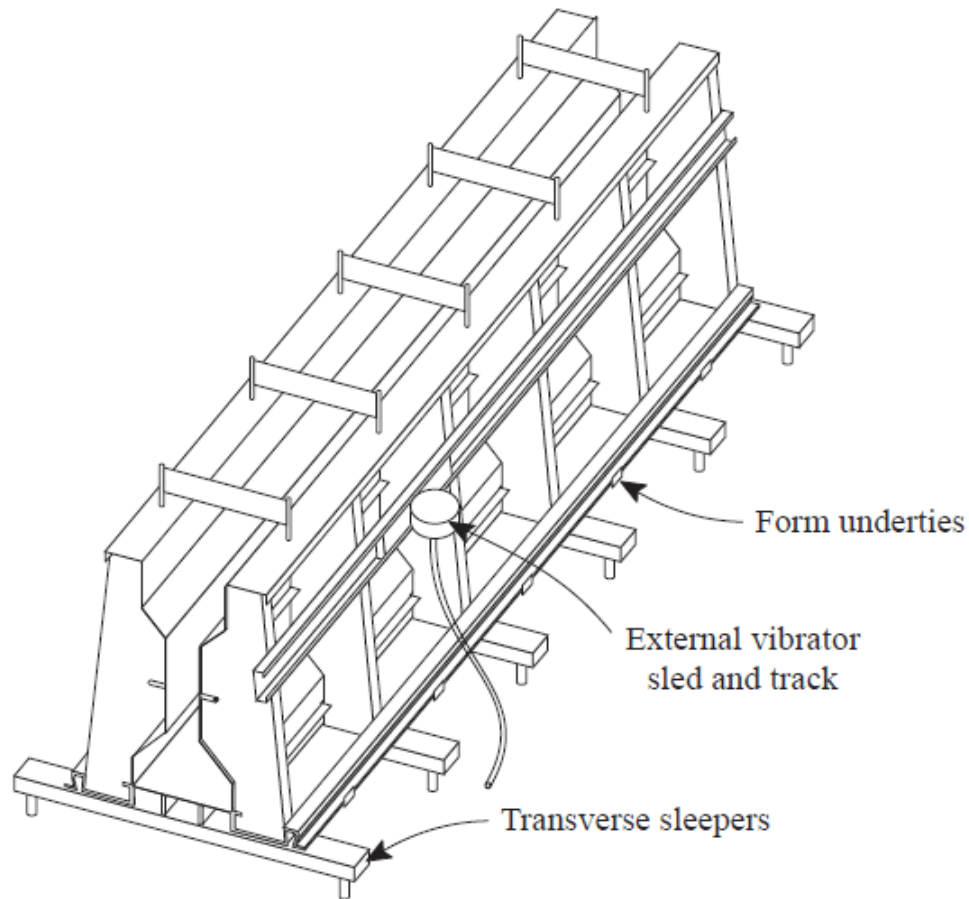


Fig. 5. Steel mould for a bridge I-beam

The forms are removed after curing the concrete and before releasing the tendons. They should be loosened or stripped in such a way that they do not restrain any longitudinal movement or vertical deflection of the member which may take place when the tendons are released.

VI. Grout

For post-tensioning construction, when the tendons are to be bound, grout is needed to transfer loads and to protect the tendons from corrosion. Grout is made of water, sand, and cements or epoxy resins. AASHTO-LRFD requires that details of the protection method be indicated in the contract documents.

2.2 Design Consideration of Precast Pre-stressed Elements and System for Bridges

Structural Design specifications

The design specification herein used is AASHTO LRFD Bridge design specification 2005 together with ERA Bridge design manual 2002. Other design standards such as PCI manuals are also used.

The design vehicle used is HL-93. This consists of “Design Truck” or “Design Tandem” and a lane load at the same place.

Losses of Pre-stress

Loss of pre-stress refers to the reduced tensile stress in the tendons. Although this loss does affect the service performance (such as camber, deflections, and cracking), it has no effect on the ultimate strength of a flexural member unless the tendons are unbounded or the final stress is less than $0.5f_{pu}$. It should be noted, however, that an accurate estimate of pre-stress loss is more pertinent in some pre-stressed concrete members than in others. Pre-stress losses can be divided into two categories:

- Instantaneous losses including losses due to anchorage set (Δf_{pA}), friction between tendons and surrounding materials (Δf_{pF}), and elastic shortening of concrete (Δf_{pES}) during the construction stage;
- Time-dependent losses including losses due to shrinkage (Δf_{pSR}), creep (Δf_{psR}), and relaxation of the steel (Δf_{pR}) during the service life.

The total pre-stress loss (Δf_{pT}) is dependent on the pre-stressing methods.

For pre-tensioned members [12]:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR} \dots\dots\dots 12$$

For post-tensioned members [13]:

$$\Delta f_{pT} = \Delta f_{pA} + \Delta f_{pF} + \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR} \dots\dots\dots 13$$

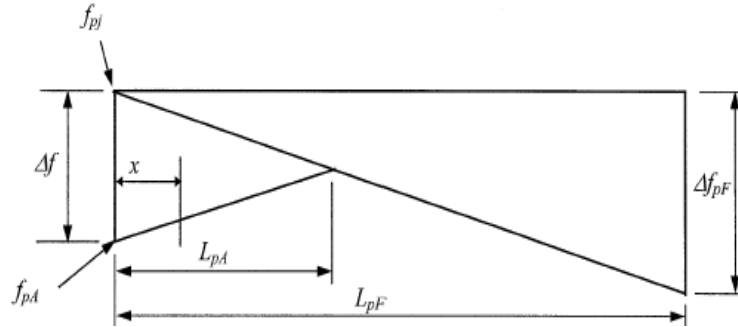


Fig. 6 Anchorage set loss model.

TABLE 1.5 Friction Coefficients for Post-Tensioning Tendons

Type of Tendons and Sheathing	Wobble Coefficient K (1/mm) ×(10 ⁻⁶)	Curvature Coefficient μ(1/rad)
Tendons in rigid and semi rigid galvanized ducts, seven-wire strands	0.66	0.05 ~ 0.15
Pre-greased tendons, wires and seven-wire strands	0.98 ~ 6.6	0.05 ~ 0.15
Mastic-coated tendons, wires and seven-wire strands	3.3 ~ 6.6	0.05 ~ 0.15
Rigid steel pipe deviations	66	0.25, lubrication required

Instantaneous Losses

Anchorage Set Loss

As shown in Figure 6, assuming that the anchorage set loss changes linearly within the length (L_{pA}), the effect of anchorage set on the cable stress can be estimated by the following formula [14]:

$$\Delta f_{pA} = \Delta f \left(1 - \frac{x}{L_{pA}} \right) \dots\dots\dots 14$$

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}} \dots\dots\dots 15$$

$$\Delta f = \frac{2\Delta f_{pF}L_{pA}}{L_{pF}} \dots\dots\dots 16$$

Where ΔL is the thickness of anchorage set; E is the modulus of elasticity of anchorage set; Δf is the change in stress due to anchor set; L_{pA} is the length influenced by anchor set; L_{pF} is the length to a point where loss is known; and x is the horizontal distance from the jacking end to the point considered.

Friction Loss

For a post-tensioned member, friction losses are caused by the tendon profile *curvature effect* and the local deviation in tendon profile *wobble effects*. AASHTO-LRFD specifies the following formula [17]:

$$\Delta f_{pF} = f_{pj}(1 - e^{-(kx+\mu\alpha)}) \dots\dots\dots 17$$

where K is the wobble friction coefficient and μ is the curvature friction coefficient (see Table 1.5);

x is the length of a pre-stressing tendon from the jacking end to the point considered;

and α is the sum of the absolute values of angle change in the pre-stressing steel path from the jacking end.

Elastic Shortening Loss Δf_{pES}

The loss due to elastic shortening can be calculated using the following formula [18]

$$\Delta f_{pES} = \begin{cases} \frac{E_p}{E_{ci}} f_{cgp} & \text{for pretensioned members} \\ \frac{N-1}{2N} \frac{E_p}{E_{ci}} f_{cgp} & \text{for post tensioned members} \end{cases} \dots\dots\dots 18$$

TABLE 1.6 Lump Sum Estimation of Time-Dependent Pre-stress Losses

Type of Beam section	Level	For wires & Strands with $f_{pu}=1620, 1725,$ or 1860MPa	For Bars with $f_{pu}=1000$ or 1100MPa
Rectangular beams and solid slab	Upper bound	200+28PPR	130+41PPR
	Average	180+28PPR	
Box Girder	Upper Bound	145+28PPR	100
	Average	130+28PPR	
I-Girder	Average	$230 \left[1.0 - 0.15 \frac{f'_c - 41}{41} \right] + 41PPR$	130+41PPR
Single-T, Double-T hollow core & Voided slab	Upper Bound	$230 \left[1.0 - 0.15 \frac{f'_c - 41}{41} \right] + 41PPR$	$230 \left[1.0 - 0.15 \frac{f'_c - 41}{41} \right] + 41PPR$
	Average	$230 \left[1.0 - 0.15 \frac{f'_c - 41}{41} \right] + 41PPR$	

Note:

1. PPR is partial pre-stress ratio $= (A_{ps}f_{py}) / (A_{ps}f_{py} + A_s f_y)$.
2. For low-relaxation strands, the above values may be reduced by
 - 28 MPa for box girders

- 41 MPa for rectangular beams, solid slab and I-girders, and
- 55 MPa for single-T, double-T, hollow-core and voided slabs.

where E_{ci} is modulus of elasticity of concrete at transfer (for pre-tensioned members) or after jacking (for post-tensioned members);

N is the number of identical pre-stressing tendons; and

f_{cgp} is sum of the concrete stress at the center of gravity of the pre-stressing tendons due to the pre-stressing force at transfer (for pretensioned members) or after jacking (for post-tensioned members) and the self-weight of members at the section with the maximum moment. For post-tensioned structures with bonded tendons, f_{cgp} may be calculated at the center section of the span for simply supported structures, at the section with the maximum moment for continuous structures.

Time-Dependent Losses

➤ *Lump Sum Estimation*

AASHTO-LRFD provides the approximate lump sum estimation (**Table 1.6**) of time-dependent losses Δf_{pTM} resulting from shrinkage and creep of concrete, and relaxation of the pre-stressing steel. While the use of lump sum losses is acceptable for “average exposure conditions,” for unusual conditions, more-refined estimates are required.

➤ *Refined Estimation*

a. **Shrinkage Loss:** Shrinkage loss can be determined by formulas [19]

$$\Delta f_{pSR} = \begin{cases} 93 - 0.85H & \text{for pretensioned members} \\ 11 - 1.03H & \text{for post tensioned members} \end{cases} \dots \dots \dots 19$$

Where H - is average annual ambient relative humidity (%).

b. **Creep Loss:** Creep loss can be predicted by [20]

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp} \geq 0 \dots \dots \dots 20$$

where f_{cgp} is concrete stress at center of gravity of pre-stressing steel at transfer, and Δf_{cdp} is concrete stress change at center of gravity of pre-stressing steel due to permanent loads, except the load acting at the time the pre-stressing force is applied.

c. **Relaxation Loss:** The total relaxation loss (Δf_{pR}) includes two parts: relaxation at time of transfer Δf_{pR1} and after transfer Δf_{pR2} . For a pretensioned member initially stressed beyond $0.5 f_{pu}$, AASHTO-LRFD specifies [21]

$$\Delta f_{pR1} = \begin{cases} \left[\frac{\log 24t}{10} \left[\frac{f_{pi}}{f_{py}} - 0.55 \right] \right] f_{pi} & \text{for stress - relieved strand} \\ \left[\frac{\log 24t}{40} \left[\frac{f_{pi}}{f_{py}} - 0.55 \right] \right] f_{pi} & \text{for low - relaxation strand} \end{cases} \dots\dots\dots .21$$

For stress-relieved strands [22]

$$\Delta f_{pR2} = \begin{cases} 138 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}) & \text{for stress - relieved strand} \\ 138 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}) & \text{for post - tensioning} \end{cases} \dots\dots\dots .22$$

Where t is time estimated in days from testing to transfer. For low-relaxation strands, Δf_{pR2} is 30% of those values obtained from Eq. [22].

Stress computation in pre-stressed concrete section

For the pre-stressed concrete member section shown in Figure 7, the stress at various load stages can be expressed by the following formula [23]:

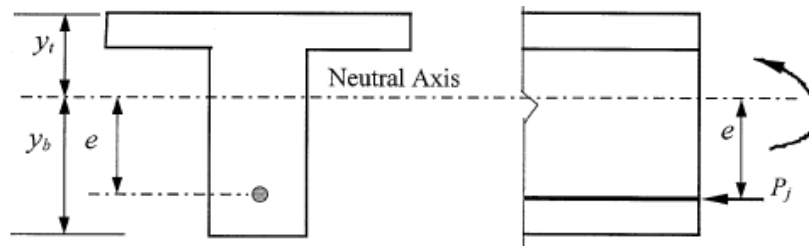


Fig. 7 Pre-stressed concrete member section at Service Limit State.

$$f = \frac{P_j}{A} \pm \frac{P_j e y}{I} \pm \frac{M y}{I} \dots\dots\dots .23$$

TABLE 1.7 Stress Limits for Pre-stressing Tendons

		Pre-stressing Tendon Type		Deformed
		Stress Relieved Strand and Low Relaxation		
Stress Type	Pre-stressing Method	Plain High-Strength Bars Strand		High-Strength Bars
At jacking, f_{pj}	Pretensioning	$0.72f_{pu}$	$0.78f_{pu}$	—
	Post-tensioning	$0.76f_{pu}$	$0.80f_{pu}$	$0.75f_{pu}$
After transfer, f_{pt}	Pretensioning	$0.70f_{pu}$	$0.74f_{pu}$	—
	Post-tensioning — at anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$
	Post-tensioning — general	$0.70f_{pu}$	$0.74f_{pu}$	$0.66f_{pu}$
At Service Limit	After all losses	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
State, f_{pc}				

TABLE 1.8 Temporary Concrete Stress Limits at Jacking State before Losses due to Creep and Shrinkage — Fully Pre-stressed Components

Stress Type	Area and Condition	Stress (MPa)
Compressive	Pretensioned	$0.60 f_{ci}'$
	Post-tensioned	$0.55 f_{ci}'$
Tensile	Precompressed tensile zone without bonded reinforcement	N/A
	Area other than the precompressed tensile zones and without bonded auxiliary reinforcement	$0.25\sqrt{f_{ci}} \leq 1.38$
	Area with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of uncracked section	$0.58\sqrt{f_{ci}}$
	Handling stresses in pre-stressed piles	$0.415\sqrt{f_{ci}}'$

where P_j is the pre-stress force;

A is the cross-sectional area;

I is the moment of inertia;

e is the distance from the center of gravity to the centroid of the pre-stressing cable;

y is the distance from the centroidal axis; and

M is the externally applied moment.

Section properties are dependent on the pre-stressing method and the load stage. In the analysis, the following guidelines may be useful:

- Before bounding of the tendons, for a post-tensioned member, the net section should be used theoretically, but the gross section properties can be used with a negligible tolerance.
- After bounding of tendons, the transformed section should be used, but gross section properties may be used approximately.
- At the service load stage, transformed section properties should be used.

Stress Limits

The stress limits are the basic requirements for designing a pre-stressed concrete member. The purpose for stress limits on the pre-stressing tendons is to mitigate tendon fracture, to avoid inelastic tendon deformation, and to allow for pre-stress losses. Tables 1.7 lists the AASHTO-LRFD stress limits for pre-stressing tendons.

TABLE 1.9 Concrete Stress Limits at Service Limit State after All Losses — Fully Pre-stressed Components

Stress type	Area and Condition		Stress (MPa)
Compressive	Non-segmental bridge at service stage		$0.45f_c'$
	Non segmental bridge during shipping and handling		$0.60f_c'$
	Segmental bridge during shipping and handling		$0.45f_c'$
Tensile	Pre-compressed tensile zone assuming uncracked section	With bonded pre-stressing tendons other than piles	$0.50\sqrt{f_c'}$
		Subjected to severe corrosive conditions	$0.25\sqrt{f_c'}$
		With unbounded pre-stressing tendon	No tension

The purpose for stress limits on the concrete is to ensure no overstressing at jacking and after transfer stages and to avoid cracking (fully pre-stressed) or to control cracking (partially pre-stressed) at the service load stage. Tables 1.8 and 1.9 list the AASHTO-LRFD stress limits for concrete.

A pre-stressed member that does not allow cracking at service loads is called a fully pre-stressed member, whereas one that does is called a partially pre-stressed member. Compared with full pre-stress, partial pre-

stress can minimize camber, especially when the dead load is relatively small, as well as provide savings in pre-stressing steel, in the work required to tension, and in the size of end anchorages and utilizing cheaper mild steel. On the other hand, engineers must be aware that partial pre-stress may cause earlier cracks and greater deflection under overloads and higher principal tensile stresses under service loads. Non-pre-stressed reinforcement is often needed to provide higher flexural strength and to control cracking in a partially pre-stressed member.

Cable Layout

A cable is a group of pre-stressing tendons and the center of gravity of all pre-stressing reinforcement. It is a general design principle that the maximum eccentricity of pre-stressing tendons should occur at locations of maximum moments. Although straight tendons (Figure 8a) and harped multi-straight tendons (Figure 8b and c) are common in the precast members, curved tendons are more popular for cast-in-place post-tensioned members. Typical cable layouts for bridge superstructures are shown in Figure 8.

To ensure that the tensile stress in extreme concrete fibers under service does not exceed code stress limits, cable layout envelopes are delimited. Figure 9 shows limiting envelopes for simply supported members. From Eq. [23], the stress at extreme fiber can be obtained

$$f = \frac{P_j}{A} \pm \frac{P_j e C}{I} \pm \frac{M C}{I} \dots\dots\dots.24$$

where C is the distance of the top or bottom extreme fibers from the center gravity of the section (y_b or y_t as shown in Figure 7).

When no tensile stress is allowed, the limiting eccentricity envelope can be solved from Eq. [24] with

$$e_{limit} = \frac{I}{AC} \pm \frac{M}{IP_j} \dots\dots\dots.25$$

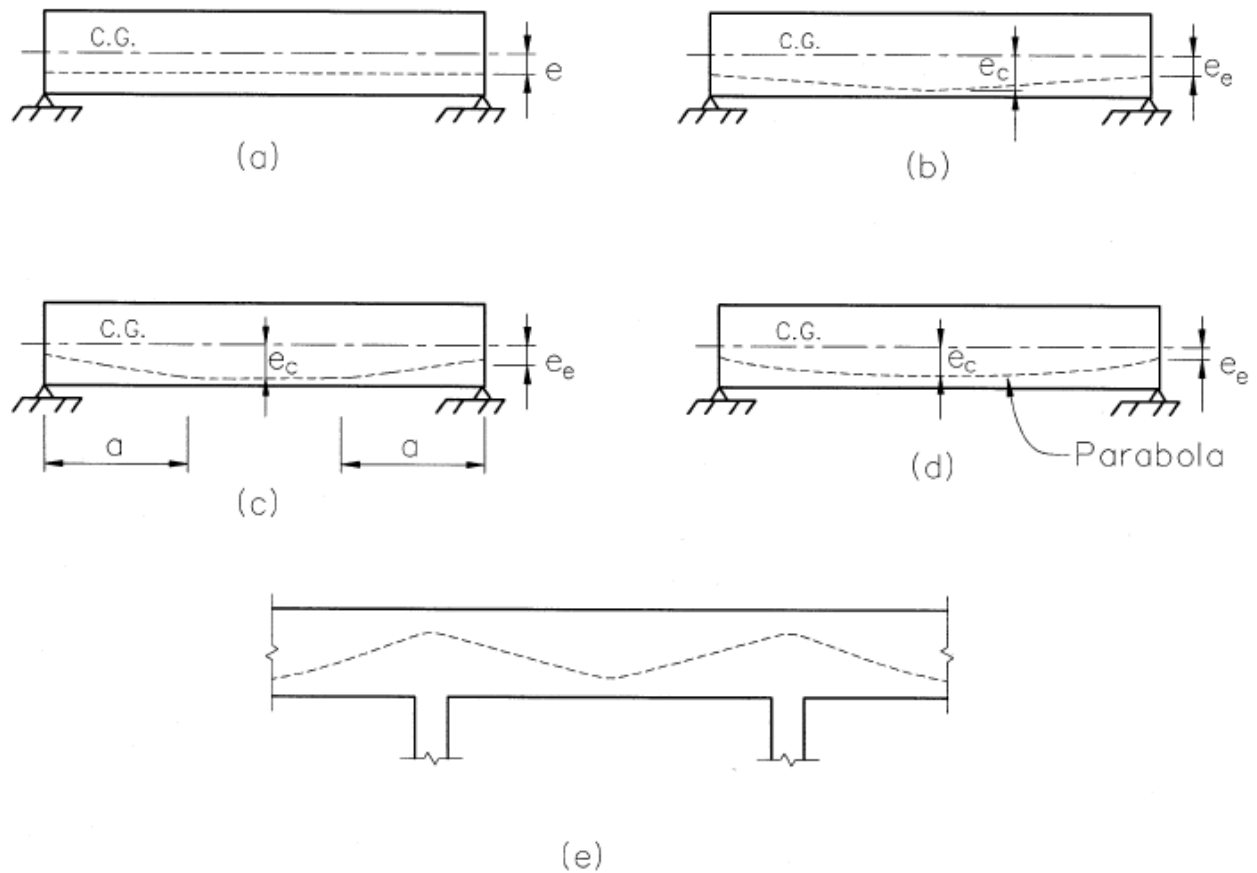


Fig. 8 Cable layout for bridge superstructures.

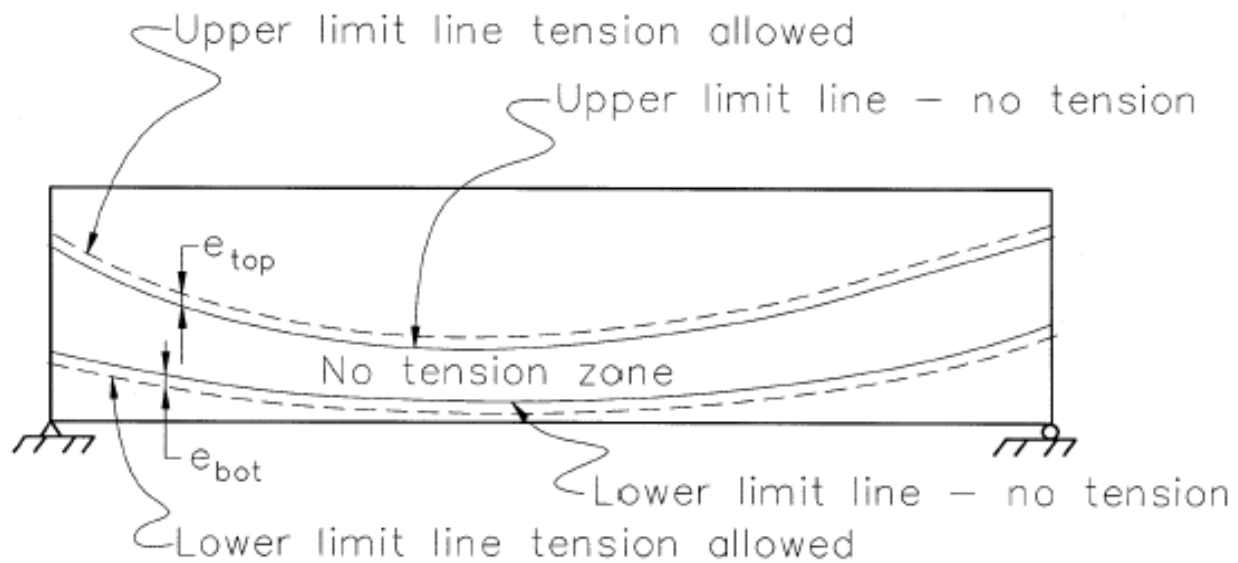


Fig. 9 Cable layout envelopes.

For limited tension stress f_t , additional eccentricities can be obtained:

$$e' = \frac{f_t I}{P_j C} \dots\dots\dots 26$$

Secondary Moments

The primary moment ($M_1 = P_j e$) is defined as the moment in the concrete section caused by the eccentricity of the pre-stress for a statically determinate member. The secondary moment M_s (Figure 10d) is defined as moment induced by pre-stress and structural continuity in an indeterminate member. Secondary moments can be obtained by various methods. The resulting moment is simply the sum of the primary and secondary moments.

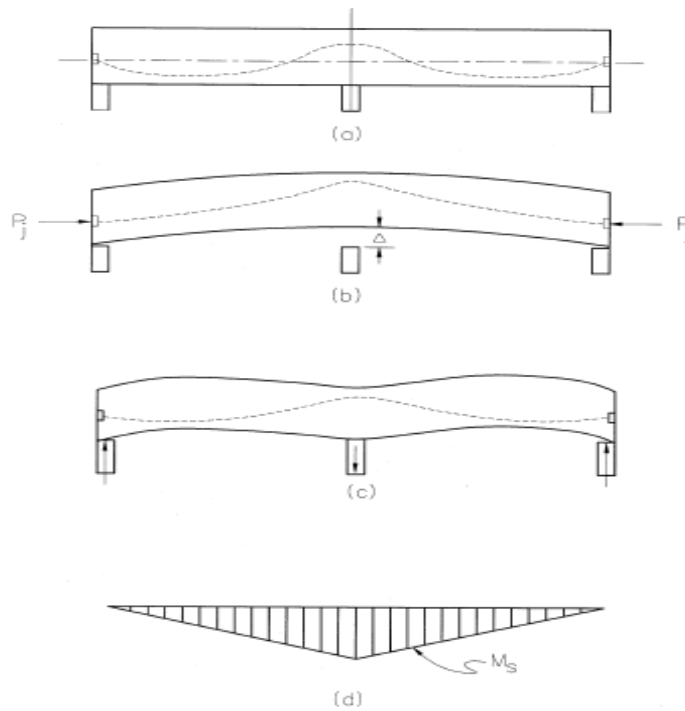


Fig. 10 Secondary moments.

Flexural Strength

Flexural strength is based on the following assumptions:

- For members with bonded tendons, strain is linearly distributed across a section; for members with unbonded tendons, the total change in tendon length is equal to the total change in member length over the distance between two anchorage points.
- The maximum usable strain at extreme compressive fiber is 0.003.
- The tensile strength of concrete is neglected.

- A concrete stress of $0.85f_c'$ is uniformly distributed over an equivalent compression zone.
- Non pre-stressed reinforcement reaches the yield strength, and the corresponding stresses in the pre-stressing tendons are compatible based on plane section assumptions.

For a member with a flanged section (Figure 11) subjected to uniaxial bending, the equations of equilibrium are used to give a nominal moment resistance of [27]

$$M_n = A_{ps}f_{ps} \left(d_p - \frac{a}{2} \right) + A_s f_y \left(d_s - \frac{a}{2} \right) - A_s' f_y' \left(d_s' - \frac{a}{2} \right) + 0.85f_c' (b - b_w) \beta_1 h_f \left(\frac{a}{2} - \frac{h_f}{2} \right) \dots \dots 27$$

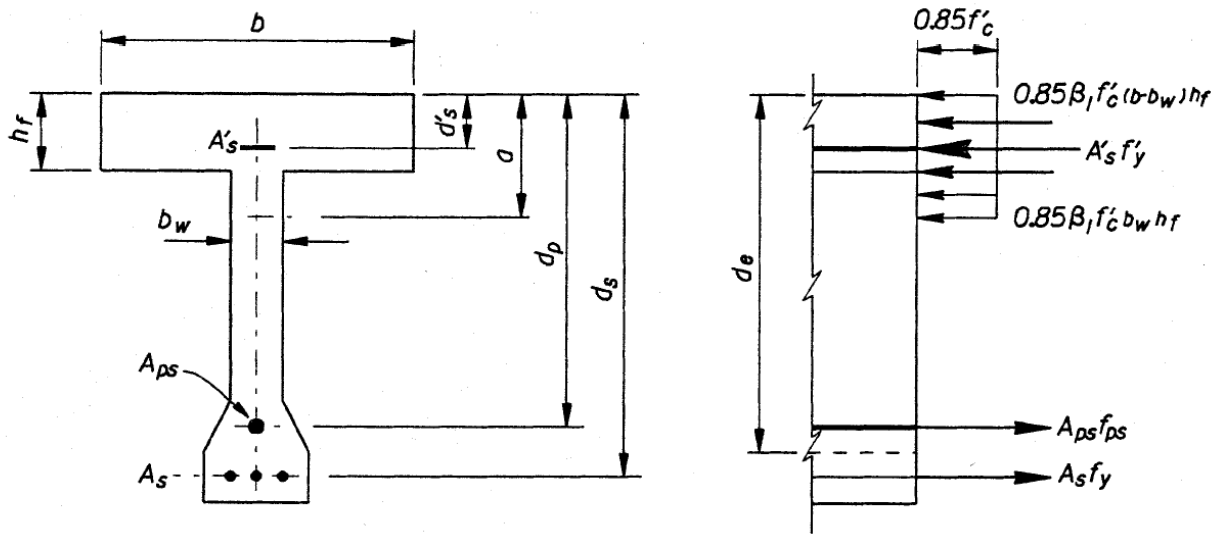


Fig. 11 A flanged section at nominal moment capacity state.

$$a = \beta_1 c \dots \dots \dots 28$$

For bonded tendons

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y' - 0.85\beta_1 f_c' (b - b_w) h_f}{0.85\beta_1 f_c' b_w + k A_{ps} \frac{f_{pu}}{d_p}} \geq h_f \dots \dots \dots 29$$

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right) \dots \dots \dots 30$$

$$k = 2 \left(1.04 - \frac{f_{py}}{f_{pu}} \right) \dots \dots \dots 31$$

$$0.85 \geq \beta_1 = 0.85 - \frac{(f_c' - 28)(0.05)}{7} \geq 0.65 \dots \dots \dots 32$$

where

- A represents area;
- f is stress;
- b is the width of the compression face of member;
- b_w is the web width of a section;
- h_f is the compression flange depth of the cross section;
- d_p and d_s are distances from extreme compression fiber to the centroid of pre-stressing tendons and to centroid of tension reinforcement, respectively;
- subscripts c and y indicate specified strength for concrete and steel, respectively;
- subscripts p and s mean pre-stressing steel and reinforcement steel, respectively;
- subscripts p_s , p_y , and p_u correspond to states of nominal moment capacity, yield, and specified tensile strength of pre-stressing steel, respectively;
- Superscript ' represents compression.

The above equations also can be used for rectangular section in which $b_w = b$ is taken.

For unbound tendons [33]:

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y - 0.85\beta_1 f'_c (b - b_w) h_f}{0.85\beta_1 f'_c b_w} \geq h_f \quad \dots \dots \dots 33$$

$$f_{ps} = f_{ps} + \Omega_u E_p \varepsilon_{cu} \left(\frac{d_p}{c} - 1.0 \right) \frac{L_1}{L_2} \leq 0.94 f_{py} \quad \dots \dots \dots 34$$

where L_1 is length of loaded span or spans affected by the same tendons;

L_2 is total length of tendon

Between anchorage; Ω_u is the bond reduction coefficient given by [35]

$$\Omega_u = \begin{cases} \frac{3}{L/d_p} & \text{for uniform and near third point loading} \\ \frac{1.5}{L/d_p} & \text{for near midspan loading} \end{cases} \quad \dots \dots \dots 35$$

in which L is span length.

Maximum reinforcement limits [36]:

$$\frac{c}{d_e} \leq 0.42 \dots\dots\dots 36$$

$$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y} \dots\dots\dots 37$$

Minimum reinforcement limits [38]:

$$\phi M_n \geq 1.2M_{cr} \dots\dots\dots 38$$

in which ϕ is flexural resistance factor 1.0 for pre-stressed concrete and 0.9 for reinforced concrete;

M_{cr} is the cracking moment strength given by the elastic stress distribution and the modulus of rupture of concrete.

$$M_{cr} = \frac{I}{y_t} (f_r + f_{pe} - f_d) \dots\dots\dots 39$$

Where

- f_{pe} is compressive stress in concrete due to effective pre-stresses; and
- f_d is stress due to unfactored self-weight;
- Both f_{pe} and f_d are stresses at extreme fiber where tensile stresses are produced by externally applied loads.

Shear Strength

The shear resistance is contributed by the concrete, the transverse reinforcement and vertical component of pre-stressing force. The modified compression field theory-based shear design strength was adopted by the AASHTO-LRFD and has the formula [40]:

$$V_n = \text{the lesser of } \begin{cases} V_c + V_s + V_p \\ 0.25f'_c b_v d_v + V_p \end{cases} \dots\dots\dots 40$$

Where

$$V_c = 0.083\beta\sqrt{f'_c} b_v d_v \dots\dots\dots 41$$

$$V_s = \frac{A_v f_y d_v (\cos \theta + \cot \alpha) \sin \alpha}{s} \dots\dots\dots 42$$

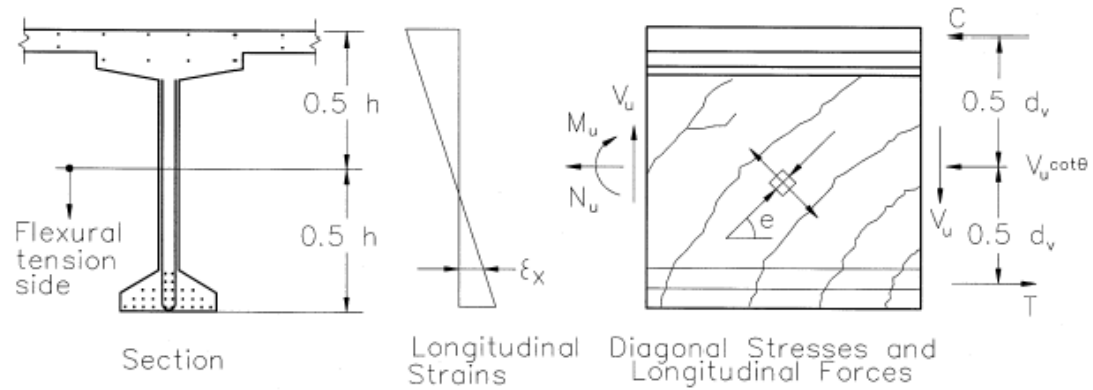


Fig. 12 Values of θ and β for Sections with Transverse Reinforcement

Where

- b_v is the effective web width determined by subtracting the diameters of ungrouted ducts or one half the diameters of grouted ducts;
- d_v is the effective depth between the resultants of the tensile and compressive forces due to flexure, but not to be taken less than the greater of $0.9de$ or $0.72h$;
- A_v is the area of transverse reinforcement within distance s ;
- s is the spacing of stirrups;
- α is the angle of inclination of transverse reinforcement to longitudinal axis;
- β is a factor indicating ability of diagonally cracked concrete to transmit tension;
- θ° is the angle of inclination of diagonal compressive stresses (Figure 12).

TABLE 1.10 Values of θ and β for Sections with Transverse Reinforcement

$v f_c'$	Angle (degree)	$\epsilon_x \times 1000$										
		-0.2	-0.15	-0.1	0	0.125	0.25	0.50	0.75	1.00	1.50	2.00
≤ 0.05	θ	27.0	27.0	27.0	27.0	27.0	28.5	29.0	33.0	36.0	41.0	43.0
	β	6.78	6.17	5.63	4.88	3.99	3.49	2.51	2.37	2.23	1.95	1.72
0.075	θ	27.0	27.0	27.0	27.0	27.0	27.5	30.0	33.5	36.0	40.0	42.0
	β	6.78	6.17	5.63	4.88	3.65	3.01	2.47	2.33	2.16	1.90	1.65
0.100	θ	23.5	23.5	23.5	23.5	24.0	26.5	30.5	34.0	36.0	38.0	39.0
	β	6.50	5.87	5.31	3.26	2.61	2.54	2.41	2.28	2.09	1.72	1.45
0.127	θ	20.0	21.0	22.0	23.5	26.0	28.0	31.5	34.0	36.0	37.0	38.0
	β	2.71	2.71	2.71	2.60	2.57	2.50	2.37	2.18	2.01	1.60	1.35
0.150	θ	22.0	22.5	23.5	25.0	27.0	29.0	32.0	34.0	36.0	36.5	37.0
	β	2.66	2.61	2.61	2.55	2.50	2.45	2.28	2.06	1.93	1.50	1.24
0.175	θ	23.5	24.0	25.0	26.5	28.0	30.0	32.5	34.0	35.0	35.5	36.0
	β	2.59	2.58	2.54	2.50	2.41	2.39	2.20	1.95	1.74	1.35	1.11
0.200	θ	25.0	25.5	26.5	27.5	29.0	31.0	33.0	34.0	34.5	35.0	36.0
	β	2.55	2.49	2.48	2.45	2.37	2.33	2.10	1.82	1.58	1.21	1.00
0.225	θ	26.5	27.0	27.5	29.0	30.5	32.0	33.0	34.0	34.5	36.5	39.0
	β	2.45	2.38	2.43	2.37	2.33	2.27	1.92	1.67	1.43	1.18	1.14
0.250	θ	28.0	28.5	29.0	30.0	31.0	32.0	33.0	34.0	35.5	38.5	41.5
	β	2.36	2.32	2.36	2.30	2.28	2.01	1.64	1.52	1.40	1.30	1.25

The values of β and θ for sections with transverse reinforcement are given in Table 1.10. In using this table, the shear stress v and strain ϵ_x in the reinforcement on the flexural tension side of the member are determined by [43]

$$v = \frac{V_u - \phi V_p}{\phi b_v d_v} \dots\dots\dots 43$$

$$\epsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + 0.5V_u \cot \theta - A_{ps} f_{po}}{E_s A_s + E_p A_{ps}} \leq 0.002 \dots\dots\dots 44$$

where

M_u and N_u are factored moment and axial force (taken as positive if compressive) associated with V_u and f_{po} is stress in pre-stressing steel when the stress in the surrounding concrete is zero and can be conservatively taken as the effective stress after losses f_{pe} . When the value of ϵ_x calculated from the above equation is negative, its absolute value shall be reduced by multiplying by the factor F_ϵ taken as [45]

$$F_\epsilon = \frac{E_s A_s + E_p A_{ps}}{E_c A_c + E_s A_s + E_p A_{ps}} \dots\dots\dots 45$$

where E_s , E_p , and E_c are modulus of elasticity for reinforcement, pre-stressing steel, and concrete, respectively; A_c is area of concrete on the flexural tension side of the member as shown in Figure 12.

Minimum transverse reinforcement [46]:

$$A_{vmin} = 0.083 \sqrt{f'_c} \frac{b_v s}{f_y} \dots\dots\dots 46$$

Maximum spacing of transverse reinforcement [47]&[48]:

$$\text{for } V_u < 0.1f'_c b_v d_v \quad s_{max} = \text{the smaller of } \begin{cases} 0.8d_v \\ 600mm \end{cases} \dots\dots\dots 47$$

$$\text{for } V_u \geq 0.1f'_c b_v d_v \quad s_{max} = \text{the smaller of } \begin{cases} 0.4d_v \\ 300mm \end{cases} \dots\dots\dots 48$$

I. Camber and Deflections

As opposed to load deflection, camber is usually referred to as reversed deflection and is caused by pre-stressing. A careful evaluation of camber and deflection for a pre-stressed concrete member is necessary to meet serviceability requirements. The following formulas developed by the moment–area method can be used to estimate mid-span immediate camber for simply supported members as shown in Figure 8.

For straight tendon (Figure 8a) [49]:

$$\Delta = \frac{L^2}{8E_c I} M_e \dots\dots\dots 49$$

For one-point harping tendon (Figure 8b) [50]:

$$\Delta = \frac{L^2}{8E_c I} (M_c + \frac{2}{3} M_e) \dots\dots\dots 50$$

For two-point harping tendon (Figure 8c) [51]:

$$\Delta = \frac{L^2}{8E_c I} (M_c + M_e - \frac{M_e}{3} (\frac{2a}{L})^2) \dots\dots\dots 51$$

For parabola tendon (Figure 8d) [52]:

$$\Delta = \frac{L^2}{8E_c I} (M_e + \frac{5}{6} M_c) \dots\dots\dots 52$$

Where

- M_e is the primary moment at end,
- P_{je} end and M_c is the primary moment at mid-span P_{jec} .

Uncracked gross section properties are often used in calculating camber. For deflection at service loads, cracked section properties, i.e., moment of inertia I_{cr} , should be used at the post-cracking service load stage. It should be noted that long term effect of creep and shrinkage shall be considered in the final camber calculations. In general, final camber may be assumed 3 times as great as immediate camber.

II. Anchorage Zones

In a pretensioned member, pre-stressing tendons transfer the compression load to the surrounding concrete over a length L_t gradually. In a post-tensioned member, pre-stressing tendons transfer the compression directly to the end of the member through bearing plates and anchors. The anchorage zone, based on the principle of St.Venant, is geometrically defined as the volume of concrete through which the pre-stressing force at the anchorage device spreads transversely to a more linear stress distribution across the entire cross section at some distance from the anchorage device

For design purposes, the anchorage zone can be divided into general and local zones. The region of tensile stresses is the general zone. The region of high compressive stresses (immediately ahead of the anchorage device) is the local zone. For the design of the general zone, a “strut-and-tie model,” a refined elastic stress analysis or approximate methods may be used to determine the stresses, while the resistance to bursting forces is provided by reinforcing spirals, closed hoops, or anchorage transverse ties. For the design of the local zone, bearing pressure is a major concern.

2.3 Construction consideration of Precast Pre-stressed Elements and System for Bridges

2.3.1 Product handling

The loads and forces on precast and prestressed members during production, transportation or erection will frequently require a separate analysis because concrete strengths are lower and support points and orientation are usually different from the girder in its final load carrying position.

The Most economical element for a project is usually the largest, considering

1. Stability and stressed on the element during handling
2. Transportation size and weight regulations and equipment restrictions.
3. Available crane capacity at both the point and the project site. Position of the crane must be considered, sine capacity is a function of reach.
4. Storage space, track turning radius, and other site restrictions.

Member shapes must be such that reinforcement and concrete placement can be accomplished effectively. To remove a member from a form without partially dismantling the form, and to reduce trapping air bubbles, the sides must have adequate draft. (See figure below)



Fig. 13 draft on side of forms

2.3.2 Structural design criteria

Precast products must be designed for the loading which occur during each phase or their existence. The items which affect the forces imposed during each phases are:-

- a. Stripping
- b. Yard handling and storage
- c. Transportation to the project site
- d. Erection

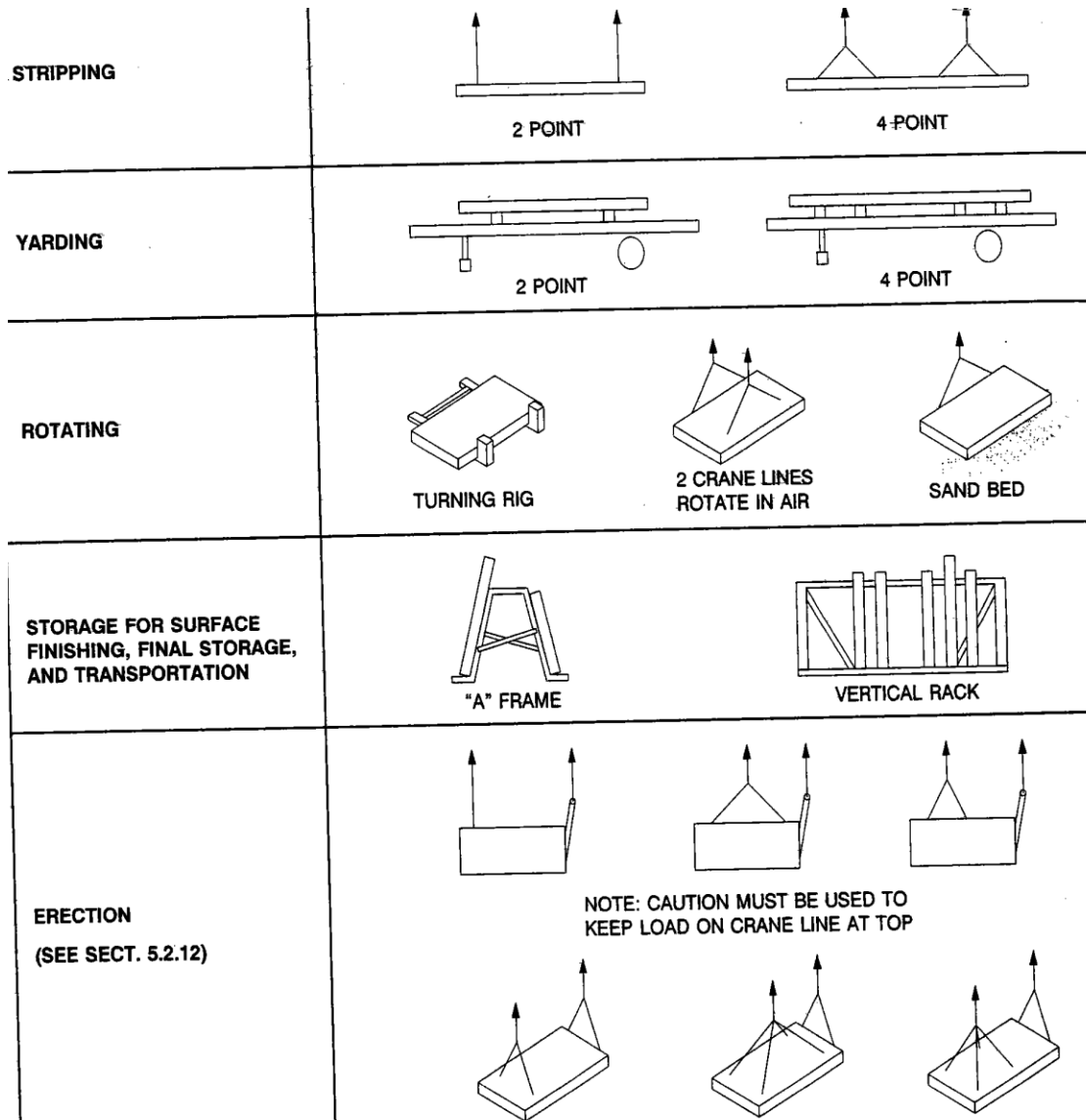


Fig. 14 typical handling methods

2.4 Environmental and social impact concern

The bridge construction process has temporary negative impacts on environment and local community. The following are the major ones.

- Traffic disruption
- Air quality
- Noise and vibration
- Hazardous wastes
- Water resource and water quality

The magnitudes of these effects are highly magnified when the construction project is located within big cities. This is due to construction process itself requires working spaces for storage of materials and maneuvering of equipments thus by requiring more frequent trips to the site for incremental deliveries of material and equipment as well as the removal of construction debris.

In developing countries, such as in Ethiopia, the growing need of population living in big cities is demanding more construction activities for expansion of cities. This is one of the prime reasons for traffic congestions and air pollution occurrence in Addis Ababa.

In the case of precast concrete construction, prefabrication of structural and architectural concrete products is performed at more accessible sites away from crowded city centers. These precast elements for bridge construction are fabricated under controlled conditions in areas with adequate operational and storage space.

After the production of bridge elements is completed, they are transported directly to the project site for erection. These finished components can either make only one trip to the construction site through the congested city traffic or can be transported during off peak hours.

2.5 Comparison of PPC and RC Bridges

One of the major differences between pre-stressed concrete and reinforced concrete, with regard to their physical attributes, is that higher strength materials (for both concrete and steel) are used for pre-stressed concrete. In pre-stressed concrete the high-strength steel is tensioned and anchored against the concrete, which produces a number of desirable effects:

- The high strength of the steel can be properly used, even at service load levels.
- The pre-stressing tends to neutralize tensile stresses and strains induced by the load, so that cracking of the section is eliminated and, as a result, the full concrete section becomes active in resisting the load. This mechanism is much more effective than is the case for reinforced concrete where only the uncracked part of the section in the compression zone participates in resisting the load.
- The deformations induced by the pre-stressing serve to offset those produced by the load, and can be used by the designer to control deflections.

Higher strength concrete may be used to obtain more economic sections than with reinforced concrete.

The following advantages of pre-stressed concrete are often put forward when compared to reinforced concrete

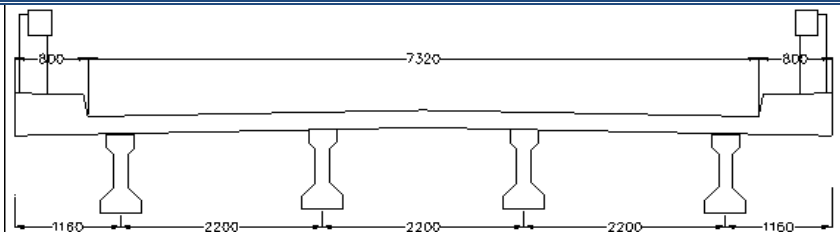
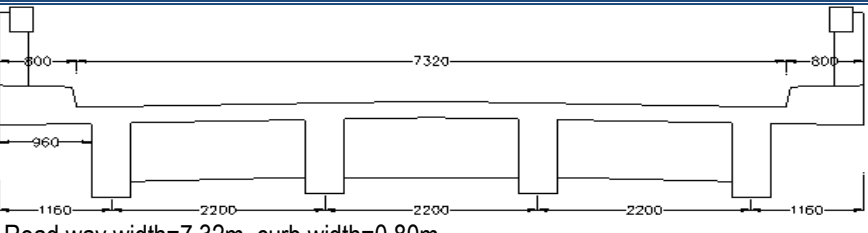
- Pre-stressed concrete requires smaller quantities of material than reinforced concrete because high-strength materials are efficiently and effectively used and because it uses the entire section to resist the load. This means that pre-stressed concrete members are lighter and more slender than their reinforced concrete counterparts.
- The fact that members are lighter and more slender if pre-stressed concrete rather than reinforced concrete is used, leads to other advantages:
 - Savings can be realized in the reduced cost of lighter supporting structures and, in the case of precast elements, in the reduced handling and transportation costs.
 - Aesthetically pleasing structures are more readily provided.
 - Longer spans are possible because of the reduced self weight.
 - Innovative construction methods are facilitated.
 - Thinner slabs result in reduced building heights and consequent savings in the cost of finishes.

These advantages are particularly evident in the case of long span bridges and multi-storey buildings.

- Pre-stressed concrete generally provides better corrosion protection to the reinforcement than does reinforced concrete. This advantage is significant for structures subjected to aggressive environments and for fluid-retaining structures.
- Improved deflection control is possible with pre-stressed concrete.
- Pre-stressed concrete members will require less shear reinforcement than reinforced concrete members. This follows from the fact that the shear capacity of a pre-stressed member is increased by curved tendons, which carry some of the shear, and by the pre-compression, which reduces the principal tension.
- It often happens that the worst service load condition for a pre-stressed concrete structure occurs during the pre-stressing operation. In such a case, it can be claimed that the safety of the structure has been partially tested: If the structure successfully withstands the effects of the pre-stressing operation, chances are good that it will perform well during its service life.

A comparison of the economic advantages or disadvantages of pre-stressed concrete with those of reinforced concrete is complicated by the fact that each has a range of applicability, depending on the type of structure and the specific design requirements. However, if such a comparison is made where the ranges of applicability overlap, care must be taken to include not only the cost of the materials but also to include the additional costs associated with pre-stressed concrete, such as the use of specialized equipment and hardware, greater design effort, more supervision and the use of specialized personnel. Such a comparison should also reflect the relative performance and cost advantages inherent in each type of structure.

3. Analysis and design of Precast Prestressed Concrete girder and RC girder Bridges

Steps	Design of Precast Prestressed Girder Bridge	Design of RC Deck Girder Bridge
1. Develop Geometry	 <p>Road way width=7.32m, curb width=0.80m C/C Girder Spacing=2.20m Deck Slab thickness, ts=0.20m Girder Depth=0.045*S</p>	 <p>Road way width=7.32m, curb width=0.80m C/C Girder Spacing=2.20m Deck Slab thickness, ts=0.20m Girder Depth=0.07*S (AASHTO 2.5.2.6.3.1)</p>
2. Select Material	<p>a) Concrete: - C-30 for composite deck, C-40 for precast girder b) Reinforcement – Prestressing steel-Grade 250, Reinforcing steel- Grade 60 for bar dia >20mm Grade 40 for bar dia ≤20mm c) Wearing surface: - unit weight 22.5KN/m³</p>	<p>a) Concrete: - C-30 for all sections, b) Reinforcement: - Reinforcing steel- Grade 60 for bar dia >20mm Grade 40 for bar dia ≤20mm c) Wearing surface: - unit weight 22.5KN/m³</p>
3. Compute Section properties	<p>a) Cross sectional Properties for a Single Beam</p> <ul style="list-style-type: none"> ▪ Compute cross-sectional Area, A ▪ Moment of inertia about the centroid of the non-composite precast girder, I_{xx} (In strong Direction) ▪ Distance from centroid to the extreme bottom fiber of the non-composite precast girder, y_b ▪ Distance from centroid to the extreme top fiber of the non-composite precast girder, y_t ▪ Section modulus referenced to the extreme bottom fiber of the non-composite precast girder, $S_b=I/y_b$ ▪ Section modulus referenced to the extreme top fiber of the non-composite precast girder, $S_t=I/y_t$ <p>b) Composite section</p> <ul style="list-style-type: none"> ▪ Effective Flange Width: - As per AASHTO LRFD Art. 4.6.2.6.1 Provision ▪ Modular Ratio between Slab and Girder Concrete, n:- $n=(E_c \text{ for Slab}/E_c \text{ for Girder})$ ▪ Transformed Section Properties:- compute the following <ul style="list-style-type: none"> - Total area of composite section, A_c - Total height of composite section, h_c - Moment of inertia about the centroid of the composite Section, I_c - Distance from the centroid of the composite section to extreme bottom fiber of the precast girder, y_{bc} - Distance from the centroid of the composite section to extreme top fiber of the precast girder, y_{tg} - Distance from the centroid of the composite section to extreme top fiber of the slab, y_{tc} - Section modulus of composite section referenced to the extreme bottom fiber of the precast girder, S_{bc} - Section modulus of composite section referenced to the top fiber of the precast girder, S_{tg} - Section modulus of composite section referenced to the top fiber of the slab, S_{tc} 	<p>Not Applicable for this design,</p> <p>Not Applicable for this design,</p>
4. Perform Structural analysis	<ul style="list-style-type: none"> ▪ Calculate dead loads (DC and DW) <ul style="list-style-type: none"> - The self-weight of the girder and the weight of the slab act on the non-composite simple span structure, while the weight of the barriers, future wearing surface, live load, and dynamic load act on the composite simple span structure. - The superimposed dead loads placed on the bridge, including loads from railing and wearing surface, can be distributed uniformly among all girders [AASHTO LRFD Art. 4.6.2.2.1] ▪ Calculate shear force and bending moment due to DC and DW $V_x = w \left(\frac{L}{2} - x \right) \text{ \& } M_x = \frac{wx(L-x)}{2}$ ▪ Calculate shear force and bending moment due to live loads <ul style="list-style-type: none"> - Compute LL distribution factors for Shear and bending moment as per the conditions in AASHTO LRFD Art. 4.6.2.2 ▪ Load combinations <ul style="list-style-type: none"> - In an LRFD design, the total factored loads are taken as follows: $Q = n \sum y_i Q_i$ (AASHTO 3.4.1-1) - Check compressive stress in prestressed concrete components for Service I $Q = 1.00(DC + DW) + 1.00(LL + IM)_{HL-93}$ (AASHTO Table 3.4.1-1) - Check tensile stress in prestressed concrete components for Service III $Q = 1.00(DC + DW) + 0.80(LL + IM)$ (AASHTO Table 3.4.1-1) - Check resistances for Strength I $Q = 1.25DC + 1.5DW + 1.75(LL + IM)_{HL-93}$ (AASHTO Table 3.4.1-1 & -2) 	<p>Applicable & similar procedure is followed</p> <p>Applicable & similar procedure is followed</p> <p>Applicable & similar procedure is followed, load combination and distribution factors for live load computation is similar.</p>

<p>5. Determine prestress force</p>	<ul style="list-style-type: none"> - The preliminary prestressing force is usually determined on the basis of the service limit state service III load condition at mid-span. The tensile stress limit for Service III is $0.5\sqrt{f_c'}$. - Assume the center of gravity of the strands, y_{bs}, at midspan is assumed to be located at 5% of the girder depth from the bottom fiber. - Eccentricity of prestressing steel at midspan $ec = y_b - y_{bs}$ $f_b = \frac{(M_{DC1} + M_{DC2})}{S_b} + \left(\frac{M_{DC3} + M_{DW} + 0.8(M_{LL+IM}HL93)}{S_{bc}} \right) - \frac{P}{A} - \frac{P * e_c}{S_b} \leq 0.5\sqrt{f_c'}$ <ul style="list-style-type: none"> - From the above equation obtain preliminary prestressing force, P_i 	<p>Not Applicable for this design,</p>
<p>6. Estimate prestress loss</p>	<ul style="list-style-type: none"> ▪ Total prestressing losses:- $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ where - Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads. - Δf_{pLT} = losses due to long-term shrinkage and creep of concrete and relaxation of prestressing steel ▪ Net Prestressing Force:- $P = P_i - \Delta f_{pT}$ 	<p>Not Applicable for this design,</p>
<p>7. Check stresses under Service limit state</p>	<ul style="list-style-type: none"> ▪ Check concrete stress limit at <u>transfer</u> condition <ul style="list-style-type: none"> - concrete compressive stress limit (MPa):- Stress limit = $0.6f_{ci}$ (AASHTO Table 5.9.4.1.1-1) - Concrete tensile stress limits Stress limit = $0.25\sqrt{f_c'}$ (AASHTO Table 5.9.4.1.2-1) ▪ Check concrete stress at service condition(MPa) <ul style="list-style-type: none"> - Concrete compressive stress limit Stress limit = $0.5\sqrt{f_c'}$ (AASHTO Table 5.9.4.2.1-1) - Concrete tensile stress limits Stress limit = $0.45f_c'$ (AASHTO Table 5.9.4.2.2-1) ▪ Fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to service III limit state with in the tensile stress limit specified in [Table 5.9.4.2.2.-1] 	<ul style="list-style-type: none"> ▪ Crack control:- Flexural cracking is controlled by limiting the spacing "S" in the reinforcement closest to the tension face under service load stress f_s ▪ Investigate fatigue:-Allowable fatigue stress range f_r in reinforcement is given by [AASHTO 5.5.3.2] $f_r = 145 - 0.33f_{min} + 55(r/h), \text{ MPa}$
<p>8. Strength limit state-Flexure</p>	<ul style="list-style-type: none"> ▪ Maximum factored moment, M_u <ul style="list-style-type: none"> - The maximum moment for a simple span structure occurs at the midspan. Strength-I $M_u = 1.25 [M_{DC1} + M_{DC2} + M_{DC3}] + 1.5M_{DW} + 1.75[M_{(LL+IM)HL93}]$ 	<p>Applicable to this design, similar load combination is used to obtain the maximum moment at mid span.</p>
<p>8. Strength limit state-Flexure</p>	<ul style="list-style-type: none"> ▪ Average prestressing steel stress $f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right)$ (AASHTO 5.7.3.1.1-1) In which, $k = 2 \left(1.04 - \frac{f_{py}}{f_{pu}} \right)$ <ul style="list-style-type: none"> - Assume that the compressive area is a rectangular section, and assume $c/d_x \geq 0.6$, thus $f_s = f_y$. (AASHTO 5.7.2.1). The distance from neutral axis to extreme compressive fiber is as follows: $c = \frac{A_{ps}f_{pu} + A_s f_s - A_s' f_s'}{0.85 f_c' \beta_1 b + k A_{ps} \left(\frac{f_{pu}}{d_p} \right)}$ (AASHTO 5.7.3.1.1-4) <ul style="list-style-type: none"> - Depth of compression block: $a = \beta_1 c$, check $a < t_s$. ok 	<p>Not applicable for this design</p>
<p>9. Check reinforcement limit</p>	<ul style="list-style-type: none"> ▪ The amount of prestressed tensile reinforcement at any section of a flexure member is adequate to develop a factored flexural resistance, M_r, $M_r = \min(1.33M_u, M_{cr})$ (AASHTO 5.7.3.3.2) Where $M_{cr} = \gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left(\frac{\epsilon_c}{\epsilon_{nc}} - 1 \right) \right]$ (AASHTO 5.7.3.3.2-1) 	<p>Applicable for this design, except the resistance from the prestressing steel is zero.</p> $\phi M_n = \phi \left[A_s f_y \left(d_p - \frac{a}{2} \right) \right]$
<p>9. Check reinforcement limit</p>	<ul style="list-style-type: none"> ▪ The amount of prestressed tensile reinforcement at any section of a flexure member is adequate to develop a factored flexural resistance, M_r, $M_r = \min(1.33M_u, M_{cr})$ (AASHTO 5.7.3.3.2) Where $M_{cr} = \gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left(\frac{\epsilon_c}{\epsilon_{nc}} - 1 \right) \right]$ (AASHTO 5.7.3.3.2-1) 	<p>Applicable in this design, for normal reinforced concrete beams, M_{cr} is obtained as:-</p> <p>where $M_{cr} = f_r I_g / y_t$ $f_r = 0.63 \text{ sqrt}(f_c')$</p>
<p>10. Design strength limit state-shear</p>	<ul style="list-style-type: none"> ▪ Critical section for shear design <ul style="list-style-type: none"> - The critical section for shear design is located at d_v from the internal face of the support $d_v \geq \begin{cases} d_g - \frac{a}{2} \\ 0.9d_g \\ 0.72h \end{cases}$ (AASHTO 5.8.2.9) Where d_e= effective depth from extreme compression fiber to centroid of tensile reinforcement = $H - y_{bs}$. 	<p>Similar provision of d_v maintained in the shear design for reinforced concrete girder bridges.</p>
<p>10. Design strength limit state-shear</p>	<ul style="list-style-type: none"> ▪ Contribution of concrete to nominal shear resistance, V_c $V_c = 0.083 \beta \sqrt{f_c'} b_v d_v$ (AASHTO 5.8.3.3-3) Requirement for shear reinforcement <ul style="list-style-type: none"> - if $V_u \geq 0.5\phi(V_c + V_p)$, transverse shear reinforcement is needed - Required area of transverse shear reinforcement $\frac{V_u}{\phi} \leq V_n = V_c + V_s$ (AASHTO 5.8.3.3-1) 	<p>Applicable in this design</p> <ul style="list-style-type: none"> ▪ Requirement for shear reinforcement <ul style="list-style-type: none"> - if $V_u \geq 0.5\phi(V_c)$, transverse shear reinforcement is needed - Required area of transverse shear reinforcement $\frac{V_u}{\phi} \leq V_n = V_c + V_s$ (AASHTO 5.8.3.3-1)

<p>11. Verify longitudinal reinforcement</p>	<ul style="list-style-type: none"> the amount of longitudinal reinforcement (on flexural tension side) at all locations along the girder are proportioned to satisfy the following: $A_{ps}f_{ps} + A_s f_y \geq \frac{ M_u }{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\left \frac{V_u}{\phi_v} - V_p \right - 0.5V_s \right) \cot \theta$ (AASHTO 5.8.3.5-1) Where A_s = area of non-prestressed tensile reinforcement 	<ul style="list-style-type: none"> the amount of longitudinal reinforcement (on flexural tension side) at all locations along the girder are proportioned to satisfy the following: $A_s f_y \geq \frac{ M_u }{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\left \frac{V_u}{\phi_v} \right - 0.5V_s \right) \cot \theta$ (AASHTO 5.8.3.5-1) Under this design the contribution of prestressing steel is omitted.
<p>12. Anchorage zone design</p>	<ul style="list-style-type: none"> Vertical reinforcement be provided within the distance $h/4$ from the end of the girder. $P_r = f_s A_s$ (AASHTO 5.10.10.1-1) Where f_s = stress in mild steel = 140MPa A_s = total of vertical reinforcement $P_r = 0.04 P_i$ 	<p>Not Applicable for this design,</p>
<p>13. Deflection and cambers</p>	<ul style="list-style-type: none"> Camber due to prestressing force and deflection due to self-weight of girder, slab, and haunch are calculated using the initial modulus of elasticity of concrete and section properties of the noncomposite girder. Deflections due to concrete barrier and future wearing surface are calculated using <i>gross composite section</i> properties. Instantaneous deflection due to prestressing force and girder weight is calculated at transfer. Long-term deflection of precast concrete girders could be computed as the instantaneous deflection multiplied by a factor. Camber due to prestressing force at midspan $\Delta_p = \frac{P_i}{E_{ci}(I)} \left(\frac{e_c L^2}{8} - \frac{e' a^2}{6} \right) \uparrow$ Deflection due to girder self-weight, wearing surface.... at midspan $\Delta_g = \frac{5w_g L^4}{384E_{ci}(I)} \downarrow$ Deflection at midspan at the time of girder erection Girder deflection = $\Delta_p + \Delta_g$ 	<p>Applicable in this design, Instantaneous deflection due to girder self-weight, wearing surface.... at midspan, $y_c = 5w_l^4 / (384 * E * I_e) \downarrow$</p> <p>Long-term deflection= $y_c * (3 - 1.2 (A_s'/A_s))$ Camber= $3y_c$</p>

4. Discussion

4.1 Design aspect

i. Dead load comparison

Comparison with regard to dead load bending is presented as follows

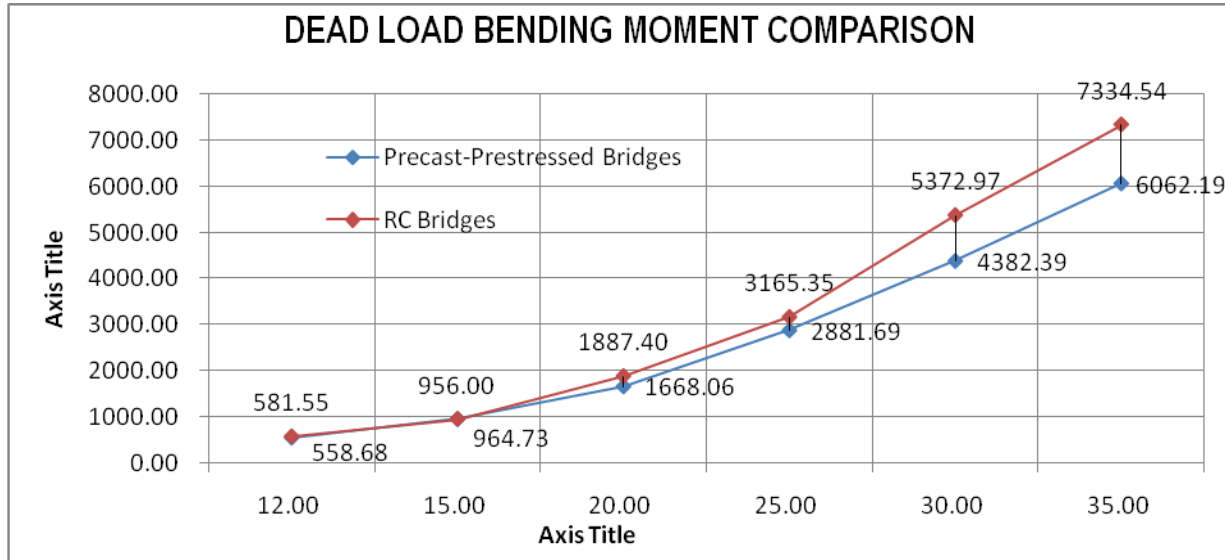


Fig. 15 Dead Load bending moment comparison

From the graph RC bridges have higher dead load bending moment when compared with precast prestressed bridge counterparts. This indicates RC bridges are heavier than precast prestressed bridge having similar span and carriage way width.

ii. Shear force comparison

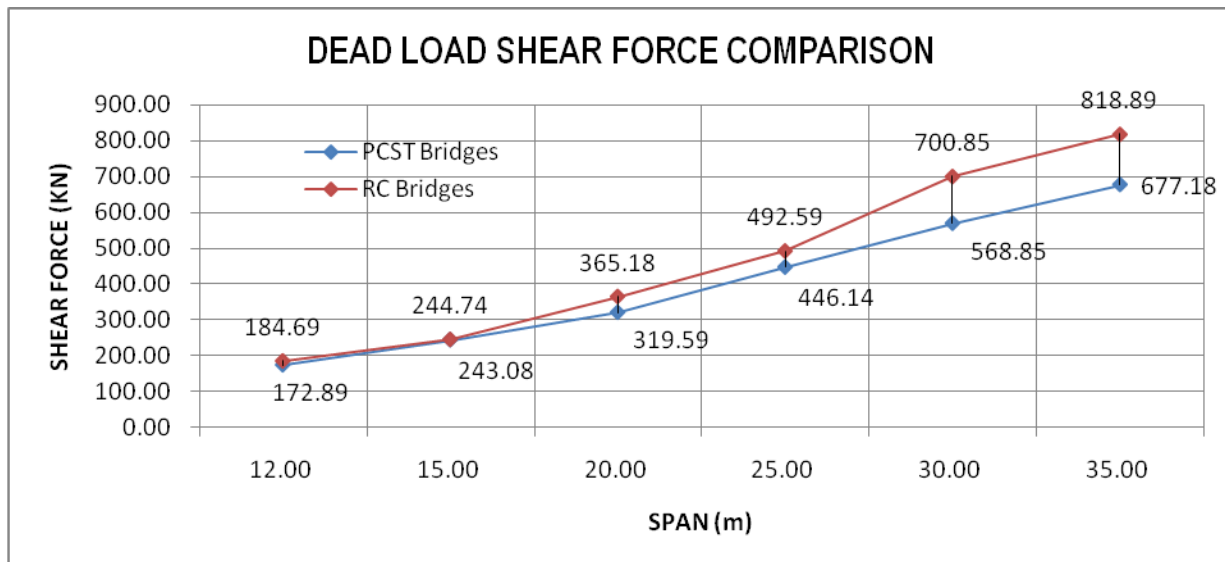


Fig. 16 Dead Load shear force comparison

On fig 16 RC bridges exhibit higher shear force than Precast- prestressed concrete counterparts. Similar to the bending moment comparison diagram this is also as a result of the requirement of bigger concrete cross-sectional area in RC bridges to resist applied shear force.

iii. **Moment capacity and shear resistance**

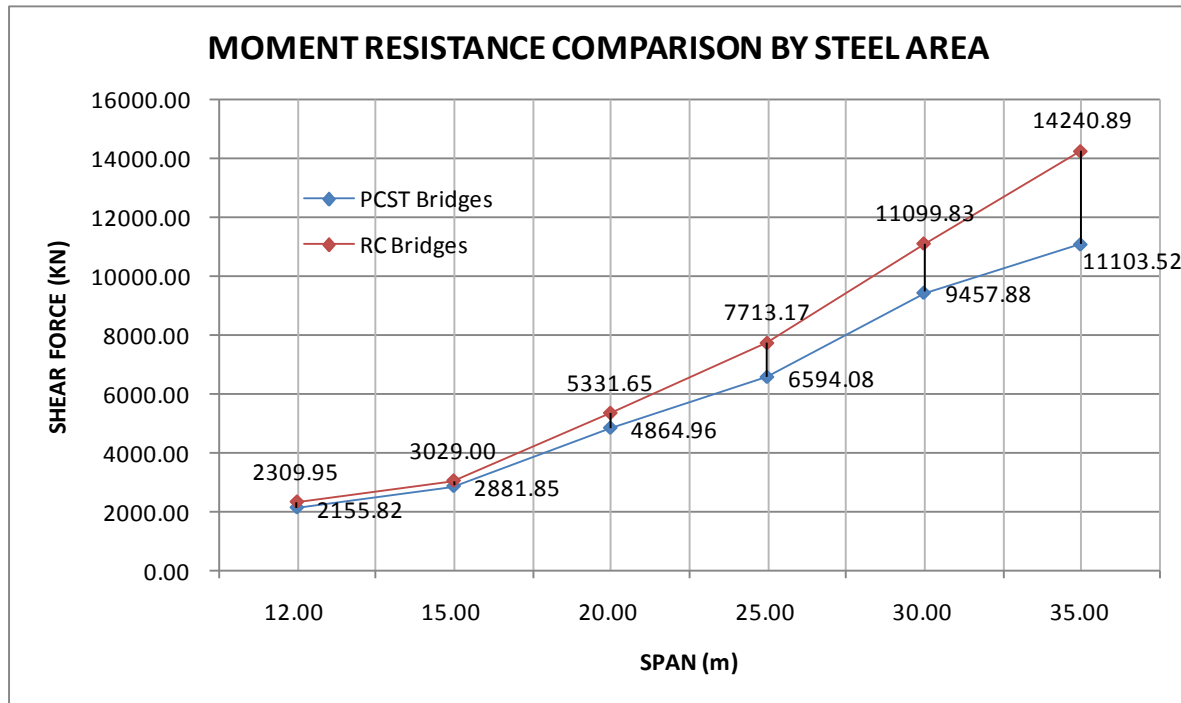


Fig. 17 moment resistance comparison by steel area

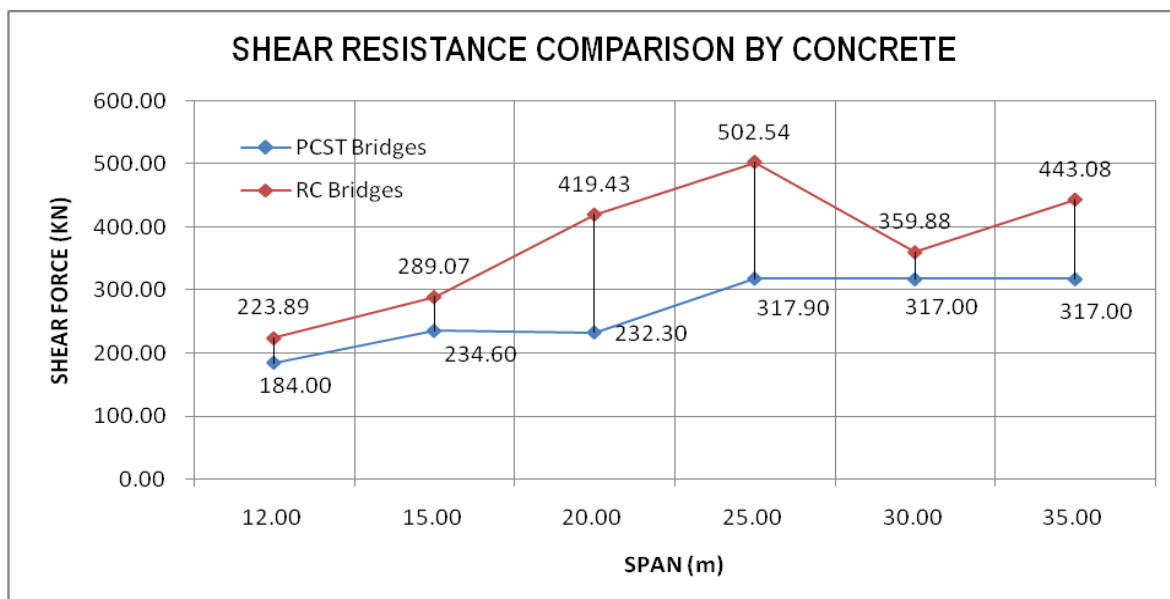


Fig. 18 shear resistance comparison by concrete

Figure 18 illustrates the moment resistance capacity by steel area for various span ranges of RC girder and Precast prestressed girder bridges. From the chart, prestressed members demand lesser steel area to resist the design bending moment rather than reinforced concrete girder bridges of similar span length. Similarly as shown on figure 19, prestressed concrete girder bridges are able to resist applied shear force with less amount of concrete area when compared with RC girder bridges.

4.2 Construction aspect

a. Construction cost

After detailed designs of both types of bridges has been carried out. Subsequently, quantities of construction materials were estimated to compare the construction cost between the two types of bridge construction. for precast prestressed girder bridges, the cost of special equipments related to precast element production, hauling and erection is included.

Table 1: Summary of RC girder Superstructure cost.

			Estimated Quantity				Structure cost
span range (m)	Span Arrangement	Number of Pier	Formwork (m ²)	Rebars (ton)	Concrete (m ³)	Bearing (no)	Birr ((mil))
12	1x12	0	196.26	8.57	42.03	8	0.75
15	1x15	0	277.7	11.16	58.23	8	0.97
20	1x20	0	417.33	14.55	85.73	8	1.28
25	1x25	0	583.88	18.85	242	8	1.97
30	1x30	0	942.36	48.01	214.72	8	3.59
35	1x35	0	1090.74	55.06	251.4	8	4.12
40	1x40	0	1,489.03	59.92	294.37	8	4.68
45	1x20+1x25	1	1001.21	33.3976	327.73	16	4.25
50	2x25	1	1167.76	37.6992	484	16	4.94
60	2x30	1	1884.72	96.02	429.44	16	8.19
80	2x40	1	2978.06	119.84	588.74	16	10.36
100	2x40+1x20	2	3395.39	134.388	674.47	24	12.64
120	3x40	2	4467.09	179.76	883.11	24	16.04
150	3x40+1x30	3	5409.45	227.77	1097.83	32	20.63
180	4x40+1x20	4	6373.45	254.228	1263.21	40	23.99
200	5x40	4	7445.15	299.6	1471.85	40	27.39
240	6x40	5	8934.18	359.52	1766.22	48	33.07
280	7x40	6	10423.21	419.44	2060.59	56	38.75
300	7x40+1x20	7	10840.54	433.988	2146.32	64	41.03

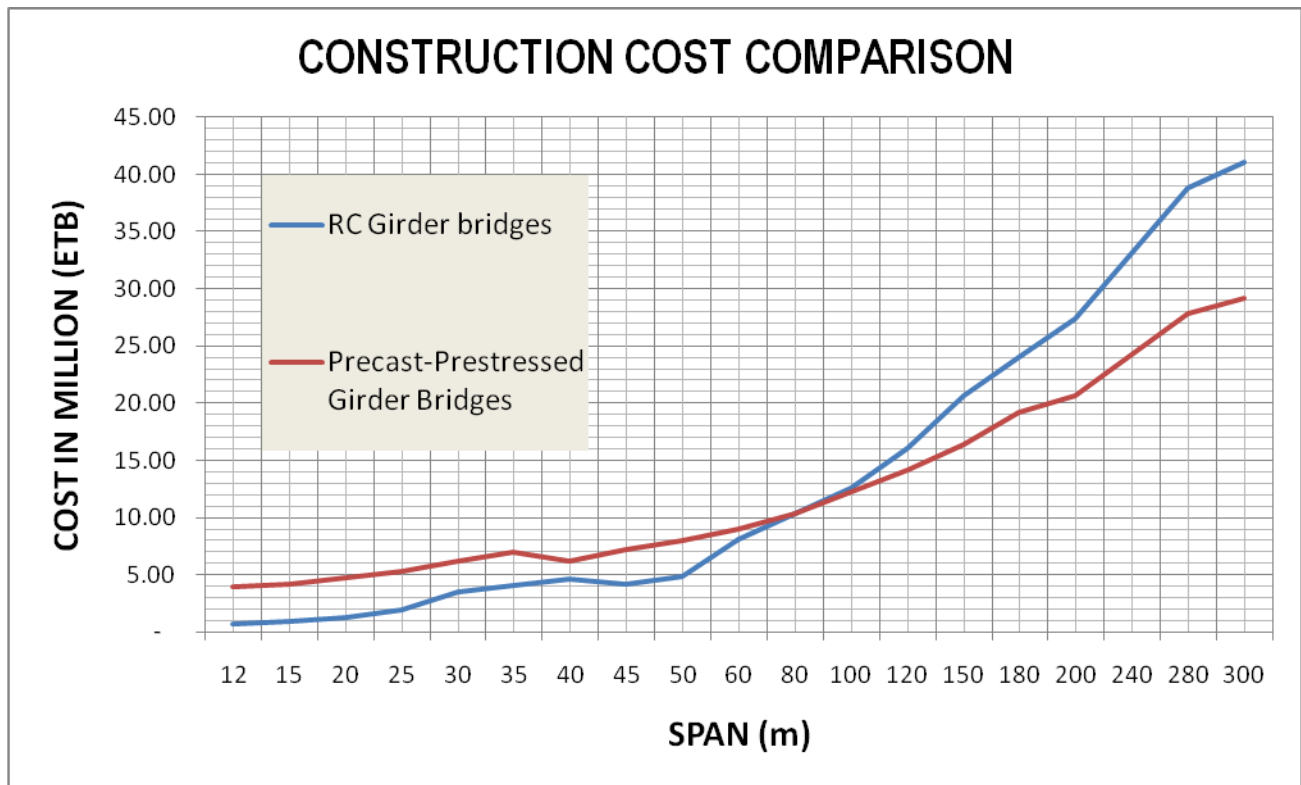
Table 2: Summary of Precast-Prestressed girder Superstructure cost.

span		# Pier	Estimated Quantity					1	2	1+2
m	Span Arrangement		Rebars (Ton)	Tendos (m)	FW (m ²)	Concrete (m ³)	Bearing (No)	Birr (mil)	additional cost (mil)	Total cost (mil)
12	1x12	0	4.48	416.00	32.18	35.752	8.000	0.54	3.45	3.99
15	1x15	0	5.73	640.00	49.60	51.919	8.000	0.71	3.56	4.26
20	1x20	0	7.76	1,512.00	65.10	68.144	8.000	0.96	3.78	4.74
25	1x25	0	10.03	2,080.00	96.72	99.840	8.000	1.27	4.06	5.34
30	1x30	0	14.08	3,720.00	153.45	135.780	8.000	1.81	4.40	6.21
35	1x35	0	17.71	5,184.00	190.80	162.720	8.000	2.25	4.79	7.04
40	1x40	0	20.16	5,740.00	130.20	185.320	8.000	2.46	3.78	6.24
45	1x45	0	22.62	6,440.00	203.05	207.920	8.000	2.80	4.40	7.19
50	1x50	0	25.08	8,568.00	240.40	230.520	8.000	3.21	4.79	8.00
60	2x30	1	28.15	7,440.00	306.90	271.56	16.00	4.61	4.40	9.01
80	2x40	1	40.33	11,480.00	372.00	370.64	16.00	6.02	4.40	10.41
100	2x50	1	50.16	17,136.00	535.05	461.04	16.00	7.48	4.79	12.26
120	2x50+1x20	2	57.92	18,648.00	622.00	529.18	24.00	9.45	4.79	14.24
150	3x50	2	75.24	25,704.00	753.60	691.56	24.00	11.67	4.79	16.46
180	3x50+1x30	3	89.32	29,424.00	893.40	827.34	32.00	14.46	4.79	19.25
200	4x50	3	100.32	34,272.00	1070.10	922.08	32.00	15.95	4.79	20.74
240	4x50+1x40	4	120.49	40,012.00	1298.25	1,107.40	40.00	19.50	4.79	24.29
280	5x50+1x30	5	139.48	46,560.00	1526.40	1,288.38	48.00	23.03	4.79	27.82
300	6x50	5	150.48	51,408.00	1591.50	1,383.12	48.00	24.42	4.79	29.20

Detailed cost analysis is attached on Annex 1.

Hence from the above table, plotting the final costs of each spans on a cost Vs Span graph yields the following:-

Fig.19 cost comparison b/n RC and Precast-Prestressed bridges



From the above graph one can understand that the reduction in construction materials such as cement, stone, sand, steel, formwork and other ingredients brings significant savings. As a result from the comparison graph on figure 21, the use of precast-prestressed concrete bridges becomes significantly economical for spans from 80m (2x40m) and above.

b. Construction time

It is evident that with the application of precast technology, faster and on time delivery can be achieved for any type of bridge construction. Contractors and consultants tend to avoid the method since it requires higher initial project cost and skilled manpower. However, the high initial cost of prefabricated systems is related to the lack of standardization and because these systems are innovative and imply new expensive materials and specialized equipment. However, the introduction of the life-cycle cost attenuates the effect of the initial cost inconvenience.

When using precast prestressed girder elements enables to run parallel work activities at a time. Which means the production and curing of these precast elements can be completed while other site activities are on

progress. In contrast if the construction is using reinforced concrete superstructures, it is difficult to obtain such advantages.

4.3 Social and environmental Aspect

The advantage of using precast prestressed bridge systems instead of cast-in-place concrete bridges is that the girders are constructed off-site under controlled conditions and brought to the site ready for installation. Using precast girders one can significantly reduce construction time, thereby decreasing delays to motorists. It also increases work zone safety by reducing the number of and exposure time of workers operating near moving traffic, reduces environmental impacts by minimizing the site access footprint, and improves the constructability of bridge designs.

In contrary it is difficult to obtain the above mentioned benefits from cast in place RC bridges. Instead intense activity at construction site results in untidiness, dust, noise air pollution, traffic congestion and other inconveniences, all of which present constant nuisance to the people living around the project area as well as aggravations to motorists driving through the area.

5. Conclusion and Recommendations

The objective of this research project was to design and evaluate precast prestressed concrete girder bridges as economical solutions for longer span bridges. In this respect, the research has carried out detailed design of single spans bridges whose span ranges between 12 to 50m.

- Precast prestressed superstructures have less dead weight compared with RC bridge counterparts. This allows utilization of less construction materials. Moreover lightweight and strength are desired combinations to withstand earthquake loads.
- As shown on the analysis result the use of precast prestressed I-concrete girders is economical for a total spans more than 80m, i.e 2x40m span arrangement. The research also concluded construction material savings is very significant if precast-prestressed concrete girders are used for long span bridges.
- For bridge constructions in cities the use of precast prestressed girder bridges offers long span Vs. less dead weight capabilities and more esthetically pleasing layouts with great reduction of traffic congestion during construction. Hence with this added benefits, the span range to be used is unlimited.
- Beside from the economic advantages and high quality product and speed of construction, Environmental and Social concern is now an additional motive for using precast construction method. Therefore from the advantages and mitigating factors that a precast concrete could provide in overcoming economic and environmental issues, the precast-prestressed technology must be well recognized by the government.

6. References

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ANNEX-1

DESIGN OF PRECAST PRESTRESSED CONCRETE GIRDER BRIDGES (12,15,20,25,30&35)

ANNEX-2

DESIGN OF REINFORCED CONCRETE GIRDER BRIDGES (12,15,20,25,30&35)

ANNEX-3

QUANTITY ESTIMATION

Comparison between RC Girder Bridge & Precast/Prestressed Concrete Bridges

1 RC Girder Bridge (superstructure only)

Cost of Pier-(LS) 1,000,000 ETB
 Cost of FW 492.96 ETB
 Cost of Rebar 51931.5 ETB
 Cost of Concrete C30 2489.49 ETB
 Rubber Bearing= 13230 ETB

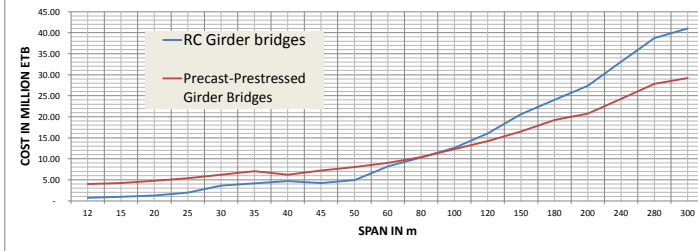
Analysis of results			Estimated Quantity				Structure cost
span range	Span Arrangement	Number of Pier	Formwork	Rebars	Concrete	Bearing	birr
m			m ²	ton	m ³	no	(mil)
12	1x12	0	196.26	8.57	42.03	8	0.75
15	1x15	0	277.7	11.16	58.23	8	0.97
20	1x20	0	417.33	14.55	85.73	8	1.28
25	1x25	0	583.88	19.85	242	8	1.97
30	1x30	0	942.36	48.01	214.72	8	3.59
35	1x35	0	1090.74	55.06	251.4	8	4.12
40	1x40	0	1489.03	59.92	294.37	8	4.68
45	1x20+1x25	1	1001.21	33.3976	327.73	16	4.25
50	2x25	1	1167.76	37.6992	484	16	4.94
60	2x30	1	1884.72	96.02	429.44	16	8.19
80	2x40	1	2978.06	119.84	588.74	16	10.36
100	2x40+1x20	2	3395.39	134.388	674.47	24	12.64
120	3x40	2	4467.09	179.76	883.11	24	16.04
150	3x40+1x30	3	5409.45	227.77	1097.83	32	20.63
180	4x40+1x20	4	6373.45	254.228	1263.21	40	23.99
200	5x40	4	7445.15	289.6	1471.85	40	27.39
240	6x40	5	8934.18	359.52	1766.22	48	33.07
280	7x40	6	10423.21	419.44	2060.59	56	38.75
300	7x40+1x20	7	10840.54	433.988	2146.32	64	41.03

2 Precast/Prestressed concrete bridges (superstructure only)

			Qty	Price (ETB)		
Cost of Pier	1,000,000 ETB		80Ton lifting crane	1	2,400,000.0 (Source CCCC)	
Formwork (mould)	900.00 ETB		prestressing bed	1	800,000.00 (Source CCCC)	
Bearing	13230 ETB		15x15m	1	247,500	
Cost of Rebar	51931.5 ETB		18x18m	1	356,400	
Cost of Concrete c40	3985.79 ETB		Casting bed	23x23m	1	581,900
Cost of prestressing tendon=	78.22 ETB		preparation,	28x28m	1	862,400
				33x33m	1	1,197,900
				38x38m	1	1,688,400

span	m	Span configuration	Number of Pier	Estimated Quantity					1		2		Total cost
				Rebars	Tendos (m)	Formwork	Concrete	Bearing	birr	additional cost			
				Ton	m	m ²	m ³	No	(mil)	(mil)	(mil)		
12	12	1x12	0	4.48	416.00	32.18	35.752	8,000	0.54	3.45	3.99		
15	15	1x15	0	5.73	640.00	49.60	51.919	8,000	0.71	3.56	4.26		
20	20	1x20	0	7.76	1,512.00	65.10	68.144	8,000	0.96	3.78	4.74		
25	25	1x25	0	10.03	2,080.00	96.72	99.840	8,000	1.27	4.06	5.34		
30	30	1x30	0	14.08	3,720.00	153.45	135.780	8,000	1.81	4.40	6.21		
35	35	1x35	0	17.71	5,184.00	190.80	162.720	8,000	2.25	4.79	7.04		
40	40	1x40	0	20.16	5,740.00	130.20	185.320	8,000	2.46	3.78	6.24		
45	45	1x45	0	22.62	6,440.00	203.05	207.600	8,000	2.80	4.40	7.19		
50	50	1x50	0	25.08	8,568.00	240.40	230.620	8,000	3.21	4.79	8.00		
60	60	2x30	1	28.15	7,440.00	306.90	271.56	16,000	4.61	4.40	9.01		
80	80	2x40	1	40.33	11,480.00	372.00	370.64	16,000	6.02	4.40	10.41		
100	100	2x50	1	50.16	17,136.00	535.05	461.04	16,000	7.48	4.79	12.26		
120	120	2x50+1x20	2	57.92	18,648.00	622.00	529.18	24,000	9.45	4.79	14.24		
150	150	3x50	2	75.24	25,704.00	753.60	691.56	24,000	11.67	4.79	16.46		
180	180	3x50+1x30	3	89.32	29,424.00	893.40	827.34	32,000	14.46	4.79	19.25		
200	200	4x50	3	100.32	34,272.00	1070.10	922.08	32,000	15.95	4.79	20.74		
240	240	4x50+1x40	4	120.49	40,012.00	1,288.25	1,107.40	40,000	19.50	4.79	24.29		
280	280	5x50+1x30	5	139.48	46,560.00	1,526.40	1,288.38	48,000	23.03	4.79	27.82		
300	300	6x50	5	150.48	51,408.00	1,591.50	1,383.12	48,000	24.42	4.79	29.20		

CONSTRUCTION COST COMPARISON



Design of || span Precast Prestressed Girder Bridge

1 Design Data and Specification

i. Subject Information

Superstructure type:- Simple Span RC Precast girder

Clear Span, S=	12.5 m		
Clear Rdwy: =	7.3 m		
Sidewalk:=	0.8 m		
Total bridge width:=	8.9 m		
Girder web width, bw=	0.457 m		
Number of girders=	4		
Girder C/C spacing=	2.2 m	<	20hf= 4 m
clear length=	2.5 m		
Cantiliver length=	1 m		
Clear Span(Cantiliver)=	1.25 m		
C/C distance b/n diaphragms=	8.49 m		
total Girder Depth=	0.914 m		

ii. Material Properties:

Concrete:-Grade C-30 concrete (section 9.3)

f'_c =	24 Mpa	(fc' cylinder)
$f_c'0.4 \cdot f'_c$ =	9.6 Mpa	
$E_c=4800\sqrt{f'_c}$ =	23,515.1 MPa	
γ_c =	25 KN/m ³	
wearing surface unit wt= γ_w =	22.5 KN/m ³	

iii. Reinforcement steel:

Grade 60 steel: For rebars dia. 20mm and above

f_y =	420 Mpa
f_s =	165 MPa
E_s =	200,000 MPa

Grade 40 steel: For rebars dia less than 20mm

f_y =	275.78 Mpa
f_s =	137.89 Mpa
E_s =	200,000 MPa

Modular ratio, n= E_s/E_c 8.51 use 9.00

iv. Loading

Design Live Loading:

The design live loading is HL-93, which consists of a combination of

- 1) Design truck or
- 2) Design Tandem

N.B: Lane Loading is uniformly distributed over a width of 3.00m.

Dynamic Load Allowance is not applied for lane Loading

V. Load Modifiers

	Strength	Service	Fatigue
Ductility, h_D	0.95	1.00	1.00
Redundancy, h_R	1.00	1.00	1.00
Importance, h_I	1.05	N/A	N/A
$h = h_D + h_R + h_I$	1.00	1.00	1.00

d. Others

Modular ratio n = E_s/E_c =	8.5	
f_{moment} =	0.90	
f_{shear} =	0.90	
b =	0.85	
Modulus of rupture = $f_r = 0.63\sqrt{f'_c}$ =	3.09 MPa	Eq. 9.5
z in Eq. 9.14 =	30,000 kN/m	

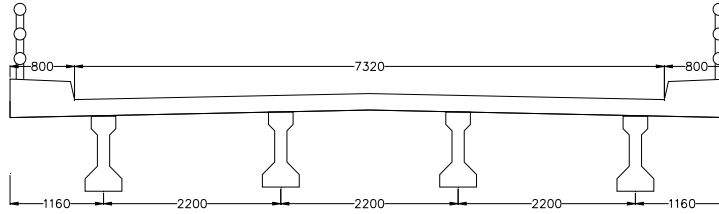
$$r_b = 0.85b^2 f'_c / f_y (599.843 / (599.843 + f_y)) = \mathbf{0.0243}$$

Design Method: Load and Resistance Factor Design (LRFD)

References: ERA's Bridge Design Manual 2002

AASHTO LRFD Bridge Design Specifications-2007

2 Analysis and Design of Precast - Cast insitu composite concrete deck
2.1 Precast section of the deck



a. Depth Requirement

clear span of inner deck= 1.7 m
Clear Span of Cantiliver Deck= 1.25 m

$$\text{Depth Requirement} = D_s = L_e / 20 = 0.085 \text{ m}$$

$$D_c = L_c / 10 = 0.125 \text{ m}$$

$$d = (0.4 + 0.6 \frac{L_c}{400}) \frac{L_c}{f_y} \quad (5.3)$$

where f_y is the characteristic strength of the reinforcement (MPa)
 L_c is the effective span and, for two-way slabs, the shorter span.
 d is the appropriate constant from Table 5.1, and for slabs carrying partition walls likely to crack, shall be taken as $d \le 150L_c$.
 L_e is the distance in meter between points of zero moments; and for a cantilever, twice the length to the face of the support.

(EBCS 2,1995)

Note, d is the effective depth of the precast part of the deck for both precast types

$$D = \frac{2.58}{31.7} = 8.12 \text{ cm} \quad (\text{Type A-precast panel})$$

hence use $t_p = 10 \text{ cm}$ for precast part of the deck

b. Depth requirement for composite Deck

AAShto table 8.9.2

$$D = (s + 3.05) / 30 = 0.2 \text{ m}$$

use 0.20 m

Depth of cast insitu= 10.0 cm

Min depth of cast insitu= 2.5*Max. Agg size+ Bar dia + Top clear cover = 9.6 cm

therefore thickness of deck, $d = 19.6 \text{ cm}$
Provided Depth, $D = 20 \text{ cm}$

c. Loading

Precast section of the deck system shall be designed for the larger of the following loading conditions

*Self weight plus 100% of self weight for dynamic effect

*Self weight plus load of concrete plus 4KN/m² uniform load to account for labourours load

$$WDL = 2 * t_p * \gamma_c = 5 \text{ KN/m}^2$$

Self weight plus load of concrete

$$WDL = D * \gamma_c = 10 \text{ KN/m}^2$$

$$\text{Factored load on the deck, } W_d = 1.25 * G_k + 1.75 * Q_k = 19.5 \text{ KN/m}^2$$

Computation of internal actions

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

$$MDL = W_d * S^2 / 8 = 15.23 \text{ KNm/m} \quad W_d = 19.5 \text{ KN/m}^2$$

$$S = 2.5 \text{ m}$$

Shear due to dead load

$$VDL = W_d * S / 2 = 24.38 \text{ KN/m}$$

d. Vertical Shear

$$V = WL / 2 = 21.45 \text{ KN/m}$$

* check punching shear developed at the hook during transportation

$$V_p = L * t_p * \gamma_c / 4 = 2.417 \text{ KN} \quad \text{Assuming Precast width, } b = 2.035 \text{ m}$$

* Check Section capacity of precast section for the above loading

Shear capacity of precast concrete

$$V_u = \phi V_n$$

Where,

V_u = Factored shear force at the section considered

V_n = Nominal shear strength computed

ϕ = Reduction factor for shear, 0.85

V_c = nominal shear strength provided by concrete

$$V_c = 0.166 * \text{sqrt}(f_c) * b_w * d$$

$$V_c = 52.860 \text{ KN/m} \quad b_w = 1000 \text{ mm}$$

$$d = 65 \text{ mm}$$

$V_c > V_d$, OK!!

e. Provide Reinforcement for Flexure (As1)

Assume a=	13.5 mm	Mu=	15.23 KNm/m
As=M _u /φf _y (d-a/2)=	637 mm ²	φ=	0.9
a=As*f _y /(0.85*f _c *b)=	13.12 mm	b=	1000 mm
		f _y =	420 N/mm ²
Assume a=	13.12 mm	f _c '=	24 N/mm ²
As=M _u /φf _y (d-a/2)=	635 mm ²	D=	100.00 mm
a=As*f _y /(0.85*f _c *b)=	13.1 mm	dia=	10 mm
	ok!	cover=	25 mm
Required As=	635 mm ²	d=	70 mm
required Spacing=	124 mm		

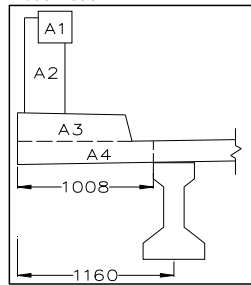
f. Punching Shear

V _p =γ _c *L*b*tp/4=	2.875 KN	γ _c =	25 KN/m ³
		L=	2.3 m
Including 100% dynamic effect,		b=	2 m
V _{pd} = 2*V _p =	5.75 KN	tp=	0.1 m
Let L be horizontal projection of the hook, then the punching area is			
A _p =2*tp*L =	200 L	tp=	0.1 m
V _c =0.166*sqrt(f _c)=	0.813 Mpa	f _c '=	24 N/mm ²
V _c =v _c *A _p =	162.65 L	φ=	0.85
V _u =φV _c	5.75		
	138.25 L		
L=	41.59 mm		

2.2 Design of Composite section of the deck

2.2.1 Design of overhang slab

2.2.1.1 Dead Load



total overhang length =	1 m
depth of railing =	0.3 m
breadth of railing =	0.25 m
distance b/n post and curb end =	0.05 m
distance b/n back of railing and end of curb =	0.15 m
depth of post =	0.25 m
width of post =	0.3 m
c/c of post =	1.5 m
height of post =	0.7 m
Concrete unit weight=	25 KN/m ³

Item	weight [KN/m]	moment arm [m]	Moment [KNm/m]
A1	1.88	0.730	1.369
A2	0.88	0.805	0.704
A3	5.25	0.600	3.150
A4	5.25	0.500	2.63
total	13.25		7.85

2.2.1.2 Live Load

a) Railing load

Railing loads shall be applied on an effective length of $E = 1140 + 0.833X$, [Eq. 7.14]

$E = 1140 + 0.833X$ [mm] where: X- is the distance in mm from the center of the post to the point under investigation
 $= 1.773$ m
 $= 760$ mm

According to Art. 2.7 of the AASHTO 1996, the design load, P is ,
 $P = 44.51$ kN

height of top of rail from top of curb = 0.87 m
 Moment arm = 1.070 m

Therefore the railing live load is as follows:

$M_{RLL} = 26.86$ kNm/m

b) Truck load

According to Art. 7.4 Slabs/Longitudinal Edges, the wheel is put 300mm from face of rail

$$E_{\text{overhang}} = 1140 + 0.833X \quad \text{Where : } X = \text{support [mm]}$$

$$= 1827.2 \text{ mm} \quad = 825 \text{ mm}$$

The multiple presence factor m is 1.20 for one loaded lane. $P = 72.50 \text{ kN}$
 $m = 1.2 \text{ m}$

$$M_{\text{TLL}} = 39.28 \text{ kNm/m}$$

c) Total Moment

i) Dead load plus rail live load

$$M_{\text{TR}} = 1.0 * (1.25M_{\text{DL}} + 1.75M_{\text{RLL}})$$

$$= 56.82 \text{ kNm/m}$$

ii) Dead load plus truck live load

$$M_{\text{TT}} = 1.0 * (1.25M_{\text{DL}} + 1.75M_{\text{LL}})$$

$$= 78.55 \text{ kNm/m}$$

iii) Design moment

$$M_{\text{D}} = \max(M_{\text{TR}}, M_{\text{TT}})$$

$$= \underline{78.55 \text{ kNm/m}}$$

2.2.1.3 Calculate reinforcement

Assume a=	26.87 mm	Mu=	78.55 KNm/m
As=M _u /φfy(d-a/2)=	1353 mm ²	φ=	0.9
a=As*fy/(0.85*fc*b)=	27.86 mm	b=	1000 mm
		fy=	420 N/mm ²
Assume a=	27.86 mm	fc=	24 N/mm ²
As=M _u /φfy(d-a/2)=	1358 mm ²	D=	200.00 mm
a=As*fy/(0.85*fc*b)=	27.95 mm	dia=	16 mm
	ok!	cover=	25 mm
Required As=	1358 mm ²	d=	167 mm
required Spacing=	148 mm		

Use φ 16 C/C 150mm Top Reinforcement

2.2.2 Design of Interior Slab

2.2.2.1 Dead Loads

* RC Slab = 0.20*25 = 5 kN/m²

* Wearing Surface -5cm thick AC pavement
and 3cm damp proof

$$\text{Total} \quad \underline{\underline{7.925 \text{ kN/m}^2}}$$

For slabs monolithic with beams or slabs monolithic with walls without haunches

S = girder spacing = 2.2 m

The dead load moment

$$M_{\text{DL}} = (WS^2 / 8) * 0.80 = (WS^2) / 10 ; \{ 0.80 \text{ is continuity factor} \}$$

$$M_{\text{DL}} = 3.84 \text{ kNm/m}$$

2.2.2.2 Live Load Moments

An approximate analysis of strips perpendicular to girders is considered. The extreme positive moment in any deck panel b/n girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any girder shall apply to all negative moment regions. The strips shall be treated as continuous beams with span lengths equal to the c/c distance b/n girders.

Slabs with more than two girders

Distribution Widths

$$E_{\text{overhang}} = 1140 + 0.833X \quad \text{Where : } X = \text{Distance from load to point of support [mm]}$$

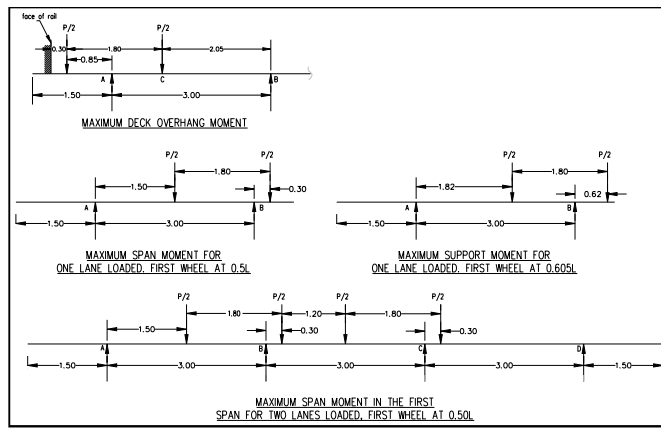
$$= 1435.7 \text{ mm} \quad = 355 \text{ mm}$$

$$E_{\text{positive}} = 660 + 0.55 * S \quad S = \text{Girder Spacing (mm)}$$

$$= 1870.0 \text{ mm} \quad = 2200 \text{ mm}$$

$$E_{\text{negative}} = 1220 + 0.25 * S$$

$$= 1770.0 \text{ mm}$$



$M_{\text{Overhang}} = 42.42 \text{ kNm/m}$ $X = 700 \text{ mm}$
 $M_{\text{positive}} = 30.56 \text{ kNm/m}$ $P = 72.5 \text{ KN}$
 $M_{\text{negative}} = 34.2 \text{ kNm/m}$ $m = 1.2$

 $M_{\text{positive}} = 16.34 \text{ kNm/m}$ (Modified with distribution widths)
 $M_{\text{negative}} = 19.32 \text{ kNm/m}$

Dynamic load Allowance= 0.33

Total Moment

i) Positive Moment at Mid Span, Dead load plus Live (Truck) Load

$$M_u^+ = 1.00 \cdot (1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL+}) \quad 1.50 M_{DW} = 1.25 \cdot (1.20 M_{DW})$$

$$= 42.83 \text{ kNm/m}$$

ii) Negative Moment at Support, Dead load plus Live (Truck) Load

$$M_u^- = 1.00 \cdot (1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL+})$$

$$= 49.77 \text{ kNm/m}$$

2.2.2.3 Determination of Reinforcement for Negative moment

Assume a=	17.1 mm	$M_u =$	49.77 KNm/m
$A_s = M_u / \phi f_y (d - a/2) =$	831 mm ²	$\phi =$	0.9
$a = A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) =$	17.1 mm	b=	1000 mm
		$f_y =$	420 N/mm ²
Assume a=	17.11 mm	$f_c' =$	24 N/mm ²
$A_s = M_u / \phi f_y (d - a/2) =$	831 mm ²	D=	200.00 mm
$a = A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) =$	17.1 mm	dia=	16 mm
	ok!	cover=	25 mm
Required $A_s =$	831 mm ²	dt=	167 mm
required Spacing=	242 mm		

Use $\phi 16$ C/C 240mm

Top Reinforcement

2.2.2.4 Determination of Reinforcement for Positive moment (A_s2)

Assume a=	14.6 mm	$M_u =$	42.83 KNm/m
$A_s = M_u / \phi f_y (d - a/2) =$	710 mm ²	$\phi =$	0.9
$a = A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) =$	14.61 mm	b=	1000 mm
		$f_y =$	420 N/mm ²
Assume a=	14.61 mm	$f_c' =$	24 N/mm ²
$A_s = M_u / \phi f_y (d - a/2) =$	710 mm ²	D=	200.00 mm
$a = A_s \cdot f_y / (0.85 \cdot f_c' \cdot b) =$	14.6 mm	dia=	16 mm
	ok!	cover=	25 mm
Required $A_s =$	710 mm ²	dt=	167 mm
required Spacing=	283 mm		
$A_s1 + A_s2 =$	1345 mm ²		
required Spacing=	150 mm		

Use $\phi 16$ C/C 150mm

As provided = 1,340 mm²

2.2.2.5 Distribution Reinforcements- (Eq. 12.12 ,ERA2002)

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

$$As(\text{distr.}) \% = \frac{3840}{\sqrt{S}} \leq 67\%$$

Therefore, % As distr. = $\frac{81.869}{100} \% (S \text{ is in mm})$ where $S = 2200.00 \text{ mm}$
 % As (distr.) = 67.000%
 As dist. = $0.67 * As \text{ provided} = 898 \text{ mm}^2/\text{m}$
 spacing = $\frac{223.88}{16} \text{ mm}$ diam. of bar = 16 mm

16 mm bars c/c	220 mm (bottom reinf. In longitudinal direction)
As provided =	914 mm²

2.2.2.6 Temperature and shrinkage reinforcements- Art. 5.10.8, AASHTO 1998

Reinforcement for temperature and shrinkage stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete

For components less than 1200mm thick, the area of reinforcement in each direction shall not be less than:

$$As_{(T+Sh)} \geq \frac{0.11Ag}{fy}$$

where $Ag(\text{mm}^2) = 200,000 \text{ mm}^2/\text{m}$
 $fy(\text{Mpa}) = 420 \text{ Mpa}$
 $Dia = 12 \text{ mm}$

Top layer of $As_{(T+Sh)} = 1/2 * As(\text{tot}) = 26.19 \text{ mm}^2/\text{m}$

Maximum Spacing:-

$$S_{max} = \min(3D \text{ or } 450\text{mm})$$

$S_{max} = 450 \text{ mm}$

2.2.2.7 Horizontal shear

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

$$V_u \leq \phi V_{nh} \quad \text{where } V_u = \text{factored shear force}$$

In composite structure only the load that comes after curing of the cast insitu part of the deck produces horizontal shear

$As1 = 1345 \text{ mm}^2$ $fyd = 420 \text{ Mpa}$
 $As2 = 1358 \text{ mm}^2$

the force developed for a meter width

$C1 = As1 * fyd = 564.82 \text{ KN}$
 $T2 = As2 * fyd = 570.20 \text{ KN}$

Total Horizontal Shear = $K(C1 + T2) = 2270.04 \text{ KN}$

Assume the concrete is intentionally roughened

$V_c = 0.7 \text{ MPa}$ $S = 2.20 \text{ m}$
 The length of the interface is, $L_{vh} = S/2 = 1.1$ $b = 1.00 \text{ m}$

Total shear force carried by the concrete

$V_c = 770 \text{ KN}$

Provide reinforcement for horizontal shear

$V_{nh} = \phi(V_c + V_s)$
 $V_s = (V_u / \phi - V_c) = 1900.63 \text{ KN}$

the area of steel for reinforcement for shear

$A_v = V_s / fy = 4525.31 \text{ mm}^2$

take the pitch distance equal to the distribution bars=

Number of distribution bars = 5

Assume ϕ is used, $a_v = 50.27 \text{ mm}^2$ bar dia = 8 mm

Area provided by single row of reinforcement = 251.33 mm²

Spacing, $S = 55.54 \text{ mm}$

Let the spacing be coincident with the spacing of bottom reinforcement

$S = 280 \text{ mm}$

the minimum area of tie reinforcement shall not be less than A_v ,

where, $A_v = 0.345 b_v S / fy = 230.00 \text{ mm}^2$ where, $b_v =$ width of crossection at Contacts surface
 $s = \text{spacing} = \min(4b_w, 610\text{mm})$

Horizontal Spacing

Provide $\phi 8$ Zigzag bar having a pitch distance of 220mm at c/c spacing 300mm

3 Analysis and Design of Precast Concrete Girder

3.1 Effective Flange Width

3.1.1 Interior Girder

The effective flange width for an interior girder is the lesser of the following criteria:

- a) One quarter the effective span:
- b) The greater of the following:
 - Twelve times the effective slab depth plus the web width:
 - twelve times the effective slab depth plus one-half the top flange width:
- c) The spacing between the girders:

$$\text{Eff. Flange width hence=} \min(1/4 * L, (\max(12 * hf + bw, 12hf + 0.5 * b_f)), s)$$

$$b_{eff} = \underline{2.2} \text{ m}$$

3.1.2 Exterior Girder

The effective flange width for an exterior girder is one-half the girder spacing for an interior girder plus the lesser of the following criteria:

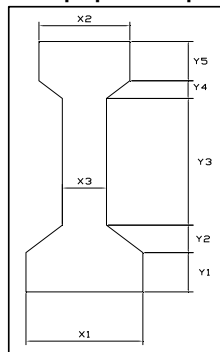
- a) One eighth the effective span:
- b) The greater of the following:
 - Six times the effective slab depth plus one-half the web width:
 - six times the effective slab depth plus one-quarter the top flange width
- c) The overhang dimension:

$$\text{Eff. Flange width hence=} \min(1/8 * L, (\max(6 * hf + 0.5bw, 6hf + 0.25 * b_f)), s) + 0.5s$$

$$b_{eff} = \underline{2.1} \text{ m}$$

Net and transformed section properties will be used for the structural design but gross section properties will be used for live load distribution and deflection calculations.

3.2 Gross Section properties of precast girder



AASHTO GIRDER TYPE II

Input lengths (cm)

X1	45.7
X2	30.5
X3	15.2
	0
Y1	15.2
Y2	15.2
Y3	38.2
Y4	7.6
Y5	15.2

Calculation of I_{xx}

A(cm ²)	y _b (cm)	A*y _b	I _x	A*(y _b -y _b) ²
694.64	7.6	5279.264	13374.13547	738092.639
231.8	20.2666667	4697.813333	2975.281778	92073.7267
231.04	22.8	5267.712	4448.290133	69924.2912
580.64	49.5	28741.68	70607.75947	50253.6528
58.14	73.6666667	4282.98	186.5648	65130.119
115.52	72.4	8363.648	556.0362667	119799.243
463.6	83.8	38849.68	8925.845333	881412.678
	Σ	95482.77733	101073.9132	2016686.35

XX -->
 Strong axis

A	2375.38	cm ²
y _b '	40.20	cm
I _{XX}	2.118E+06	cm ⁴

3.7 DESIGN FOR FLEXURE (AASHTO GIRDER)

Geometrical Properties

Clear span (m)	L=	12	15	20	25	30	35
Girder Effective Span Length (m)	S=	12.5	15.5	20.5	25.5	30.5	35.5
Girder Depth (cm)	h=	114.3	114.3	114.3	137.2	137.2	137.2
Spacing of Girders (m)	S=	2.2	2.2	2.2	2.2	2.2	2.2
Deck Thickness (cm)	T _D =	20	20	20	20	20	20
Haunch Thickness (cm)		0	0	0	0	0	0
Haunch Width (cm)		0	0	0	0	0	0
Recommended type of AASHTO I-girder to be used use section property listed under	Type II	Type III	Type III	Type IV	Type V	Type VI	
	3	4	4	5	1	2	

Cross sectional Properties for a Single Beam

Span (m)	Type	Area (cm ²)	I _{strong} (cm ⁴)	I _{weak} (cm ⁴)	bw (cm)	y _b (cm)
12	3	2375.4	2117760.3	221148.7	15.2	40.2
15	4	3612.4	5678124.7	603173.4	17.8	51.5
20	4	3612.4	5678124.7	603173.4	17.8	51.5
25	5	5087.1	13463521.4	1536679.2	20.3	62.8
30	1	6453.5	23293670.6	2464389.8	20.3	82.0
35	2	6814.9	29624496.0	2476798.6	20.3	90.6

Cross Diaphragms

width	25	cm
height	94.3	cm
Quantity	3	two at ends and one at mid-span

Cross sectional Properties for the Composite Beam

Descript.	Area (cm ²)	y _b (cm)	A.y _b (cm ³)	A(y _{cb} -y _b) ² (cm ⁴)	I _{strong} (cm ⁴)	I _{str} +AΔy ² (cm ⁴)
Beam	2375.38	40.2	95483	6375580	2.12E+06	8.49E+06
Haunch	0.00	0	0	0	0	0.00E+00
Deck	3810.51	124.3	473647	3974381	127017	4.10E+06
Sum	6185.89		5.69E+05			1.26E+07

Section, = y_{cb}

(Span=12)	92.0	cm
(Span=15)	88.9	cm
(Span=20)	88.9	cm
(Span=25)	99	cm
(Span=30)	106.2	cm
(Span=35)	110.9	cm

Prestressing steel (1/2 in. Dia. low relaxation)

	Span 12	Span 15	Span 20	Span 25	Span 30	Span 35	
# of strands	10	12	15	18	23	32	
Area of 1 strand (A _b)	126.68	126.68	126.68	126.68	126.68	126.68	mm ²
Spacing for prestressing strands	10	5	5	5	5	5	cm
Layer 1 - # of strands	4	5	5	10	10	10	
Layer 2 - # of strands	3	5	5	5	10	10	
Layer 3 - # of strands	3	2	3	3	3	10	
Layer 4 - # of strands	0	0	2	0	0	2	
Layer 5 - # of strands	0	0	0	0	0	0	
Layer 6 - # of strands	0	0	0	0	0	0	
Layer 7 - # of strands	0	0	0	0	0	0	
Layer 8 - # of strands	0	0	0	0	0	0	
Layer 9 - # of strands	0	0	0	0	0	0	
c.g of prest. tendons from bottom	19.00	8.75	10.67	8.06	8.48	10.63	cm

Prestressing force		12m	15m	20m	25m	30m	35m	
Ultimate strength	fpu	1861.65	1861.65	1861.65	1861.65	1861.65	1861.65	MPa
Yield strength	fy	1675.485	1675.485	1675.485	1675.485	1675.485	1675.485	MPa (5.4.4.1-1)
Initially (=0.75 fpu)	fpi	1396.2	1396.2	1396.2	1396.2	1396.2	1396.2	MPa (5.9.3-1)
Initial loss		0.7	0.8	0.8	0.7	0.7	0.9	%
Initial loss		9.8	10.5	10.5	9.1	9.8	12.8	MPa
At Transfer after initial losses		1386.5	1385.8	1385.8	1387.2	1386.5	1383.4	MPa
Total Prestressing Force		1756.3	2106.5	2633.2	3163.0	4039.6	5607.8	kN

Reinforcing Bars

Yield strength	fy	420	MPa
----------------	----	-----	-----

STRESSES AT TRANSFER

	12m	15m	20m	25m	30m	35m	
Moment due to prestressing (Mp) at c/g of beam	372.29	899.89	1074.40	1732.87	2969.34	4484.13	kN-m
of beam	152.78	276.11	478.68	1034.62	1864.99	2661.50	kN-m

Stress check at transfer - midspan

Bottom Fiber - Compression = $-P/A - Mp/S_b + Mb/S_b$

					remark
12m	-11.560 MPa	<	-18.000 MPa		OK
15m	-11.486 MPa	<	-18.000 MPa		OK
20m	-12.689 MPa	<	-18.000 MPa		OK
25m	-9.477 MPa	<	-18.000 MPa		OK
30m	-10.146 MPa	<	-18.000 MPa		OK
35m	-13.802 MPa	<	-18.000 MPa		OK

Top Fiber - Tension Check = $-P/A + Mp/S_t - Mb/S_t$

12m	0.287 MPa	<	1.369 MPa		OK
15m	1.071 MPa	<	1.369 MPa		OK
20m	-0.697 MPa	<	1.369 MPa		OK
25m	-2.361 MPa	<	1.369 MPa		OK
30m	-3.642 MPa	<	1.369 MPa		OK
35m	-5.361 MPa	<	1.369 MPa		OK

Check Total Loss due to Initial Prestressing

Total prestress loss: $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ [LRFD Eq. 5.9.5.1-1]

where

Δf_{pES} = loss of prestress due to elastic shortening

Δf_{pSR} = loss of prestress due to concrete shrinkage

Δf_{pCR} = loss of prestress due to creep of concrete

Δf_{pR2} = loss of prestress due to relaxation of steel after transfer

$$\Delta f_{pES} = n * f_{cgp} \quad \text{[LRFD Art. 5.9.5.2.3a]}$$

Where f_{cgp} = sum of concrete stresses at the center of gravity of the prestressing steel due to prestressing force at transfer and self weight of the member at sections of maximum moment

$$n = E_s/E_c = 6.69$$

$$\text{Elastic Shortening Stress } (f_{cgp}) = (-P/A) - (Mp^*e)/I + (Mb^*e) - 9.591 \quad \text{MPa}$$

Shrinkage

$$\Delta f_{pSR} = 17 - 0.15 H$$

[LRFD Eq. 5.9.5.4.2-1]

where H is relative humidity = 60%

$$\Delta f_{pSR} = [17 - 0.15(60)] = \underline{\underline{16.91}} \text{ Mpa}$$

Creep of Concrete

$$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp}$$

[LRFD Eq. 5.9.5.4.3-1]

$$\Delta f_{cdp} = M_s.ec/l + M_{sDL}(Y_{bc}-Y_{bs})/I_c$$

Ms= 362.0 KNm

e= 71.59 cm

l= 2.96E+07 cm4

Ic= 1.26E+07 cm4

M_{sDL}= 43.95 KNm

Y_{bc}= 92.0 cm

Y_{bs}= 19.00 cm

$$\Delta f_{cdp} = \underline{\underline{11.29}} \text{ Mpa}$$

where Δf_{cdp} = change of stresses at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied calculated at the same section as f_{cgp}

MS = slab moment

M_{sDL} = superimposed dead load moment

l = moment of inertia of the non-composite section

Ic = moment of inertia of composite section

Summary

Span (m)	12	15	20	25	30	35
Elastic Shortening Stress (fcgp)	-9.59	-10.52	-11.57	-9.06	-9.74	-13.15
Shrinkage	16.91	16.91	16.91	16.91	16.91	16.91
Creep of Concrete	11.29	21.75	37.15	45.58	62.92	80.86

Relaxation of Prestressing Strands

Relaxation before Transfer

Initial loss due to relaxation of prestressing steel is accounted for in the beam fabrication process.

Therefore, loss due to relaxation of the prestressing steel prior to transfer is not computed, i.e. $\Delta f_{pR1} = 0$.

Recognizing this for pretensioned members, LRFD Article 5.9.5.1

allows the portion of the relaxation loss that occurs prior to transfer to be neglected in computing the final loss.

Relaxation after Transfer

$$\Delta f_{pR2} = 30\% [20.0 - 0.4 \Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$

$$= 3.16$$

The instantaneous loss of prestress is estimated using the following expression:

$$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1}$$

$$= -9.59 \text{ Mpa}$$

Span (m)	Instantaneous Loss (Mpa)	Initial loss (Mpa)	Remark
12	9.59	9.8	acceptable
15	10.52	10.5	acceptable
20	11.57	10.5	acceptable
25	9.06	9.1	acceptable
30	9.74	9.8	acceptable
35	13.15	12.8	acceptable

Check Stresses at Transfer Length Section

$$l_d \geq \kappa (0.15 f_{ps} - 0.097 f_{pe}) d_b$$

Where d_b = nominal strand diameter (mm)

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (Mpa)

f_{pe} = effective stress in prestressing steel after loss (Mpa)

κ = 1.6 for pretensioned members with a depth greater than 600mm

$$L_d = 62.51 \times \text{dia.}$$

Transfer Length, L_d = 793.94 mm LRFD Art. 5.8.2.3

Span	12m	15m	20m	25m	30m	35m	
Debonded strands	6	7	9	8	10	10	
Mbeam @ end of	27.07	49.75	66.62	117.59	179.33	221.21	kN-m
P =	702.53	877.72	1053.27	1757.21	2283.23	3855.36	kN
Mp at c.g of beam =	148.91	374.96	429.76	962.71	1678.33	3082.84	kN-m

Bottom Fiber Stresses = $= -P/A - M_p/S_b + M_b/S_b$

12m	-5.270	MPa	<	-18	MPa	OK
15m	-5.378	MPa	<	-18	MPa	OK
20m	-6.207	MPa	<	-18	MPa	OK
25m	-7.399	MPa	<	-18	MPa	OK
30m	-8.814	MPa	<	-18	MPa	OK
35m	-14.408	MPa	<	-18	MPa	OK

Top fiber Stresses = $= -P/A + M_p/S_t - M_b/S_t$

12m	1.306	MPa	<	1.369	MPa	OK
15m	1.169	MPa	<	1.369	MPa	OK
20m	1.103	MPa	<	1.369	MPa	OK
25m	1.213	MPa	<	1.369	MPa	OK
30m	0.015	MPa	<	1.369	MPa	OK
35m	-1.155	MPa	<	1.369	MPa	OK

STRESSES AT SERVICE LOADS

Total Losses at Service Loads= $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} + \Delta f_{pR2}$
 = 34.6 Mpa

Span (m)	12	15	20	25	30	35	
Δf_{pES} =	9.591	10.52	11.57	9.06	9.74	13.15	MPa
Δf_{pSR} =	16.91	16.91	16.91	16.91	16.91	16.91	MPa
Δf_{pCR} =	11.29	21.75	37.15	45.58	62.92	80.86	MPa
Δf_{pR1} =	0	0.00	0.00	0.00	0.00	0.00	MPa
Δf_{pR2} =	-3.16	-2.42	-1.37	-1.16	-0.04	1.44	MPa
Δf_{pT} =	34.638	46.77	64.26	70.38	89.53	112.37	MPa
fpi =	1396.24	1396.24	1396.24	1396.24	1396.24	1396.24	MPa
Service =	34.64	46.77	64.26	70.38	89.53	112.37	MPa
Total Prestress loss (%) =	2.5	3.35	4.60	5.04	6.41	8.05	
Total Prestress Stress after losses =	1430.88	1443.01	1460.50	1466.62	1485.77	1508.61	MPa
Total Prestress Force =	1812.6	2193.55	2775.17	3344.16	4328.89	6115.37	kN
Mp =	384.2	937.07	1132.34	1832.14	3182.03	4890.00	kN-m
Mdc =	332.6	600.17	1045.55	1911.72	3119.78	4361.42	kN-m
Mdw =	162.6	249.97	437.26	676.57	967.90	1311.26	kN-m
Mll =	314.3	402.41	517.95	657.45	754.74	843.96	kN-m
Mll+im =	418.0	535.21	688.88	874.41	1003.80	1122.47	kN-m

Compute stresses using
 non-composite
 non-composite
 non-composite
 composite
 from Load analysis
 composite

Stresses at Mid-Span

Service I = 1.00(DC+DW) + 1.00 (LL+IM)

Beam Top Fiber Stresses =

Check compressive Stresses in prestressed comp.

Span (m)	P/A (MPa)	Mp (MPa)	Mdc (MPa)	Mdw (MPa)	(Mll+im) (MPa)	Total (MPa)	Remark
12	-7.63	13.44	-11.64	-0.29		-6.11	< -18.00 OK
	-7.63	13.44	-11.64	-0.29	-0.74	-6.85	< -18.00 OK
15	-6.07	10.37	-6.64	-0.41		-2.75	< -18.00 OK
	-6.07	10.37	-6.64	-0.41	-0.87	-3.62	< -18.00 OK
20	-7.68	12.53	-11.57	-0.71		-7.43	< -18.00 OK
	-7.68	12.53	-11.57	-0.71	-1.12	-8.55	< -18.00 OK
25	-6.57	10.12	-10.56	-0.89		-7.90	< -18.00 OK
	-6.57	10.12	-10.56	-0.89	-1.15	-9.05	< -18.00 OK
30	-6.71	7.54	-7.40	-0.89		-7.45	< -18.00 OK
	-6.71	7.54	-7.40	-0.89	-0.93	-8.38	< -18.00 OK
35	-8.97	7.69	-6.86	-0.92		-9.06	< -18.00 OK
	-8.97	7.69	-6.86	-0.92	-0.79	-9.85	< -18.00 OK

Top of Deck Fiber Stresses

Span (m)	P/A (MPa)	Mp (MPa)	Mdc (MPa)	Mdw (MPa)	(MII+im) (MPa)	Total (MPa)	Remark
12				-0.47	-1.22	-1.69	< -13.5 OK
15				-0.63	-1.35	-1.98	< -13.5 OK
20				-1.10	-1.73	-2.83	< -13.5 OK
25				-1.17	-1.52	-2.69	< -13.5 OK
30				-1.27	-1.32	-2.59	< -13.5 OK
35				-1.40	-1.20	-2.60	< -13.5 OK

Service III : 1.00(DC+DW) + 0.80 (LL+IM)

Beam Bottom Fiber Stresses =

Check tensile stresses in prestressed concrete comp.

Span (m)	P/A (MPa)	Mp (MPa)	Mdc (MPa)	Mdw (MPa)	(MII+im) (MPa)	Total (MPa)	Remark
12	-7.63	-7.29	6.31	1.19		-7.42	< 3.16 OK
	-7.63	-7.29	6.31	1.19	2.44	-4.98	< 3.16 OK
15	-6.07	-8.49	5.44	1.42		-7.71	< 3.16 OK
	-6.07	-8.49	5.44	1.42	2.43	-5.27	< 3.16 OK
20	-7.68	-10.26	9.48	2.48		-5.99	< 3.16 OK
	-7.68	-10.26	9.48	2.48	3.13	-2.85	< 3.16 OK
25	-6.57	-8.55	8.92	2.30		-3.90	< 3.16 OK
	-6.57	-8.55	8.92	2.30	2.38	-1.52	< 3.16 OK
30	-6.71	-11.20	10.98	3.06		-3.87	< 3.16 OK
	-6.71	-11.20	10.98	3.06	2.54	-1.33	< 3.16 OK
35	-8.97	-14.95	13.34	3.87		-6.72	< 3.16 OK
	-8.97	-14.95	13.34	3.87	2.65	-4.07	< 3.16 OK

STRENGTH LIMIT STATE

$$\mu_{u1} = 1.25(DC)+1.5(DW)+1.75(LL+IM)$$

$$\mu_{u2} = 0.9(DC)+0.65(DW)+1.75(LL+IM)$$

Span	12m	15m	20m	25m	30m	35m	
$\mu_{u1} =$	1391.18	2061.79	3168.36	4934.72	7108.23	9382.98	kN-m
$\mu_{u2} =$	1136.57	1639.25	2430.75	3690.54	5193.60	6741.91	kN-m
dp =	115.30	125.55	123.63	149.14	148.72	146.58	cm

β	k
0.75	0.38

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right) \quad (5.7.3.1.1-1)$$

$c = 5.50 \text{ cm} < 20 \text{ cm}$

Rectangular

in which:

Top flange thickness of the PC beam = 15.2 cm

$$k = 2 \left(1.04 - \frac{f_{ps}}{f_{pu}} \right) \quad (5.7.3.1.1-2)$$

$c =$ apply rectangular NA = 35.2 cm

for T-section behavior:

Average stress in prestressing tendons

$f_{ps} = 1827.884 \text{ MPa}$

$M_n = 2615.622 \text{ kN-m}$ for rectangular

$M_n = \text{#####} \text{ kN-m}$ for T-section

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f'_y - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_b + k A_{ps} \frac{f_{ps}}{d_p}} \quad (5.7.3.1.1-3)$$

for rectangular section behavior:

$$M_r = \phi M_n$$

$$\phi = 1 \quad \text{LRFD 5.5.4.2.1}$$

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f'_y}{0.85 f'_c \beta_b + k A_{ps} \frac{f_{ps}}{d_p}} \quad (5.7.3.1.1-4)$$

Span (m)	Mr (kN-m)	Mu (kN-m)	Remark
12	2615.62	1391.2	OK
15	3404.40	2061.8	OK
20	4143.15	3168.4	OK
25	5999.37	4934.7	OK
30	7530.973	7108.2	OK
35	10044.53	9383.0	OK

SHEAR DESIGN

$V_p = 0 \text{ kN}$ no draped tendons exist

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

$$V_u > 0.5\phi(V_c + V_p) \quad (5.8.2.4-1) \quad V_u = V_c + V_s + V_p \quad (5.8.3.3-1)$$

$$V_c = 0.083\beta\sqrt{f'_c} b_w d \quad (5.8.3.3-3)$$

Critical shear section approx. = $d_e = h - y_{bs} = 115.30 \text{ cm}$

Span	12	15	20	25	30	35
V _{dc} =	91.1	134.0	183.4	269.9	374.2	456.1
V _{dw} =	42.4	54.1	75.0	93.7	114.6	135.6
V _{ll} =	237.70	259.43	288.49	313.16	335.61	356.77
V _{ll+im} =	301.4	265.3	295.0	320.2	343.2	364.8

V _u =	694.3	699.3	839.1	1014.9	1211.6	1378.1
V _c =	184.0	234.6	232.3	317.9	317.0	310.1
0.5 ϕ (V _c +V _p) =	82.8	105.6	104.5	143.0	142.6	139.53
V _u >0.5 ϕ (V _c +V _p)	Prov. Str	Prov. Str	Prov. Str	Prov. Str	Prov. Str	Prov. Str
V _u / ϕ =	771.4	777.0	932.4	1127.6	1346.2	1531.21
Req'd V _s = V _u / ϕ -V _c =	587.4	542.4	700.1	809.8	1029.2	1221.15

$$\theta = 45 \text{ deg} \quad V_s = \frac{A_s f_y d_v \cot \theta}{s}$$

Span =	12	15	20	25	30	35
Av/s =	1.213	1.029	1.341	1.293	1.648	1.999
Say S =	120	130	110	110	130	110
Required Av =	145.6	133.7	147.512	142.197	214.204	219.839
Bar Dia =	10	10	10	10	12	12
2 Bars, Av =	157	157	157	157	226.08	226.08
	OK	OK	OK	OK	OK	OK
Use stirrups	Ø10@120	Ø10@130	Ø10@110	Ø10@110	Ø12@130	Ø12@110

Check for maximum spacing of transverse reinforcement

$$0.125 f_c' = 5 \text{ Mpa}$$

$$\text{Shear stress on concrete, } V_u = \frac{|V_u - \phi V_p|}{\phi b_v d_v} \text{ (Mpa)}$$

• If $V_u < 0.125 f_c'$ then:

$$s_{max} = 0.8d_v \leq 600 \text{ mm} \quad (5.8.2.7-1)$$

Where $V_u = 694.3 \text{ kN}$
 $V_p = 0$
 $b_v = 15.2 \text{ cm}$
 $d_e \text{ or } d_v = 115.30 \text{ cm}$
 $\phi = 0.9$
 $V_u = 4.40 \text{ Mpa}$

• If $V_u \geq 0.125 f_c'$ then:

$$s_{max} = 0.4d_v \leq 300 \text{ mm} \quad (5.8.2.7-2)$$

therefore, $S_{max} = 0.8d_e \leq 600 \text{ mm}$

$0.8d_e = 92.24 \text{ cm} > S \text{ provided Ok!}$

Design Anchorage Zone Reinforcement

The provision in AASHTO Art. 5.10.10 requires that the following vertical reinforcement be provided within the distance $h/4$ from the end of the girder.

$$Pr = fsAs \quad 5.10.10.1-1$$

$fs = \text{stress in mild steel} = 140 \text{ Mpa}$

$As = \text{total of vertical reinforcement}$

$Pr = 0.04 Pi$

$Pr = 70.75 \text{ KN}$

Requires $As = Pr/fs = 505.35 \text{ mm}^2$

using dia 12 with 2 vertical legs, number of bars required = 2.23

Provide 3-Ø12mm vertical bars (2 legs) within $h/4$ (3m) from the end of the beam.

Calculate Deflection and Camber

(a) Camber due to prestressing force at midspan (initial camber)

$$\Delta_p = \frac{P_i}{E_c(I)} \left(\frac{e_c L^2}{8} - \frac{e' a^2}{6} \right)$$

$\Delta_p = 1.98 \text{ mm} \uparrow$

$P_i = 1768.7 \text{ Kpa}$

$E_c I = 30406 \text{ Mpa}$

$I = 1.32E+07 \text{ cm}^4$

$e_c = 21.20 \text{ cm}$

$e_e = 21.20 \text{ cm}$

$L_g = 13 \text{ m}$

$e' = 0 \text{ m}$

$a = 0 \text{ m}$

$w_g = \text{uniform distributed load} = 26.7 \text{ KN/m}$

(b) Deflection due to UDL at midspan

$$\Delta_w = \frac{5w_g L^4}{384 E_c(I)}$$

$\Delta_w = 2.48 \text{ mm} \downarrow$

final camber = 1.51 mm \uparrow

summary

Span (m)	12	15	20	25	30	35
Deflection (mm) \downarrow	2.48	6.30	12.96	17.21	36.30	66.26
camber due to prestressing (mm) \uparrow	1.98	5.67	16.83	23.99	44.87	74.75
Final camber (mm) \uparrow	1.51	1.89	11.61	20.34	25.70	25.47

Case || Box Girder SUPERSTRUCTURE DESIGN BY LRFD METHOD

1 GENERAL REFERENCE

Specification used : ERA Bridge Design Manual 2002

a. Data:

Clear Span of Bridge, m=	30.0 m
c/c of support, L =	30.5 m
Clear bridge width, B =	7.32 m
No of lane =	2.00
Face of railing/curb to end of slab, X =	0.80 m
Total top width =	8.92 m
Skew angle, θ =	0°
	0.000 rad

b. Material Properties

Type of concrete:	30Mpa concrete cube or 24Mpa cylinder	
f_c' =	24.0 Mpa	
$f_c=0.4f_c'$ =	9.6 Mpa	
γ_c =	25.0 KN/m ³	
$E_c = 0.043 \cdot \gamma_c \cdot 1.5 \cdot \sqrt{f_c'}$ =	26332 Mpa	Eq 9.3

Type of steel :

For Reinforcement use Deformed bars of G60 Steel

f_{yk} =	414.0 MPa
f_s =	165.6 Mpa
E_s =	217185 Mpa

Wearing Surface and Damp proof

γ =	22.5 kN/m ³
------------	------------------------

c. Resistance Factors

i. Strength Limit State	Φ
Flexure and Tension =	0.90
Shear and Torsion =	0.90
Axial compression =	0.75
Bearing =	0.70

ii. Non strength limit states =	1.00
---------------------------------	------

d. Load Modifiers

	Strength	Service	Fatigue
Ductility, η_D	0.95	1.00	1.00
Redundancy, η_R	1.00	1.00	1.00
Importance, η_I	1.05	N/A	N/A
$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.00	1.00	1.00

e. Limit State Load Factors

	Maximum	Minimum		
Dead load Strength I factor, D_c =	1.25	0.9	Live load Strength I factor =	1.75
Dead load Strength I factor, D_w =	1.5	0.65	Live load Service I factor =	1.00
Dead load Service I factor =	1.00		Fatigue Live load factor =	0.75

f. Loading

Rear Axle load = P =	145.00 kN
=	72.50 kN
Front axle, truck =	35.00 kN
Transverse axle spacing, truck=	1.8 m
Longitudinal axle spacing, truck=	4.3 m
Lane load in the transverse direction =	9.30 kN/3m
=	3.10 kN/m
Design tandem axle load =	110.00 kN
Axle spacing, tandem=	1.2 m
Highway railing design loading, P_H =	44.48 kN

g. Applicable Load Combinations

Strength I Limit State

$$U = \eta * [1.25DC + 1.50DW + 1.75(LL+IM) + 1.00(WA+FR) + \dots]$$

Service I Limit State

$$U = 1.00 (DC + DW) + 1.00(LL+IM) + 1.00WA+0.30(WS + WL) + \dots$$

Fatigue Limit State

$$U = 0.75(LL+IM)$$

h. Multiple presence factor, m

No of loaded lanes	m
1	1.2
2	1

i. Dynamic load allowance

* Not applied to the design lane load.

Component	IM	1+IM
Deck joints	0.75	1.75
Fatigue	0.15	1.15
All other	0.33	1.33

j. Others

modular ratio $n=Es/Ec=$	8	
$\beta =$	0.85	
Modulus of rupture $=f_r = 0.63\sqrt{f_c'} =$	3.09 Mpa	Eq. 9.5
$\rho_b = 0.85b_1f_c'/f_y*(599.843/(599.843+f_y)) =$	0.02478	
Z in Eq. 9.14 =	30,000.00 kN/m	
Concrete cover for unprotected main reinforcing steel (Cover to ties and stirrups 12mm less)		
Deck surfaces subject to tire stud/ chain wear =	60 mm	
Exterior other than above =	50 mm	
Bottom of CIP slabs =	25 mm	

2 DETERMINATION OF SECTION FOR SUPERSTRUCTURE

a. Structural Depth, d

According to Table 2.5.2.6.3-1, $D = 0.06 \cdot S$

	30m	35m
Simple Span box Girder:	$0.060L = 1.8$	2.1

Girder depth provided = $D_{girder} = \underline{1.80} \quad \underline{2.10}$ m

b. Girder Spacing, S:

The Spacing of girders is generally taken no more than one and half their depth

$$a_{MAX} < 1.5D = 2.7$$

By using an overhang of 2.1 m, the c/c distance b/n two exterior girders is 4.72 m

- (i) Try 2 girders and 1 bay, a= 4.72 **NG**
 (ii) Try 3 girders and 2 bays, a= 2.36 **OK**
 (iii) Try 4 girders and 3 bays, a= 1.57 **OK**

Use **iii** i.e. $a = 1.573$ m
 Bottom flange width = 1.573
 Overhang length, C = 1.160
 USE Bottom flange width = 2.200 m
 Use Overhang length, C = 1.160 m

c. Top Flange

Thickness of top flange serving as deck slabs shall be the greater of

- Minimum depth of concrete deck = 175mm As determined in Section 9 [A9.7.1.1.]
- $h_{tf} \geq 1/20$ (Clear span fillets) = 63.67 mm
- Effective len_{empirical design} = 1273.33 mm
- $h_{tf} \geq 1/18$ (Effective len) = 70.74

Traditional minimum depths of slabs are based on the deck span

length S to control deflection to give = 173.33 mm [Table A9.7.1.1]; i.e. $(S + 3)/30 =$
 Top slab thickness = $h_{tf} = \underline{165.00}$ mm
 Use $h_{tf} = \underline{0.20}$ m

d. Bottom Flange

The bottom flange thickness shall be not less than:

- 140mm As determined in Section 9 [A9.7.1.1.]
- $h_{bf} \geq 1/30$ (Clear span b/n webs) = 42.44 mm

From practical point of view for placing two layers of large reinforcement = $2 \cdot 0.07 + 0.06 = 0.20$ m

Bottom slab thickness = $h_{bf} = \underline{170.00}$ mm

Use $h_{bf} = \underline{0.20}$ m

e. Webs

For adequate field placement and consolidation of concrete

- Minimum of 200mm without prestress ducts
- Minimum of 300mm with only longitudinal and vertical prestress ducts and 380mm for both ducts.

Use $b_w = \underline{0.30}$ m

- For girders with over 2400mm of depth, the web dimensions should be increased to compensate for increased difficulty of concrete placement.

f. Reinforcement limits

i. Deck reinforcement:

At least 1/3 of transverse reinforcement shall be extended into the slab overhang and shall have an anchorage beyond the exterior face of the web not less in resistance than that provided by a standard hook.

ii. Minimum reinforcement:

The lesser of $\Phi M_n > 1.2M_{cr}$ or $\Phi M_n > 1.33$ (the factored moment required for the strength I limit state)
 $\rho_{min} \geq 0.03f_c'/f_y = 0.001739$

iii. Crack control

$$f_s \leq f_{sa} = Z/(d_c A)^{1/3} \leq 0.6f_y = 248.40 \text{ MPa}$$

iv. Flanges in tension

Tension reinforcement shall be distributed over the lesser of the effective flange width or a width equal to 1/10 of the average of adjacent spans between bearings.

Dist. Width = 2110 mm

v. Longitudinal skin reinforcement

Required if web depth > 900mm. It should be uniformly distributed on both side faces for a distance d/2 nearest the flexural tension reinfor in mm²/mm of height on each side face shall not be less than:

$$A_{sk} \geq 0.001(de-760) \leq (A_s/1200)$$

The maximum spacing shall not exceed d/6 or 300mm.

vi. Effective Flange Width

Effective span length of simple span = Actual span = 30.50 m

Interior Girder

For interior girder, the effective flange width may be taken as the least of:

- One-quarter of the effective span length 7.63 m
- 12.0 times the average thickness of the slab, plus the greater of half the web thickness or one quarter of the width of the top flange of the girder, 2.70 m
- The average spacing of the adjacent beams 2.20 m

$$b_{eff} = \underline{\underline{2.20 \text{ m}}}$$

Exterior Girder

The effective flange width may be taken as one-half the effective width of the adjacent interior beam plus the least of:

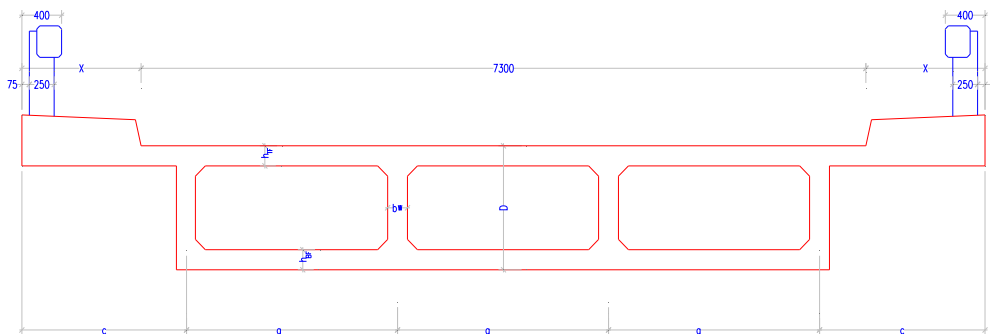
- one-eighth of the effective span length 3.81 m
- 6.0 times the average thickness of the slab, plus the greater of half the web thickness or one quarter of the width of the top flange of the basic girder, 1.35 m
- The width of the overhang 1.01 m

$$b_{eff} = \underline{\underline{2.11 \text{ m}}}$$

vii. Crown and Wearing Surface

* Use 2% crown in the transverse direction.

* Use Asphalt as wearing surface.



Typical Cross Section of Box Girder

h. Distribution Factors

MOMENT

Condition		Applicability
2100 ≤ S ≤ 4000	S: Spacing of beams, mm= 2200.00	ok
-330 ≤ d _c ≤ 1700		ok
18001 ≤ L ≤ 73000	L: Span of beam, mm= 30500.00	ok
N _c ≥ 3	N _c : Number of Cell= 3.00	ok
Cross section type	(d), ok	

d_c is the distance in mm from the face exterior face of exterior girder. It is positive if exterior web is inboard and vice versa

i. Interior Girder

The live load flexural moment for interior beams with concrete decks may be determined by applying the lane load fraction specified as follows:

For preliminary design

$$k_g/Lt_s^3 \text{ and } I/J = 1.0$$

One Design lane loaded

$$mg_M^{SI} = (1.75 + S/1100) \cdot (300/L) \cdot 0.35 \cdot (1/N_c) \cdot 0.45$$

$$= \underline{\underline{0.454}}$$

Two Design lane loaded

$$mg_M^{MI} = (13/N_c) \cdot 0.3 \cdot (a/430) \cdot (1/L) \cdot 0.25$$

$$= \underline{\underline{0.601}}$$

For interior girder, DF is governed by

Two Design lane loaded

$$DF = \underline{\underline{0.601}}$$

ii. Exterior Girder

One Design lane loaded

$$W_e = (\text{One half of the web spacing} + \text{total overhang}) \leq S$$

$$= 2200 \text{ mm}$$

$$mg_M^{SE} = 1.2 \cdot (W_e/4300)$$

$$= \underline{\underline{0.614}}$$

Two Design lane loaded

$$e = 0.77 + d_c/2800$$

$$= 0.99$$

$$mg_M^{ME} = e \cdot mg_M^{MI}$$

$$= \underline{\underline{0.594}}$$

For exterior girder, DF is governed by

One Design lane loaded

$$DF = \underline{\underline{0.614}}$$

SHEAR

Condition		Applicability
1800 ≤ s ≤ 4000	a: Spacing of beams, mm= 2200.00	ok
6000 ≤ L ≤ 73000	L: Span of beam, mm= 30500.00	ok
890 ≤ d ≤ 2800	d: Depth of Beam, mm= 1800.00	ok
N _c ≥ 3	N _c : Number of Cell= 3.00	ok

i. Interior Girder

One Design lane loaded

$$mg_V^{SI} = (S/2900) \cdot 0.6 \cdot (d/L) \cdot 0.1$$

$$= \underline{\underline{0.638}}$$

Two Design lane loaded

$$mg_V^{MI} = (a/2200) \cdot 0.9 \cdot (d/L) \cdot 0.1$$

$$= \underline{\underline{0.754}}$$

For interior girder, DF is governed by

Two Design lane loaded

$$DF = \underline{\underline{0.754}}$$

ii. Exterior Girder

One Design lane loaded

Lever rule to be used

$$\text{Furthest wheel dist.} = 1960.00$$

$$\text{Closest wheel dist.} = 160.00$$

$$R = 0.482$$

$$mg_M^{SE} = 0.578$$

Two Design lane loaded

$$-600 \leq d_e \leq 1500$$

$$d_e = 610$$

$$e = 0.801$$

$$mg_M^{ME} = e \cdot mg_M^{MI}$$

$$= \underline{\underline{0.603}}$$

For exterior girder, DF is governed by

Two Design lane loaded

$$DF = \underline{\underline{0.603}}$$

iii. Skewed Bridges

Condition		Applicability
0° ≤ θ ≤ 60°	θ: Skew angle, ° = 0	ok
1800 ≤ s ≤ 4000	s: Spacing of beams, mm= 2200	ok
6000 ≤ L ≤ 73000	L: Span of beam, mm= 30500	ok
900 ≤ d ≤ 2700	d: Depth of Beam, mm= 1800	ok
N _c ≥ 3	N _c : Number of Cell= 3	ok

$$r_{\text{SKEW}} = 1.0 + (0.25 + L/70d) \cdot \tan \theta$$

$$= \underline{\underline{1.000}}$$

Skew correction is not applicable and hence correction factor = 1.000

Summary of Distribution Coefficients

	Exterior		Interior	
	Shear	Moment	Shear	Moment
DF	0.603	0.614	0.754	0.601

Total DF 1.960 1.829 Assuming the box to act as a single entity, the total DF on the three girders will be the sum of DFs.

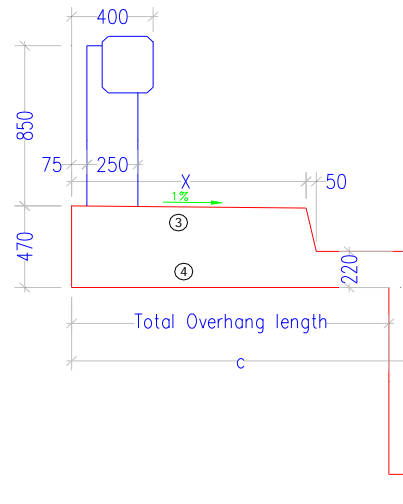
3 DESIGN OF DECK SLAB, TOP

3.1 DESIGN OF OVERHANG SLAB

3.1.1 Dead load

Item	weight [kN/m]	moment arm [m]	Moment [kNm/m]
Walkway	9.699	0.760	7.371
Slab	1.980	0.180	0.356
Damp proof	0.648	0.180	0.117
Railing	1.875	0.735	1.378
Post	1.063	0.835	0.887
total	15.265		10.110

total overhang length =	1.010 m
depth of railing =	0.300 m
breadth of railing =	0.250 m
distance b/n post and curb end =	0.050 m
distance b/n post and railing start =	0.050 m
distance b/n back of railing and end of curb =	0.15 m
depth of post =	0.250 m
width of post =	0.300 m
c/c of post =	1.500 m
height of post =	0.850 m
depth of curb at the end =	0.490 m
depth of curb at the inner =	0.480 m



3.1.2 Live load

a) Railing load

Railing loads shall be applied on an effective length of $E = 1140 + 0.833X$,

Eq. 7.14

$$E = 1140 + 0.833X \text{ [mm]}$$

$$= \underline{1702} \text{ m}$$

where:

X - is the distance in mm from the center of the post to the point under investigation = $850/2 + 250 =$

675 mm

According to Art. 2.7 of the AASHTO 1996, the design load, P is ,

$$P = 44.51 \text{ kN}$$

height of top of rail from top of curb =

0.85 m

Moment arm =

1.050 m

Therefore the railing live load is as follows:

$$M_{RLL} = \underline{27.45} \text{ kNm/m}$$

b) Truck load

According to Art. 7.4 Slabs/Longitudinal Edges, the wheel is put 300mm from face of rail

$$E_{\text{overhang}} = 1140 + 0.833X$$

$$= 1398.2 \text{ mm}$$

Where: X = Distance from load to point of support [mm]

$$= 310 \text{ mm}$$

The multiple presence factor m is 1.20 for one loaded lane.

$$P = 72.50 \text{ kN}$$

$$m = 1.2$$

$$M_{TLL} = \underline{19.29} \text{ kNm/m}$$

$$\text{oad Moment} = \underline{0.78} \text{ kNm/m}$$

$$M_{TLL} = \underline{20.070} \text{ kNm/m}$$

c) Total Moment

i) Dead load plus rail live load

$$M_{TR} = 1.0 \cdot (1.25M_{DL} + 1.75M_{RLL})$$

$$= 60.68 \text{ kNm/m}$$

ii) Dead load plus truck live load

$$M_{TT} = 1.0 \cdot (1.25M_{DL} + 1.75M_{LL})$$

$$= 47.76 \text{ kNm/m}$$

iii) Design moment

$$M_D = \max(M_{TR}, M_{TT})$$

$$= \underline{60.68} \text{ kNm/m}$$

3.1.3 Design for flexure

D =	0.30 m	Reinf Φ	16 , Grade 60	Φ (moment) =	0.90
Cover = 1" =	2.54cm (bottom)	as =	201.02 mm ²		
	2" = 5.08cm (top)	use cover:	2.5 cm bottom 5.0 cm top		
	d_{bot} =	267 mm (bottom)			
	d_{top} =	242 mm (top)			
f_y =	414 Mpa	f_c' =	24 MPa		
M_u =	60.68 kNm/m				
$A_s = M_u / (\phi f_y (d-a/2))$		$a = A_s * f_y / 0.85 * f_c' * b$			
Try a =	32				
$A_s =$	720.63	$A_s =$	693.16		
a =	14.62	a =	14.07		
$A_s =$	693.96	$A_s =$	693.13		
a =	14.08	a =	14.07		
$A_s =$		693.13 mm ² / m	Spacing =	290 mm	
		Use \emptyset 16.00 c/c	280.00 mm	(Top & Bottom Slab Reinf.)	

3.2 DESIGN OF INTERIOR SLAB

3.2.1 Dead Loads

* RC Slab = 0.20*25 =		5 kN/m ²
* Wearing Surface -5cm thick AC pavement and 3cm damp proof		2.16 kN/m ²
	Total	7.16 kN/m²

For slabs monolithic with beams or slabs monolithic with walls without haunches
S = girder spacing = 2.2 m
The dead load moment
 $M_{DL} = (WS^2 / 8) * 0.80 = (WS^2) / 10 ; \{ 0.80 \text{ is continuity factor} \}$
 $M_{DL} = 3.47 \text{ kNm/m}$

3.2.2 Live Load Moments

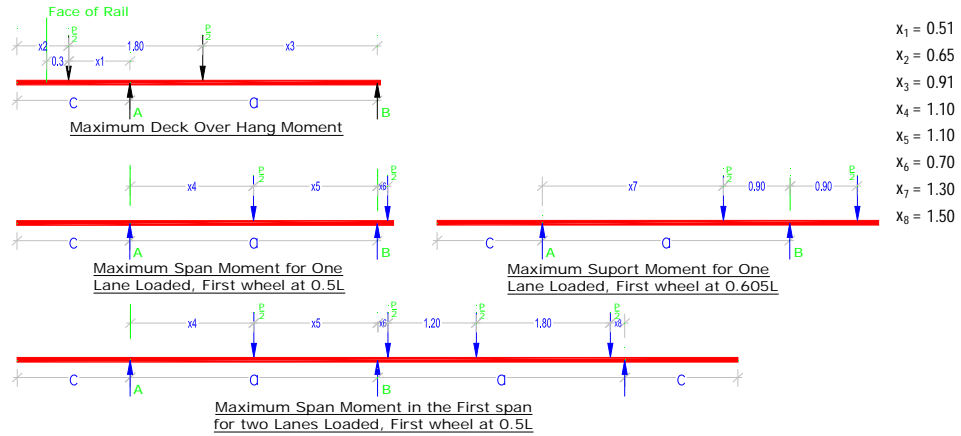
An approximate analysis of strips perpendicular to girders is considered. The extreme positive moment in any deck panel b/n girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any girder shall apply to all negative moment regions. The strips shall be treated as continuous beams with span lengths equal to the c/c distance b/n girders.

The design truck shall be positioned transversely to produce maximum force effects such that the center of any wheel load is not closer than 300mm from the face of the curb/rail for the design of the deck overhang and 600mm from the edge of the 3600mm wide design lane for the design of all other components. Where a sidewalk is not separated from the roadway by a crashworthy traffic barrier, consideration should be given to the possibility that vehicles can mount the sidewalk.

Slabs with more than two girders

Distribution Widths

$E_{overhang} = 1140 + 0.833X$	Where: X is the distance from the wheel to centerline of support
= 1523.2 mm	= 460 mm
$E_{positive} = 660 + 0.55 * S$	S = Girder Spacing (mm)
= 1870.0 mm	= 2200 mm
$E_{negative} = 1220 + 0.25 * S$	
= 1770.0 mm	



$M_{\text{positive}} =$	30.8 kNm/m	SAP Output
$M_{\text{negative}} =$	34.00 kNm/m	SAP Output
$M_{\text{positive}} =$	16.47 kNm/m	(Modified with distribution widths)
$M_{\text{negative}} =$	19.21 kNm/m	

Total Moment

i) Positive Moment at Mid Span, Dead load plus Live (Truck) Load

$$M_{TR} = 1.00 * (1.25 * M_{DC} + 1.50 * M_{DW} + 1.75 * M_{LL}) \quad 1.50 M_{DW} = 1.25 * (1.20 M_{DW})$$

$$= \mathbf{42.56 \text{ kNm/m}}$$

ii) Negative Moment at Support, Dead load plus Live (Truck) Load

$$M_{TT} = 1.00 * (1.25 * M_{DC} + 1.50 * M_{DW} + 1.75 * M_{LL})$$

$$= \mathbf{48.92 \text{ kNm/m}}$$

3.2.3 Design for flexure

a. Main Slab Reinforcement

D =	0.20 m	Reinf Φ	16 , Grade 60	Φ (moment) =	0.90
Cover = 1" =	2.54cm (bottom)	as =	201.02 mm ²		
2" =	5.08cm (top)	use cover:	2.5 cm bottom		
			5.0 cm top		
d = 220 - 25 - 16/2 =	167 mm (bottom)	fc' =	24 Mpa		
d = 220 - 50 - 16/2 =	142 mm (top)				
fy =	414 Mpa				
Mu =	42.56 kNm/m	(bottom)			
Mu =	48.92 kNm/m	(top)			
As = Mu / (Φ fy(d-a/2))		a = As * fy / 0.85 * fc' * b			

For Bottom

Try a =	12.93
As =	711.53
a =	14.44

As =	714.89
a =	14.51

As =	715.04
a =	14.51

As =	715.05
a =	14.51

As =	715.05 mm² / m
Spacing =	281 mm

For Top

Try a =	29
As =	1029.72
a =	20.90

As =	998.00
a =	20.25

As =	995.57
a =	20.20

As =	995.38
a =	20.20

As =	995.38 mm² / m
Spacing =	202 mm

Use \emptyset	16.00	c/c	200.00 mm	(Top Slab Reinf.)
As =	1005.12	mm ² / m		
Use \emptyset	16.00	c/c	280.00 mm	(Bottom Slab Reinf.)
As =	717.94	mm ² / m		

b. Distribution Reinforcement (Art. 3.24.10.)

For main reinforcement perpendicular to traffic

Percentage = $220 / \sqrt{S} \leq 67\%$

81.9 > 67% ; Use 67% of main reinf as distribution reinforcement

$$A_{S_{dist}} = 481.02 \text{ mm}^2 / \text{m}$$

$$\text{Spacing} = 418 \text{ mm}$$

Use Ø	16.00	c/c	200.00 mm	(Bottom Slab Reinf.)
As =		1005.12	mm² / m	

c. Check for maximum Reinforcement (Art. 8.20.)

* Check ρ_{max} for main slab reinf.

$$\rho_{max} = 0.75 \cdot p_{bal} = 0.75 \cdot \left\{ \frac{0.85 \cdot \beta_a \cdot f_c}{f_y} \right\} \cdot \left[\frac{87000}{87000 + f_y} \right]$$

$$\rho = A_s / b d = 0.007078 \ll \rho_{max} = 0.01859 \quad \text{Ok}$$

d. Shrinkage and Temperature Reinforcement (Art. 8.20.)

The minimum amount of reinforcement in each direction shall be

$$\text{Temp } A_s \geq 0.75 A_g / f_y \quad h = 200.00 \text{ mm}$$

$$= 362.32 \text{ mm}^2 / \text{m}$$

For members greater than 150mm in thickness, the shrinkage and temperature reinforcement is to be distributed equally on both faces.

Maximum spacing of primary reinforcement is

$$S_{max} = 3 \times \text{slab thickness or } 450 \text{ mm}$$

$$450 \text{ mm}$$

$$1/2 \text{ Temp } A_s = 181.16 \text{ mm}^2 / \text{m}$$

$$\text{Spacing} = \pi \cdot d_b^2 \cdot 1000 / (4 \cdot A_s)$$

Spacing using Ø	12.00	c/c	312.149 mm	
Use Ø	12.00	c/c	250.00 mm	(Top Slab Reinf.)

4 DESIGN OF DECK SLAB, BOTTOM

4.1 DESIGN OF BOTTOM SLAB, LONGITUDINAL DIRECTION

Bottom Slab Reinforcement in Cast-in-Place Box Girders shall be uniformly distributed reinforcement of 0.4% of the flange area and be placed in the bottom slab parallel to the girder span, either in single or double layers. The spacing of such reinforcement shall not exceed 450mm.

$$A_{sll} = 0.800 \text{ m}^2 / \text{m}$$

$$\text{Spacing} = \pi \cdot d_b^2 \cdot 1000 / (4 \cdot A_{sll}) \quad \emptyset = 16 \text{ mm}$$

$$= 250.00 \text{ mm}$$

Use Ø	16.00	c/c	250.00 mm	Top and Bottom slab reinforcement
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4.2 DESIGN OF BOTTOM SLAB, TRANSVERSE DIRECTION

A uniformly distributed reinforcement of 0.5% of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 450mm. All transverse reinforcement in the bottom slab shall be extended to the exterior face of the outside web in each group and shall be anchored by a standard 90° hook.

$$A_{s-t} = 1.000 \text{ m}^2 / \text{m}$$

$$\text{Spacing} = \pi \cdot d_b^2 \cdot 1000 / (4 \cdot A_{s-t})$$

$$= 200.00 \text{ mm}$$

Use Ø	16.00	c/c	200.00 mm	Top and Bottom slab reinforcement
-------	-------	-----	-----------	-----------------------------------

The transverse reinforcement shall be anchored in the outside face of the exterior face of exterior web with 90° hook.

4.3 Shrinkage and Temperature Reinforcement (Art. 8.20.)

The minimum amount of reinforcement in each direction shall be

$$\text{Temp } A_s \geq 0.75 A_g / f_y \quad h = 200.00 \text{ mm}$$

$$= 362.32 \text{ mm}^2 / \text{m}$$

For members greater than 150mm in thickness, the shrinkage and temperature reinforcement is to be distributed equally on both faces.

Maximum spacing of primary reinforcement is

$$S_{max} = 3 \times \text{slab thickness or } 450 \text{ mm}$$

$$450 \text{ mm}$$

$$1/2 \text{ Temp } A_s = 181.16 \text{ mm}^2 / \text{m}$$

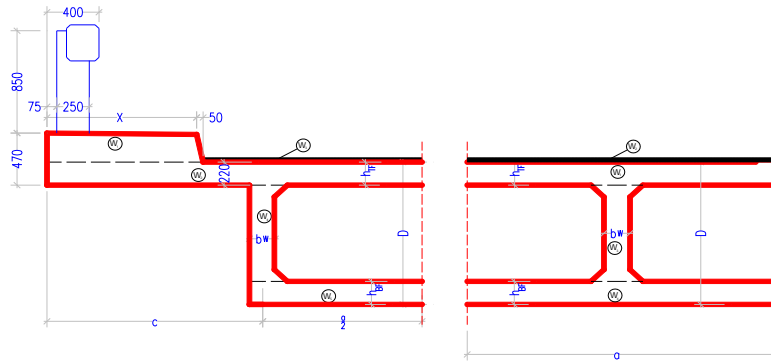
$$\text{Spacing} = \pi \cdot d_b^2 \cdot 1000 / (4 \cdot A_s)$$

Spacing using Ø	12.00	c/c	312.149 mm	
Use Ø	12.00	c/c	280.00 mm	(Top Slab Reinf.)

5 DESIGN OF GIRDER

5.1 DESIGN LOADS

5.1.1 Dead Loads



a. Exterior Girder

* Dead Load per linear meter span

$W_1 = \text{Curb Load} =$	5.00
$W_2 = \text{Top Slab L.} =$	11.30
$W_{3,1} = \text{Damp proof} =$	1.18
$W_{3,2} = \text{Pavement} =$	1.97
$W_4 = \text{Girder Load} =$	10.50
$W_5 = \text{Bott. Slab L.} =$	6.25
$W_6 = \text{Railing Load} =$	3.47
$W =$	<u>45.71 kN/m</u>

* Diaphragm Load

$$P = 8.75 \text{ kN}$$

* Support Reaction

$$R_A = R_B = 723.33 \text{ kN}$$

b. Interior Girder

* Dead Load per linear meter span

$W_1 = \text{Top Slab L.} =$	11
$W_{2,1} = \text{Damp proof} =$	1.782
$W_{2,2} = \text{Pavement} =$	2.97
$W_3 = \text{Girder Load} =$	10.5
$W_4 = \text{Bott Slab Load} =$	11
$W =$	<u>37.25 kN/m</u>

* Diaphragm Load

$$P_3 = 17.50 \text{ kN}$$

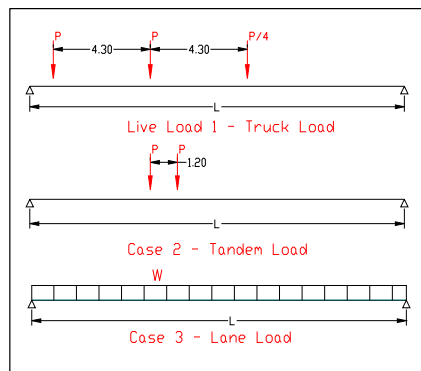
* Support Reaction

$$R_A = R_B = 620.59$$

5.1.2. Live Loads

a. HL-93 Truck Load

$$P \text{ (kN)} = 145$$



b. HL-93 Tandem load

$$P \text{ (kN)} = 110$$

c. Lane Loads

$$w \text{ (kN/m)} = 9.3$$

5.2. BENDING MOMENTS AND SHEAR FORCES

5.2.1. Dead Load Moments and Shear Forces

a. Exterior Girder

For $0 \leq x \leq S/2$

$$V_{DL}(X) = R_A - P - 0.3 \cdot W - W \cdot x$$

$$M_{DL}(X) = V_{DL} \cdot x - (W \cdot x^2)/2$$

Span (m)	30	35
$V_{DL}(X)$	700.86	818.89
$M_{DL}(X)$	5372.97	7334.54

b. Interior Girder

For $0 \leq x \leq S/2$

$$V_{DL}(X) = R_A - P - W \cdot x$$

$$M_{DL}(X) = R_A \cdot x - (W/2) \cdot x^2$$

Span (m)	30	35
$V_{DL}(X)$	591.92	731.81
$M_{DL}(X)$	4695.03	6766.83

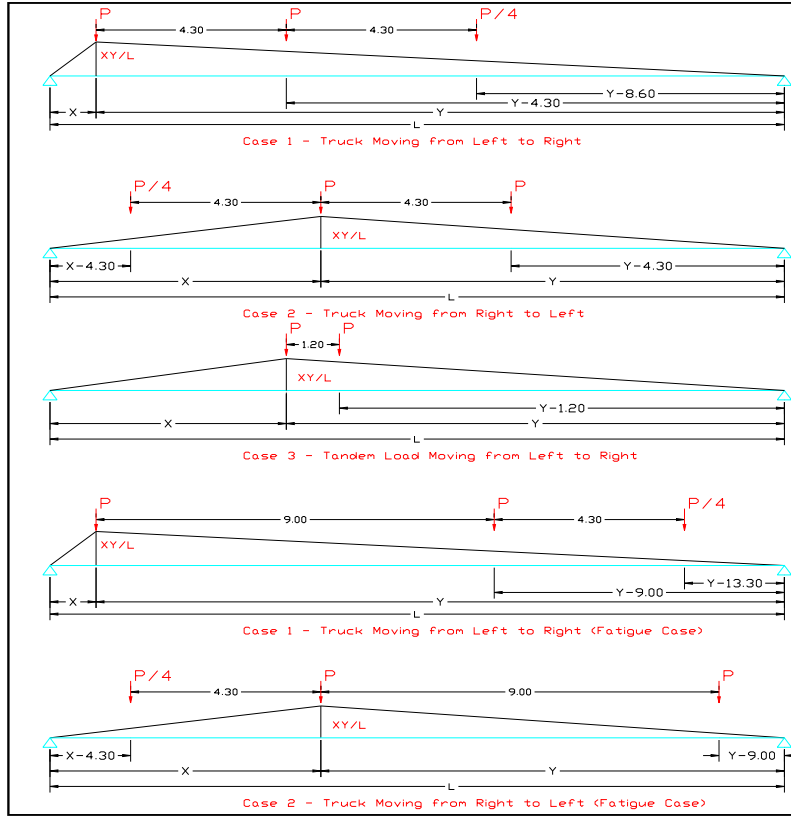
5.2.2. Influence Lines for Live Loads
5.2.2.1 Influence Line for Bending Moment

a. Loading 1 (HL-93 Truck Vehicle moving from left to right)

$$M_{LL}(X) = P'xy/L [1+(y-4,30)/y + (y-8,60)/4.1y] =$$

b. Loading 2 (HL-93 Truck Vehicle moving from right to left)

$$M_{LL}(X) = P'xy/L [1+(y-4,30)/y + (x-4,30)/4.1x] =$$



For Flexure

x/L	Mmax		
	30	35	
0.000	0.000	0.000	*P'
0.050	2.931	3.464	*P'
0.100	5.520	6.530	*P'
0.150	7.766	9.197	*P'
0.200	9.671	11.466	*P'
0.250	11.233	13.337	*P'
0.300	12.453	14.809	*P'
0.350	13.383	15.936	*P'
0.400	14.076	16.769	*P'
0.450	14.427	17.204	*P'
0.465	14.473	17.273	*P'
0.500	14.435	17.240	*P'

For Fatigue

x/L	Mmax		
	30	35	
0.000	0.000	0.000	*P'
0.050	2.639	3.172	*P'
0.100	4.935	5.945	*P'
0.150	6.889	8.320	*P'
0.200	8.501	10.297	*P'
0.250	9.796	11.899	*P'
0.300	10.938	13.294	*P'
0.350	11.738	14.291	*P'
0.400	12.196	14.889	*P'
0.450	12.312	15.089	*P'
0.465	12.217	15.017	*P'
0.500	12.085	14.890	*P'

c. Loading 3 (Tandem Load)

$$M_{LL}(X) = P''xy/L [1+(y-1.20)/y] =$$

$$L = 30.5 \text{ m}$$

For Flexure

x/L	Mmax		
	30	35	
0.000	0.000	0.000	*P''
0.050	2.838	3.313	*P''
0.100	5.370	6.270	*P''
0.150	7.598	8.873	*P''
0.200	9.520	11.120	*P''
0.250	11.138	13.013	*P''
0.300	12.450	14.550	*P''
0.350	13.458	15.733	*P''
0.400	14.160	16.560	*P''
0.450	14.558	17.033	*P''
0.465	14.650	17.146	*P''
0.500	14.650	17.150	*P''

5.2.2 Influence Line for Shear Force

a. Loading 1

(HL-93 Truck Vehicle moving from left to right)

$$V_{LL}(X) = P'y/L + P'(y-4,30)/L + P'/4.1(y-8,60)/L =$$

Span(m)= x/L	Vmax		
	30	35	
0.000	2.034	2.064	*P'
0.050	1.922	1.951	*P'
0.100	1.810	1.839	*P'
0.150	1.698	1.727	*P'
0.200	1.585	1.615	*P'
0.250	1.473	1.503	*P'
0.300	1.361	1.391	*P'
0.350	1.249	1.278	*P'
0.400	1.137	1.166	*P'
0.450	1.024	1.054	*P'
0.465	0.957	0.987	*P'
0.500	0.912	0.942	*P'

b. Loading 2 (Tandem Load)

$$V_{LL}(X) = P''y/L + P''(y-1.20)/L =$$

x/L	Vmax		
	12	15	
0.00	1.961	1.966	*P''
0.63	1.861	1.866	*P''
1.25	1.761	1.766	*P''
1.88	1.661	1.666	*P''
2.50	1.561	1.566	*P''
3.13	1.461	1.466	*P''
3.75	1.361	1.366	*P''
4.38	1.261	1.266	*P''
5.00	1.161	1.166	*P''
5.63	1.061	1.066	*P''
5.52	1.001	1.006	*P''
6.25	0.961	0.966	*P''

5.2.3 Bending Moment

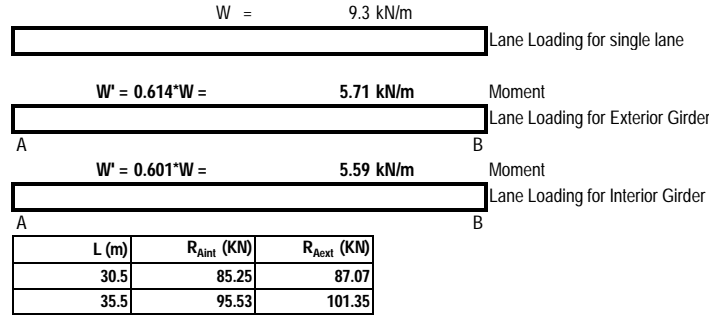
a. Bending Moment due to HL-93 Truck

Span(m)	X/L	Live Load Moment, M _{LL}	
		MExt, max	MInt, max
30.00	0.50	1288.43	1261.40
35.00	0.50	1534.78	1446.62

b. Bending Moment due to Tandem

Span(m)	X/L	Live Load Moment, M _{LL}	
		MExt, max	MInt, max
30.00	0.50	989.39	968.63
35.00	0.50	1158.22	1091.69

c. Bending Moment due to Lane Loading



Span(m)	X/L	Live Load Moment, M _{LL}	
		Exterior	Interior
30	0.50	663.94	650.01
35	0.50	899.47	847.80

d. Governing Bending Moment

X	Exterior Girder				Interior Girder			
	M _{LL(Truck)}	M _{LL(Tandem)}	Governing	M _{LL(Gov.)}	M _{LL(Truck)}	M _{LL(Tandem)}	Governing	M _{LL(Gov.)}
15.25	1285.08	989.39	Truck	1285.08	1258.12	968.63	Truck	1258.12
17.75	1534.78	1158.22	Truck	1534.78	1446.62	1091.69	Truck	1446.62

FLEXURAL DESIGN IS GOVERNED BY TRUCK LOADING

e. Design Bending Moment

Total Design Moment

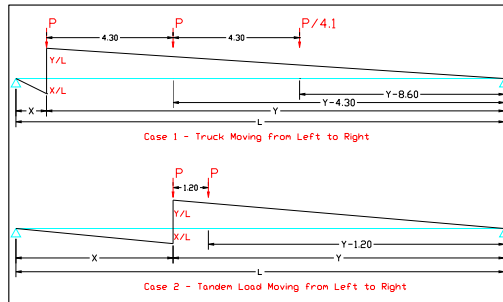
According to subchapter 3.3, the coef. for load factor service load design are as follows:

For standard truck & lane , and design tandem loading

Group I Load Combination, the factored moment $M_u = 1.00 \cdot (1.25 \cdot M_{DC} + 1.50 \cdot M_{DW} + 1.75 \cdot M_{LL+I})$

X	Exterior Girder				Interior Girder			
	M _{DL}	M _{LL(Truck+Lane)}	M _{LL+I}	M _U	M _{DL}	M _{LL(Truck+Lane)}	M _{LL+I}	M _U
15.25	5,372.97	1,949.02	2,373.10	10,841.96	4,695.03	1,908.13	2,323.31	9,909.74
17.75	7,334.54	2,434.25	2,940.73	14,278.66	6,766.83	2,294.42	2,771.81	13,275.92

5.2.4 Shear Force



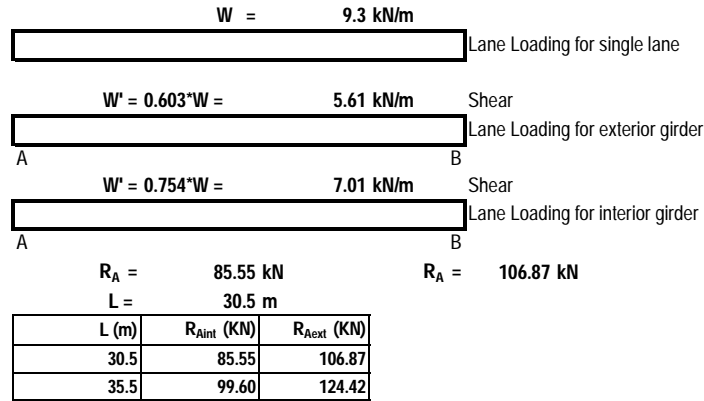
a. Shear Force due to HL-93 Truck

Span	X/l	Live Load Shear Force, V _{LL}	
		Exterior	Interior
30.00	0.00	177.92	222.25
35.00	0.00	180.55	225.53

b. Shear Force due to Tandem

Span	X/l	Live Load Shear Force, V _{LL}	
		Exterior	Interior
30.00	0.00	130.10	162.51
35.00	0.00	130.50	163.01

c. Shear Force due to Lane Loading



Live Load Shear F, V_{LL}			
Span	X/L	Exterior	Interior
0.00	0.00	85.55	106.87
0.05	0.00	99.60	124.42

d. Governing Shear Force

Span(m)	Exterior Girder				Interior Girder			
	$V_{LL(Truck)}$	$V_{LL(Tandem)}$	Governing	$V_{LL(Gov.)}$	$V_{LL(Truck)}$	$V_{LL(Tandem)}$	Governing	$V_{LL(Gov.)}$
30.00	177.92	130.10	Truck	177.92	222.25	162.51	Truck	222.25
35.00	180.55	130.50	Truck	180.55	225.53	163.01	Truck	225.53

SHEAR DESIGN IS GOVERNED BY TRUCK LOADING

e. Design Shear Force

Total Design Shear

According to subchapter 3.3, the coef. for load factor service load design are as follows:

For standard truck & lane, and design tandem loading

Group 1 Load Combination, the factored shear, $V_u = 1.00 \cdot (1.25 \cdot V_{DC} + 1.50 \cdot V_{DW} + 1.75 \cdot V_{LL+i})$

Group I Load Combination, the service shear, $V_u = 1.00 \cdot (V_{DC} + V_{DW} + V_{LL+i})$

Span(m)	Exterior Girder					Interior Girder				
	V_{DL}	V_{LL} (Truck+Lane)	V_{LL+I}	V_S	V_U	V_{DL}	V_{LL} (Truck+Lan)	V_{LL+I}	V_U	V_U
30.00	700.86	263.47	322.18	1020.49	1436.30	591.92	329.12	402.46	991.90	1440.60
35.00	818.89	280.14	339.72	1155.72	1614.09	731.81	349.95	424.38	1153.30	1653.28

5.3. DESIGN FOR FLEXURE

3.3.1. Exterior Girder Design

3.3.1.1. Effective flange width for flexural compression

The total width of slab effective as a T-girder flange shall not exceed one fourth of the span length of the girder, c/c spacing b/n girders (if total No of girders is greater or equal to 3), half of c/c spacing b/n girders plus over hang length (where the effective flange width overhanging on one side of the web shall not exceed six times the thickness of slab) plus half web width.

$$b_{\text{eff}} = \underline{\underline{2.11 \text{ m}}}$$

Design Moment

Span(m)=	30	35		
M, EXT MAX =	10,841.96	14,278.66	kNm	Ø = 32
D =	1.80	2.10	m	as = 804.10
Demp =	1.83	1.83	m	Φ (moment) = 0.90
beff =	2.11	2.11	m	
fy =	414	414	Mpa	
fc' =	24	24	Mpa	

Steel Area Required

Span(m)	Mu	d(mm)	a(mm)	As(mm ²)	No of reinforcement
30	10841.96	1700.00	173.48	18036.82	Required 22 Bars dia 32mm
35	14278.66	2001.20	193.54	20122.38	Required 25 Bars dia 32mm

Extension length required at bar cutoff

Extension length required beyond the point at which it is no longer required is given by:

$$L_{\text{ext}} \text{ required} = \max (15 \cdot \text{dia of bar, deff, } 1/20 \text{ of clear span length})$$

Span(m)	L _{ext} (mm)
30	1701
35	2001

$$\begin{aligned} \text{Length of standard hook} &= 0.63 \text{ m} \\ &= 640 \text{ mm} \end{aligned}$$

3.3.2. Interior Girder Design

3.3.2.1. Effective flange width for flexural compression

The total width of slab effective as a T-girder flange shall not exceed one fourth of the span length of the girder, average spacing of the adjacent beams, 12 times the average thickness of the slab, plus the greater of half the web thickness or one quarter of the width of the top flange of the girder.

$$b_{\text{eff}} = \underline{\underline{2.20 \text{ m}}}$$

Design Moment

Span(m)=	30	35	m	
MU MAX =	9,909.74	13,275.92	kNm	Ø = 32 mm
D =	1.80	2.10	m	as = 804.10 mm ²
fy =	414	414	Mpa	Φ (moment) = 0.90
fc' =	24	24	Mpa	

Steel Area Required

Span(m)	Mu	d(mm)	a(mm)	As(mm ²)	No of reinforcement
30	9909.74	1700.00	151.03	16372.06	Required 20 Bars dia 32mm
35	13275.92	1996.09	172.08	18654.24	Required 23 Bars dia 32mm

Spacing of bars

Half bottom slab width (without overhang), mm =	3440
Total no bars in bottom layer, Int & Ext Girders =	22
Total no bars in top layer, Int & Ext Girders =	22
Spacing _{bot} =	156
Spacing _{top} =	156.36

Use Spacing 150mm for bott and 150mm for top

3.3.3. Skin Reinforcement

If the effective depth, d_e , of RC members exceeds 900mm, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance $d/2$ nearest the flexural tension reinforcement.

$$A_{sk} \geq 0.001(d_e - 760) \leq A_s/1200 \quad \text{mm}^2/\text{mm}$$

$$A_{sk} = 0.970 \text{ mm}^2/\text{mm} \quad \leq \quad 10.498 \quad A_s = 12597.5 \text{ mm}^2$$

$$A_{sk} = 970.0 \text{ mm}^2/\text{m} \quad \emptyset = 16 \text{ mm}$$

$$\text{max spacing} = d/6 \text{ or } 300\text{mm} = 288.33 \text{ mm} \quad a_s = 201.1 \text{ mm}^2$$

$$\text{Spacing} = 1000 \cdot a_s/A_s = 207.3 \text{ mm} \quad \text{No of bars reqd.} = 5$$

$$d/2 = 865 \text{ mm}$$

Provide 8 Ø 16 at 200mm

3.4. DESIGN FOR SHEAR

$$V_u \leq \Phi V_n \quad V_n = V_c + V_s$$

V_c = nominal shear strength provided by concrete
 V_s = nominal shear strength provided by shear reinf.

$$V_c = 2(f_c' \cdot 0.5) b_w \cdot d = 0.166(24.15 \cdot 0.5) \cdot 300 \cdot d = 244.73 \cdot d \text{ [N]}$$

$$4 \cdot \text{Sqrt}(f_c) b w d = 4 \cdot \text{sqrt}(4000) \cdot 300 / 25.4 \cdot d / 25.4 = 203511.9 \text{ lb} = 905.22 \text{ kN}$$

if $V_s > 905.22$, then $S_{max} = d/4$ or $24"/2$

$\Phi = 0.85$ for shear

$$\Phi V_c = 0.85 \cdot 244.73 \cdot d = 208.02 \cdot d$$

$$S_{req} = A_v \cdot f_y \cdot d / V_s \quad S_{max} = d/2 \text{ or } 24" \text{ (} d/2 \text{ or } 609\text{mm)}$$

$$f_y = 414 \text{ Mpa} \quad S_{max} = A_v \cdot f_y / 0.34475 \cdot b w = 905.43 \text{ mm}$$

$$V_{Umax} = 1436.30 \text{ kN} \quad \emptyset = 12 \text{ mm}$$

$$D = 1.80 \text{ m} \quad A_v = 2 \cdot 113 \cdot 10^{-6} = 0.000226 \text{ mm}^2$$

Span m	V_{max} kN	d m	ΦV_c kN	ΦV_s kN	V_s kN	S_{req} mm	S_{max} mm
30.00	1436.30	2.13	443.08	993.22	1168.49	170.70	609.00
35.00	1614.09	2.13	443.08	1171.00	1377.65	144.79	609.00

3.4.1. Serviceability Requirements

For flexural members designed with reference to load factor and strengths by strength design method, stresses at service load shall be limited to satisfy the requirements for fatigue and for distribution of reinforcement. The requirements for control of deflection shall also be checked.

Service Load Bending Moment

Group I Load Combination, the service load moment $M_s = M_{DL} + M_{LL+I}$ Take $I = 0.15$

X	Exterior Girder				Interior Girder			
	M_{DL}	$M_{LL(Truck)}$	M_{LL+I}	M_s	M_{DL}	$M_{LL(Truck)}$	M_{LL+I}	M_s
0.00	-	-	-	-	-	-	-	-
1.53	1,015.67	260.92	300.06	1,315.73	859.36	255.45	293.77	1,153.12
3.05	1,925.03	491.39	565.09	2,490.12	1,632.08	481.08	553.24	2,185.32
4.58	2,728.09	691.39	795.09	3,523.18	2,318.17	676.88	778.41	3,096.58
6.10	3,424.84	860.92	990.06	4,414.90	2,917.62	842.86	969.28	3,886.91
7.63	4,015.29	999.99	1,149.99	5,165.28	3,430.44	979.01	1,125.86	4,556.30
9.15	4,499.43	1,108.60	1,274.89	5,774.32	3,856.63	1,085.34	1,248.14	5,104.77
10.68	4,877.27	1,191.41	1,370.13	6,247.40	4,196.18	1,166.41	1,341.38	5,537.56
12.20	5,148.81	1,253.10	1,441.07	6,589.87	4,449.10	1,226.81	1,410.83	5,859.93
13.73	5,314.04	1,284.32	1,476.97	6,791.01	4,615.38	1,257.37	1,445.98	6,061.36
14.64	5,362.15	1,288.43	1,481.70	6,843.85	4,673.57	1,261.40	1,450.61	6,124.18
15.25	5,372.97	1,285.08	1,477.85	6,850.81	4,695.03	1,258.12	1,446.84	6,141.87

$$P'_{Ext} = 0.614 \cdot P \quad 89.02 \text{ kN} \quad P'_{Int} = 0.601 \cdot P \quad 87.16 \text{ kN}$$

$$M_{Ext, max} = 6,850.81 \text{ kNm} \quad M_{Int, max} = 6,141.87 \text{ kNm}$$

Fatigue stress limits

Spacing b/n the 35kN and 145kN axels = 4.3 m
Spacing b/n the 145kN and 145kN axels = 9.0 m
L = 30.50 m

Impact

Take I = 0.15
1+I = 1.15

The stress range (f_i) in straight reinforcement resulting from the fatigue load combination, specified in Table 3-2, shall not exceed :

$$f_i = 145 \cdot 0.33 \cdot \text{min} + 55 \cdot (r/h) \quad [\text{MPa}] \quad (\text{Eq. 9.19}) \quad \rho/h = 0.30$$

Reinforcement Proportioning

Mmax = **3,697.96** KNm coef. of d = 1.000

The following parameters are read from trendline eq. on the moment diagram charts.

y = ax ² +bx+c	a	b	c
Service Load Moment	-29.885	904.463	5.538
Dead Load Moment	-22.855	700.864	0.000

	No of bar(or N)	31	24	20	16	11
bar size [mm]		32	32	32	32	32
As[mm ²]		24927.0	19298.3	16081.9	12865.5	8845.1
No of row (or k)		2	2	2	1	1
y bar [m]		0.0989	0.0833	0.0780	0.0700	0.0700
d [m]		1.701	1.717	1.722	1.730	1.730
ρ		0.0069	0.0053	0.0044	0.0035	0.0024
fy[MPa]		414.00	414.00	414.00	414.00	414.00
a[mm]		239.75	185.61	154.68	123.74	85.07
As[mm ²]		16053.91	276930.00	0.00	0.00	0.00
As-As[mm ²]		8873.06	-257631.70	16081.92	12865.54	8845.06
a'[mm]		600	-17428	0	0	0
Mu[KNm]		14864.90	11966.00	9855.01	7996.51	5561.32
Half length		0.00	5.46	7.31	10.64	15.25
Remark	T-Beam	T-Beam	Rect. Beam	Rect. Beam	Rect. Beam	
ρn	0.0573	0.0439	0.0365	0.0291	0.0200	
t/d	0.106	0.105	0.105	0.104	0.104	
k	0.386	0.332	0.298	0.259	0.205	
kd [mm]	655.84	570.39	512.43	448.14	354.26	
j	0.950	0.951	0.901	0.914	0.932	
Distance from support	15.250	9.785	7.942	4.610	0.000	
M _s	6848.49	5994.45	5303.66	3539.89	0.00	
M _{DL}	5372.97	4669.76	4124.58	2745.19	0.00	
fsmax	170.03	190.30	212.60	174.07	0.00	
fsmin	133.39	148.24	165.34	134.99	0.00	
fsmax - fsmin	36.63	42.05	47.27	39.08	0.00	
f _i	117.48	112.58	106.94	116.95	161.50	
fs	248.40	248.40	248.40	248.40	248.40	

n = 8
hf = 0.2 m
b_{eff} = 2.11
bw = 0.30
fc' = 24.00

Ok Ok Ok Ok Ok f_i > (fsmax - fsmin) Ok !!
Ok Ok Ok Ok Ok fs > fsmax Ok !!
fs = z/(dcA)^{1/3} <= 0.6fy

Where

y [m] = Centroid of reinforcement from bottom of girder

d [m] = D - y

$$\rho = A_s / b_{eff} \cdot d \quad m = \rho \cdot n$$

$$a = A_s \cdot f_y / 0.85 \cdot f_c' \cdot b_{eff} \quad M_s = A_s \cdot f_y \cdot j \cdot d$$

T-Beam (if a > h_f)

$$As[mm^2] = 0.85 \cdot f_c' \cdot (b_{eff} - b_w) \cdot h_f / f_y$$

$$a' = (A_s - A_{st}) \cdot f_y / 0.85 \cdot f_c' \cdot b_w$$

$$M_u = M_{df} + M_{uw}$$

$$M_{df} = \Phi A_{st} \cdot f_y \cdot (d - h_f / 2)$$

$$M_{uw} = \Phi (A_{st} - A_s) \cdot f_y \cdot (d - a / 2)$$

$$j = (6 + 6 \cdot h_f / d + 2 \cdot (h_f / d)^2 + (h_f / d)^3 \cdot (1 / 2m)) / (6 \cdot 3 \cdot h_f / d)$$

Rectangular-Beam (if a < h_f)

$$As[mm^2] = 0.85 \cdot f_c' \cdot (b_{eff} - b_w) \cdot h_f / f_y$$

$$a' = (A_s - A_{st}) \cdot f_y / 0.85 \cdot f_c' \cdot b_w$$

$$M_u = \Phi A_s \cdot f_y \cdot (d - a / 2)$$

$$k = (m + 0.50 \cdot (h_f / d)^2) / (m + h_f / d)$$

$$j = 1 - k / 3$$