ECONOMIC DESIGN COMPARISON OF PRE-STRESSED CONCRETE SLAB & BOX-GIRDER RAILWAY BRIDGES

Ibrahim Abdoulkader

Thesis Submitted to
The School of Civil and Environmental Engineering

Presented in Fulfillment of the Requirements for the Degree of Master of Science
(**Civil Railway Engineering**)

Addis Ababa University
Addis Ababa Institute of Technology
Addis Ababa, Ethiopia
JUNE 2017

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

It is to certify that the thesis prepared by Ibrahim Abdoulkader, entitled: *Economic Design Comparison of Pre-Stressed Concrete Slab & Box-Girder Railway Bridges* and submitted in partial fulfillment of the requirements for the degree of Master of Science (Civil and Environmental Engineering) complies with the regulations of the University and meets the accepted standards respect to originality and quality

Approved by the Examining Committee:

Dr. Abrham Gebre		
Internal Examiner	Signature	Date
Dr-Ing. Bedilu Habte		
External Examiner	Signature	Date
Dr. Asnake Adamu		
Advisor	Signature	Date

School Chairperson

UNDERTAKING

I certify that research work titled "**ECONOMIC DESIGN COMPARISON OF PRE-STRESSED CONCRETE SLAB & BOX-GIRDER RAILWAY BRIDGES**" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been properly acknowledged.

Name: Ibrahim Abdoulkader

Signature:

Place Addis Ababa University
Institute of Technology

Date of submission: June 15, 2017

To my parents for believing in me…

ABSTRACT

An investigation has been undertaken to study the design comparison of pre-stressed concrete Slab and Box-girder railway bridges based on economy of the structure.

Though, the Prestress concrete bridges are the most widely used bridges types, nowadays it’s an alternative solution for reducing the dead load of the structure and allows the possibility to build longer span bridges and to carry heavy loads.

The condition survey site visit during this study also reveals that railway bridges structure has to be built properly and the contractors have to carry about economy, efficiency of the structures without forgetting maintainability. Slab bridges are to be employed for span less than 15m, though not considered in the actual practice so far.

The objective of this study is to develop a reliable engineering Computer program using excel spread sheets template for analysis, design and cost estimate of superstructures of pre-stressed Slab and Box-girder railway bridges. Hence, identifying a clear demarcation of span length for selecting between Slab and Box-girder bridges on the basis of economy, happens to be found indispensable.

For the design of superstructure pre-tensioned system is used for Voided Slab bridges and post-tensioned for Box-girder bridges. A comparative study on the basis of cost of materials is conducted to select and identify the economical span range of Voided Slab and Box-Girder bridges. The developed program is illustrated using particular example of both Slab and Box-girder bridges.

The analysis results show that precast pre-stressed Voided Slab bridge is economical up to 15m and that of the Box-girder is economical beyond 15m.

Keywords: Voided, Slab, Box-girder, Economy, Excel, Pre-stressed, Bridge, Railway, Precast.

ACKNOWLEDGMENTS

First and foremost, I would like to thank the Almighty God for giving me the strength to carry on and show me the path of rightness in my life.

Further, I would like to express my most sincere gratitude to my supervisor Dr. Asnake Adamu. He has been a precious source of knowledge, guidance, and inspiration during this research. It has been a great honor for me to work with a world-class Doctor of his caliber, and this will remain as an unforgettable experience in my memory.

My sincere acknowledgement is to the Société Djiboutienne de Chemin de Fer (SDCF) and Ethiopian Railways Corporation (ERC) for their wonderful support, and to providing me a scholarship with covering all the expenses during my studies in Addis Ababa.

Special thanks to the staff in the Civil and Environmental Engineering Department for their helpful assistance.

Thanks to my Family and friends, whose constant love and prayers encouraged me to pursue this study.

I am indebted to my parents for always supporting and their continuous love, patience, encouragement, and motivation in this study and through all my life. I would like to dedicate this thesis to them.

I.A.M.K.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATIONS

A = gross area of Section

A_c = Total concrete area (m^2)

A_{gb} = gross area of the bearing plate

A_s = area of non-prestressed tension reinforcement

A_s * = area of prestressing steel

A_{sr} = steel area required to develop the compressive strength of the web of a flanged section

A_v = area of web reinforcement

B_w = Bridge width

b = width of flange of flanged member or width of rectangular member

b_w = web width

b' = width of a web of a flanged member

B = buoyancy

CF = centrifugal Force

C_{Rc} = loss of prestress due to creep of concrete

C_{Rs} = loss of prestress due to relaxation of prestressing steel

D = dead load

D = degree of curve

D_w = deck width

d = distance from extreme compression fiber to centroid of the prestressing force

d d = outside diameter of post-tensioning duct

d_t = distance from the extreme compressive fiber to the centroid of the non-prestressed tension reinforcement

E = earth pressure

E a = actual super elevation

e = base of Naperian logarithms

e p = prestressing strands eccentricity

e_{max} = maximum eccentricity

e_{min} = minimum eccentricity

E_c = modulus of elasticity of concrete

E_{ci} = modulus of elasticity of concrete at transfer

E_s = modulus of elasticity of prestressing steel

E_{si} = modulus of elasticity of steel reinforcement

ES = elastic shortening of concrete

f_b = bottom fiber stress in concrete

f_{br} = concrete bearing compressive strength

f'_c = compressive strength of concrete at 28 days

f'_{ci} = compressive strength of concrete at time of initial prestress

f_{cir} = average concrete stress at the center of gravity of the prestressing steel at time of release

f_{cds} = average concrete compressive stress at the center of gravity of the prestressing steel under full dead load

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads

f_{se} = effective prestress

Δf_s = total prsetress loss, excluding friction

f_t = top fiber stress in concrete

f_{fl} = friction loss in post tensioned members

f_{pe} = compressive stress in concrete due to effective prestress forces only

f_{po} = stress in the prestressing tendon at the jacking end

f_r = modulus of rupture of concrete

f'_s = ultimate strength of prestressing steel

f_{se} = effective stress prestress after losses

f_{sy} = yield strength of non-prestressed conventional reinforcement in tension

f^*_y = yield point stress of prestressing steel

f^*_{su} = average stress in prestressing steel at ultimate load

F = longitudinal force due to friction or shear resistance at expansion bearings

FR = friction loss

I = impact load

I_T = Second moment of area of T- section (mm4)

ICE = ice pressure

K = friction wobble coefficient

K_s = a constant for the determination of a stream pressure

L = the span length in meters

L_F = longitudinal force from live load

L = live load

M_{cr} = moment causing flexural cracking at section due to externally applied loads

$M_{d/nc}$ = non-composite dead load moment at the section

M_n = nominal moment strength of a section

p = ratio of non-prestressed tension reinforcements

P = Prestressing Force

p^* = A_s^*/bd ratio of prestressing steel

P_{avg} = average stream pressure

P_{max} = maximum stream flow pressure
P_{se} = Effective prestressing force
P_{v} = Vertical Prestressing Force at section considered
R = annual average ambient relative humidity
s = longitudinal spacing of the web reinforcement
S = permissible speed
SF = stream flow pressure
SH = loss of prestress due to concrete shrinkage
Tw = Track width
V_{avg} = average velocity of water
V_{c} = nominal shear strength provided by concrete
V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment
V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web
V_{d} = shear force at section due to unfactored dead load
V_{i} = factored shear force at section due to externally applied loads occurring
V_{p} = vertical component of effective prestress force at section
V_{s} = nominal shear strength provided by shear reinforcement
V_{u} = the factored shear force at the section considered
VI = Shear force due to Impact load (kN)
w = width of a section
w t = equivalent tendon load
W = wind load on structure
w c = unit density (weight) of concrete
W_L = wind load on live load
x = length of a prestressing tendon from the jacking end to point considered
y_{t} = distance of the top from the from centroidal of the section
y_{b} = distance of the bottom from centroidal axis of the section
Z_{b} = section modulus of bottom fiber
Z_{t} = section modulus of top fiber
C° = degree Celsius
ϕ = Strength Reduction Factor
α = total angular change of prestressing steel profile in radians from jacking end to point x
ß 1 = factor for concrete strength
μ = friction curvature coefficient

LIST OF ABBREVIATIONS

AASHTO:	American Association of State Highway Officials
ACI:	American Concrete Institute
AREMA:	American Railway Engineering and Maintenance-of-Way Association
ASTM:	American Society for Testing and Materials
CCECC:	China Civil Engineering Construction Corporation
CRCC:	China Railway Construction Corporation
ERC:	Ethiopian Railway Corporation
HSR:	Hight-Speed Rail
LFD:	Load Factor Design
LRT-AA:	Light Rail Transit of Addis Ababa

Chapter 1: Introduction

1.1 Background

When it comes to decision-making for infrastructural projects, it often tends to be the design alternative with the lowest investment cost that is adopted when it stands between a number of different possible options. The National new Addis Ababa ~ Djibouti network was built in standard gauge (1,435 mm). The Project was to be constructed from 2013 to 2017, 48 months in total, constructed at one time. Though, the total investment amount for such project is estimated about 5,047,356,900USD with an estimated cost per km of $6 million, [8]. A well-known fact more expensive in railroad investment are that Bridge and Tunnel structures, [7].

Lately the Ethiopian government has been spending more money on rail project whether for both the Federal railway and the LRT-AA because of need of reliable transportation systems in order to transport materials, finished goods, and people over greater distances in shorter times with lowest transport cost rather than plane and car.

The world’s first high-speed rail line, known as the Shinkansen, was built in 1964 between Tokyo and Osaka, Japan. The earliest routes, such as Tokyo-Osaka, cost less than $3.13 million per km but most newer routes cost at least $6.25 million per km to construct. From a financial standpoint, only two HSR lines in the world are profitable: Paris-Lyon in France and Tokyo-Osaka in Japan, [7]. European network used the standard gauge (1,435 mm) moreover the so-called HS2, between London and Edinburgh, is under study to introduce a new HSR track and the cost estimated per km is $74 million, [13].

However, the engineer must choose the shape he considers is best in all the circumstances. The decision is not an easy task. It depends on several criteria for example they are used to dealing with issues of performance, efficiency, cost, less maintenance costs and longer life cycle of the structures.

Thus, this research shall focus first on the selection of demarcation point according to the span length between Slab and Box-girder bridges which will save more money of the project cost and secondly it may affect upon construction time and finally gathering these two points it will influence on economies aspect of the structures which is selecting the most economical shape of the structure.

1.2 Objectives

The objective of present study is to compare Slab and Box-girder bridges, with parameter such as economy by using a reliability engineering Computer program. Though the aim of this thesis was to develop a Computer program using excel spread sheets template which is going to provide a curve of cost verses span length, the detail will show up a demarcation point and a most economical bridge between the two structures and also the economical approach of construction. Finally, an investigation on the condition survey site visit and brief discussion on findings is made to identify if the structures were made properly by the contractor and also to check the performance of the structure and the carrying capacity of the formwork equipment.

The research objectives are:

1. Analysis and design of Slab and Box-girder Railway following AREMA.
2. Make the manually computed routine calculations fast and as simple as possible.
3. Provide discussion on shortcoming in design and quality of construction on site findings.
4. Find out an accurate demarcation point with span length between Slab bridge and Box-girder bridge.

1.3 Scope and limitations of the Study

1.3.1 Scope

- This study cover computer program applicable for Analysis and Design of Slab and Box-Girder railway bridges for any span length and perform the selection of bridge type either Slab or Box-Girder based on economy.
- Furthermore, the school may use this for forthcoming research in line with upgrading the spreadsheet to handle many other aspects of the bridge design thus enlarges the scope.

1.3.2 Limitations of the study

The following parameters are only considered in this study.

- Prestressed post-tensioned system is used for Box-girder Bridge & prestressed pre-tensioned system for Slab Bridge.
- The cost analysis of bridges does consider only cost of construction without time costs.
- The superstructure cost of the bridge is only considered for economic design since it is assumed that the substructures are almost the same for both Slab and box-girder railway bridges for same span length.
- The program is developed for single span railway bridges and for single track.
- The loading condition is assumed to be the same for both structures with the same span length.
- The computation will proceed a span at time.
- Live load is assumed to be the same for both structures with same span length.

1.4 Outline of the thesis

To get a better overview of this thesis, a short description of the chapters is presented here.

The first chapter begins by shortly describing the general background of railway bridge economy, the objectives of the thesis, scope and limitations of the study and highlights the outcome of the study.

The second chapter concerns on the literature review where, advantages and disadvantages of prestressed concrete as well as prestressed concrete over reinforced concrete in addition to explaining the pre-tensioning and post-tensioning systems, economies of the structures, selecting Design standards more suitable for Designing, structure form and bridge type, problems associated with selection of bridge type, especially around the boundary, Bridges deck and materials, aspects for Railway Bridge construction.

The third chapter highlights the condition survey of selected bridges. It gives an overview of the condition of existing constructed bridges as per the discussion of site findings of the inspection carried.

The fourth chapter deals with the detailed Analysis and Design of Slab & Box-Girder bridges, with specific attention given to various assumption on loadings, material properties and design. Moreover, it also described the design specification to be considered during analysis and design of bridges as per the standards stated in the Design Manuals. The computer Program for Design of Slab and Box-Girder Bridges using “excel" software is presented in the annex. The flow charts and Algorithm for analysis and design of the two types of bridges under considerations are also presented in this chapter.

The chapter five outlines the Basics of Demarcation and practical example of different selected girder bridges. Furthermore, results and discussion are also included.

Last chapter gathers the conclusion and evokes possible recommendations for further work.

Chapter 2: Literature review

2.1 General

Economic design comparison analysis according to the shape of structure is very important factor because it may save money of the project cost.

In the 1930s, Eugène Freyssinet invented prestressed concrete. High tensile steel cables were substituted for the bars. These cables were tensioned by jacks and were then locked to the concrete. Thus, they compressed the concrete, ridding it of its cracks, improving both its appearance and its resistance to deterioration, [17].

Prestressing is more a philosophy than a specific technique. It means preparing a structure to receive a load by applying a pre-emptive countervailing load. For instance, if it is known that a column will be deflected 100 mm to the left by applied loads, the designer can arrange to bend it 50 mm to the right; the column then only has to be designed to resist a deflection of ± 50 mm, rather than the full 100 mm, [19]. Nowadays Prestressed technology is an alternative solution for reducing the dead load of the structure and allows the possibility to build longer span bridges.

During the last 25 years, prestressed concrete has been widely used for the construction of long-span bridges, industrial shell roofs, marine structures, nuclear pressure vessels, water-retaining structures, transmission poles, railway sleepers and a host of other structures, [17].

The search for economy in materials and in methods of construction (Pre-tensioned and Post-tensioned), in order to be able to resolve the tension between appearance and economy that is the source of creativity in the design of bridges.

So far, Prestressed technology are suitable for railway bridges and selecting an appropriate superstructure type is a critical factor in the planning and design process.

2.2 Advantage and Limitation of pre-stressed concrete

2.2.1 Advantage

The prestressing of concrete has several advantages as compared to traditional reinforced concrete (RC) without prestressing. A fully prestressed concrete member is usually subjected to compression during service life. This rectifies several deficiencies of concrete, [5].

1) Section remains uncracked under service loads

- Reduction of steel corrosion
  - Increase in durability.
- Full section is utilised
  - Higher moment of inertia (higher stiffness)
  - Less deformations (improved serviceability).
- Increase in shear capacity.
- Suitable for use in pressure vessels, liquid retaining structures.
- Improved performance (resilience) under dynamic and fatigue loading.
- Excellent fire resistance, low maintenance costs, elegance, high corrosion-resistance, adaptability.

2) High span-to-depth ratios

- Larger spans possible with prestressing (bridges, buildings with large column-free spaces)
- For the same span, less depth compared to RC member.
  - Reduction in self-weight
  - More aesthetic appeal due to slender sections
  - More economical sections.

3) Suitable for precast construction

The advantages of precast construction are as follows.

- Rapid construction
- Better quality control
- Reduced maintenance
- Suitable for repetitive construction
- Multiple use of formwork
  - Reduction of formwork
- Availability of standard shapes.

2.2.2 Limitations of Prestressing

Although prestressing has advantages, some aspects need to be carefully addressed.

- Prestressing needs skilled technology. Hence, it is not as common as reinforced concrete.
- The use of high strength materials is costly.
- There is additional cost in auxiliary equipment.
- There is need for quality control and inspection, [5].

2.3 Advantage of Prestressed Concrete over Reinforced Concrete

- In prestressed concrete member steel plays, active role. The stress in steel prevails whether external load is there or not. But in R.C.C., steel plays a passive role. The stress in steel in R.C.C members depends upon the external loads. i.e., no external load, no stress in steel.
- In prestressed concrete, the stresses in steel is almost constant where as in R.C.C the stress in steel is variable with the lever arm.
- Prestressed concrete has more shear resistance, whereas shear resistance of R.C.C is less.
- In prestress concrete members, deflections are less because the eccentric pre-stressing force will induce couple which will cause upward deflections, whereas in R.C.C deflections are more.
- In prestress concrete fatigue resistance is more compare to R.C.C. because in R.C.C. stress in steel is external load dependent where as in P.S.C member it is load independent.
- Prestress concrete is more durable as high grade of concrete is used which are denser in nature. R.C.C. is less durable.
- In prestress concrete dimensions are less because external stresses are counterbalance by the internal stress induced by prestress. Therefore, reactions on column & footing are less as a whole the quantity of concrete is reduced by 30% and steel reduced by about 60 to 70%. R.C.C. is uneconomical for long span because in R.C.C. dimension of sections are large requiring more concrete & steel. Moreover, as self-weight increases more reactions acted on columns & footings, which requires higher sizes, [22].

2.4 Pre-tensioning and Post-tensioning

There are two principal families of prestressed bridge decks systems: pre-tensioned and post-tensioned, [19].

- Pre-tensioning involves tensioning the cables before the concrete is cast. The cables are anchored to a strengthened mould. Once the concrete has hardened, the cables are released, and maintain their tension by their adherence to the concrete. This technique is used principally for the construction of relatively short span bridge decks using standard bridge beams.
- Post-tensioning involves first placing of ducts cables with unstressed tendon in the forms & then casting the concrete deck, which are then, stressed by proprietary jacks reacting against the concrete. The cables are then locked to the concrete by anchors wedges or nuts. This technique is ideally suited for medium to long-span in situ work where the tensioning cost is only a small proportion of the cost of the hole job.

In this work, the Pre-tensioning system is used for Slab bridge while Post-tensioning is used for Box-girder. These systems are respectively chosen on basis of a short literature study where handbooks and articles concerning the design structures.

- ➢ The following equation of sling load for lifting Precast bridge shall be considered as follow:

To figure out the stress that’s on each of this sling:

Reach at 90 degrees: $L = D$ (2.1)

Reach at 60 degrees: $L = 2 \times D$ (2.2)

Reach at 45 degrees: $L = 1.4 \times D$ (2.3)

$L/H = X$ (2.4)

$X*[(1/2) * W]$ = Stress on each sling (2.5)

1. Vertical Safe working load (SWL)
2. Weight (Total weight of bridge) (W)
3. Length of the reach (L)
4. Height (H)
5. D is the width of the structure
6. X results of equation

Finally, compare the result (stress on each sling) with safe working Load at (90°, 60°, 45°) because it's very important to make sure that strap is able and capable of lifting the amount that we calculate.

2.5 Economies of the Structures

Structural types, span lengths, and materials shall be selected with due consideration of projected cost. The cost of future expenditures during the projected service life of the bridge should be considered. Regional factors, such as availability of material, fabrication, location, shipping, and erection constraints, shall be considered, [3].

The use of precast, prestressed concrete products for the construction of bridges results in very economical, high quality structures. This is due to several factors, [18]:

- Mass production of standardized, low maintenance sections
- A factory environment that includes quality control
- Speed of erection and construction
- The use of high quality, inexpensive and locally available materials for production.

Prestressed members are shallower in depth than their reinforced concrete counterparts for the same span and loading conditions. In general, the depth of a prestressed concrete member is usually about 65 to 80 percent of the depth of the equivalent reinforced concrete member. Hence, the prestressed member requires less concrete, and about 20 to 35 percent of the amount of reinforcement. Unfortunately, this saving in material weight is balanced by the higher cost of the higher quality materials needed in prestressing. Also, regardless of the system used, prestressing operations themselves result in an added cost; formwork is more complex, since the geometry of prestressed sections is usually composed of flanged sections with thin webs, [11].

In instances where economic studies do not indicate a clear choice, the Owner may require that alternative contract plans be prepared and bid competitively. Designs for alternative plans shall be of equal safety, serviceability, and aesthetic value, [3].

Economy in this context is not simply saving money; it is a concept of rationality and frugality. It is fundamental to engineering design that the designer is constantly planning how he can save materials, and how he can make the construction process simpler, even if many of these design decisions in isolation would not register on the overall balance sheet of a project, [19].Consequently, although the search for economy is at the heart of this design, it cannot be used as an alternative to aesthetic judgement; the engineer must choose the shape he considers is best in all the circumstances, [19].

2.6 Design Standards

Engineers are using various national standards to achieve an optimum design. Some countries are using their own codes to achieve better economy and better standards. Design decisions is to ensure reliability and durability during their service life, regular service and maintainability. The design standards provide minimum design program such as minimum dimension, concrete grade, steel ratio, loadings, load factors, Flexural strength, etc.

2.6.1 Computation of Loads based on the Chinese Standard

1. Dead Loads: They are volumetric in nature and are prominently due to the mass and the action of gravity. For computation of dead loads the Chinese standard uses the following unit weights for common materials.

Table 2.1 Unit Weight of common construction material

MATERIAL	KN/m^3
Steel	78.5
Cast iron	72.5
Lead	114
Reinforced concrete (reinforcement ration is within 3%)	25
Concrete and rubble concrete	23
Mortar rubble ashlars	25
Mortar block stone	23
Mortar rubble	22
Dry rubble	20.2
Earth fill	17
Rock fill (make use of spoil)	19
Crushed stone ballast	21
Cast asphalt	15
Compacted asphalt	20
Untreated timber	7.5
Treated timber	9

2. Live load actions: The model used in PRC for computation of railway live load effects is:

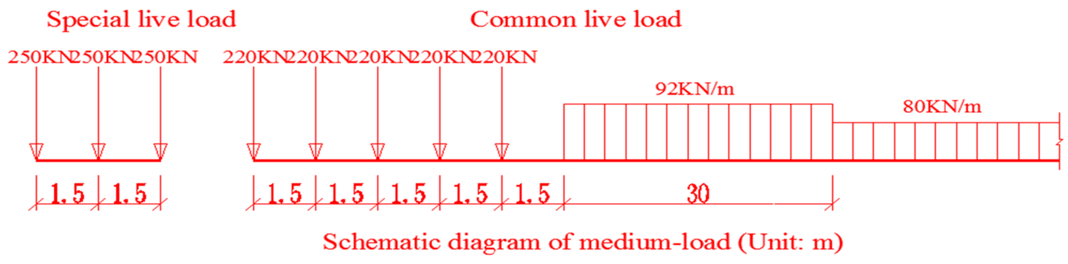


Figure 2.1: Graphic equation of Chinese Standard Railway Live Load

2.6.2 Computation of Loads based on the AREMA Standard

For computation of dead loads the AREMA Manual uses the following unit weights for common materials,

1. The unit weight of materials comprising the dead load, except in special cases involving unusual conditions or materials, shall be assumed as follows:

Table 2.2: Unit Weight of common construction material

MATERIAL	WEIGHT	UNIT
Track rails, guardrails, fastenings	3	kN / m
Ballast	19	kN / m^3
Reinforced concrete	24	kN / m^3
Waterproofing	0.90	kN / m
Catenaries	0.02	kN / m
Future utilities	0.05	kN / m

2. Live Load Computations in AREMA

The recommended live load model for main line structures is given below with the axel spacing shown in the fig. below:

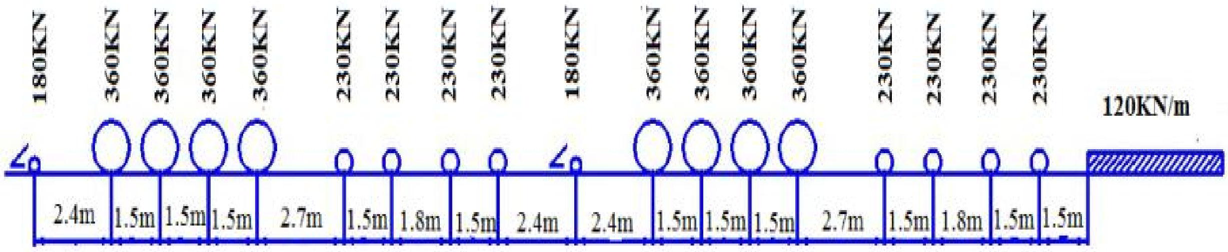


Figure 2.2 Cooper E 80 (EM 360) Axle Load Diagram

From the above comparison of Chinese and AREMA standards it is observed that they have almost same unit weight of materials comprising dead load. Whereas, the Live Load Computations in AREMA which is Cooper E 80 has eighteen axle load and Chinese standard has only eight axle loads.

It is noted that AREMA standard is more applicable than Chinese because it has easiest tabular analysis axle load to use. Perhaps it’s uneconomical because it is conservative with many axles loads but it guaranteed safety for example if Ethiopian government want to increase in the Future their importation from Djibouti port that’s mean increasing load to carry upon railway bridges or maybe other trains heavier may show up in the upcoming years which is what it will be wise to consider this all things in order to provide a better and high structural efficiency railway bridges design.

Finally, AREMA Standard provides all the necessary procedural guidance, dimensions, and materials and allow the designer to develop the solution of the problem rather than other standards. thus, AREMA standard will be followed in this work.

2.7 Structure and Bridge Types

2.7.1 Structure Type of bridges

Bridges may be classified by how the forces of tension, compression, bending, torsion and shear are distributed through their structure. Most bridges will employ all of the principal forces to some degree, but only a few will predominate. The separation of forces may be quite clear, [23].

- Beam:

Beam bridges are horizontal beams supported at each end by substructure units and can be either simply supported when the beams only connect across a single span, or continuous when the beams are connected across two or more spans, [23].

- Truss:

A truss bridge is a bridge whose load-bearing superstructure is composed of a truss. This truss is a structure of connected elements forming triangular units, [23].

- Cantilever:

Cantilever bridges are built using cantilevers-horizontal beams supported on only one end. Most cantilever bridges use a pair of continuous spans that extend from opposite sides of the supporting piers to meet at the center of the obstacle the bridge crosses, [23].

- Arch:

Arch bridges have abutments at each end. The weight of the bridges is thrusted into the abutments at either side, [23].

- Suspension:

Suspension bridges are suspended from cables. In modern bridges, the cables hang from towers that are attached to caisssons or cofferdams. The caisssons or cofferdams are implanted deep into the bed of the lake, river or sea, [23].

- Cable-stayed:

Cables- stayed bridges, like suspension bridges, are held up by cables. However, in a cable-stayed bridge, less cable is required and the towers holding the cables are proportionately higher, [23].

2.7.2 Bridge Types

- **Slab bridge**

Slab bridges are under-used principally because of the lack of refinement of the preliminary costings carried out by most contractors' estimators. The unit costs of formwork, concrete, reinforcement and prestress tendons should clearly be lower for a solid slab deck than for more complex cross sections such as voided slab or box girder decks. Contractors who have experience of building prestressed concrete solid slabs and who have evidence for the low costs and for the reliability and speed of the construction process, tend to use such structures more readily. Slabs allow the designer to minimise the depth of construction and provide a flat soffit where this is architecturally desirable. Their use is limited principally by their high self-weight. For the designer, the main difference between prestressed and reinforced concrete solid-slab bridge decks is that the self-weight deflections of the former are entirely cancelled by the effect of the prestress; fully prestressed slabs deflect upwards slightly under long-term permanent loads. Prestressed concrete slabs are very valuable for bridge decks where the supports are skew or for irregularly shaped decks where the columns do not define organised spans. As the prestress opposes the deflections due to permanent loads, the transverse bending moments and torques due to these loads are also eliminated or reversed. An additional advantage of slender prestressed continuous slabs is their great flexibility. In areas where settlement of supports is to be feared, either through mining subsidence, variable ground or due to settlement of the abutments under the weight of fill, these slabs may be designed to accept large movements without the need to resort to hinged structures, [19].

Although the cast-in-place prestressed slab is more expensive than a reinforced concrete slab, the precast prestressed slab is economical when many spans are involved. Common spans range from 6 to 15 m, [20].

- **T-girder**

The T-beam construction consists of a transversely reinforced slab deck which spans across to the longitudinal support girders. These require a more-complicated formwork, particularly for skewed bridges, compared to the other superstructure forms. T-beam bridges are generally more economical for spans of 12 to 18 m.

Prestressed concrete T-beams with depths from 1.75 to 2.6m are applicable for spans from 16.5 to 27.6m for railway Bridge. The girder stem thickness usually varies from 35 to 55 cm and is controlled by the required horizontal spacing of the positive moment reinforcement. Optimum lateral spacing of longitudinal girders is typically between 1.8 and 3.0 m for a minimum cost of formwork and structural materials. However, where vertical supports for the formwork are difficult and expensive, girder spacing can be increased accordingly, [20].

The T-girder has a big disadvantage because it has no bottom flange with which to deal with tensile forces and it isn’t appropriate for curved structures or skews.

- **Box-girder**

The box girder is the most flexible bridge deck form. It can cover a range of spans from 25 m up to the largest non-suspended concrete decks built, of the order of 300 m. Single box girders may also carry decks up to 30 m wide. For the longer span beams, beyond about 50 m, they are practically the only feasible deck section.

The advantages of the box form are principally its high structural efficiency, which minimises the prestress force required to resist a given bending moment, and its great torsional strength with the capacity this gives to re-centre eccentric live loads, minimising the prestress required to carry them. One of the main disadvantages of box decks is that they are difficult to cast in-situ due to the inaccessibility of the bottom slab and the need to extract the internal shutter. Either the box has to be designed so that the entire cross section may be cast in one continuous pour, or the cross section has to be cast in stages, [19]. For a box section deck of constant depth, it is likely to lie between 30 m and 45 m, while if the deck is provided with a haunch, the most economical span will lie between 40 m and 60 m, [19]. Precast pre-stressed box girder sections are used frequently for simple spans of over 30 m and are particularly suitable for widening bridges to control deflections, [20].

This study shall focus only on design of prestressed concrete precast pre-tensioning Slab & Post-tensioning Box-girder Railway Bridges. First because the Box-girder has more advantage which is more Flexible, suitable on curve with high structural efficiency and its great in torsional strength, and most economical for longer span, though prestressed concrete slabs girder are very valuable for bridge decks where the supports are skew or for irregularly shaped decks, more flexibility, economical for short span and suitable on curve rather than prestressed T-girder bridges.

2.8 Form of Prestressed Slab & Box-Girder Railway Bridges

Bridge designers have a variety of options available when considering the advantages of precast concrete construction.

2.8.1 Form of Prestressed Slab Girder

In general terms, prestressed concrete decks become more economical as they become deeper, with span/depth of 15–18 being typical. The solid slab is the only exception to this rule; its economy derives from its extreme simplicity. However, when a prestressed solid slab exceeds a thickness of 700 mm, providing a span of up to 23 m, its weight starts to become excessive, and the designer should consider alternative deck types. The most logical alternative is to abandon the slab form and adopt a ribbed slab deck. The depth of such a deck is likely to increase substantially, a 23m span requiring a depth generally in excess of 1.15 m. However, the depth allowable may be limited, or a flat soffit may be required. For these special cases, a voided slab deck may well be suitable. Voids may be circular, quasi-circular such as octagonal, or rectangular. The diameter of circular voids should be a maximum of 240 mm less than the deck thickness, and preferably 300 mm, and not greater than 0.7 of the deck depth. The ribs between voids should generally be 250–300 mm wide. The voids are usually stopped short of the lines of piers, abutments and construction joints to create incorporated crossbeams. Intermediate crossbeams are not required in such decks. Skews may be easily accommodated, [19]. Voided bridges have shown a tendency to crack along the line of the voids, principally on the bridge soffit. This is due either to a heat of hydration effect, or more probably, to the expansion of the void due to this heat. In order to prevent this, it is necessary to reinforce the concrete above and below the voids, [19].

Concreting voided decks requires more care than for solid slabs, as there is the risk of incomplete filling and compaction beneath the voids. Consequently, it is a slower and more labour-intensive operation. The normal method for decks with circular voids is to work across the deck, filling one rib and observing the concrete flowing out from beneath the adjacent void, [19].

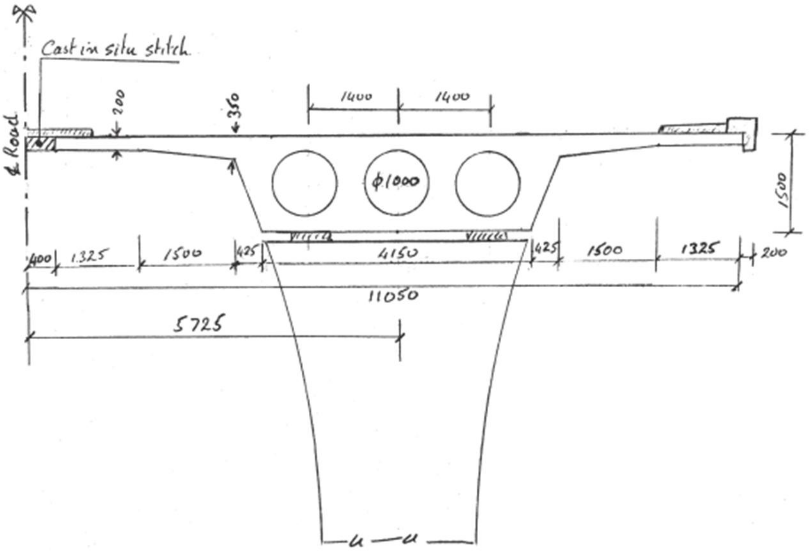


Figure 2.3 Voided ribbed slab deck (Source: Robert Benaim, [19].).

2.8.2 Form of Prestressed Box Girder

A box section deck consists of side cantilevers, top and bottom slabs of the box itself and the webs. For a good design, there must be a rational balance between the overall width of the deck, and the width of the box. Box sections suffer from a certain blandness of appearance; the observer does not know whether the box is made of an assemblage of thin plates, or is solid concrete, [19]. Box girder can be constructed as single cell, double or multicellular it may be monolithically constructed with the deck, called closed box girder or the deck can be separately constructed afterwards called open box girder and it can be rectangular, trapezoidal and circular means the girder web can be vertical or inclined which reduces the width of the bottom flange, [20].

Boxes may be rectangular or trapezoidal, with the bottom flange narrower than the top. Rectangular box sections are easier to build, and are virtually essential for the longest spans due to the great depth of the girders. However, they have the disadvantages that their appearance is somewhat severe, and that their bottom slabs may be wider than necessary, [19].

The trapezoidal cross section is frequently economical as well as good looking. In general, the width of the top of the box is determined by the need to provide points of support to the top slab at suitable intervals. The cross-section area of the bottom slab is logically determined at mid-span by the need to provide a bottom modulus sufficient to control the range of bending stresses under the variation of live load bending moments. For a box of rectangular cross section of span/depth ratio deeper than about 1/20, the area of bottom slab is generally greater than necessary, resulting in redundant weight. Choosing a trapezoidal cross section allows the weight of the bottom slab to be reduced. Close to the piers, the area of bottom slab is determined by the need to limit the maximum bending stress on the bottom fibre and to provide an adequate ultimate moment of resistance. If the narrow bottom slab defined by mid-span criteria is inadequate, it is simple to thicken it locally. For a very wide deck that has a deep span/depth ratio, this logic may give rise to webs that are inclined at a very flat angle. The designer should be aware of the difficulties in casting such webs, and make suitable allowances in specifying the concrete and in detailing the reinforcement.

Also, an important consideration in the design of box section decks is the distortion of the cross section under the effect of eccentric live loads. The effect of this distortion is reduced in a trapezoidal cross section. Boxes may have a single cell or multiple cells. it was explained how important it is for economy to minimise the number of webs. Furthermore, it is more difficult to build multi-cell boxes, and it is worthwhile extending the single-cell box as far as possible before adding internal webs, [19]. Once the span of a box section deck exceeds about 45 m, it becomes relevant to consider varying the depth of the beam. This is not an automatic decision as it depends on the method of construction. Clearly, this decision also has an aesthetic component. The depth may be varied continuously along the length of the beam, adopting a circular, parabolic, elliptical or Islamic profile. These attributes depend on the quality of the conceptual design. Economy in this context is not simply saving money; it is a concept of rationality and frugality. It is fundamental to engineering design that the designer is constantly planning how he can save materials, and how he can make the construction process simpler, even if many of these design decisions in isolation would not register on the overall balance sheet of a project, [19].

Therefore, my design shall focus only on Analysis and Design of precast prestressed concrete Rectangular Box-Girder who are easier to build, and are virtually essential for the longest spans due to the great depth of the girders and Voided Slab bridge who are very valuable for bridge decks where the supports are skew or for irregularly shaped decks where the columns do not define organised spans or when the deck weight starts to become excessive in addition of this both structures are suitable in torsion field.

2.9 Selection of bridges

Bridge design is a complex engineering problem, [17]. Generally, all the factors are related to the economy, safety, and aesthetics. The design process includes consideration of other important factors, such as choice of bridge system, materials, dimensions, foundations, aesthetics, and local landscape and environment, [17].

So, that sometimes the selection is complicated by other consideration such as, [17]:

- The deflection limits.
- Life cycle cost.
- Maintenance cost
- The unit costs of formwork
- The reliability and speed of the construction
- Seismicity at the site.
- Suitability for future widening.

Furthermore, the choice of span length can also be affected by the cost of substructure units. Where the foundation conditions are poor or the piers are tall, it could be more economical to use longer spans. The choice of span length should result from the lowest combined cost of the superstructure and substructure, [18].

2.10 Bridges deck

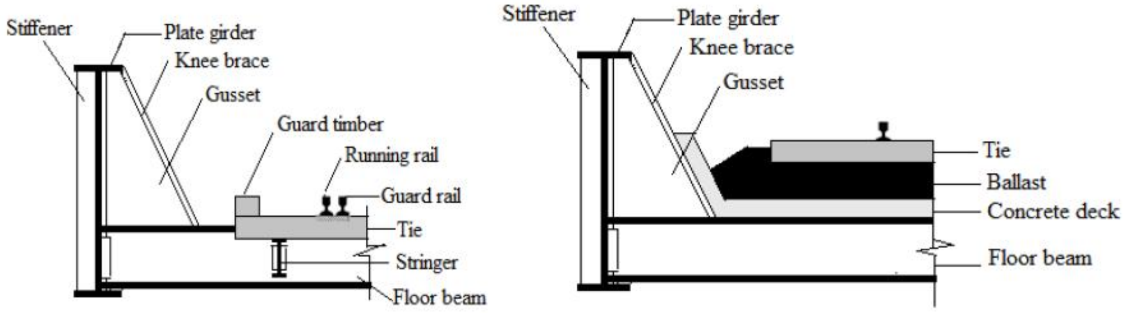
The engineer experienced in highway bridge design may not think of the typical railway bridge as having a deck. However, it is essential to have a support system for the rails. Railway bridges typically are designed as either open deck or ballast deck structures, [20].

2.10.1 Open Deck

Open deck bridges have ties supported directly on load-carrying elements of the structure (such as stringers or girders). The dead loads for open deck structures can be significantly less than for ballast deck structures. Open decks, however, transfer more of the dynamic effects of live load into the bridge than ballast decks. In addition, the bridge ties required are both longer and larger in cross section than the standard track ties. This adds to their expense. Bridge tie availability has declined, and their supply may be a problem, particularly in denser grades of structured timber, [20].

2.10.2 Ballast Deck

Ballast deck bridges have the track structure supported on ballast, which is carried by the structural elements of the bridge. Typically, the track structure (rails, tie plates, and ties) is similar to track constructed on grade. Ballast deck structures offer advantages in ride and maintenance requirements. Unlike open decks, the track alignment on ballast deck spans can typically be maintained using standard track maintenance equipment. If all other factors are equal, most railroads currently prefer ballast decks for new structures. In ballast deck designs, an allowance for at least 6 in. of additional ballast is prudent. Specific requirements for additional ballast capacity may be provided by the railroad. In addition, the required depth of ballast below the tie should be verified with the affected railroad. Typical values for this range from 8 to 12 in. or more. The tie length used will have an effect on the distribution of live-load effects into the structure. Ballast decks are also typically waterproofed. The weight of waterproofing should be included in the dead load, [20].



(a) (b)

Figure 2.4: (a) Open-deck through-plate girder (b) Ballasted-deck through-plate girder. (source: Robert Benaim, [19].)

2.11 Materials

The concrete materials shall be selected for strength, durability and chemical resistance, and ability to attain specified properties as required, in accordance with this recommended practice and as approved by the Engineer. They shall be combined in such a manner as to produce uniformity of color and texture in the surface of any structure or group of structures in which they are to be used. No change shall be made in the brand, type, source or characteristics of cementitious materials, the character and source of aggregate or water, or the class of concrete and method of transporting, placing, finishing or curing without approval of the Engineer, [4].

2.11.1Concrete

High strength concrete is used, concrete grade larger than C-30 class I works High compressive strength at a reasonably early age, & comparatively higher tensile strength as compared with ordinary RC member, low shrinkage, minimum creep characteristics and high young’s modulus are necessary, [6].

Concrete grade larger than, [6].

- C-40, for pre-tensioned,
- C-30, for post-tensioned members

2.11.2 Reinforcement

- **Pre-stressing Steel**

The members prestressed with wire, strands, or bars shall conform to one of the following specifications, [4]:

- Standard Specification for Steel Strand Uncoated Seven-Wire for Prestressed Concrete (ASTM A416).
- Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete (ASTM A421).
- Standard Specification for Uncoated High-Strength Steel Bar for Pre-stressing Concrete (ASTM A722).

➢ **Reinforcing Steel**

Non-prestressed reinforcement generally consists of deformed bars or welded wire reinforcement, [14].

Design shall not be based on a yield strength fy in excess of 60.000 Psi (420Mpa), [4].

Bars, wire, welded wire fabric, prestressing tendons, structural steel, steel pipe and tubing shall conform to one of the ASTM specifications found in Table below, [4].

Table 2.3: Recapitulative of ASTM Specifications for Reinforcement [AREMA, Table 8-1-7]

Type	Designation
Bars, Wire and Fabric	
Deformed and Plain Billet-Steel Bars	A615
Deformed and Plain Low-Alloy Steel Bars	A706
Deformed Rail-Steel and Axle-Steel Bars	A996
Deformed and Plain Stainless-Steel Bars	A955
Welded or Forged Headed Bars	A970
Steel Wire, Plain (wire shall not be smaller than size W4 (0.226 inch (5.74 mm) dia.))	A82
Steel Welded Wire Fabric, Plain	A185
Steel Wire, Deformed (wire shall not be smaller than size D4 (0.225 inch (5.72 mm) dia.))	A496
Steel Welded Wire Fabric, Deformed (welded intersections shall not be spaced farther apart than 16 inches (400 mm) in direction of primary flexural reinforcement)	A497
Prestressing Tendons	
Uncoated Seven-Wire Steel Strand	A416
Uncoated Stress-Relieved Steel Wire	A421
Uncoated High-Strength Steel Bar	A722
Structural Steel, Steel Pipe and Tubing	
Structural-Steel	A36, A242, A529, A572, A588 or A709 (Grade 36, 50 or 50W)
Steel Pipe	A53 (Grade B)
Steel Tubing	A500, A501 or A618

Chapter 3: Condition Survey of Selected Bridges

3.1 Condition Survey Site Visit

Although in Ethiopia railway bridges are freshly built it seems crucial to investigate on it, so as to ensure public safety and to protect public investment. The basic theory of condition survey in this work is to identify if the structures were made properly by the contractor and also to check the performance of the structure and the carrying capacity of the formwork equipment.

The National existing Addis Ababa ~ Djibouti New Standard-gauge Railway line with electric traction has a total length of 781km, [8], whereas the Addis Ababa Light Rail Transit is a standard gauge with a total length of 34.25 km double-track electrified, [12].

In fact, the area of visited site was located within Addis Ababa City exactly the Addis Ababa LRT bridges around St. Stefanos to St. Lideta and other area was located in the vicinity of Djibouti port for the Federal railway bridges line was also visited. However, from Weldia-Mekele railway project a drawing picture data is presented in the underneath because it’s currently under construction.

Condition Survey has been carried in order to assess the practice in selecting appropriate type of bridge structure, which is related to the study work.

Brief surveyed summary and condition of The Federal and LRT-AA line is provided below.

3.1.1. LRT St. Estfanos Bridge

Bridge Name:	LRT Bridge Saint Estefanos
Crossing:	Highway of Africa Road
Bridge Location:	Around St. Estfanos station
Bridge Type:	Precast Prestressed Simply support Post-Tensioned Box-Girder
Track Type:	Double Track
Form:	2cells Box-Girder
Number of spans:	1
Span Length:	30 m
Distance between Centers track:	4 m
Deck Type:	Ballastless Track
Deck width:	9.20 m
Deck Thickness:	0.20 m
Girder Depth:	1.80 m
Present Status:	In good condition
Abutments:	None
Piers:	Reinforced concrete pier
Bearings Type:	Not accessible
Problem observed:	None

Figure 3.1: Saint Stefanos Bridge

3.1.2. LRT Leghar Bridge

Bridge Name:	LRT Leghar Bridge
Crossing:	Highway
Bridge Location:	Around Leghar Station
Bridge Type:	Precast Prestressed Simply support Post-Tensioned Box-Girder
Track Type:	Double Track
Form:	single cell Box-Girder (for each track)
Number of spans:	1
Span Length:	20 m
Distance between Centers track:	4m
Deck Type:	Ballastless Track
Deck width:	7.14 m
Deck Thickness:	0.30 m
Girder Depth:	1.60 m
Present Status:	In good condition
Abutments:	None
Piers:	Reinforced concrete pier
Bearings Type:	Not accessible
Problem observed:	Short span length for Box-girder Bridge.

Figure 3.2: Leghar Bridge

3.1.3. LRT Tegbared Station Bridge

Bridge Name: LRT Tegbared Station Bridge
Crossing: Highway
Bridge Location: Around Tegbared Station
Bridge Type: Refuge Platform for sidewalk under stairs
Deck Type: Ballastless Track
Present Status: In good condition
Piers: Reinforced concrete pier
Problem observed: Water leakage, Corrosion of reinforced bars and chloride attack.

Figure 3.3: Tegbared Station Bridge

3.1.4. LRT Tegbared Bridges

Bridge Name: LRT Tegbared Bridges
Crossing: Highway
Bridge Location: Around Tegbared Station
Bridge Type: Precast Prestressed Simply support Post-Tensioned Box-Girder

Track Type: Double Track
Form: single cell Box-Girder (for each track)
Number of spans: Many simply support
Span Length: 30m (each span)
Deck Type: Ballastless Track
Deck width: 9.20 m
Deck Thickness: 0.20 m
Girder Depth: 1.80 m
Present Status: In good condition
Piers: Reinforced concrete pier
Problem observed: There is many reinforced concrete piers with short span length of simply support Post-Tensioned Box-Girder.

Figure 3.4: Tegbared Bridges

3.1.5. LRT St. Lideta Bridge

Bridge Name: LRT St. Lideta Bridge
Crossing: Highway
Bridge Location: Around St. Lideta
Bridge Type: Precast Prestressed Simply support Post-Tensioned Box-Girder
Distance between Centers track: 4m
Track Type: Double Track
Form: 2cells Box-Girder
Number of spans: 1
Span Length: 30 m
Deck Type: Ballastless Track
Deck width: 9.20 m
Deck Thickness: 0.20 m
Girder Depth: 1.80 m
Present Status: In good condition
Abutments: None
Piers: Reinforced concrete pier
Problem observed: The location of drain pipe isn’t properly adjusted according to its location of PVC pipe so there is water leakage on deck and pier.

Figure 3.5: Saint Lideta Bridge

3.1.6. LRT Tegbared Bridge

Bridge Name: LRT Tegbared Bridges
Crossing: Highway
Bridge Location: Tegbared Station
Bridge Type: Precast Prestressed Simply support Post-Tensioned Box-Girder.
Track Type: Double Track
Form: Single cell Box-Girder (for each track)
Distance between Centers track: 4m
Deck Type: Ballastless Track
Deck width: 9.20 m
Deck Thickness: 0.20 m
Girder Depth: 1.80 m
Present Status: In good condition
Piers: Reinforced concrete piers
Problem observed: Water leakage, chloride attack and water stagnation stains.

Figure 3.6: Tegbared Bridge

3.1.7. National Bridge of route d'Arta (RN1)

Bridge Name: Route d'Arta (RN1)
Crossing: Highway of Route d'Arta (RN1).
Bridge Location: Near Camp Cheik Osman in the north of Nagad Station.
Bridge Type: Precast Prestressed Simply support Post-Tensioned T-Girder
Track Type: Single Track
Number of spans: 3
Span Length: 32m (each span)
Deck Type: Ballast Track
Deck width: 4.9 m
Deck Thickness: 0.30 m
Girder Depth: 1.5 m
Present Status: In good condition
Abutments: Stone masonry
Piers: Reinforced concrete piers
Bearings Type: Elastomeric Bearing
Problem observed: Short Girder depth

Figure 3.7: Route d'Arta bridge.

3.1.8. National Bridge of Hayableh

Bridge Name: Bridge of Hayableh
Crossing: Highway of Hayableh.
Bridge Location: about 3.8km of Doraleh Multi-Purpose Port of Djibouti
Bridge Type: Precast Prestressed Simply support Post-Tensioned T-Girder
Track Type: Single Track
Number of spans: 3
Span Length: 32m (each span)
Deck Type: Ballast Track
Deck width: 4.9 m
Deck Thickness: 0.30 m
Girder Depth: 1.5 m
Present Status: In good condition
Abutments: Stone masonry
Piers: Reinforced concrete piers

Bearings Type: Elastomeric Bearing
Problem observed: Short girder depth.

Figure 3.8: Hayableh Bridge.

3.1.9. <u>Federal Bridge of Weldia-Mekele section</u>

Bridge Name: Aroley Major bridge
Crossing: River
Bridge Location: In Weldia-Mekele section about 60km from Mekele.
Bridge Type: Precast Prestressed Simply support Post-Tensioned T-Girder
Track Type: Single Track
Number of spans: 10
Span Length: 8N° 32m span & 2N° 24m span
Deck Type: Ballast Track
Deck width: 4.9 m
Deck Thickness: 0.30 m
Girder Depth: 1.5 m
Abutments: Stone masonry
Piers: Reinforced concrete piers
Bearings Type: Elastomeric Bearing
Problem observed: T-Girder is used on curve with short girder depth.

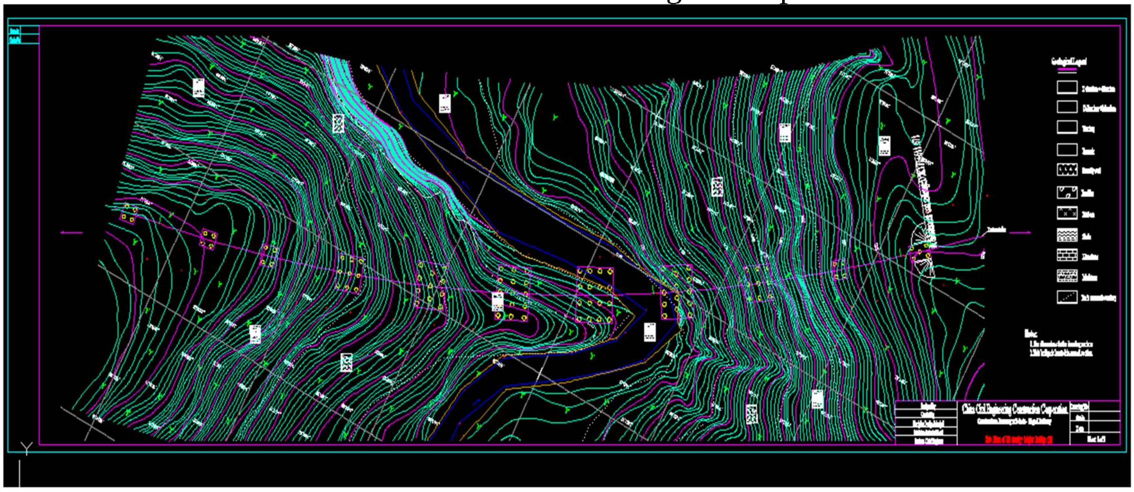


Figure 3.9: Drawing of Aroley Major bridge (*Source*: ERC).

Photo: Not Attached but drawing picture is presented.

3.2 Discussion

3.2.1 Design & Construction

According on the site survey and discussion with the ERC Professional Engineer on how designs of bridges have been carried are described below.

Case 1: Information regarding the Federal Railway of Djibouti-Addis Ababa line.

- China railway standard is adopted for Bridge Design.
- The railway line section from Sebeta-Mieso is characterized by Double-Track and Single-Track is adopted for Mieso to Djibouti Doraleh port.
- Ballast Deck is adopted For the National railway of Djibouti-Addis Ababa line.
- New culvert or Bridge for flood drainage is generally determined by flow size.
- The bridges are to cross various levels of highways, local roads or the area with local road planning requirements are required to meet the planning requirements, and sufficient clearance shall be reserved. When bridges are subject to the flyover, the R.C. frame bridge or other bridge structures are used to achieve the clear height or span.
- Simply-supported T-beams are collectively prefabricated and erected with bridge erection equipment; special structures are constructed with cast-in-situ or cantilever casting construction method and connection of decks will be cast in-situ concrete.
- In combination of the design and construction experience of railway bridges in China by CCECC and CREC main Contractors, the pre-stressed concrete simply-supported T-shaped beam by post-tensioning are mainly adopted for the whole route.
- Beam selection: 24m and 32m pre-stressed concrete simply-supported T beam is adopted as the priority for the superstructure of bridge for the whole railway line. It should be designed as mixed passenger and cargo line.
- For the bridge with span below 20m, the frame structure should be preferably used according to landform, geology, elevation of the structure, construction conditions.

- Based on engineering conditions of this line, solid or hollow pier with round-ended shape and T-type abutment are adopted.

Case 2: Information regarding The Addis Ababa Light Rail Transit Line.

- China railway standard is adopted for Bridge Design.
- Ballastless Deck is used for AA-LRT.
- The pre-stressed concrete simply-supported post-tensioning Box-girder are mainly adopted on the AA-LRT, some Box-girder are constructed continuously supported when the passageway is obstructed by curve, or in consideration of the topography. Basically, it's characterized by safe, simple structure, economical, standardized fabrication, easy centralized pre-fabrication, convenient construction and easy maintenance over the rest bridge types like T-girder, truss girder, arch, cable stayed. Topography, traffic flow and convenient of equipment construction are also considered in bridge Design.
- The Box-girder railway elevated structures were manufactured as pre-cast and assembled in the construction site with epoxy joint.
- Beam selection: depending on the alignments, straight and curve. 20m and 30m simply supported pre-stressed concrete Box-girder and 30m continuously supported pre-stressed concrete Box-girder are adopted as the priority for the superstructure of AA-LRT bridges.

3.2.2 Discussion of Site Findings

During the investigation, the followings were observed:

1. In the National railway line, a Simply-supported T-girder bridges are actually under construction and it constructed on curve way, at the Aroley in Weldia-Mekele section about 60km from Mekele crossing river. The relevant reason why these types of bridges are selected instead of the other is because they gave only the machinery erection equipment of T-shaped girder by post-tensioning for the whole route in order to save transportation costs from China to Ethiopia and also for trim transportation costs inside the city from the manufactory to the construction site and finally to decreasing time cost.

2. In the Federal railway line, bridge is a structure providing passage over an obstacle for example a short river with 10 m of length however the contractors gave only the machinery erection equipment of 24m and 32m pre-stressed concrete simply-supported T beam so they digging the river in order to increase the size of the obstacle and then put down the beam.

3. In the AA-LRT bridges project, there are many piers closely nearby each other because of the short span length of Box-girder who is 20m. consequently it may affect the economy structure.
4. Special finishing is not considered in the Design of the AA-LRT bridges i.e. elevator doesn’t work since the starting work of the AA-LRT until now and also bridge decks and approach structures don’t meet quality standards etc.…
5. During the protective layer construction, 2% horizontal drainage slope isn’t adjusted to the location of drain pipes as mentioned in the project that’s why we observe stagnation of rain water on deck and leakage mostly in rainy season. Thus, it will affect on the design durability of service life. Adequate superstructure drainage must be provided, and must ensure chloride-laden water does not come in contact with the structure.
6. Stain of chloride on deck and corrosion has been found on the AA-LRT bridges. They have to be regularly inspected and appropriately maintained because many prestressed concrete bridges have been found deteriorated due to this factor.

A survey performed in this study showed an overview of the actual new modern railway line of Ethiopia. Thus, the predominant type of bridge for the National railway is the T-girder whereas for the Addis Ababa Light Rail Transit (AA-LRT) is the Box-girder. Designing alone is not enough but they have to be regularly inspected and appropriately maintained in order to provide the necessary level of protection for durability.

Finally, the main finding of the survey was that no clear demarcation for Slab, T and Box-girders were set but my attention is given only to the Slab and Box-girders. Thus, there is a need of identifying a clear demarcation of span length to achieve better discount.

Chapter 4: Analysis & Design of Slab & Box-girder Bridges

This chapter treats the analysis and design of prestressed pre-tensioning Slab-girder & post-tensioning Box-girder railway bridges. AREMA manual is intended to be the only Standards used in this study for the design of structures.

4.1 Loadings, Material properties, Design assumptions

4.1.1 Loadings

The following loads and forces shall be considered in the design of railway concrete structures supporting tracks, [1].

- Dead load (D).
- Live Load (L).
- Impact Load (I).
- Centrifugal Force (CF).
- Earth Pressure (E).
- Buoyancy (B).
- Wind Force on Structures (W).
- Wind Force on Live Load (WL).
- Longitudinal Force from Live Load (LF).
- Longitudinal force due to friction resistance or shear resistance at expansion bearings (F).
- Earthquake Seismic (EQ).
- Stream Flow Pressure (SF).
- Ice Pressure (ICE).
- Other forces (rib, shortening, shrinkage, temperature and /or settlement of supports) (OF).

Thus, there are many loads that the bridge must support in addition to its primary loads and should be considered during design, they are highlighted in detail in the underneath.

DEAD LOAD (D)

The dead load shall consist of the estimated weight of the structural member, plus that of the track, ballast, fill, and other portions of the structure supported thereby. In order to estimate the weight for the purpose of computing dead load stresses, the unit weights found in Table 4.1 shall be used, [1].

Table 4.1: Unit Weight for Dead Load Stresses

MATERIAL	WEIGHT	UNIT
Track rails, guardrails, fastenings	3	kN / m
Ballast	19	kN / m^3
Reinforced concrete	24	kN / m^3
Waterproofing	0.90	kN / m
Catenaries	0.02	kN / m
Future utilities	0.05	kN / m

LIVE LOAD (L)

Railroad locomotives and equipment (box and flat cars, commodity gondolas, tank cars) vary greatly with respect to weight, number of axles, and axle spacing as shown in Figure 4-1.

The recommended live load for each track of main line structure is Cooper E 80 (EM 360) load, which is based on two consolidation-type steam locomotives with trailing cars represented by a uniformly distributed load. The maximum locomotive axle load is 360KN and fright equipment is represented by a uniform load of 120KN/m of track.

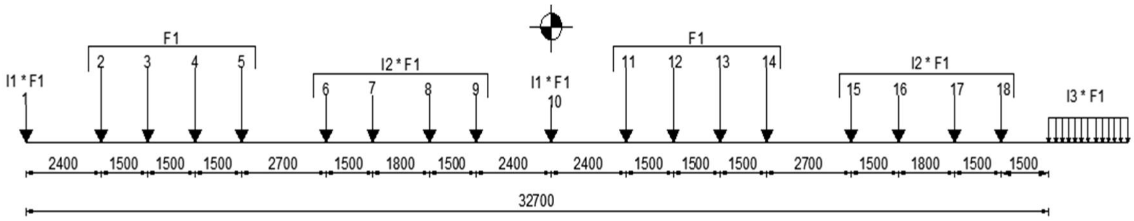


Figure 4.1: E-Cooper series loading

Where

I1 = F3/F1 = 180/360 = 0.5

I2= F2/F1 = 230/360 =0.64

I3= F4/F1= 120/360 = 0.33

F1= 360 KN/m

F2= 230 KN/m

F3= 180 KN/m

F4= 120 KN/m

IMPACT LOAD (I)

Impact forces, applied at the top of rail, shall be added to the axle loads specified. For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.), the impact shall be equal to the following percentages of the live load: [Note: This formula is intended for ballasted-deck spans and substructure elements as required]

For L ≤ 4 meters $I = 60\%$ (4-1)
For 4 < L ≤ 39 meters $I = \frac{125}{\sqrt{L}}\%$ (4-2)
For L > 39 meters $I = 20\%$ (4-3)

CENTRIFUGAL FORCE (CF)

On curves, a centrifugal force corresponding to each axle load shall be applied horizontally through a point 2450 mm above the top of rail measured along a line perpendicular to the line joining the tops of the rails and equidistant from them. This force shall be the percentage of the live load computed from the formulas below. On curves, each axle load on each track shall be applied vertically through the point defined in the first paragraph of this article. The greater of loads on high and low sides of a superelevated track shall be used for the design of supports under both sides.

The relationships between speed, degree of curve, centrifugal force and a superelevation which is 75 mm less than that required for zero resultant flange pressure between wheel and rail are expressed by the formulas:

$$C = 0.000452S^2D \quad (4\text{-}4)$$

$$E_a = 0.0068S^2D - 75 \quad (4\text{-}5)$$

$$S = \frac{(E_a-75)^{0.5}}{(0.0068D)^{0.5}} \quad (4\text{-}6)$$

C = Centrifugal force in percentage of the live load

D = Degree of curve (Degrees based on 30 m chord)

E = Actual super elevation in (mm)

S = Permissible speed in (km/hr)

Assuming straight section in this developing computer program shall be taken, so CF = 0

BUOYANCY (B)

Buoyancy shall be considered as it affects the design of either substructure, including piling, or the superstructure.

WIND LOAD ON THE STRUCTURE (W)

The base wind load acting on the structure is assumed to be (2160 Pa) on the vertical projection of the structure applied at the center of gravity of the vertical projection in any horizontal direction. A base wind velocity of (160 km/h) was used to determine the base wind load. If an increase in the design wind velocity is made, the design wind velocity and design wind load shall be shown on the plans.

For Group II and Group V loadings, when a design wind velocity greater than 100 miles per hour (160 km/h) is advisable the base wind load may be increased by the ratio of the square of the design wind velocity to the square of the base wind velocity. This increase shall not apply to Group III and Group VI Loadings, [AREMA, Table8-2-4].

WIND LOAD ON LIVE LOAD (WL)

A wind load of 4.4 kN/m on the train shall be applied 2450 mm above the top of rail in a horizontal direction perpendicular to the centerline of the track.

LONGITUDINAL FORCE (LF)

Longitudinal forces, due to train braking and locomotive tractive effort are considerable for modern railway freight equipment, for E-80 (EM 360) loading shall be taken as the larger of:

Force due to braking, as prescribed by the following equation, acting (2450 mm) above top of rail.

Longitudinal braking force (kN)

$$L_{F1} = 200 + 17.5L \tag{4-7}$$

Force due to traction, as prescribed by the following equation, acting (900 mm) above top of rail.

Longitudinal traction force (kN)

$$L_{F2} = 200\sqrt{L} \tag{4-8}$$

Where L is the length in (meters) of the portion of the bridge under consideration.

$$\boldsymbol{L_F = Max(L_{F1}; L_{F2})} \tag{4-9}$$

LONGITUDINAL FORCE DUE TO FRICTION OR SHEAR RESISTANCE AT EXPANSION BEARINGS (F)

Provisions shall be made to accommodate forces due to friction or shear resistance due to expansion bearings.

EARTHQUAKE (EQ)

In regions where earthquakes may be anticipated, structures may be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure. Refer to Chapter 9 Seismic Design for Railway Structures for additional guidance, [4].

<u>STREAM FLOW PRESSURE</u> (SF)

All piers and other portions of structures which are subject to the force of flowing water or drift shall be designed to resist the maximum stresses induced thereby.

(1) Stream Pressure

The effect of flowing water on piers and drift build up, assuming a second-degree parabolic velocity distribution and thus a triangular pressure distribution, shall be calculated by the formula:

$$\mathbf{P_{avg} = K\left(V_{avg}\right)^2} \qquad (4\text{-}10)$$

Where:

P_{avg} = average stream pressure, in, (Pa)

V_{avg} = average velocity of water in feet per second, (m/s) computed by dividing the flow rate by the flow area,

K = a constant, being (or 725 for metric) for all piers subjected to drift build up and square-ended piers, (or 360 for metric) for circular piers, and (or 260 for metric) for angle-ended piers where the angle is 30 degrees or less.

The maximum stream flow pressure, P_{max}, shall be equal to twice the average stream flow pressure, P_{avg}, computed by EQ above. Stream flow pressure shall be a triangular distribution with P_{max} located at the top of water elevation and a zero pressure located at the flow line.

(2) The stream flow forces shall be computed by the product of the stream flow pressure, taking into account the pressure distribution, and the exposed pier area. In cases where the corresponding top of water elevation is above the low beam elevation, stream flow loading on the superstructure shall be investigated. The stream flow pressure acting on the superstructure may be taken as P_{max} with a uniform distribution.

(3) Pressure Components

When the direction of stream flow is other than normal to the exposed surface area, or when bank migration or a change of stream bed meander is anticipated, the effects of the directional components of stream flow pressure shall be investigated.

(4) Drift Lodge against Pier

Where a significant amount of drift lodge against a pier is anticipated, the effects of this drift build up shall be considered in the design of the bridge opening and the bridge components. The overall dimensions of the drift build up shall reflect the selected pier locations, site conditions, and known drift supply upstream. When it is anticipated that the flow area will be significantly blocked by drift build up, increases in high water elevations, stream velocities, stream flow pressures, and the potential increases in scour depths shall be investigated.

ICE PRESSURE (ICE)

The effects of ice pressure, both static and dynamic, shall be accounted for in the design of piers and other portions of the structure where, in the judgment of the Engineer, conditions so warrant.

OTHER FORCES (OF)

The structure shall be designed to resist the forces caused by rib shortening, shrinkage, temperature rise and/or drop and the anticipated settlement of supports. The range of temperature shall generally be as shown in Table below.

Table 4.2: Temperature ranges

Climate	Temperature Rise	Temperature Fall
Moderate	17°	22°
Cold	20°	25°

4.1.2 Material properties

According to AREMA and AASHTO LRFD Bridge Construction Specifications tests of materials, concrete, reinforcement shall be made in accordance with appropriate standards of the ASTM - International as specified. Therefore, the materials used in designing railway bridges of this program are as follows:

a. The specified compressive strength for prestressed concrete and decks shall not be less than 28 MPa but in this design 50MPa shall be taken.
   Where: $f'_c = 50MPa$

so, $f_{ck} = 0.8* f'_c = 40MPa$ (4-11)

b. The density of concrete is taken as 2400 kg/m^3 for the computation of the Modulus of elasticity of concrete and Unit weight of 25 kN/m^3 for dead load computations.

c. Modulus of elasticity (E_c) in MPa.

$$E_c = \gamma_c^{1.5} * 0.043\sqrt{f_c^{'}} \qquad (4\text{-}12)$$

Where:

$\gamma_c =$ unit density of cocncrete (kg/m^3)

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

d. The modulus of elasticity, E_S, of steel reinforcing shall be assumed as 200 GPa.

e. Grade 270 is used in pre-stressed low relaxation seven wire strands steel with a minimum ultimate strength of 1890 MPa.

4.1.3 Design Assumptions

This section describes the assumptions that needed to be performed before being able to run the developing computer program analysis.

- The abutments are considered to be the same for both Slab and Box-girder railway bridges but the design of abutments and it is associated costs are not taken into account for cost analysis.
- Assuming straight section bridges with zero curvature in plan and 0° skew.
- The deck width is constant for both structures and along the bridge.
- The dead load, loads of columns carrying the catenary, curb, railings are considered during analysis and design and in quantity computations, it is assumed that these loads are the same for the same span length of Slab and Box-girder.

The bearings were chosen to be the same for the same structures span length.

4.2 Design specifications

4.2.1 Design loads

Loads taken in to account for the design are:

- Dead load including (rail, tie, ballast, curbs, guard rail, self-weight of the superstructure)
- Live Load (Cooper E-80) loading
- Impact forces

All types of load taken in to account in the analysis of Voided Slab Railway Bridge will be taken for Box-girder.

4.2.2 Load factors and load combinations

The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition. (LRFD)

The structure as a whole and its components shall be designed to resist sliding, overturning, uplift and buckling. Effects of eccentricity of loads shall be considered in the analysis and design. (LRFD)

Pre-stressed members shall have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations for the load groups that are applicable. For the design of post-tensioned anchorage zones, a load factor of 1.2 shall be applied to the maximum tendon jacking force [2]. Several types of loads may be applied to a structure simultaneously. These loads are combined in a prescribed manner to produce design loads for the bridge. The AREMA Manual for railway Engineering recognize that it is unlikely that the maximum values of all loads will be applied concurrently to a structure, load combination methods are given to develop maximum credible design forces on the structure [2]. As a general rule, the section determined by a load combination should never be smaller than the section required for dead load, live load. Moreover, Load factors and load combinations are considered accordingly.

Table 4.3: Group loading combinations-Service load design (AREMA Table 8-2-4)

Group	**Item**	**Allowable Percentage of basic Unit stress**
I	D + L + I + CF + E + B + SF	100
II	D + E + B + SF + W	125
III	Group I + 0.5W + WL + LF + F	125
IV	Group I + OF	125
V	Group II + OF	140
VI	Group III + OF	140
VII	Group I + ICE	140
VIII	Group II + ICE	150

Table 4.4: Group loading combinations load factor design (AREMA Table 8-2-5)

Group	**Item**
I	1.4 (D + 5/3 (L + I) + CF + E + B + SF)
IA	1.8 (D + L + I + CF + E + B + SF)
II	1.4 (D + E + B + SF + W)
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)
IV	1.4 (D + L + I + CF + E + B + SF + OF)
V	Group II + 1.4 (OF)
VI	Group III + 1.4 (OF)
VII	1.0 (D + E + B + EQ)
VIII	1.4 (D + L + I + E + B + SF + ICE)
IX	1.2 (D + E + B + SF + W + ICE)

The following strength capacity reduction factors shall be used in accordance with AREMA:

(1) For flexure $\varphi = 0.95$

(2) For shear $\varphi = 0.90$

(3) For anchorage zones $\phi = 0.85$ for normal weight concrete and $\varphi = 0.70$ for lightweight concrete.

4.2.3 Distribution of Live load

Like highway bridge loads, which may move laterally across the bridge deck, railway live loads are generally fixed in lateral position. However, they are a longitudinal series of large magnitude concentrated wheel loads, and longitudinal and lateral distribution to the deck and supporting members must be considered. [10]

For ballasted deck bridges, longitudinal and lateral distribution of live load will follow the AREMA recommendations.

Longitudinal deck distribution width (mm)

$$Min[(915 + db); 1524] \tag{4-13}$$

Lateral deck distribution width (mm)

$$Min[(S_w + db); 4300] \tag{4-14}$$

S_w = Sleepers length

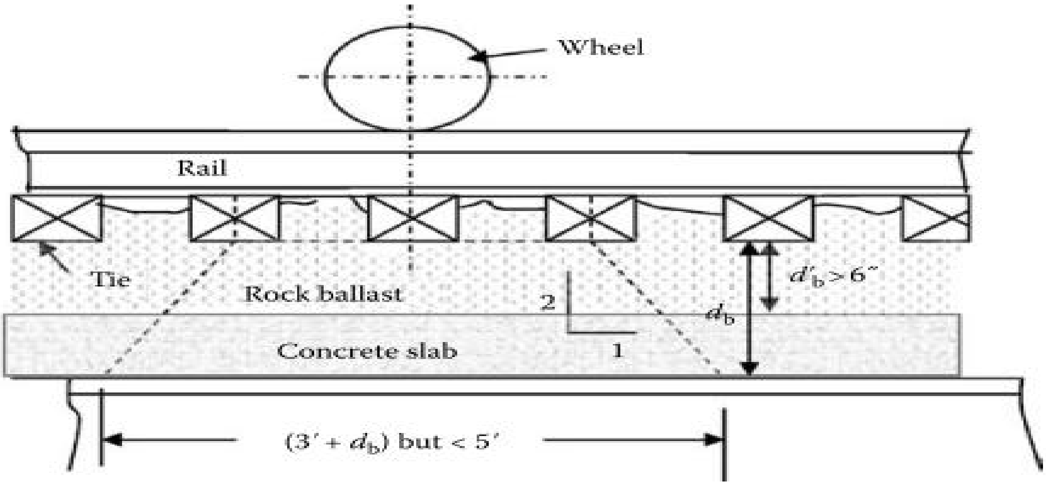


Figure 4.2: Longitudinal distribution of wheel load according to AREMA Standard

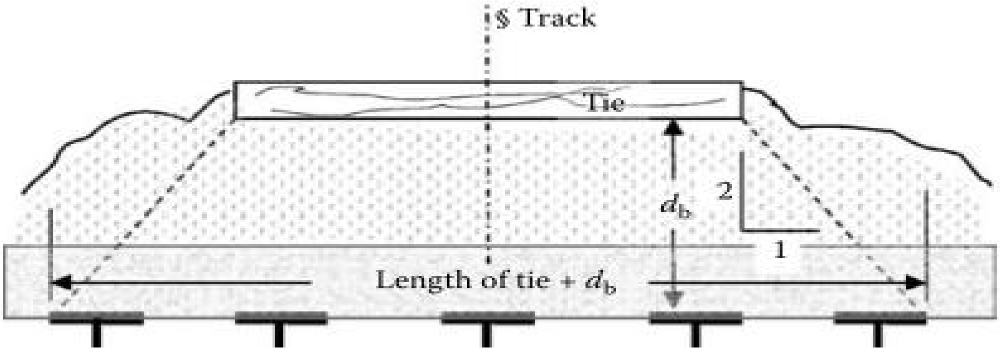


Figure 4.3: Lateral distribution of track load according to AREMA Standard

4.3 General Analysis and Design of Slab Bridge

4.3.1 Analysis procedure

Superstructure depth

For the Superstructure, the minimum depth (includes deck thickness) design for precast pre-stressed Slab bridge can then expressed according to [AASHTO LRFD, Table 2.5.2.6.3-1].

For Simple Span:

$$Hmin = 0{,}030\, L \qquad (4\text{-}15)$$

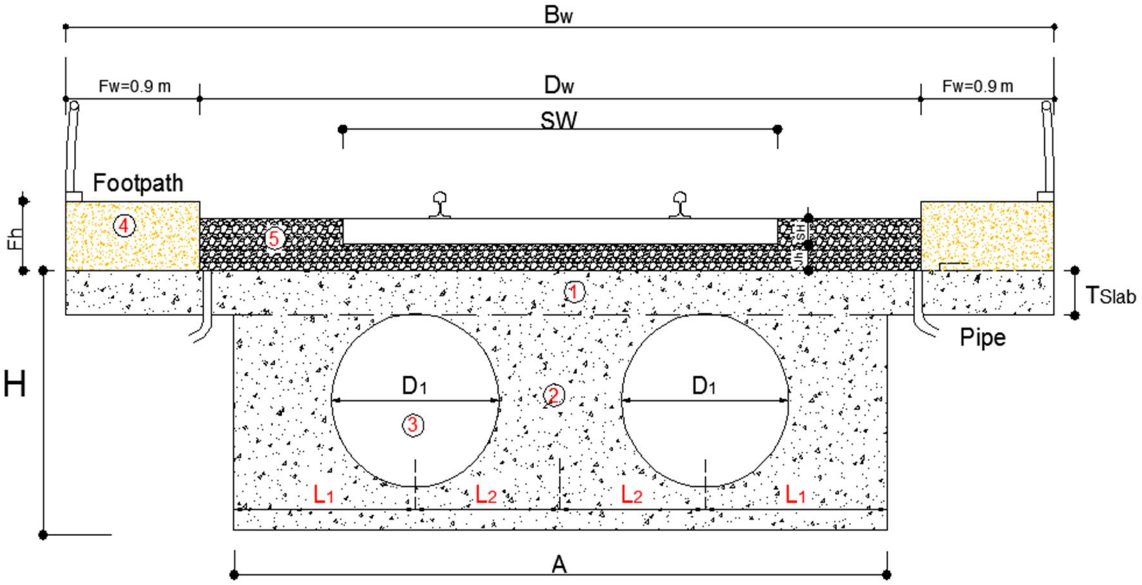


Figure 4.4: Cross section of Voided Slab Bridge.

The minimum Top Slab thickness:

Recommended Minimum Thickness for Constant Depth Members, [4].

$$T_{slab} = Max\left[\frac{(S+3000)}{20}\right] \text{, but not less than 230 mm} \qquad (4\text{-}16)$$

Where

T_{slab} = Thickness of top slab (mm)

S = Span length (mm)

The diameter of the structure:

The the minimum diameter of the slab can be expressed: [19].

$$D_1 \leq 0.5H \qquad (4\text{-}17)$$

Where

D_1 = Diameter of the voids (mm)

H = The Deck depth (mm)

And the ribs between voids should generally above 300 mm.

- **Section properties:**

From the minimum dimension of the section calculated above:
Here are the section proprieties.

$$I_s = \sum \boldsymbol{I_x} + \sum \boldsymbol{A_x d_x^2} \quad (4\text{-}18)$$

$$Z_t = I_T / Y_t \quad (4\text{-}19)$$

$$Z_b = I_T / Y_b \quad (4\text{-}20)$$

Where
A_c= Concrete area for different section (m^2)
Y_t = Distance of highest point from the centroid of concrete section (mm)
Y_b = Distance of lowest point from the centroid of concrete section (mm)
I_x = Second moment of area of Slab- section (mm^4)
Z_t = Section modulus of top fiber of Slab bridge (mm^3)
Z_b = Section modulus of bottom fiber of Slab bridge (mm^3)

- **Permanent Load:**

Permanent load is the load due to the weight of different elements in the railway bridges. These values are directly deduced from the dimensions of the different elements.
We divide the permanent load in two parts:

- Self-weight of the Slab bridge.
- Superimposed permanent load

The general formula for the dead load components of bridges is calculated as follow:

$$\boldsymbol{q\left(\frac{kN}{m}\right) = \gamma\left(\frac{kN}{m^3}\right) \times A(m^2)} \quad (4\text{-}21)$$

Self-weight of the Slab bridge:

$$q_{sw} = q_{Top\text{-}slab} + q_{Slab\ deck} + q_{diaphragms} = q_{Slab\text{-}Total} + q_{diaphragms}$$

Superimposed permanent load:

$$q_{SI} = q_{rail} + q_{Sleepers} + q_{Ballast} + q_{Curb} + q_{Water\ proofing} + q_{Catenaries}$$

Where
γ = Unit weight of the different components
q_{sw} = Total permanent load from self-weight of the slab (kN/m)
q_{SI} = Total permanent load from superimposed load (kN/m)

- **Diaphragms**

Diaphragms shall be used at span ends. Intermediate diaphragms shall be used where required in the judgment of the Engineer. Diaphragm spacing for curved girders shall be given special consideration, [4].

Diaphragms shall be provided in accordance with the underneath, except that adequate bracing of the compression flange shall be provided by a cast-in-place deck, [4].

- Diaphragms or other means shall be used at span ends to strengthen the free edge of the slab and to transmit forces to the substructure.
- For spread box beams, diaphragms shall be placed within the box and between boxes at span ends and at the points of maximum moment for spans over (24 m).
- For precast box multi-beam bridges, diaphragms are required only if necessary for slab end support or to contain or resist transverse tension ties.
- For cast-in-place box girders, diaphragms or other means shall be used at span ends to resist lateral forces and maintain section geometry. Intermediate diaphragms are not required for bridges with inside radius of curvature of (245 m) or greater.
- For all types of prestressed boxes in bridges with inside radius of curvature less than (245 m), intermediate diaphragms may be required and the spacing and strength of diaphragms shall be given special consideration in the design of the structure.

Permanent load from Top Slab +Slab deck +diaphragms

For center to center support length (X) lesser or equal to 24m

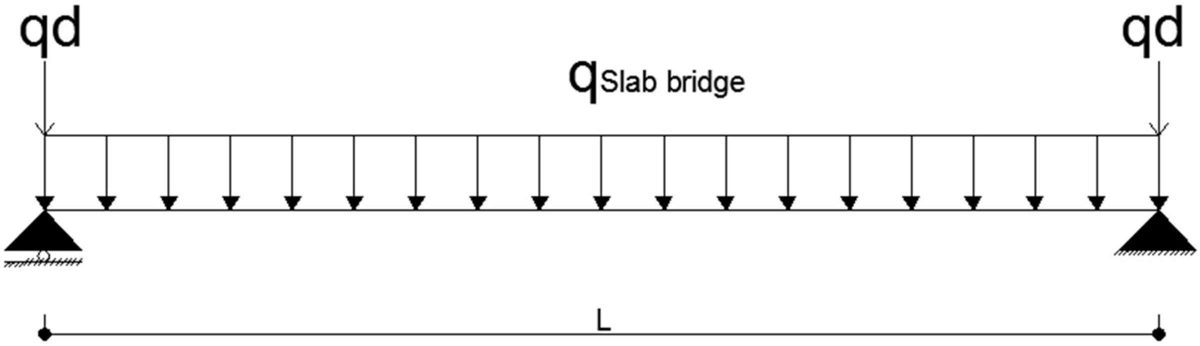


Figure 4.5: Self weight of the Slab bridge (L ≤ 24m)

The Slab bridge bending moment and shear force dead load + diaphragms at every distance (X) from the support is computed as follows:

For $0 \leq X \leq L$

$$V_{qsw}(X) = \frac{q_{\text{slab bridge}}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_{\text{slab bridge}}}{2}(LX - X^2)$$

Where $q_{\text{Slab-bridge}} = q_{\text{Top-slab}} + q_{\text{Slab deck}}$ (kN/m)

$q_d = q_{\text{ diaphragms}}$ (kN)

For center to center support length (X) greater than 24m:

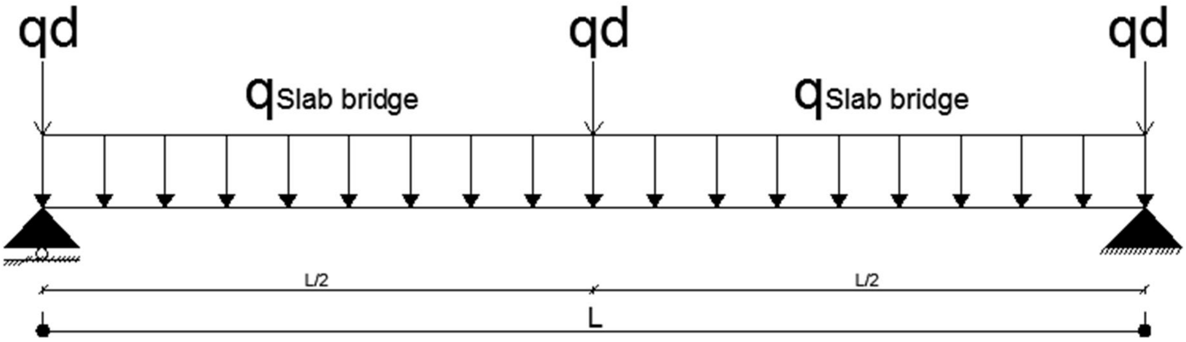


Figure 4.6: Self weight of the Slab bridge (L ≤ 24m)

For $0 \leq X \leq L/2$

$$V_{qsw}(X) = \frac{q_d}{2} + \frac{q_{\text{slab bridge}}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_d}{2}X + \frac{q_{\text{slab bridge}}}{2}(LX - X^2)$$

For $X > L/2$

$$V_{qsw}(X) = -\frac{q_d}{2} + \frac{q_{\text{slab bridge}}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_d}{2}(L - X) + \frac{q_{\text{slab bridge}}}{2}(LX - X^2)$$

Where

M_{qsw} = Moment due to Permanent load from Top Slab + Slab deck + diaphragms (kN-m)

V_{qsw} = Shear force due to Permanent load from Top Slab + Slab deck + diaphragms (kN-m).

Superimposed dead load

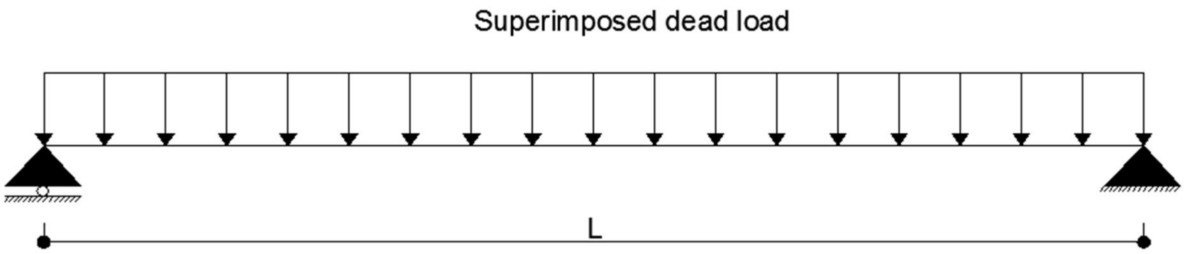


Figure 4.7: Superimposed dead load

The bending moment and shear force due to superimposed dead load at every distance (X) from the support is computed as follows:

$$V_{qsi}(X) = \frac{q_{SI}}{2}(L - X)$$

M_{qsi} (X) = $\frac{q_{SI}}{2}(LX - X^2)$

Where

M_{qsi} = Bending Moment due to superimposed load (kN-m)

V_{qsi} = Shear force due to superimposed load (kN)

Permanent load = Top Slab +Slab deck +diaphragms + superimposed DL

For center to center support length (X) lesser or equal to 24m

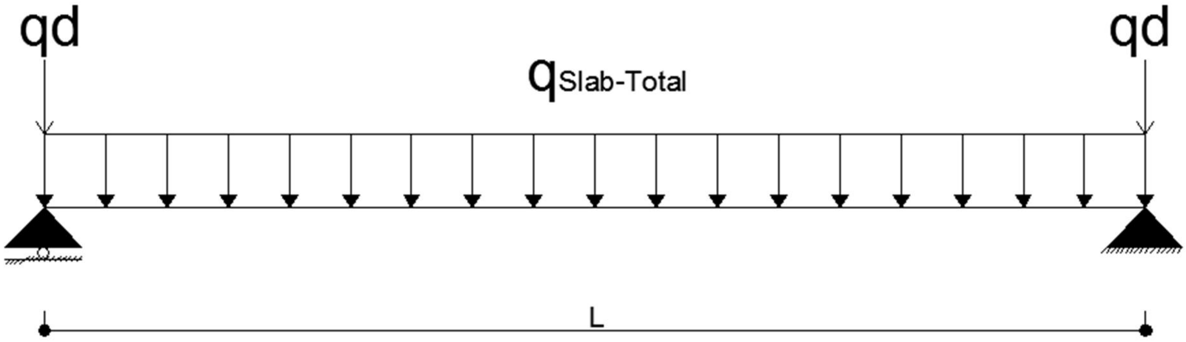


Figure 4.8: Permanent load placement (L≤24m)

The bending moment and shear force due to girder dead load +diaphragms and superimposed load at every distance (X) from the support is computed as follows:

For 0 ≤ X ≤ L

$$V_{\text{Permanent}}(X) = \frac{q_{\text{Slab-Total}}}{2}(L - 2X)$$

$$M_{\text{Permanent}}(X) = \frac{q_{\text{Slab-Total}}}{2}(LX - X^2)$$

Where $q_{\text{Slab-Total}} = q_{\text{Top Slab}} + q_{\text{Slab deck}} + q_{\text{superimposed load}}$ (q_{SI})

For center to center support length (X) greater than 24m

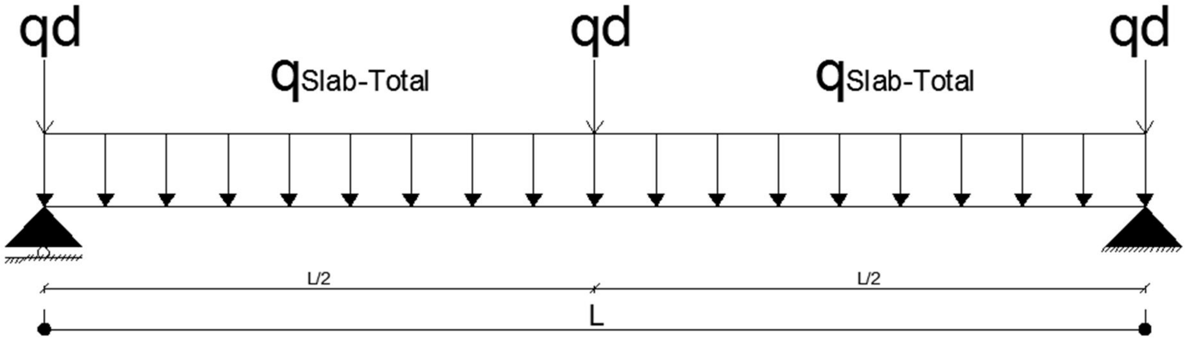


Figure 4.9: Permanent load placement (L>24m)

For 0 ≤ X ≤ L/2

$$V_{\text{Permanent}}(X) = \frac{q_d}{2} + \frac{q_{Slab\ total}}{2}(L - 2X)$$

$$M_{\text{Permanent}}(X) = \frac{q_d}{2}X + \frac{q_{Slab\ total}}{2}(LX - X^2)$$

For X > L/2

$$V_{Permanent}(X) = -\frac{q_d}{2} + \frac{q_{Slab\ total}}{2}(L - 2X)$$

$$M_{Permanent}(X) = \frac{q_d}{2}(L - X) + \frac{q_{Slab\ total}}{2}(LX - X^2)$$

Where:
$M_{Permanent}$ = Moment due to Permanent load from Slab bridge + diaphragms + superimposed DL (kN-m)
$V_{Permanent}$ = Shear force due Permanent load from Slab bridge + diaphragms + superimposed DL (kN)

- **Impact load**

Load from impact q_I, the impact shall be equal to the following percentages of the live load:
Equation: 4.1, 4.2, 4.3
The bending moment and shear force due to impact load at every distance (X) from the support is computed as follows:

$$V_I(X) = \frac{q_I}{2}(L - 2X)$$

$$M_I(X) = \frac{q_I}{2}(LX - X^2)$$

Where
q_I = Load from impact (kN/m)
M_I = Bending Moment due to impact load (kN-m)
V_I = Shear force due to impact load (kN).

- **Variable loads**

Influence lines for shear and bending moments due to live loads
The concentrated live loads due to the first locomotive include 9 wheels composed as following:
A first wheel of magnitude F3 = 180kN, followed by a four consecutive wheels load with the same spacing and magnitude respectively of 1.5m, F1= 360kN. Then another four consecutive wheels load with the same magnitude of F2= 230 kN but different spacing of 1.5m ,1.8m ,1.5m. The first locomotive are followed by a second locomotive with the same characteristics, and finally these locomotive pull a freight equipment represented by a uniform load of a magnitude F4= 120kN/m.
Those entire characteristics are represented on the figure below cooper loading.
Where:

I1 = F3/F1 = 180/360 = 0.5

I2= F2/F1 = 230/360 =0.64

I3= F4/F1= 120/360 = 0.33

F1= 360 KN/m

F2= 230 KN/m

F3= 180 KN/m

F4= 120 KN/m

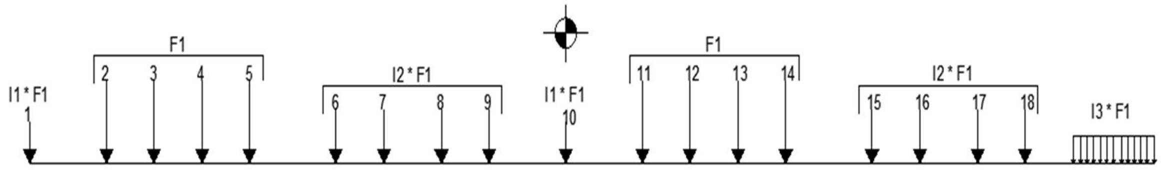


Figure 4.10: Train moving from the right to the left (wheel load in kN)

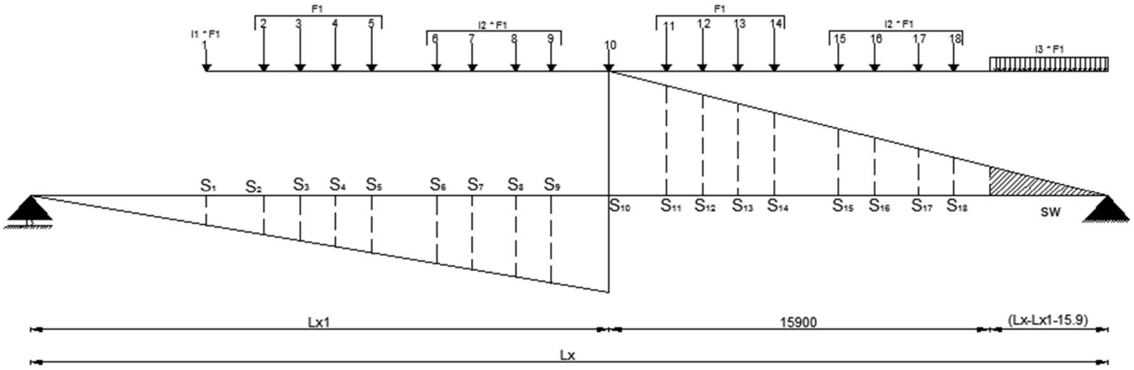


Figure 4.11: Influence Line for Shear Force at (Lx1) Distance from End Support.

Influence line coefficient for shear force: (if S< 0, we take 0).

S_1= (Lx1-16.8)/Lx
S_2= (Lx1-14.4)/Lx
S_3= (Lx1-12.9)/Lx
S_4= (Lx1-11.4)/Lx
S_5= (Lx1-9.9)/Lx
S_6= (Lx1-7.2)/Lx
S_7= (Lx1-5.7)/Lx
S_8= (Lx1-3.9)/Lx
S_9= (Lx1-2.4)/Lx
S_{10}= (Lx- Lx1)/Lx
S_{11}= (Lx- Lx1-2.4)/Lx
S_{12}= (Lx- Lx1-3.9)/Lx
S_{13}= (Lx- Lx1-5.4)/Lx
S_{14}= (Lx- Lx1-6.9)/Lx
S_{15}= (Lx- Lx1-9.6)/Lx
S_{16}= (Lx- Lx1-11.1)/Lx
S_{17}= (Lx- Lx1-12.9)/Lx
S_{18}= (Lx- Lx1-14.4)/Lx
S_w= $(Lw)^2/2Lx$= $(Lx- Lx1-15.9)^2/2Lx$

V (X) = F1 [I1(S1)+(S2+ S3+ S4+ S5)+I2*(S6+ S7+ S8+ S9)+I1*(S10)+(S11+ S12+ S13+ S14)+I2*(S15+ S16+ S17+ S18)+I3*(Sw)

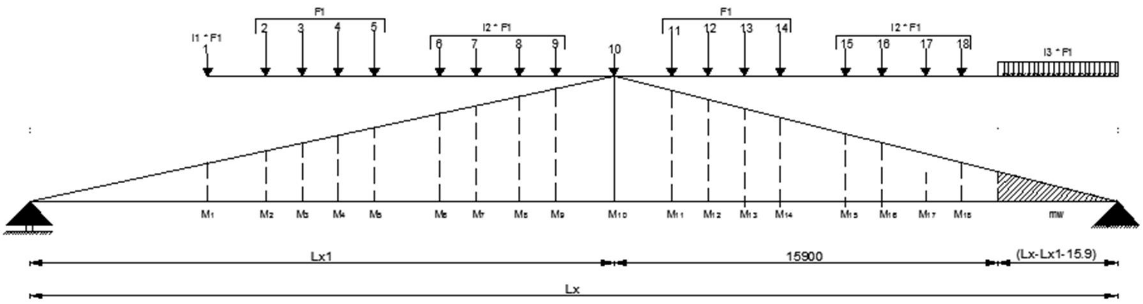


Figure 4.12: Bending Moment of influence line at (Lx1) Distance from End Support (moving from right to Left).

Influence line coefficients for bending moment: (if M< 0, we take 0)

M1 = (Lx1–16.8)*(Lx– Lx1)/Lx
M2 = (Lx1-14.4)*(Lx– Lx1)/Lx
M3 = (Lx1-12.9)*(Lx– Lx1-1.5)/Lx
M4 = (Lx1-11.4)*(Lx– Lx1-3)/Lx
M5 = (Lx1-9.9)*(Lx– Lx1-5.7)/Lx
M6= (Lx1-7.2)*(Lx– Lx1-7.2)/Lx
M7 = (Lx1-5.7)*(Lx– Lx1-9)/Lx
M8 = (Lx1-3.9)*(Lx– Lx1-10.5)/Lx
M9 = (Lx1-2.4)*(Lx– Lx1-12.5)/Lx
M10 = (Lx1)*(Lx– Lx1)/Lx
M11 = (Lx1)*(Lx– Lx1-2.4)/Lx
M12 = (Lx1)*(Lx– Lx1-3.9)/Lx
M13 = (Lx1)*(Lx– Lx1-5.4)/Lx
M14 = (Lx1)*(Lx– Lx1-6.9)/Lx
M15 = (Lx1)*(Lx– Lx1-9.6)/Lx
M16 = (Lx1)*(Lx– Lx1-11.1)/Lx
M17 = (Lx1)*(Lx– Lx1-12.9)/Lx
M18 = (Lx1)*(Lx– Lx1-14.4)/Lx
Mw = $(Lx1)*(Lw)^2/2Lx= (Lx1)*(L\text{-}X\text{-}15.9)^2/2Lx$

M (X) = F1 [I1*(M1)+((M2)+ (M3)+ (M4)+ (M5))+I2*((M6)+ (M7)+ (M8)+ (M9))+I1*(M10)+((M11)+ (M12)+ (M13)+ (M14))+ I2*((M15)+ (M16)+ (M17)+ (M18))+I3*(Mw)]

M (X) = Bending moment at a distance(Lx1) due to train load moving from right to the left case 2.

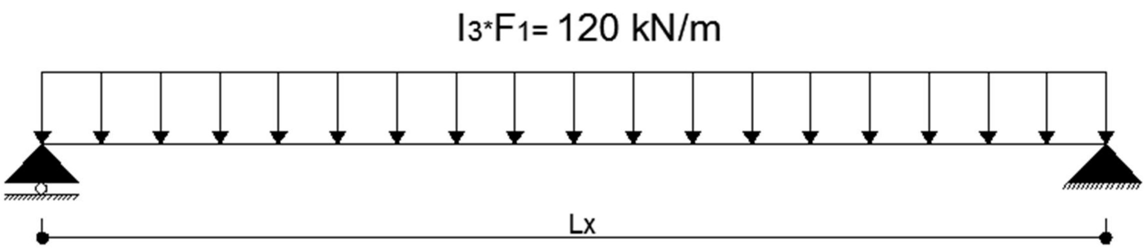


Figure 4.13: Influence Line of bending moment and shear force for moving uniform load.

$M(2) = \frac{PL^2}{8}$ (kN/m)

$V2 = \frac{PL}{2}$ (kN)

$$\boldsymbol{M_{LL} = Max[M_1;\ M_2]} \tag{4-22}$$

$$\boldsymbol{V_{LL} = Max[V_1;\ V_2]} \tag{4-23}$$

In this computer program, all negative result for dynamic load is considered as null so that means the wheel is out of the span clearance.

From the above formula (Lx1) is taken as the distance of the wheel 2 for the case 1 and for the case 2 as the wheel 10.

The maximum load will be considered for the structure design.

- **Permissible linear stress**

Concrete

- Stress in concrete at transfer
  - Extreme fiber stress in compression

Pre-tensioned	0.6 f'ci	(4-24)
Post-tensioned	0.55 f'ci	(4-25)

  - Extreme fiber stress in tension

Members with bonded auxiliary reinforcement:	$0.623\sqrt{f'ci}$	(4-26)
Members without bonded auxiliary reinforcement:	$0.25\sqrt{f'ci}$	(4-27)

- Stress in concrete at service loads

Compression	0.40 f'c	(4-28)
Tension	0	(4-29)

- Cracking stress

For normal weight concrete $0.623\sqrt{f'ci}$ (4-30)

For sand lightweight concrete $0.523\sqrt{f'ci}$ (4-31)

- Anchorage bearing **stress**

Post-tensioned anchorage at service load Max 21 MPa (4-32)

➢ Pre-stressing tendons stress

- Pre-tensioning: tensile stress in pre-stressing tendons shall not exceed the following
  - Due to tendon jacking force:

Min (0.75f’s ; 0.90f*y) (4-33)

  - Slight over stressing of pre-tensioning tendons up to 0.85f 's for short periods of time may be permitted to offset seating losses, provided the stress after seating does not exceed the value in Paragraph above.
  - Stress-relieved pre-tensioning tendons immediately after pre-stress transfer

Max (0.82f*y ; 0.70f’s) (4-34)

  - Stabilized (low-relaxation) pre-tensioning tendons immediately after pre-stress transfer

Max (0.82f*y; 0.75f’s) (4-35)

- Post-tensioning: tensile stress in post-tensioning tendons shall not exceed the following
  - Immediately after tendon anchorage

Max (0.82f*y; 0.70f’s) (4-36)

But not greater than 0.70f’s at the end anchorage

  - Over stressing of post-tensioning tendons up to 0.90f’s for short periods of time may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the value above. The stress at the end of the seating loss zone must not exceed 0.82f*y immediately after seating.

4.3.2 Design procedures

General the following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition.

Basic assumptions

- Strains vary linearly over the depth of the member throughout the entire load range.
- Before cracking, stress is linearly proportional to strain.
- After cracking, tension in the concrete is neglected.

Maximum un-factored Moment & Shear

The maximum values of the loading combinations from table 3-3 shall be used for design

$$\boldsymbol{M_{unfactored}} = \boldsymbol{Max}[\boldsymbol{Group}(\boldsymbol{I}, \boldsymbol{II}, \boldsymbol{III}, \boldsymbol{IV}, \boldsymbol{V}, \boldsymbol{VI}, \boldsymbol{VII}, \boldsymbol{VIII})] \tag{4-37}$$

$$\boldsymbol{V_{unfactored}} = \boldsymbol{Max}[\boldsymbol{Group}(\boldsymbol{I}, \boldsymbol{II}, \boldsymbol{III}, \boldsymbol{IV}, \boldsymbol{V}, \boldsymbol{VI}, \boldsymbol{VII}, \boldsymbol{VIII})] \tag{4-38}$$

Maximum factored Moment & Shear

The maximum values of the loading combinations from table 3-4 shall be used for design

$$\boldsymbol{M_{factored}} = \boldsymbol{Max}[\boldsymbol{Group}(\boldsymbol{I}, \boldsymbol{IA}, \boldsymbol{II}, \boldsymbol{III}, \boldsymbol{IV}, \boldsymbol{V}, \boldsymbol{VI}, \boldsymbol{VII}, \boldsymbol{VIII}, \boldsymbol{IX})] \tag{4-39}$$

$$\boldsymbol{V_{factored}} = \boldsymbol{Max}[\boldsymbol{Group}(\boldsymbol{I}, \boldsymbol{IA}, \boldsymbol{II}, \boldsymbol{III}, \boldsymbol{IV}, \boldsymbol{V}, \boldsymbol{VI}, \boldsymbol{VII}, \boldsymbol{VIII}, \boldsymbol{IX})] \tag{4-40}$$

Loss of Pre-stress

Loss of pre-stress refers to the reduced tensile stress in the tendons, the initial pre-stress in concrete undergoes a gradual reduction with time from the stage of transfer due to various causes. Some of these losses are immediate, affecting the pre-stress force as soon as it is transferred to the concrete member other losses occur gradually with time.

Consequently, it is important to determine the level of pre-stress force at each loading stage from the transfer of the pre-stressing force to the concrete, to various stage of pre-stressing available at service load, up to ultimate.

Table 4.5: Types of Losses of Prestress

Pre-tensioning	**Post-tensioning**
Elastic shortening of concrete	Elastic shortening of concrete
Creep of concrete	Creep of concrete
Shrinkage of concrete	Shrinkage of concrete
Relaxation of tendons stress	Relaxation of tendons stress
	Friction
	Anchorage slip

Frictional loss occurs only in post tensioned beams. When the cable is stressed, friction between the sides of the duct and the cable does not permit full tension to be transmitted. Therefore, at a point away from the jacking end pre-stress is less.

Anchorage slip losses occur in post-tensioned members due to the seating of wedges in the anchors when the jacking force is transferred to the anchorage. They can also occur in the pre-stressing casting beds of pre-tensioned members due to the adjustment expected when the pre-stressing force is transferred to these beds.

a) To determine effective pre-stress fse, allowance for the following sources of loss of pre-stress shall be considered:

$$\Delta fs = ES + CRC + SH + CRs$$

where:

ES = Elastic shortening of concrete

CRc = Creep of concrete

SH = Shrinkage of concrete

CRs = Relaxation of tendon stress

Anchorage seating and friction due to intended or unintended curvature in post-tensioning tendons shall be considered.

b) Total loss of pre-stress shall be determined in accordance with a method of calculating pre-stress losses supported by appropriate research data, representing properties of the materials to be used, methods of curing, ambient service conditions, and any pertinent structural details.

c) In lieu of the more exact procedure prescribed in Paragraph b, loss of pre-stress may be determined in accordance with either Paragraph d or Paragraph e for the conditions stated.

d) Loss of pre-stress may be determined by the following procedure for normal weight concrete and the following types of pre-stressing tendons:270 ksi (1860 MPa) uncoated seven-wire stress-relieved or low-relaxation strand; 145 to 160 ksi (1000 to 1100 MPa) uncoated high-strength steel bar (plain or deformed).

1) Elastic shortening of concrete
   - Pre-tensioned

$$ES = (Es / Eci) * fcir$$

   - Post-tensioned

$$Es = 0.5 (Es / Eci) * fcir$$

Where: fcir = 0.69 f`s for relaxation strand

= 0.63 f`s for relieved strand

2) Creep of the concrete
   - Pre-tensioned and Post-tensioned

$$CRc = 12\ fcir - 7\ fcds$$

Where:

fcds = Stress in concrete at centroid of pre-stressing reinforcement, due to all dead load not included in calculation of fcir.

3) Shrinkage of the concrete
   - Pre-tensioned

$$SH = 117 - 1.03\ R$$

- Post – tensioned

$$SH = 0.8 * (117 - 1.03R)$$

Where

R = annual average ambient relative humidity in percent. The following map may be used to determine R.

4) Relaxation of tendons
   - Pre-tensioned tendons

     1860 Mpa stress – relieved strand tensioned to 0.70 f`s

$$CRs = 138 - 0.40ES - 0.2\ (SH - CRc)$$

1860 Mpa low-relaxation strand tensioned to 0.70f`s

$$CRs = 25\% \text{ of } CRs \text{ above}$$

   - Post-tensioning tendons

     1860 Mpa stress-relieved strand anchored

$$CRs = 138 - 0.3\ FR - 0.4ES- - 0.2\ (SH+CRc)$$

1860 Mpa low relaxation strand anchored at 0.75 f`s

$$CRs = 25\ \% \text{ of } CRs \text{ above}$$

1000 Mpa a high strength steel bar

$$CRs = 21 \text{ Mpa}$$

5) Friction

However, based on past experience, some literatures generally recommend average prestress losses to be 22% for pre-tensioning, and 18% for post-tensioning assuming that over tensioning has been applied to overcome friction and anchorage set losses

Table 4.6: Approximate Prestress Loss

Type of Loss	**% Loss**	
	Pre-tensioning	**Post-tensioned**
Elastic shortening of concrete	4	1
Creep of concrete	6	5
Shrinkage of concrete	7	6
Relaxation of tendon stress	5	6
TOTAL	**22**	**18**

- **Stress conditions**

Pre-stressed sections under the action of flexure should satisfy the limits specified for permissible stresses at the stages of transfer of pre-stress and at service loads.

Pre-stressing force and the corresponding eccentricity are developed using the following equation stress relationships established for the two extreme fiber and considering the two critical combinations of pre-stress and moments.

The general critical combinations considered are:

1) The maximum pre-stressing force at transfer together with the minimum moments sustained by the section.
2) The minimum pre-stressing force after all losses in combinations with the maximum design moment for the serviceability limit state.

- At transfer

$$\boldsymbol{f_t} \geq \boldsymbol{f_{tt}} - \left[\frac{M_{permanent}}{Z_t}\right] \tag{4-41}$$

$$\boldsymbol{f_b} \leq \boldsymbol{f_{ct}} + \left[\frac{M_{permanent}}{Z_b}\right] \tag{4-42}$$

- At working load

$$\boldsymbol{f_t} \geq \left[\frac{f_{cs}}{\eta}\right] - \left[\frac{M_{permanent}+M_{LL}+M_I}{\eta * Z_t}\right] \tag{4-43}$$

$$\boldsymbol{f_b} \geq \left[\frac{f_{ts}}{\eta}\right] + \left[\frac{M_{permanent}+M_{LL}+M_I}{\eta * Z_b}\right] \tag{4-44}$$

The minimum pre-stressing force required will be obtained by selecting the maximum tensile pre-stress at the top fiber and the minimum compressive pre-stress corresponding to the bottom fiber.

$$\boldsymbol{f_t} = \frac{P}{A} - \frac{Pe}{Z_t} \tag{4-45}$$

and

$$\boldsymbol{f_b} = \frac{P}{A} + \frac{Pe}{Z_b} \tag{4-46}$$

Eliminating e from the equation

$$\boldsymbol{P} = \frac{[A(f_b * Z_b + f_t * Z_t)]}{[(Z_t + Z_b)]} \tag{4-47}$$

Similarly, eliminating P from the equation, the corresponding maximum eccentricity is given by

$$\boldsymbol{e_{max}} = \frac{[Z_t * Z_b(f_b - f_t)]}{[A*(f_t * Z_t + f_b * Z_b)]} \tag{4-48}$$

The limiting zone of Eccentricity of prestressing force For Pre-tension is defined as Follow:

$$e = (H/2) - y_b \tag{4-49}$$

- **Stresses in beam**
  - Direct Stress (N/mm^2)

$$\frac{P}{A}$$

  - Bending stress due to pre-stress (N/mm^2)

$$\frac{Pe}{A}$$

  - Bending stress due to self-weight and superimposed load (N/mm^2)

$$\frac{M_{permanent}}{Z}$$

  - Bending Stress due to Live Load (N/mm^2)

$$\frac{M_{LL}}{Z}$$

  - Bending Stress due to Impact Load (N/mm^2)

$$\frac{M_I}{Z}$$

Actual resultant fiber stress at span

- Stress in concrete at transfer

At top

$$\boldsymbol{f_t} = \left(\frac{P}{A}\right) - \left(\frac{Pe}{Z_t}\right) + \left(\frac{M_{permanent}}{Z_t}\right) \tag{4-50}$$

At bottom

$$\boldsymbol{f_b} = \left(\frac{P}{A}\right) + \left(\frac{Pe}{Z_b}\right) - \left(\frac{M_{permanent}}{Z_b}\right) \tag{4-51}$$

- Stress in concrete at service load

At top

$$\boldsymbol{f_t} = \left(\frac{P}{A}\right) - \left(\frac{Pe}{Z_t}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{Z_t}\right) \tag{4-52}$$

At bottom

$$\boldsymbol{f_b} = \left(\frac{P}{A}\right) + \left(\frac{Pe}{Z_b}\right) - \left(\frac{M_{permanent}+M_{LL}+M_I}{Z_b}\right) \tag{4-53}$$

Actual resultant fiber stress at End Anchorages (e = e_{min})

- Stress in concrete at transfer

At top

$$\boldsymbol{f_t} = \left(\frac{P}{A}\right) - \left(\frac{Pe}{Z_t}\right) + \left(\frac{M_{permanent}}{Z_t}\right) \tag{4-54}$$

At bottom

$$\boldsymbol{f_b} = \left(\frac{P}{A}\right) + \left(\frac{Pe}{Z_b}\right) - \left(\frac{M_{permanent}}{Z_b}\right) \tag{4-55}$$

- Stress in concrete at service load

At top

$$\boldsymbol{f_t} = \left(\frac{P}{A}\right) - \left(\frac{Pe}{Z_t}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{Z_t}\right) \tag{4-56}$$

At bottom

$$\boldsymbol{f_b} = \left(\frac{P}{A}\right) + \left(\frac{Pe}{Z_b}\right) - \left(\frac{M_{permanent}+M_{LL}+M_I}{Z_b}\right) \tag{4-57}$$

At the end of a simple supported beam the moment is zero so the equation is reduced to the following:

At top

$$\boldsymbol{f_t} = \left(\frac{P}{A}\right) - \left(\frac{Pe}{Z_t}\right) \tag{4-58}$$

At bottom

$$\boldsymbol{f_b} = \left(\frac{P}{A}\right) + \left(\frac{Pe}{Z_b}\right) \tag{4-59}$$

➢ Deflection

The limiting zone of Eccentricity of prestressing force For Pre-tension:

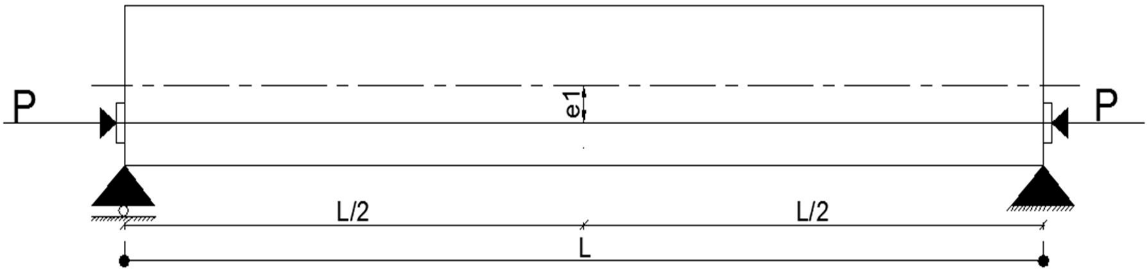


Figure 4.14: Eccentric Tendon

➢ Bottom slab reinforcement

- Minimum distributed reinforcement of 0.4% of the flange area shall be placed in the bottom slab parallel to the girder span. A single layer of reinforcement may be provided. The spacing of such reinforcement shall not exceed 18 inches (450 mm).
- Minimum 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 18 inches (450 mm). All transverse reinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook, [4].

4.4 General Analysis and Design of Box-girder

4.4.1 Analysis procedure

Structural Depth

For pre-stressed box-girder simple supported spans, the minimum structural depth including deck shall be determined as follow [3]

D = 0.045 L (4-60)

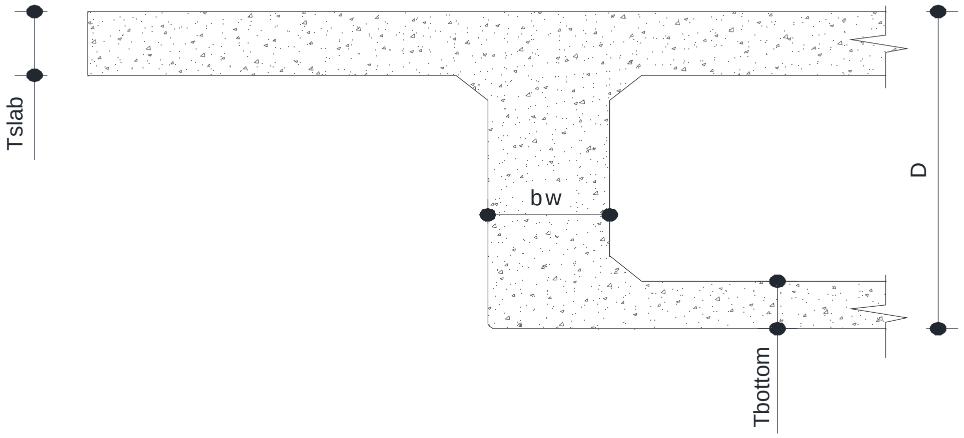


Figure 4.17: Cross section of Box-Girder bridges

The minimum top slab thickness:

The thickness of the top slab shall be designed for loads above, but shall be not less than the minimum specified as follows [1]

$$\boldsymbol{t_{slab}} = \boldsymbol{Max}\left[\frac{(\boldsymbol{S}+\mathbf{3000})}{\mathbf{17}};\mathbf{150}\right] \tag{4-61}$$

Where

t_{slab} = Thickness of top slab (mm)

S = Clear span between girders (mm)

Minimum bottom thickness

The thickness of the bottom slab shall be not less than 1/16 of the clear span between girder webs or 6 inches (150 mm), whichever is greater, except that the thickness need not be greater than the top slab unless required by design, [4].

$$\boldsymbol{t_{bottom}} = \boldsymbol{Max}\left[\frac{(\boldsymbol{S}-\boldsymbol{bw})}{\mathbf{16}};\mathbf{150}\right] \tag{4-62}$$

T_{bottom} = Thickness of bottom slab (mm)

The minimum Web Thickness

Webs with only longitudinal or vertical Post-tensioning tendons shall be:

$$b_w = 12\ inches = 305\ mm$$

Cross section properties for Box-girder

$I_T = \sum I_x + \sum A_x d_x^2$ (4-63)

$Z_t = I_T / Y_t$ (4-64)

$Z_b = I_T / Y_b$ (4-65)

Where

A_c= Concrete area for different section (m^2)

Y_t = Distance of highest point from the centroid of concrete section (mm)

Y_b = Distance of lowest point from the centroid of concrete section (mm)

I_T = Second moment of area of section (mm^4)

Z_t = Section modulus of top fiber of girder (mm^3)

Z_b = Section modulus of top fiber of girder (mm^3)

Permanent Load

Permanent load are the load due to the weight of different elements in the railway bridges, just including the bottom slab weight

There values are directly deduced from the dimensions of the different elements. We divide the permanent load in two parts

- Self-weight of the girder railway bridge
- Superimposed load

The dead load of different components of the railway bridge is calculated with the following equation.

$$q\left(\frac{kN}{m}\right) = \gamma\left(\frac{kN}{m^3}\right) \times A(m^2)$$

Self-weight of the girder

$q_{sw} = q_{Tslab} + q_{girders} + q_{Tbottom} + q_{diaphrams}$

Superimposed permanent load

$q_{SI} = q_{rail} + q_{Sleepers} + q_{Ballast} + q_{Curb} + q_{Water\ proofing} + q_{catenaries}$

γ = Unit weight of different components table (4.1)

Where

q_{sw} = Total permanent load from self-weight of the girder (kN/m)

q_{SI} = Total permanent load from superimposed load (kN/m)

Self-weight of the girder

For center to center support length (X) lesser or equal to 24m

The bending moment and shear force due to girder dead load + diaphragms at every distance L1a from the support is computed as follows:

For $0 \le X \le L$

$$V_{qsw}(X) = \frac{q_{BG}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_{BG}}{2}(LX - X^2)$$

Where $q_{TG} = q_{Tslab} + q_{\ girders}$ kN/m

$q_d = q_{\ diaphragms}$ kN

For center to center support length (X) greater than 24m

For $0 \le X \le L/2$

$$V_{qsw}(X) = \frac{q_d}{2} + \frac{q_{BG}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_d}{2}X + \frac{q_{BG}}{2}(LX - X^2)$$

For $X > L/2$

$$V_{qsw}(X) = -\frac{q_d}{2} + \frac{q_{BG}}{2}(L - 2X)$$

$$M_{qsw}(X) = \frac{q_d}{2}(L - X) + \frac{q_{BG}}{2}(LX - X^2)$$

Where

M_{qsw} = Moment due to Permanent load from Box-girder + diaphragms (kN-m)

V_{qsw} = Shear force due Permanent load from Box-girder + diaphragms (kN)

- **Superimposed dead load**

The bending moment and shear force due to superimposed dead load at every distance L from the support is computed as follows:

$$V_{qsi}(X) = \frac{q_{SI}}{2}(L - 2X)$$

$$M_{qsi}(X) = \frac{q_{SI}}{2}(LX - X^2)$$

Where

M_{qsi} = Bending Moment due to superimposed load (kN-m)

V_{qsi} = Shear force due to superimposed load (kN)

Girder dead load +diaphragms + super imposed load

The bending moment and shear force due to girder dead load +diaphragms and superimposed load at every distance (X) from the support is computed as follows:

For center to center support length (X) lesser or equal to 24m

For $0 \le X \le L$

$$V_{\text{Permanent}}\ (X) = \frac{q_{\text{TBG}}}{2}(L - 2X)$$

$$M_{\text{Permanent}}\ (X) = \frac{q_{\text{TBG}}}{2}(LX - X^2)$$

For center to center support length (X) greater than 24m

For $0 \leq X \leq L/2$

$$V_{\text{Permanent}}(X) = \frac{q_d}{2} + \frac{q_{\text{TBG}}}{2}(L - 2X)$$

$$M_{\text{Permanent}}(X) = \frac{q_d}{2}X + \frac{q_{\text{TBG}}}{2}(LX - X^2)$$

For $X > L/2$

$$V_{\text{Permanent}}(X) = -\frac{q_d}{2} + \frac{q_{\text{TBG}}}{2}(L - 2X)$$

$$M_{\text{Permanent}}(X) = \frac{q_d}{2}(L - X) + \frac{q_{\text{TBG}}}{2}(LX - X^2)$$

Where

$M_{\text{Permanent}}$ = Moment due to Permanent load from Box-girder + diaphragms + superimposed DL (kN-m)

$V_{\text{Permanent}}$ = Shear force due Permanent load from Box girder + diaphragms +superimposed load (kN)

- **Impact load**

Impact load are calculated following the equation (4-1); (4-2); (4-3)

- **Variable load**

The influence lines for shear force and moment for moving load of Box-Girder railway bridge is similar to that of Slab bridge. Moreover, the influence line coefficients, the equation of moments are the same. The effects of forces due to concentrated and distributed live loads are calculated at all distances (X), the computation stops at the end span length. The maximum value will be taken in consideration for design.

- **Compute Permissible linear stress**

The permissible linear stress for Box-girder are calculated following the equation (4-25).

4.4.2 Design procedures

Maximum un-factored Moment & Shear

The maximum value for the different group loading combination in table (4.3)

Max Un-factored moment & shear are calculated following the equation (4-37); (4-38).

Maximum factored Moment & Shear

The maximum value for the different group loading combination in table (4.4)

Max factored moment & shear are calculated following the equation (4-39); (4-40).

- **Loss of pre-stress**

For the Loss of pre-stress see Table 4.5.

- **Minimum pre-stressed force**

Minimum pre-stressed force is calculated following the equation (4-46).

- **Maximum eccentricity**

The limiting zone For Post-tensioning is defined by four equations obtained by combining the stress condition and the pre-stress equations for the top and bottom fiber:

$$e_1 \leq \left(\frac{-Z_t f_{tt}}{P}\right) + \left(\frac{Z_t}{A}\right) + \left(\frac{M_{permanent}}{P}\right) \tag{4-66}$$

$$e_2 \leq \left(\frac{Z_b f_{ct}}{P}\right) - \left(\frac{Z_b}{A}\right) + \left(\frac{M_{permanent}}{P}\right) \tag{4-67}$$

$$e_3 \geq \left(\frac{-Z_t f_{cs}}{\eta P}\right) + \left(\frac{Z_t}{A}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{\eta P}\right) \tag{4-68}$$

$$e_4 \geq \left(\frac{Z_b f_{ts}}{\eta P}\right) - \left(\frac{Z_b}{A}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{\eta P}\right) \tag{4-69}$$

Actual resultant fiber stress at span

- Stress in concrete at transfer:

  At top following the equation (4-50).

  At bottom following the equation (4-51).

- Stress in concrete at service load :

  At top following the equation (4-52).

  At bottom following the equation (4-53).

- Actual resultant fiber stress at End Anchorages ($e = e_{min}$) :
- Stress in concrete at transfer:

At top following the equation (4-54).

At bottom following the equation (4-55).

- Stress in concrete at service load:

  At top following the equation (4-56).

  At bottom following the equation (4-57).

- **Deflection**

The structural concrete members should be designed to have adequate stiffness to limit deflections, which may adversely affect the strength or serviceability of the structure at working loads.

Members having simple or continuous spans shall be designed so that the deflection due to service live load plus impact does not exceed (L/640) of the span.

$$a = L / 640$$

Suitable control on deflection is very essential for the following reasons

- Excessive, sagging of principal structural members is not only unsightly, but at times also renders the floor unsuitable for the intended use.
- Large deflections under dynamic effects and under the influence of variable loads may cause discomfort to the users.
- Excessive deflections are likely to cause damage to finish, partitions and associated structures.

Deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections and moment of inertia of gross concrete section may be used for uncracked sections.

Additional long-time deflection shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of pre-stressing steel.

Parabolic tendons (eccentric anchored, central anchors).

$$a = \left(\frac{Pl^2}{48EI}\right) * (-5e_1 + e_2)$$

In pre-stressed concrete members, unlike reinforced concrete ones, deflections under a given load can be eliminated entirely. This is achieved by the use of a suitable arrangement of pre-stressing.

Since the stress limits calculated for the members are within the allowable range under the action of all the loadings considered, there is no need to check for the deflections.

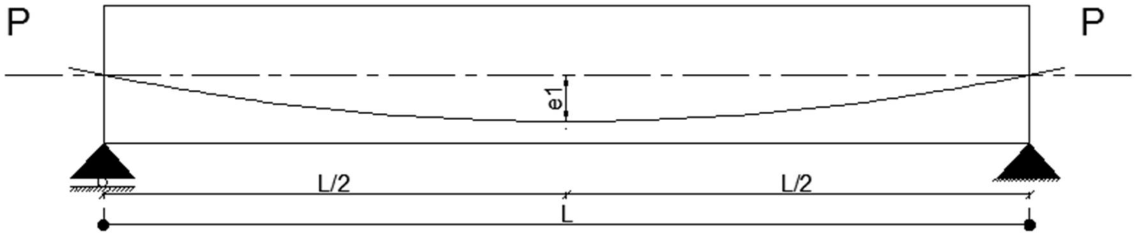


Figure 4.16: Parabolic tendons central anchors

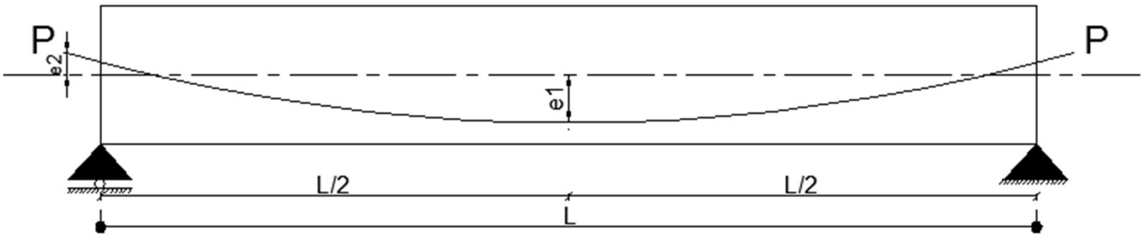


Figure 4.17: Parabolic tendons eccentric anchors

- **Flexural strength** [Note- Flexural strength is ignored for voided slab].

Pre-stressed concrete members may be assumed to act as un-cracked members subjected to combined axial and bending stresses within specified service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included in pre-tensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted. [4]

a) Rectangular sections

For rectangular or flanged sections having pre-stressing steel only, in which the depth of the equivalent rectangular

stress block, defined as (A*sf*su)/(0.85 f 'cb), is not greater than the compression flange thickness “t”, and which satisfy Eq 3-66, the design flexural strength shall be assumed as:

$$\boldsymbol{\varphi Mn = \varphi\left[A^{*}s * f^{*}su * \left\{1 - 0.6\left(\frac{P*fsu}{f'c}\right)\right\}\right]} \tag{4-70}$$

For rectangular or flanged sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as (A*s f*su + As fsy)/(0.85 f'c b), is not greater than the compression flange thickness “t” and which satisfy Eq 3-67, the design flexural strength shall be assumed as:

$$\boldsymbol{\varphi Mn = \varphi\left[A^{*}s * f^{*}su * d\left\{1-0.6\left(\left(\frac{P^{*}f^{*}su}{f'c}\right)+\left(\left(\frac{dt}{d}\right)\left(\frac{pfsy}{f'c}\right)\right)\right)\right\} + As * fsy * dt\left\{1-0.6\left(\left(\frac{d}{dt}\right)\left(\frac{P^{*}f^{*}su}{f'c}\right)+\left(\frac{pfsy}{f'c}\right)\right)\right\}\right]} \tag{4-71}$$

b) Flanged sections

For sections having pre-stressing steel only, in which the depth of the equivalent rectangular stress block, defined as (Asr f*su)/(0.85 f'cb') is greater than the compression flange thickness “t”, and which satisfy Eq 3-67 the design flexural strength shall be assumed as:

$$\boldsymbol{\varphi Mn = \varphi\left[Asr * f^{*}su * d\left\{1-0.6\left(\frac{Asr*f^{*}su}{b`df'c}\right)\right\} + 0.85f`c(b-b')(t)(d-0.5t)\right]}$$
(4-72)

For sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as (Asr f*su)/(0.85 f'cb') is greater than the compression flange thickness “t”, and which satisfy Eq 3-67, the design flexural strength shall be assumed as:

$$\boldsymbol{\varphi Mn = \varphi\left[Asr * f^{*}su * d\left\{1-0.6\left(\frac{Asr*f^{*}su}{b'df'c}\right)\right\} + As\,fsy(dt-d) + 0.85f'c(b-b')(t)(d-0.5t)\right]} \tag{4-73}$$

Where

Asr = A*s - Asf, in EQ 3-62

Asr = A*s + (Asfsy/f*su) - Asf, in EQ 3-63

Asf = 0.85f'c(b-b')t/f*su

Asf =The steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

c) Steel Stress

As an alternative to a more accurate determination of f*su based on strain compatibility, the following approximate values of f*su shall be permitted to be used in accordance with AREMA:

- Bonded Members

With pre-stressing only

$$f^{*}su = f's\left[1-\left(\frac{\gamma^{*}}{\beta_1}\right)\left(\frac{P^{*}f's}{f'c}\right)\right] \quad (4\text{-}74)$$

With non-prestressed tension reinforcement included:

$$f^{*}su = f's\left\{1-\left(\frac{\gamma^{*}}{\beta_1}\right)\left[\left(\frac{P^{*}f's}{f'c}\right)+\frac{d_t}{d}\left(\frac{Pf_{sy}}{f'c}\right)\right]\right\} \quad (4\text{-}75)$$

- Unbonded Members

f*su = fse + 100 (4-76)

Where γ*:

0.28 for low relaxation steel

0.40 for stress-relieved-steel

0.55 for bars

- **Ductility limits** [Note - Ductility limits are ignored for Slab bridge]
  - **Maximum pre-stressing steel**

For rectangular sections

$$\frac{P^{*}f^{*}su}{f'c} \quad (4\text{-}77)$$

For flanged sections

$$\frac{A_{sr}f^{*}su}{b'df'c} \quad (4\text{-}78)$$

does not exceed 0.36 β1

For members with reinforcement indices greater than 0.36β1, the design flexural strength shall be assumed not greater than:

For rectangular sections

$$\varphi Mn = \varphi\left[\left(0.36\beta_1-0.08\beta_1^2\right)f'cbd^2\right] \quad (4\text{-}79)$$

For flanged section

$$\varphi Mn = \varphi\left[\left(0.36\beta_1-0.08\beta_1^2\right)f'cbd^2+0.85f'c(b-b')t(d-0.5t)\right] \quad (4\text{-}80)$$

Minimum reinforcement steel

The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment M*cr.

$$\varphi\ Mn \geq 1.2\ M^{*}cr$$

Where:

$$\boldsymbol{M^{*}cr = \left(f_r + f_{pe}\right)S_c - M_{d/nc}\left(\frac{S_c}{S_b} - 1\right)} \tag{4-81}$$

Appropriate values for $M_{d/nc}$ and S_b shall be used for any intermediate composite sections. Where beams are designed to be non-composite, substitute S_b for S_c in the above equation for the calculation of M^{*}_{cr}

- **Non-prestressed reinforcement**

Non-pre-stressed reinforcement may be considered as contributing to the tensile strength of the beam at design flexural strength in an amount equal to its area times yield strength, provided that:

For rectangular sections:

$$\boldsymbol{\left(\frac{Pf_{sy}}{f'c}\right)\frac{d_t}{d} + \left(\frac{P^{*}f^{*}su}{f'c}\right) - \left(\frac{P'f'_y}{f'_s}\right) \leq 0.36\beta_1} \tag{4-82}$$

For flanged sections:

$$\boldsymbol{\left(\frac{A_s f_{sy}}{b'df'c}\right) + \left(\frac{A_{sr}f^{*}su}{b'df'c}\right) - \left(\frac{A'_s f'_y}{b'df'c}\right) \leq 0.36\beta_1} \tag{4-83}$$

Design flexural strength shall be calculated based on EQ (3-61) or EQ (3-63) if these values are met and on EQ (3-68) or EQ (3-69) if these values are exceeded.

- **Check the ultimate flexural strength of the section**

This is the most important of the ultimate limit states. The structure must be able to withstand, with an acceptable factor of safety against collapse, the loads likely to act upon it.

A similar procedure for Slab Bridge is carried for Box- girder to obtain the design moment and shear.

- **Bearing strength**

The effective concrete bearing compressive strength, the actual bearing stress and the reinforcement area required to resist bursting stress calculations will be similar as those of Slab Bridge.

- **Bottom slab reinforcement**

- Minimum distributed reinforcement of 0.4% of the flange area shall be placed in the bottom slab parallel to the girder span. A single layer of reinforcement may be provided. The spacing of such reinforcement shall not exceed 18 inches (450 mm).
- Minimum 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 18 inches (450 mm). All transverse reinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90-degree hook, [4].

- **Design for shear**

Prestressed concrete flexural members, except solid slabs and footings, shall be reinforced for shear and diagonal tension stresses. Voided slabs shall be investigated for shear, but shear reinforcement may be omitted if the factored shear force, Vu, is less than half the shear strength provided by the concrete

Web reinforcement shall consist of stirrups perpendicular to the axis of the member or welded wire fabric with wires located perpendicular to the axis of the member. Web reinforcement shall extend to a distance d from the extreme compression fiber and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Web reinforcement shall be anchored at both ends for its design yield strength, [4]. [Note: For Slab shear design is ignored]

Members subject to shear shall be designed so that:

$$V_u \leq \varphi (V_c + V_s)$$

Where

Vu = The factored shear force at the section considered

Vc = The nominal shear strength provided by concrete

Vs = The nominal shear strength provided by web reinforcement

- **Shear strength provided by concrete**

As our members have an effective force less than 40 percent of the tensile strength of flexural reinforcement, therefore the shear strength provided by concrete need to be analysis with more precise and shall be taken as the lesser of the values Vci or Vcw.

$$Vc = \mathrm{Min}\ [\ Vci\ ;\ Vcw\ ] \tag{4-84}$$

Flexure-shear strength V_{ci}:

$$\boldsymbol{V_{ci} = Max\left[\left(5 * 10^4 * \sqrt{f'c}b'd + V_d + \left(\frac{V_i * M_{cr}}{M_{max}}\right)\right); 220\sqrt{f'cb'd}\right]} \tag{4-85}$$

Where

The moment causing flexural cracking at the section due to externally applied loads, Mcr, shall be computed by:

$$\boldsymbol{M_{cr} = \left(\frac{I}{y_t}\right)\left(0.5\sqrt{f'e} + f_{pe} - f_d\right)} \tag{4-86}$$

The maximum factored moment and factored shear at the section due to externally applied loads, M_{max} and Vi, shall be computed from the load combination causing maximum moment at the section.

Web-shear strength V_{cw}:

$$\boldsymbol{V_{cw} = 10 * 10^5\left[\left(0.29\sqrt{f'c} + 0.3f_{pe}\right)b'd\right] + V_p} \tag{4-87}$$

and d need not be taken less than 0.8h

Shear strength provided by web reinforcement

Shear reinforcement shall consist of stirrups perpendicular to axis of member or welded wire fabric with wires located perpendicular to axis of member. Shear reinforcement shall be anchored at both ends [4].

- **The shear strength provided by web reinforcement shall be taken as:**

$$\boldsymbol{V_s = Min\left[\left(\frac{A_v f_{sy} d}{s}\right); 0.66\sqrt{f'c}b'd\right]} \tag{4-88}$$

Where

Av = Area of web reinforcement (mm^2)

s = longitudinal spacing of web reinforcement (mm)

- **Minimum shear reinforcement:**

A minimum area of shear reinforcement shall be provided in all flexural members, except: slabs, footings, and shallow beams. [Note - for Voided Slab bridge shear reinforcement are ignored]

The minimum area of web reinforcement shall be:

$$\boldsymbol{A_v} = \frac{(\boldsymbol{0.345 b_w s})}{\boldsymbol{f_{sy}}} \qquad (4\text{-}89)$$

Where b' and s are in (mm) and f_{sy} in (MPa).

The spacing of web reinforcing shall not exceed 0.75h or 24 inches (600 mm). When V_s exceeds

When: $V_s > 0.332\sqrt{f'c}b'd$

Maximum spacing s = 300 mm

When: $V_s < 0.332\sqrt{f'c}b'd$

Maximum spacing s = 600 mm

Summary

- If $\frac{V_u}{\varphi} \leq 0.5V_c$ no web shear is needed.
- If $\frac{V_u}{\varphi} < V_c$ provided minimum reinforcement.
- If $V_s = \frac{Vu}{\varphi} - V_c > 0.66\sqrt{f'c}\, b_w d$ or if $V_s > \varphi\left(V_c + 0.66\sqrt{f'c}\, b_w d\right)$ enlarge the section.
- If $\frac{V_u}{\varphi} > V_c$ and $V_s = \frac{Vu}{\varphi} - V_c \leq 0.66\sqrt{f'c}\, b_w d$ design web steel is required with maximum spacing of 600mm.
- If $\frac{V_u}{\varphi} > V_c$ and $V_s = \frac{Vu}{\varphi} - V_c \leq 0.332\sqrt{f'c}\, b_w d$ design web steel is required with maximum spacing of 300mm.
- **Flange reinforcement**

Bar reinforcement for box girder flanges shall conform to the provisions in [4] except that the minimum reinforcement in bottom flanges shall be 0.3 percent of the flange section [4].

- **Bearing strength** [Note- Bearing strength is not considered for Pre-tension Slab bridge]

The anchorages used to maintain the tendons in tension until the concrete has hardened sufficiently. In case of Post Tensioning members, where prestress is transferred to concrete by means of external anchorages, the bearing pressures developed behind the anchorages have to be investigated and suitably controlled to prevent crushing failure of the end-block zone.

Anchorage devices may be either basic anchorage devices or special anchorage devices both devices meeting the bearing compressive strength limits specified in [1].

The effective concrete bearing compressive strength fb used for design shall not exceed

$$\boldsymbol{f_b = Min\left[\left(0.7f'ci\sqrt{\frac{A}{Ag}}\right); 2.25f'ci\right]} \tag{4-90}$$

Where

f 'ci = the concrete compressive strength at stressing

A = the maximum area of the portion of the supporting surface that is geometrically similar to the loaded area and concentric with it

Ag= the gross area of the bearing plate

- **Actual bearing stress**

The main difference between the anchorages used in post-tensioning and pre-tensioned is that, in the latter, the anchorages should be reusable, but in the former the anchorages must be cast into the member and can only be used once thus regarding this point pre-tensioned appear more economical.

Prestressing force / n*Bearing plate area

The area of reinforcement required to resist bursting stresses

Area = (P * tan34°) / f_{sy}

Total = Area * number of anchorages

Chapter 5: Basics of Demarcation and Practical Example

5.1 Basics of demarcation

The basic theory of basics demarcation is that to obtain the best solution or the equilibrium point, so we can have more than one feasible alternative combinations. In this work cost comparison is considered as parameter for selecting bridges type of the same material. Though, the parameter for the construction of the bridges are total volume of concrete, total amount of pre-stressed tendons and reinforcing bars, area of formwork, and cables duct or their associated costs and anchorage devices are the only costs considered using the unit prices for the program.

5.2 Practical Example of Design of Slab & Box-Girder Railway Bridges

The figure 5.1 show the Longitudinal Section of Slab & Box-Girder Railway Bridge.

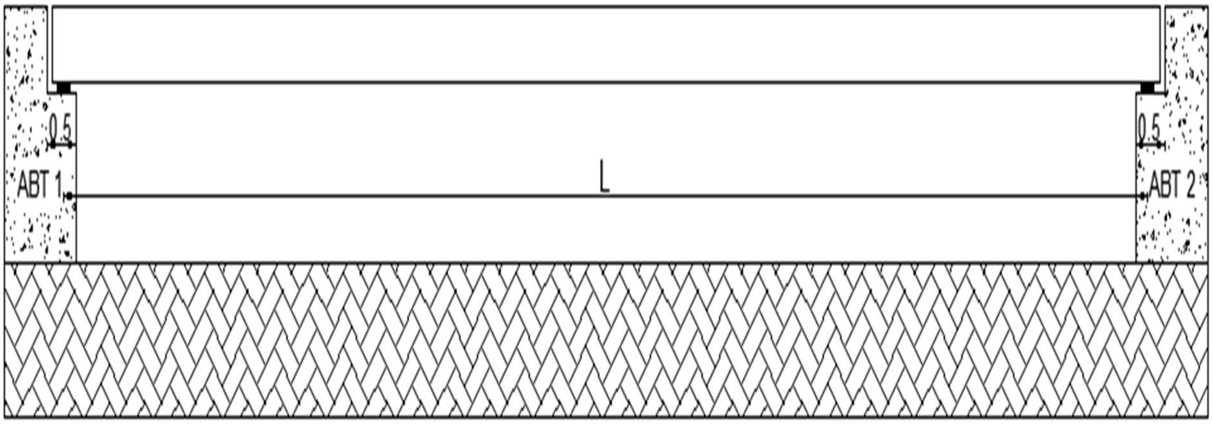


Figure 5.1: Longitudinal Section of Voided Slab & Box-Girder Railway Bridge.

5.2.1 Prestressed Slab Railway Bridges

A Practical example of the results of voided Slab bridges especially near the boundaries are summarized and tabulated in Table 5.1 below.

Table 5.1: Summary of 14m, 15m,16m span Outputs for voided Slab railway bridges

Span Length (m)	Voided Slab dimensions				Pre-stressed		Reinforcements Slab		Reinf Quantity (Kg)	Volume of concrete (m3)	Area of formwork (m2)
	Depth (mm)	Top Slab thick (mm)	Voided diameter (mm)	Spacing between diameter (m)	Pre-stressed force (kN)	Tendons	Longitudinal Direction	Transversal Direction	Grade 420		
14	910	280	500	60% L1= 4.2 40% L2= 2.8	10724	Ø = 15.24 N= 59 L= 14 Kg= 908	Ø = 16 N= 12 L= 14650m	Ø = 13 N= 8 L= 8	317.37	39.78	51.1
15	983	280	500	60% L1= 4.5 40% L2= 3	11595	Ø = 15.24 N= 64 L= 14 Kg= 1052	Ø = 16 N= 12 L= 15650m	Ø= 13 N= 8 L= 8	336	45.5	59.2
16	1055	280	530	60% L1= 4.8 40% L2= 3.2	12208	Ø = 15.24 N= 67 L=16 Kg= 1181	Ø = 16 N= 12 L= 16650m	Ø=13 N= 8 L= 8	240.12	50.82	67.7

5.2.2 Prestressed Box-Girder Railway Bridges

A Practical example of the results of the Box-Girder bridges especially near the boundaries are summarized and tabulated in Table 5.2 below.

Table 5.2: Summary of 14m, 15m,16m span Outputs for Box-Girder railway bridges.

Span Length (m)	Box-Girder dimensions					Pre-stressed				Reinforcements Slab					Reinf Quantity (Kg)	Volume of concrete (m^3)	Area of formwork (m^2)
	Dept. (mm)	Top Slab thick (mm)	Bottom Slab thick (mm)	Web width (mm)	Girder Spacing (m)	Pre-stressed force (kN)	Tendons	Cable duct (m)	Anch. steel plates	Mild Reinf			Web Reinf	Bursting Reinf			
										Bottom Slab		Top					
										Transversal Direct	Long Direct	Long Direct			Grade 420		
14	853	300	200	150	2.6	9725	Ø= 15.24 N= 53 L= 14 Kg= 823	Ø= 100 N= 2 L= 14 28	2	Ø= 13 c/c= 215 L= 2.45	Ø= 16 c/c= 133 L= 14650	Ø=16 L=14650	Ø= 10 L= 1106 N°60	Ø=10 L= 503 N°= 4	448	33	149
15	898	300	200	150	2.6	9873	Ø= 15.24 N= 54 L= 15 Kg=895	Ø= 100 N= 2 L= 15 30	2	Ø= 13 c/c= 215 L= 2.45	Ø= 16 c/c= 133 L15650	Ø=16 L=15650	Ø= 10 L=1196 N°64	Ø=10 L= 548 N°= 4	482	36	162
16	943	300	200	150	2.6	10023	Ø= 15.24 N= 64 L=14 Kg=1052	Ø= 100 N= 2 L= 16 32	2	Ø= 13 c/c= 215 L= 2.45	Ø= 16 c/c= 133 L16650	Ø=16 L=16650	Ø= 10 L=1286 N°68	Ø=10 L= 593 N°= 4	464	38	176

5.3 Results & Discussion

- **Results:**

Table 5.3: Unit Prices for materials

N°	Materials	Unit	Unit Price (Birr)
1	Concrete	m^3	4000
2	Pre-stressed tendons	Kg	400
3	Reinforcing bars	Kg	40
4	Formwork	m^2	400
5	Post-tensioned galvanized metal duct	m	350
6	Elastomeric bearing	Pcs	25000
7	Anchorages devices pre-tensioned	Pcs	1000
8	Anchorages devices post-tensioned	Pcs	5000

An output of the results of the two types of bridges of span ranges from 10 to 20 m and their total associated costs are summarized and tabulated in Table 5.4.

Table 5.4: Total Costs for different span lengths

N°	Span length (m)	Total Cost (Birr)	
		Slab Bridge	Box-girder Bridge
1	10	406 100,17	608 894,24
2	11	478 742,71	664 490,60
3	12	538 567,63	679 641,95
4	13	620 053,46	685 594,52
5	14	658 511,60	690 511,87
6	15	705 702,04	705 020,95
7	16	814 329,60	750 556,88
8	17	889 144,31	847 634,53
9	18	964 490,03	898 742,71
10	19	1 042 310,17	950 155,62
11	20	1 126 834,07	1 036 312,38

The figure 5.1 show a curve of total cost of the superstructures verses span length ranging from 10m to 20m.

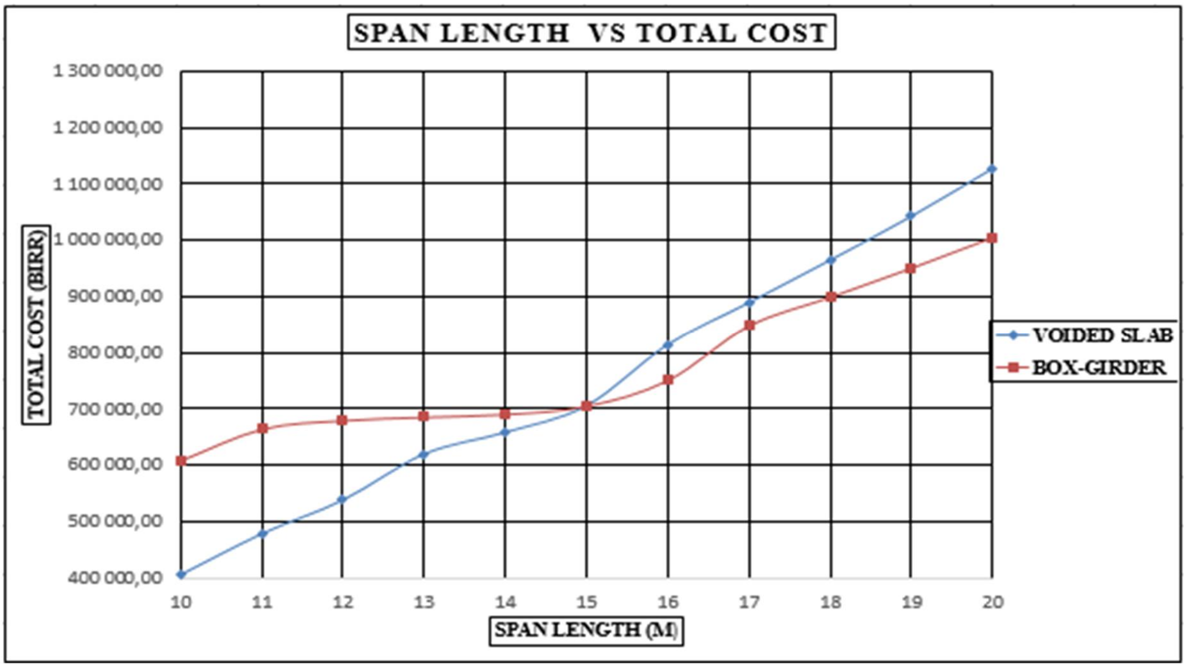


Figure 5.2: Span Length Vs Total Cost

Discussion:

- By comparing the two superstructures it can be seen that the advantage of Voided Slab bridge rather than the Box-girder is based on the anchorage device cost which is very cheap for Voided slab because in this study pre-tensioned system of construction is used for Voided Slab and post-tensioned for Box-Girder railway bridge; see table 5.3.
- However, from the table 5.1 & 5.2 we observe that the reinforced quantity and formwork of Box-Girder are larger than those for Voided slab which is what for minimum span length Voided Slab railway bridge is economical than Box-girder railway bridge.
- While the reason why after some span length the voided slab bridge become expensive from Box-Girder bridge is because of the Voided slab weight starts to become excessive due to volume of concrete; see table 5.4.
- Cost of construction differs from time to time due to the variability in price of construction materials, skilled & unskilled workers, and other associated factors. For these reason, the economical span that demarcates the two types of bridges can vary with time accordingly.
- Finally, the analysis results show that the demarcated span length is 15m.

Chapter 6: Conclusions and Recommendations

6.1 Conclusions

One of the aim of the thesis was to fix the selection problem of type of bridge for a defined as Slab or Box-girder railway bridge moreover the thesis has shown that it’s possible to produce our own computer program to facilitate, make fast and as simple as possible our daily design works.

Nevertheless, the condition survey has been carried in order to assess the practice in selecting appropriate type of bridge structure. Thus, there are in situation where a river/flow stream requires opening ranging between 6 to 15 m, the practice note is an enlarged of the opening to build T- or Box-girder or frame structure irrespectively of cost. The relevant reason is because of their formwork equipment. Designing alone is not enough but they have to be regularly inspected and appropriately maintained and this need be also accounted in future works.

Per the observation of the result from table 5.4 and figure 5.2 of cost comparison for superstructures of the two types of bridges using the prevailing cost, it is found that a span of 15m is the demarcation span for selecting Voided Slab or Box-Girder bridge. However, it has introduced Voided Slab bridge where opening up to 15m becomes more economical as compared to Box-Girder.
As the analysis results show it found that the two bridges superstructure total cost are very closer, therefore considering all cost of bridges may be offers accuracy for the selection.

Though for bridge types selection, not only the cost of construction is taken into account but also the cost of future expenditures during the projected service life of the bridge should be considered.

Finally, the conclusion that can be drawn from this study is that the engineer must choose the shape he considers is best in all the circumstances such as regional factors, maintenance costs and methods of construction.

6.2 Recommendations

Suggestions for further work within the subject:

1. Comparison to demarcate the position of T-Girder in between Slab & Box-Girder need to be carried.
2. Development of guidelines for modelling in 3D the comparative analysis of pre-stressed between Slab and Box-Girder railway bridges.
3. It would be possible better compare Slab and Box-Girder bridges with more additional parameters in the program like time cost, maintenance cost or factor influencing the degradation cost.
4. Further research is needed to understand the application comparative analysis using 3D shell and 2D frame models, a prestressed concrete bridge design with FEM.

Reference

[1] Abiy Alemu, Master`s thesis, Computer program for comparative study of the analysis and Design of T and Box-girder bridges, AAU 2013

[2] Abrham Gebre, Master`s thesis, Computer program for comparative study of the analysis and Design of Slab and T-girder bridges, AAU 2006

[3] American Association of State Highway and Transportation Officials. (2004), LRFD Bridge Design Specifications, 3rd edition, Washington, AASHTO

[4] American Railway Engineering and Maintenance-of -way Association, AREMA, 2010.

[5] Dr. Amlan K Sengupta and Prof. Devdas Menon Ppt “Prestressed Concrete structures”.

[6] Asnake Adamu (Dr.), Lecture Note of Structural Civil Engineering, AAiT.

[7] Baruch Feigenbaum, High-Speed Rail in Europe and Asia: Lessons for the United States, May 2013.

[8] CCEC Technical Proposal for Construction of Ethiopian Railway, Miesso-Dawanle Project, November 2011.

[9] CREC Report and Feasibility Study_September_2012, 2012.

[10] Charles E. Reynolds and James C. Steedman, Reinforced Concrete Designer’s Handbook, Tenth edition.

[11] Edward G.Nawy, Prestressed concrete a fundamental approach Fifth edition.

[12] Ethiopian Railways Corporation, “Ethiopia/Sebeta-Djibouti/Nagad Railway, Feasibility Study, General Specification,” Executive Edition, Part I, September 2012.

[13] Ginés de Rus, Economic Evaluation of the High-Speed Rail, University of Las Palmas de G.C. University Carlos III de Madrid, May 2012.

[14] International journal of Construction Research in Civil Engineering.

[15] John F. Unsworth, Design of Modern Steel Railway Bridges, 2010.

[16] Khaireh Mohamed, Master`s thesis, Economical Analysis & Design for the Selection of a Prestressed T- & Box-girder Railway Bridges, AAU 2016.

[17] N Krishna Raju, Prestressed concrete fourth edition, 2007.

[18] Precast/ Prestressed Concrete Institute, PCI Bridge Design Manual, oct 1997.

[19] Robert Benaim, The design of prestressed concrete bridges concepts and principles, 2007.

[20] Wai-Fah Chen & Lian Duan Bridge Engineering HandBook 2000 CRC Press LLC.

[21] www.fdlake.com/rig-slng.html.

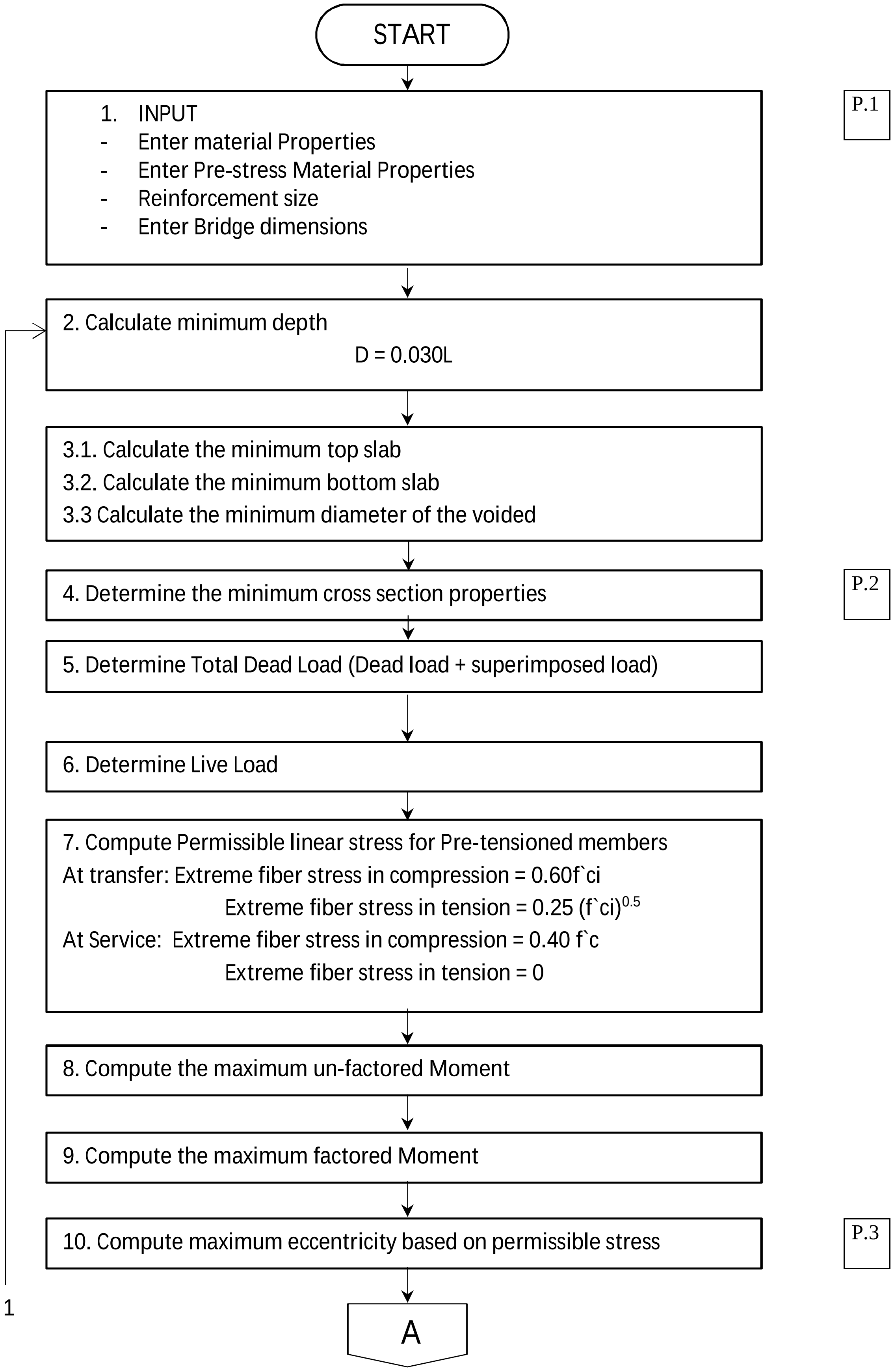
[22] www.quora.com/What -is- prestressed - concrete - and - what - is - the - advantage of - prestressed - concrete - over - R-C-C.

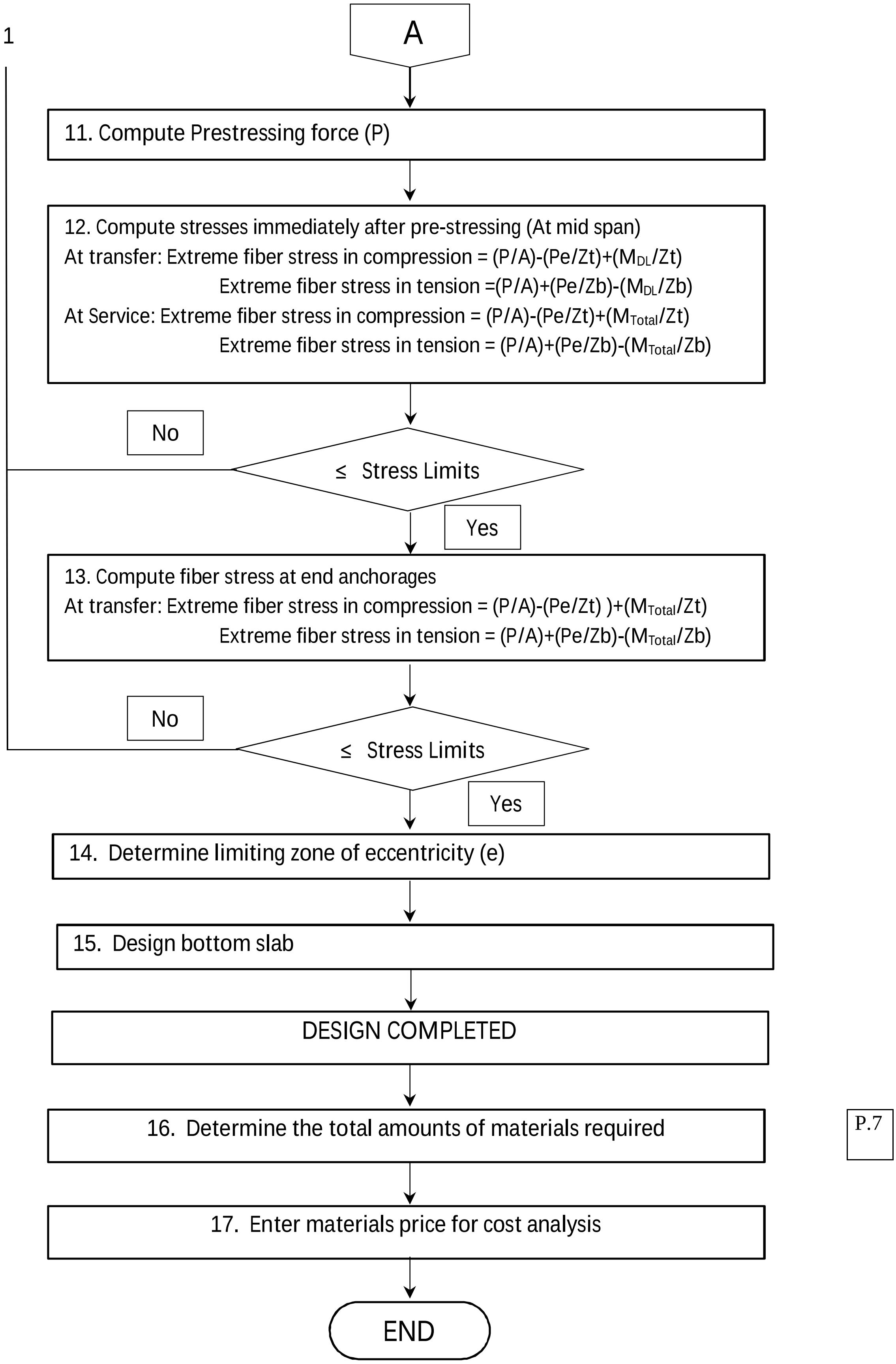
[23] www.en.wikipedia.org/wiki/Bridge.

Appendix A

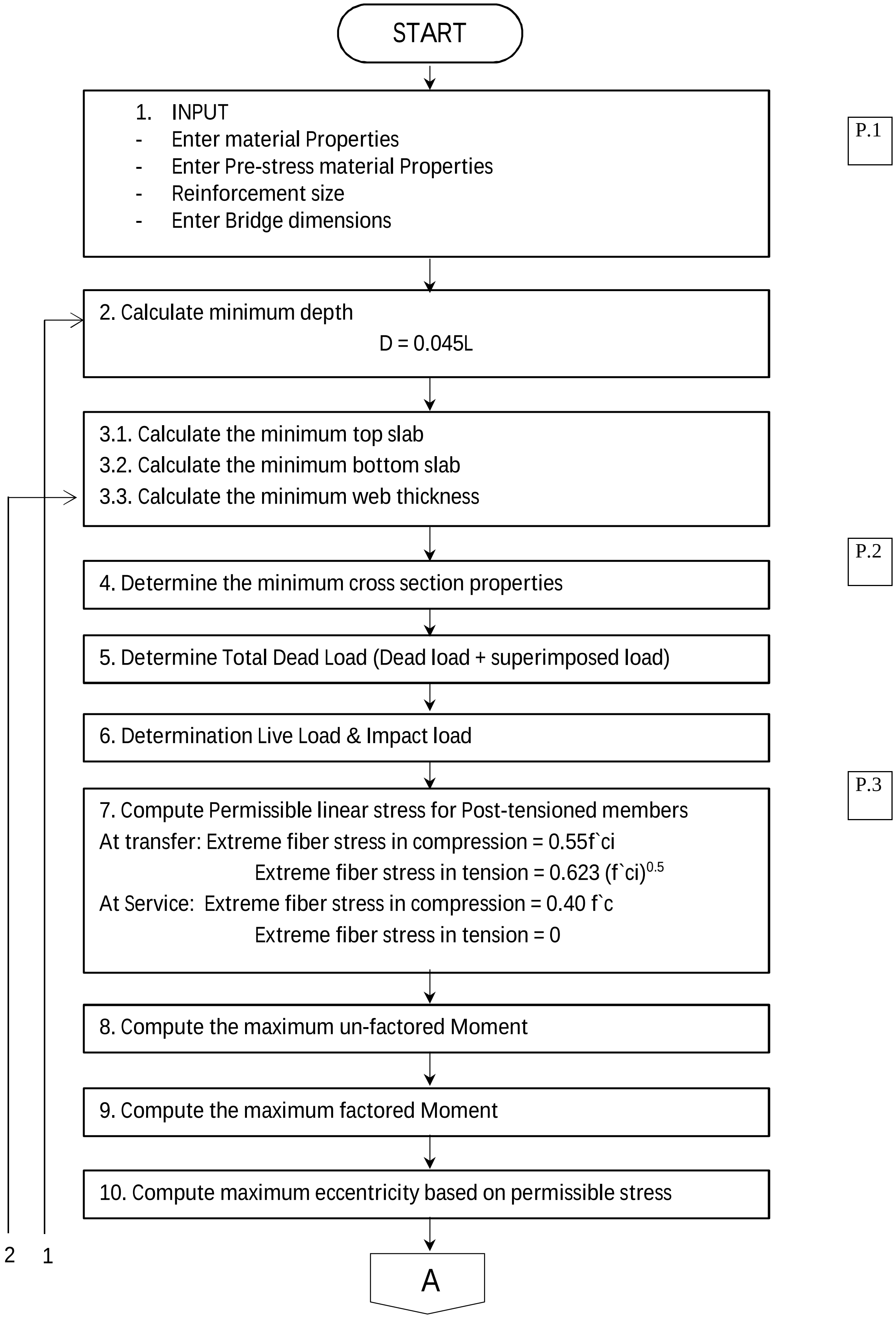
FLOW CHARTS

➢ FLOWCHART FOR PRE-STRESSED VOIDED SLAB BRIDGE




A
1
11. Compute Prestressing force (P)
12. Compute stresses immediately after pre-stressing (At mid span)
At transfer: Extreme fiber stress in compression = (P/A)-(Pe/Zt)+(M_{DL}/Zt)
Extreme fiber stress in tension =(P/A)+(Pe/Zb)-(M_{DL}/Zb)
At Service: Extreme fiber stress in compression = (P/A)-(Pe/Zt)+(M_{Total}/Zt)
Extreme fiber stress in tension = (P/A)+(Pe/Zb)-(M_{Total}/Zb)
No
≤ Stress Limits
Yes
13. Compute fiber stress at end anchorages
At transfer: Extreme fiber stress in compression = (P/A)-(Pe/Zt))+(M_{Total}/Zt)
Extreme fiber stress in tension = (P/A)+(Pe/Zb)-(M_{Total}/Zb)
No
≤ Stress Limits
Yes
14. Determine limiting zone of eccentricity (e)
15. Design bottom slab
DESIGN COMPLETED
16. Determine the total amounts of materials required
P.7
17. Enter materials price for cost analysis
END

➢ FLOW CHART FOR BOX-GIRDER BRIDGE



1. INPUT
- Enter material Properties
- Enter Pre-stress material Properties
- Reinforcement size
- Enter Bridge dimensions

2. Calculate minimum depth

$D = 0.045L$

3.1. Calculate the minimum top slab

3.2. Calculate the minimum bottom slab

3.3. Calculate the minimum web thickness

4. Determine the minimum cross section properties

5. Determine Total Dead Load (Dead load + superimposed load)

6. Determination Live Load & Impact load

7. Compute Permissible linear stress for Post-tensioned members

At transfer: Extreme fiber stress in compression = $0.55f`ci$

Extreme fiber stress in tension = $0.623\ (f`ci)^{0.5}$

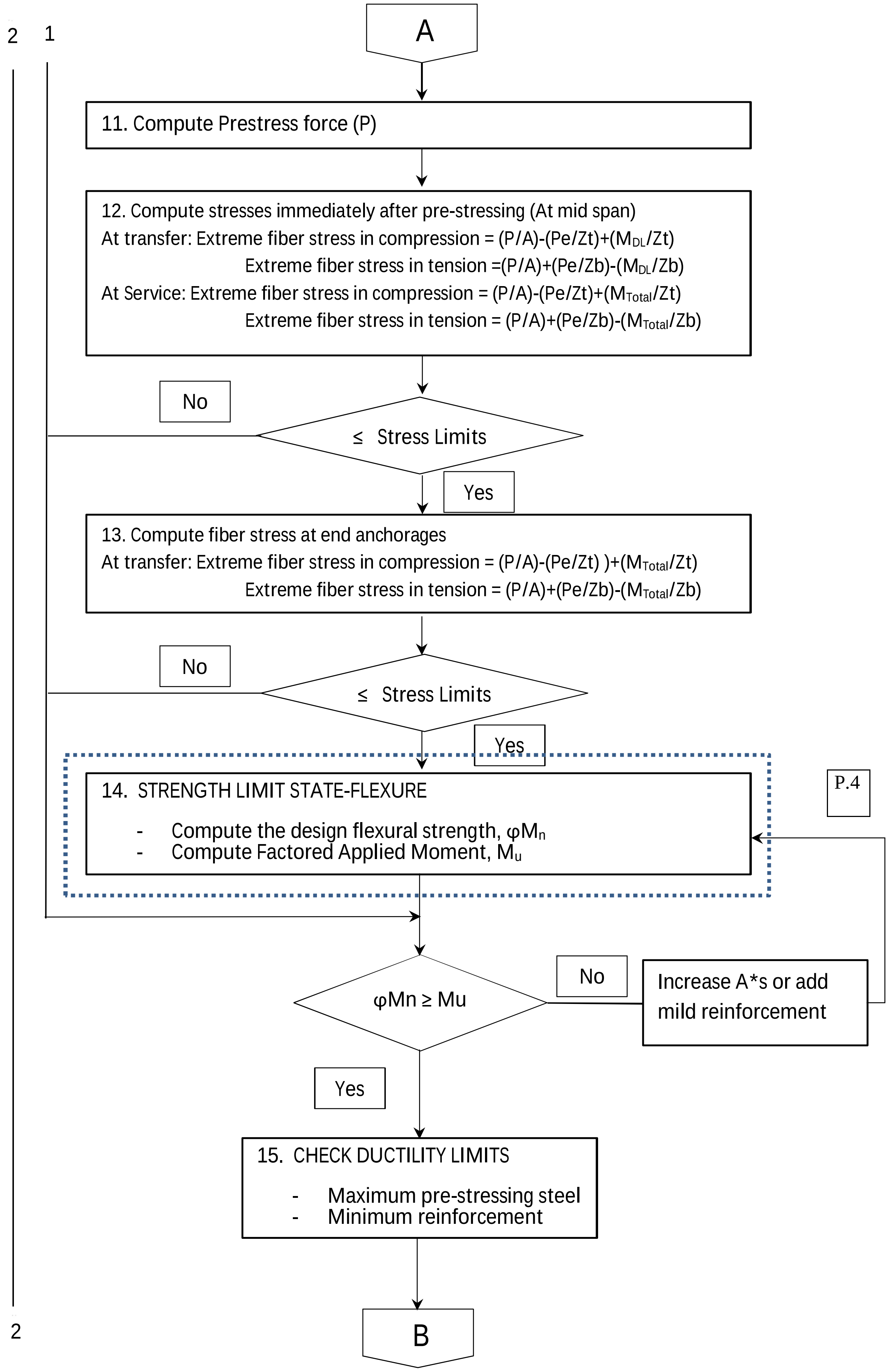
At Service: Extreme fiber stress in compression = $0.40\ f`c$

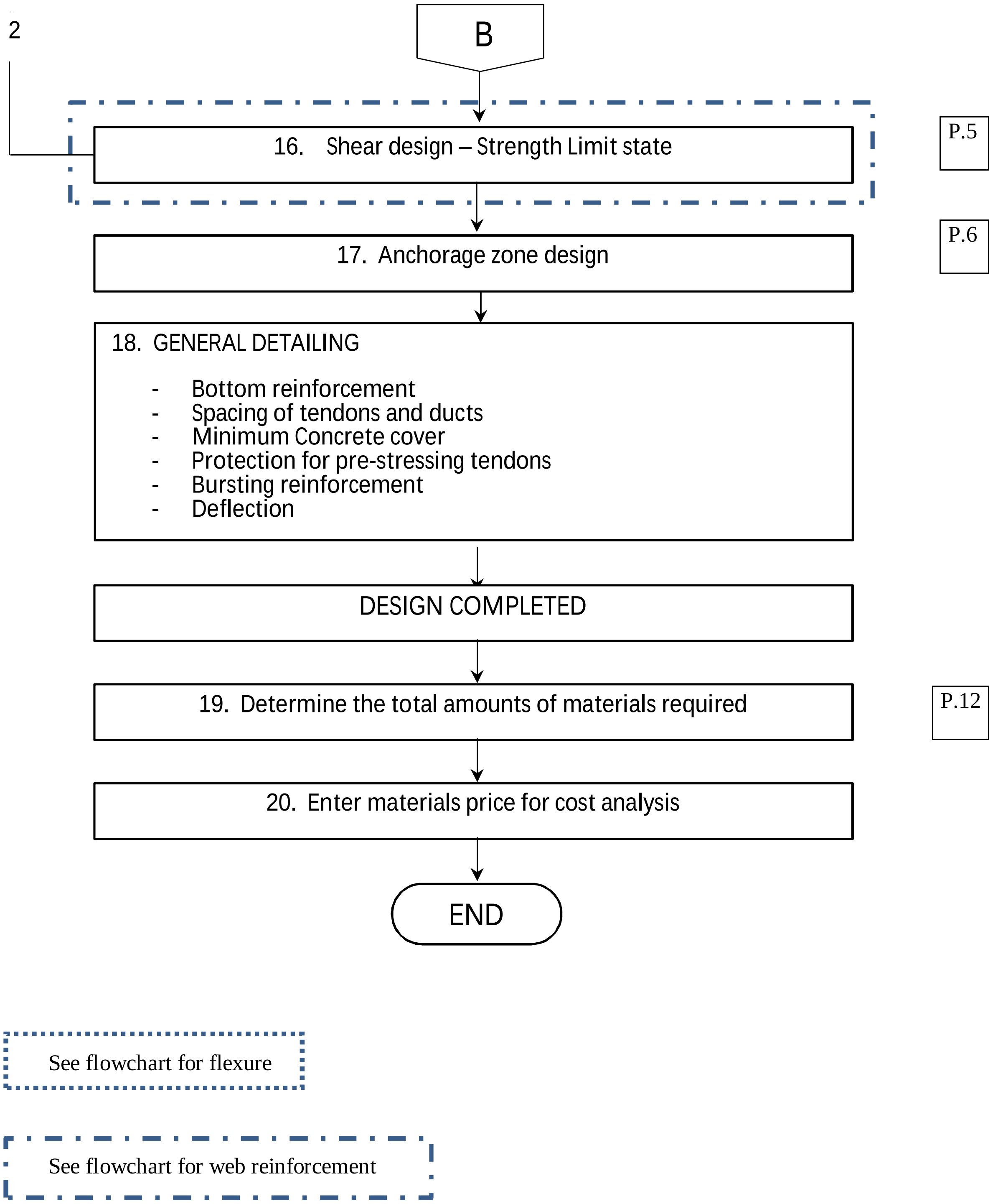
Extreme fiber stress in tension = 0

8. Compute the maximum un-factored Moment

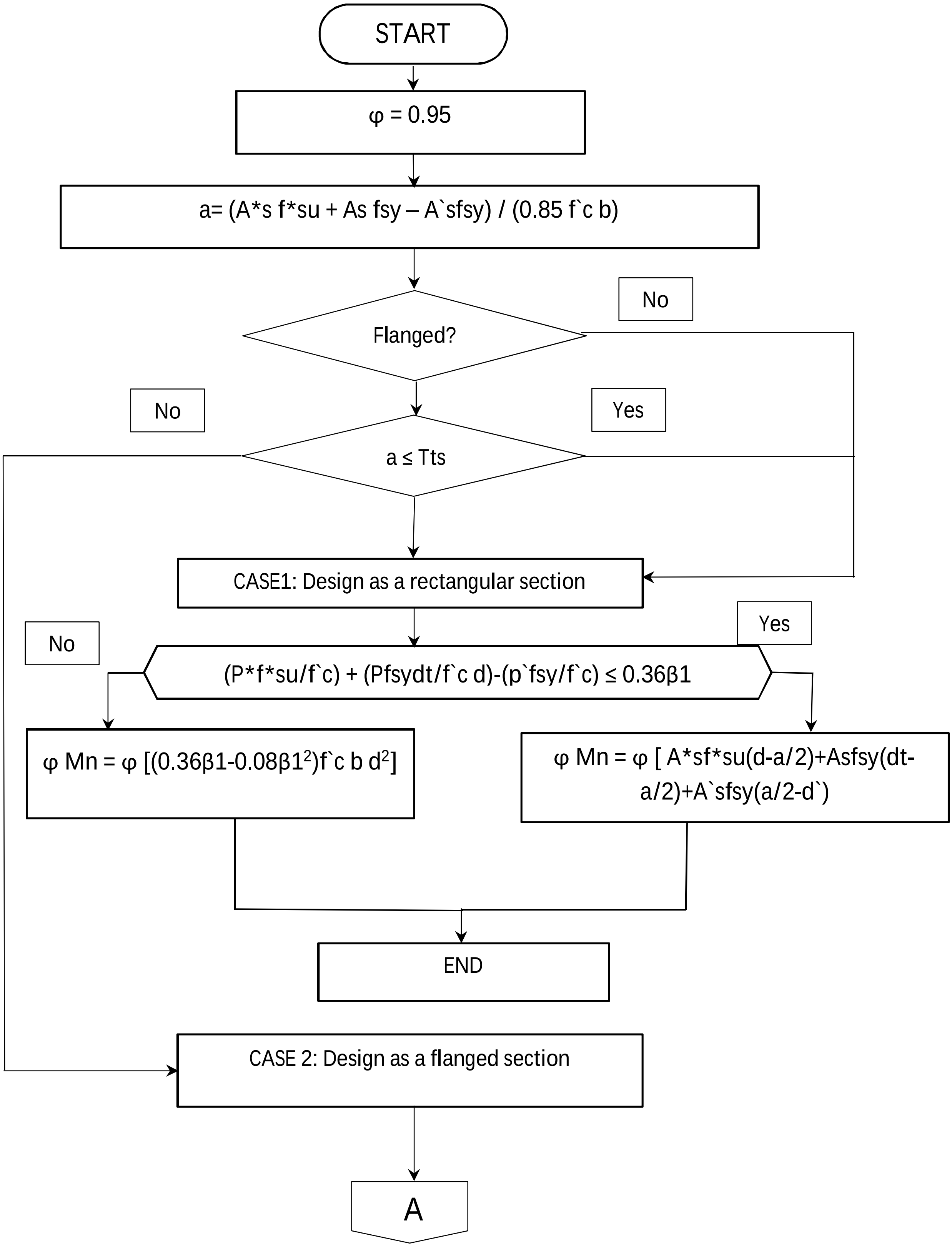
9. Compute the maximum factored Moment

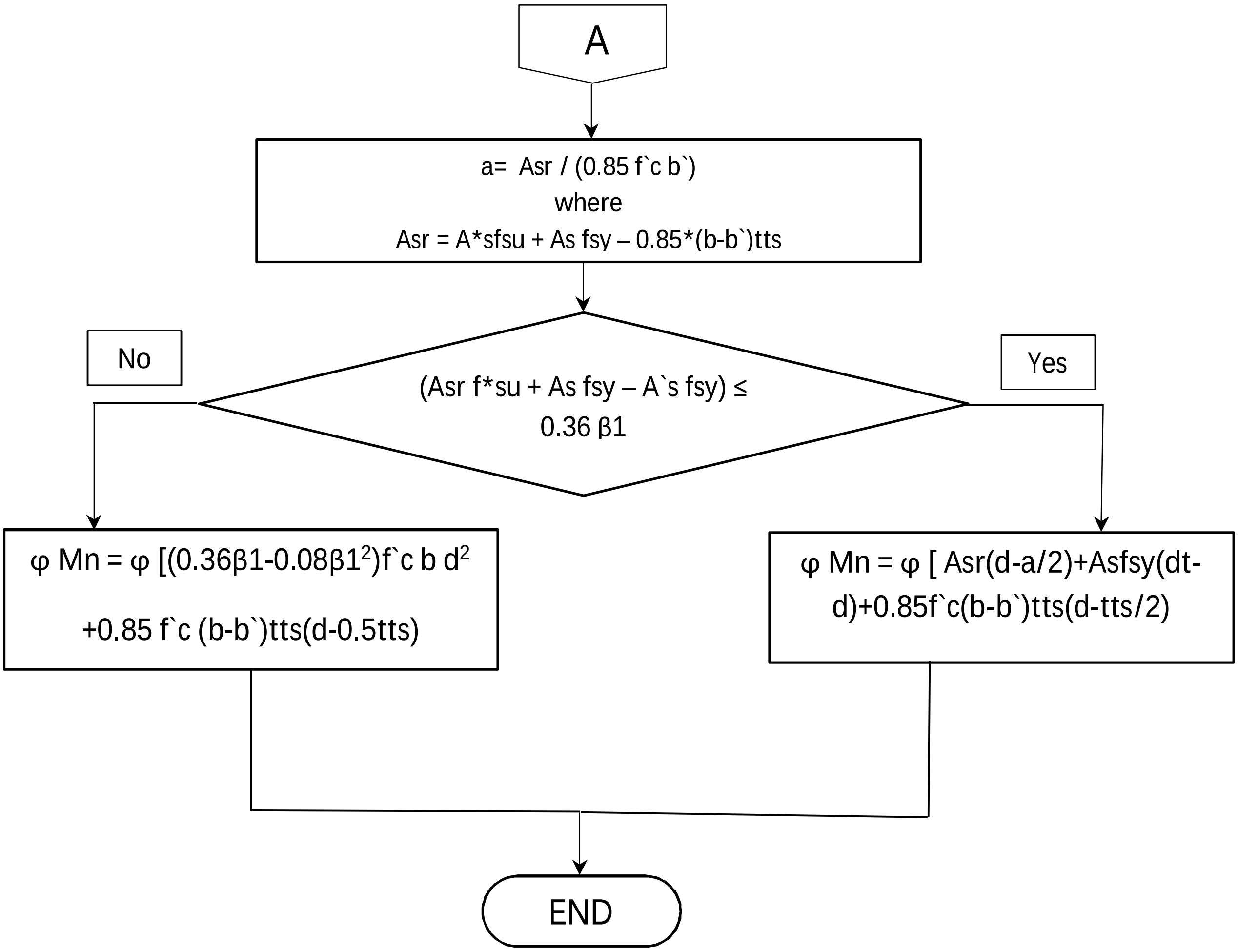
10. Compute maximum eccentricity based on permissible stress


2
1
A
11. Compute Prestress force (P)
12. Compute stresses immediately after pre-stressing (At mid span)
At transfer: Extreme fiber stress in compression = (P/A)-(Pe/Zt)+(M_{DL}/Zt)
Extreme fiber stress in tension =(P/A)+(Pe/Zb)-(M_{DL}/Zb)
At Service: Extreme fiber stress in compression = (P/A)-(Pe/Zt)+(M_{Total}/Zt)
Extreme fiber stress in tension = (P/A)+(Pe/Zb)-(M_{Total}/Zb)
No
≤ Stress Limits
Yes
13. Compute fiber stress at end anchorages
At transfer: Extreme fiber stress in compression = (P/A)-(Pe/Zt))+(M_{Total}/Zt)
Extreme fiber stress in tension = (P/A)+(Pe/Zb)-(M_{Total}/Zb)
No
≤ Stress Limits
Yes
14. STRENGTH LIMIT STATE-FLEXURE
- Compute the design flexural strength, φM_n
- Compute Factored Applied Moment, M_u
P.4
φMn ≥ Mu
No
Increase A*s or add mild reinforcement
Yes
15. CHECK DUCTILITY LIMITS
- Maximum pre-stressing steel
- Minimum reinforcement
2
B

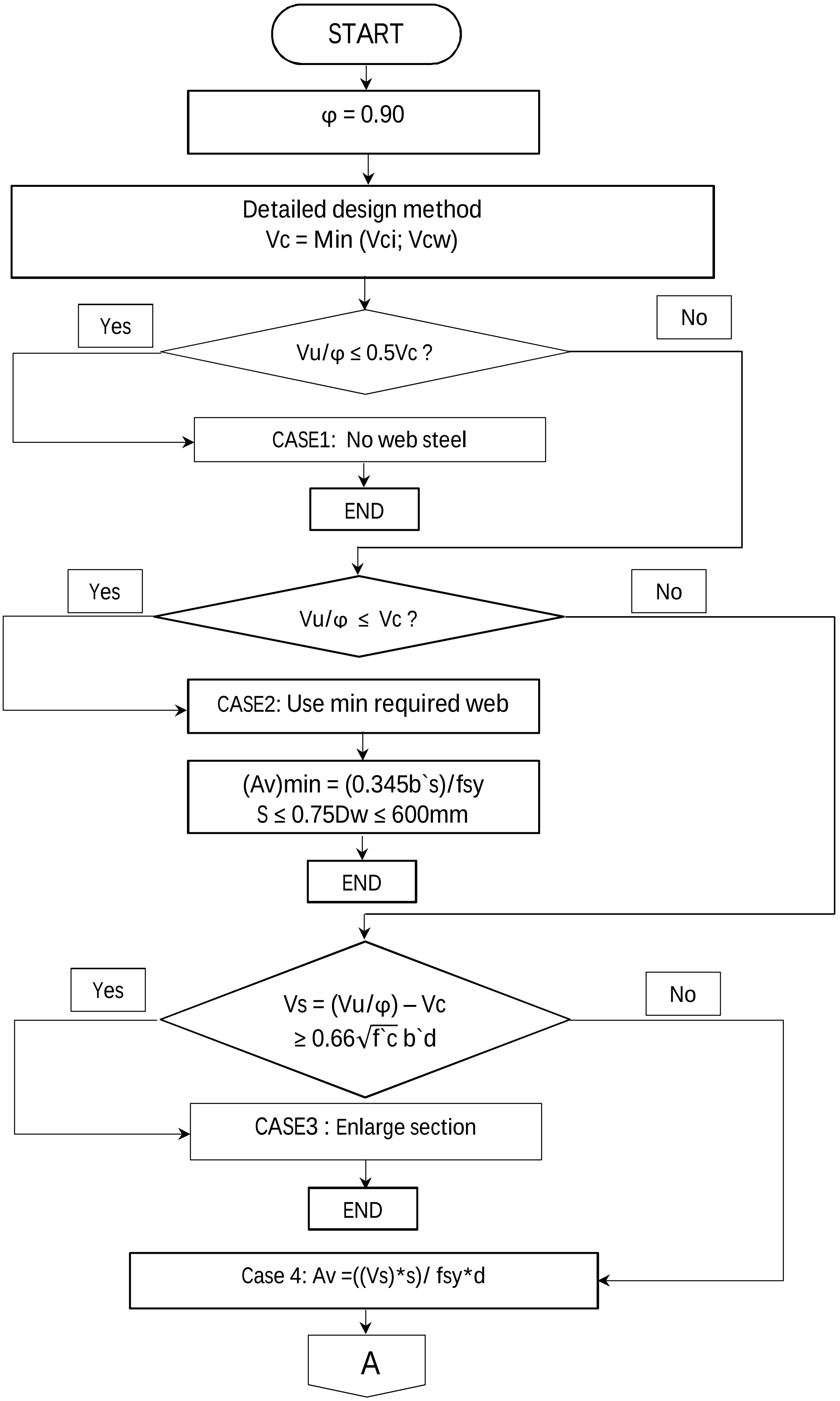

2
B
16. Shear design – Strength Limit state
P.5
17. Anchorage zone design
P.6
18. GENERAL DETAILING
- Bottom reinforcement
- Spacing of tendons and ducts
- Minimum Concrete cover
- Protection for pre-stressing tendons
- Bursting reinforcement
- Deflection
DESIGN COMPLETED
19. Determine the total amounts of materials required
P.12
20. Enter materials price for cost analysis
END
See flowchart for flexure
See flowchart for web reinforcement

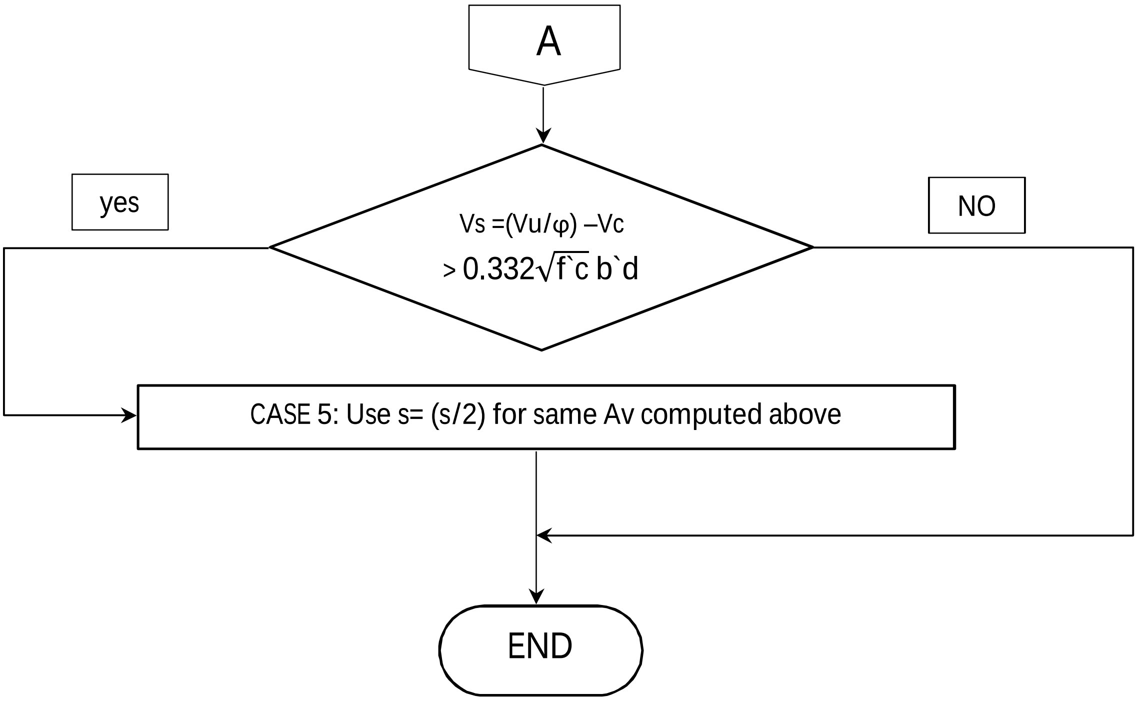
➢ Flowchart for flexural analysis of rectangular and flanged prestressed section based on cgs profile depth




A
a= Asr / (0.85 f`c b`)
where
Asr = A*sfsu + As fsy – 0.85*(b-b`)tts
No
Yes
(Asr f*su + As fsy – A`s fsy) ≤ 0.36 β1
φ Mn = φ [(0.36β1-0.08β1^2)f`c b d$^2$ +0.85 f`c (b-b`)tts(d-0.5tts)
φ Mn = φ [Asr(d-a/2)+Asfsy(dt-d)+0.85f`c(b-b`)tts(d-tts/2)
END

➢ **Flow chart for Shear web reinforcement**




A
yes
Vs =(Vu/φ) –Vc
> 0.332√f`c b`d
NO
CASE 5: Use s= (s/2) for same Av computed above
END

Appendix B

EXCEL DESIGN OUTPUTS OF PRE-STRESSED VOIDED SLAB BRIDGE

ECONOMIC DESIGN COMPARISON OF PRE-STRESSED CONCRETE SLAB & BOX-GIRDER RAILWAY BRIDGES	Prepared by	Ibrahim Abdoulkader
	Advisor :	Dr. Asnake Adamu
	Internal examiner:	
	External examiner:	
ADDIS ABABA INSTITUTE OF TECHNOLOGIE **SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING**	JUN 2017	

DESIGN OF PRE-STRESSED PRE-TENSIONED VOIDED SLAB RAILWAY BRIDGE

▨ Given cells

MATERIAL	Concrete	Grade f''c =	50	wc (kg/m^3) =	2400	f_r (Mpa)=	3,052	Cover(mm) =	50
		f'_{ck} (Mpa)=	40	E_c (Mpa)=	31,975	E_{ci} (Mpa)=	24,768		
	At transfer	f'_{ci} (Mpa)=	24	f'_{tt} (Mpa)=	2,456				
		go(N/Kg) =	10						

Pre-tens	Strand	Grade=	270	f's (N/mm^2) =	1860	LR or SR =	LR
		Nominal Ø(mm) =	15,24	f'y (N/mm^2) =	1675		
		Cross section Area(mm^2) =	140,00	Ultimate load (KN)=	260,7		
		Weight (kg/m)=	1,102				

Mild steel	Grade=	S-400	f_y (N/mm^2)=	400	fsy (N/mm2) =	347,83
	Esi (N/mm^2) =	210000				
Mild steel	Transversal Ø =	10	Area (mm^2)=	71	weight(Kg/m)=	0,56
	Prestressing system =		Pre-tensioned			

Bridge Dimension	Bridge	Span length=	20	Dw (m) =	3,8	Gauge (m) =	1,435
		Bw(m) =	5,6	ABT1 (m) =	0,5	ABT2(m) =	0,5
	Sleepers	S_w (m) =	2,2	S_d (m)=	0,2	S_h (m) =	0,25
	Curb	Fw(m) =	0,9	F_h (m) =	0,5	t_{pt} (mm) =	50
		Bottom slab (A) (m) =	2,6	S_{Lp} spacing =	0,6	W_{pw} (Kg/m^3) =	900
		Jh (mm) =	160	d_b (mm) =	440	W_{bl}(kg/m^3) =	1900
		C/C support length (m)	20,5	Cx (m) =	0,60		

Analysis	Minimum depth recommended AASHTO (D) 0,030L	1329	(mm)
	Minimum top slab thickness (T_{ts}) (S+3000)/20	280	(mm)
	Minimum Diameter of the voided (D_1) Ø ≤ 0.5H	665	(mm)

VOIDED SLAB BRIDGE

Cross Section Properties	VSB						
	Yt (mm) =	552	Zt (mm^3) =	1268404190	I_{cgc} (mm^4) =	700379753282	
	Yb (mm)=	777	Zb (mm^3) =	901591488	Ac (m^2) =	3,601	

Dead Load	Weight of the Slab per linear meter	86,418	kN/m

Diaphrams	DVS (m) =	0,5	0,000	kN
	Diaphragm is ignored for Voided Slab			

Super Imposed Load	Weight of rail guard rail fastenings (W_{RL})	3	kN/m
	Weight of sleepers (W_{SL})	4,400	kN/m
	Weight of ballast (W_{bl})	17,632	kN/m
	Weigt of curb (W_{cb})	21,6	kN/m
	Weight of water proofing (W_{wp})	1,755	kN/m
	Additionnal weight (Wa)	0,25	kN/m
	Catenarie	0,02	kN/m

Total super imposed dead Load	DSI	48,657	kN/m

Total Dead Load	Dead load of Slab + Super imposed load	135	kN/m

Live Load	Live Load Distribution width(m)=	2,64	Vmax (KN)=	1737	kN
	(Length of tie + $d_{b)}$		Mmax (KN-m) at mid =	7599,25	kN-m
			Peq LL =	144,6615	kN/m

Impact Load	For L ≤ 4m 60%	Impact factor	impact load (KN-m)	
	For 4< L ≤ 39m 125/(L$^{(1/2)}$)%	0,28	2097,9919	kN-m
	For L > 39m 20%		39,937979	kN/m
		40		

ALLOWABLE STRESSES	Concrete	Transfer	Compression (fct) =	24,00	N/mm^2
			Tension (ftt) =	1,22	N/mm^2
		Service	Compression (fcs) =	16,00	N/mm^2
			Tension (fts) =	0	N/mm^2
	Steel		Initial stress in tendons (fpi) =	1374	N/mm^2
			The effective prestress (fpe) =	1302	N/mm^2

Maximum Moments	Maximum Unfactored moments M_{DL} =	7096	kN-m
	Maximum Unfactored moments due to Total load =	16793	kN-m
	Maximum factored moments due to DL =	9934	kN-m
	Maximum factored moments due to Total Load =	32561	kN-m
	Maximum moments at end =	0	kN-m

Maximum tensile	f_{top} =	-4,37	N/mm^2
Stress	f_{bottom} =	18,63	N/mm^2

Maximum eccentricty	e_{max}=	649	mm	---

Loss	Pre-tensioned	η	22,00%	0,78
	Post-tensioned	η	18,00%	0,82

Minimum	Pmin or Peff =	31939	kN
Pre-stressing	P =	14053	kN
Force	Number of strands =	77	strands
	Area of prestressing steel =	10793	mm^2

Checking the stress mid span	Concrete	Tansfer	Top fiber =	-5001,881	N/mm^2	SAFE
			Bottom fiber =	24,00	N/mm^2	SAFE
		Service	Top fiber =	5,764	N/mm^2	SAFE
			Bottom fiber =	21,325	N/mm^2	SAFE

Checking stresses at the end	Concrete at transfer	Top fiber =	9	N/mm^2	SAFE
		Bottom fiber =	9	N/mm^2	SAFE

Limiting Zone of Eccentricity	e=	112	(mm)	OK

Design bottom slab

longitudinal

0,50%	1400	mm^2/m
S =	142,14286	mm
As =	1400	mm^2/m

Ø (mm) =	16	Number =	8	Area =	199	Length	20650

Required As =	1400	<	1592	=Provided As	OK

Nominal mass (kg/m) 1,552

Quantity (Kg) = 256,390

As provided= 1400 mm2	Use #8 Ø16 mm Rebars
C/C Spacing = 142,1 mm	Length = 20650 mm

Transversal

0,40%	1120	mm^2/m
S =	230	mm
As =	560	mm^2/m

Ø (mm) =	13	Number =	6	Area =	129	Length	5,600

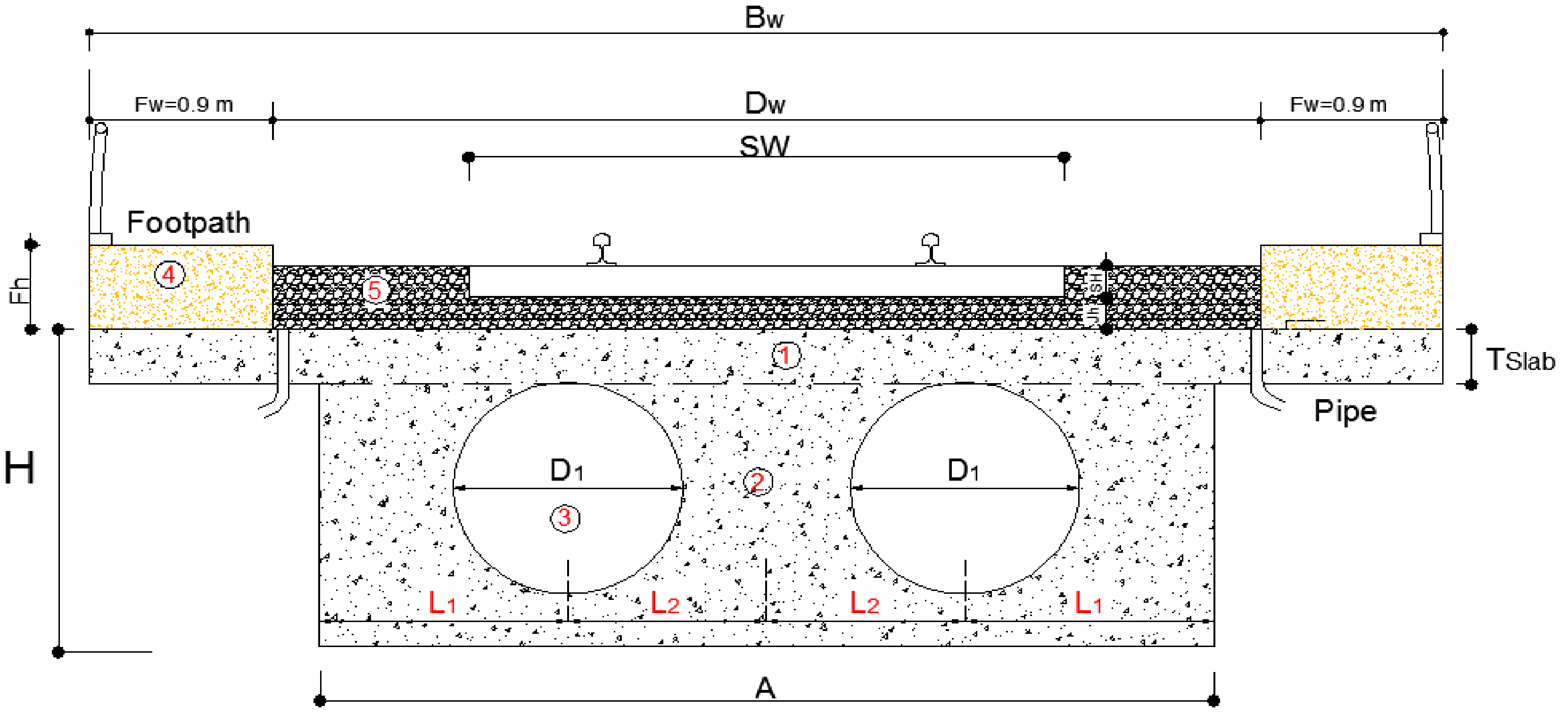
Required As =	560	<	774	=Provided As	OK

Nominal mass (kg/m) 0,994

Quantity (Kg) = 33,398

As provided= 560 mm2	Use #6 Ø13 mm Rebars
C/C Spacing = 230 mm	Length = 5,6 mm

Section Properties



Voided Slab Bridge: Section Properties

	b(mm)	b'(mm)	h(mm)	Area(mm2)	y(mm)	A*y(mm2)
A1	5600		280	1568000	1189	1864352000
A2	2600		1049	2727400	525	1430521300
A3	-	-	-	347324	716,5	248857298
-				347324	716,5	248857298,5
				3600753		2797158703

Yt (mm) =	552
Yb (mm) =	777

	b(mm)		h(mm)	Area(mm2)	Ix(mm4)	d(mm)	Ad^2(mm4)	Icgc(mm4)
A1	5600	0	280	1568000	1,024E+10	412	266383389742	276627656409
A2	2600		1049	2727400	2,501E+11	252	173649289590	423752096873
A3	-	-	-	347324	4,909E+10	60	1263990914	50351490914
-	0		0	347323,515	4,909E+10	60	1263990914	50351490914
					2,603E+11			700379753282

Zt (mm^3) =	1268404190
Zb (mm^3) =	901591487,8

ANALYSIS OF LONGITUDINAL GIRDER

Dead Load force effect

Girder dead load

MTG = DG + DP

X	M_{TG}
0	0,000
2,05	1634,274
4,1	2905,376
6,15	3813,305
8,2	4358,063
10,25	4539,649
12,3	4358,063
14,35	3813,305
16,4	2905,376
18,45	1634,274
20,5	0,000
Max	**4539,649**

X	V_{TG}
0	885,7852306
2,05	708,6281845
4,1	531,4711384
6,15	354,3140922
8,2	177,1570461
10,25	0
12,3	-177,1570461
14,35	-354,3140922
16,4	-531,4711384
18,45	-708,6281845
20,5	-885,7852306
Max	**885,7852306**

Super Imposed dead load

X	M_{SI}
0	0
2,05	920,1646913
4,1	1635,84834
6,15	2147,050946
8,2	2453,77251
10,25	2556,013031
12,3	2453,77251
14,35	2147,050946
16,4	1635,84834
18,45	920,1646913
20,5	0
Max	**2556,013**

X	V_{SI}
0	498,734
2,05	448,861
4,1	398,987
6,15	349,114
8,2	299,241
10,25	249,367
12,3	199,494
14,35	149,620
16,4	99,747
18,45	49,873
20,5	0,000
Max	**498,734**

Total Dead Load (M_{TG} + M_{SI})

X	M_{DL}
0	0,000
2,05	2554,438
4,1	4541,224
6,15	5960,356
8,2	6811,836
10,25	7095,662
12,3	6811,836
14,35	5960,356
16,4	4541,224
18,45	2554,438
20,5	0,000
Max	**7095,662**

X	V_{DL}
0	1384,519481
2,05	1107,615584
4,1	830,7116884
6,15	553,8077922
8,2	276,9038961
10,25	0
12,3	-276,9038961
14,35	-553,8077922
16,4	-830,7116884
18,45	-1107,615584
20,5	-1384,519481
Max	**1384,519481**

Impact Load

X	M_{I}
0	0,000
2,05	755,277
4,1	1342,715
6,15	1762,313
8,2	2014,072
10,25	2097,992
12,3	2014,072
14,35	1762,313
16,4	1342,715
18,45	755,277
20,5	0,000
Max	**2097,992**

X	V_{I}
0	409,364
2,05	327,491
4,1	245,619
6,15	163,746
8,2	81,873
10,25	0,000
12,3	-81,873
14,35	-163,746
16,4	-245,619
18,45	-327,491
20,5	-409,364
Max	**409,364**

VOIDED SLAB BRIDGE

		Moments		Shears	
D =	$M_{DL} = M_{TG} + M_{SI}$	7095,662	KN-m	1384,519481	KN
L =	M_{LL}	7599,250	KN-m	1736,76	KN
I =	M_{I}	2097,992	KN-m	409,364	KN

Group loading combinations-Service load design

		Moment	Shear
I	D + L + I + CF + E + B + SF =	16792,904	3530,645
II	D + E + B + SF + W =	7095,662	1384,519
III	Group I + 0.5W + WL + LF + F =	16792,904	3530,645
IV	Group I + OF =	16792,904	3530,645
V	Group II + OF =	7095,662	1384,519
VI	Group III + OF =	16792,904	3530,645
VII	Group I + ICE =	16792,904	3530,645
VIII	Group II + ICE =	7095,662	1384,519
	Unfactored Max =	**16792,904**	**3530,645**

Group loading combinations-Load Factor Design (AREMA Table 8-2-5)

		Moment	Shear
I	1.4 (D + 5/3 (L + I) + CF + E + B + SF) =	32560,825	6945,953
IA	1.8 (D + L + I + CF + E + B + SF)=	30227,228	6355,161
II	1.4 (D + E + B + SF + W)=	9933,927	1938,327
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)=	23510,066	4942,903
IV	1.4 (D + L + I + CF + E + B + SF + OF)=	23510,066	4942,903
V	Group II + 1.4 (OF)=	9933,927	1938,327
VI	Group III + 1.4 (OF)=	23510,066	4942,903
VII	1.0 (D + E + B + EQ)=	7095,662	1384,519
VIII	1.4 (D + L + I + E + B + SF + ICE)=	23510,066	4942,903
IX	1.2 (D + E + B + SF + W + ICE)=	8514,795	1661,423
	Factored Max =	**32560,825**	**6945,953**

Design Moment & shear (Unfactored)

M_{UNF} =	**16 792,90**	kN-m
V_{UNF} =	**3 530,64**	kN

Design Moment & shear (Factored)

M_{FT} =	**32 560,83**	kN-m
V_{FT} =	**6 945,95**	kN

Table 7.1 : Unit Prices for materials

20m span

N°	Materials	Unit	Unit Price (Birr)	QUANTITY	TOTAL AMOUNT SLAB-Girder
1	Concrete	m^3	4000	72,74	290 940,84
2	Pre-stressed tendons	kg	400	1 699,19	679 674,48
3	Reinforcing bars	kg	40	289,79	11 591,55
4	Formwork	m^2	400	106,57	42 627,20
5	Cable Duct	m	350	0,00	0,00
7	Steel bearings	Pcs	25000	4,00	100 000,00
8	Anchorage devices	Pcs	1000	2,00	2 000,00

TOTAL (Birr)	**1 126 834,07**

Pre-stressed Box-girder railway bridge is economical for the given span length

Appendix C

EXCEL DESIGN OUTPUTS OF PRE-STRESSED BOX-GIRDER BRIDGE

ECONOMIC DESIGN COMPARISON OF PRE-STRESSED CONCRETE SLAB & BOX-GIRDER RAILWAY BRIDGES	Prepared by	Ibrahim Abdoulkader
	Advisor :	Dr. Asnake Adamu
	Internal examiner:	
	External examiner:	
ADDIS ABABA INSTITUTE OF TECHNOLOGIE **SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING**	JUN 2017	

DESIGN OF PRE-STRESSED POST-TENSIONED BOX-GIRDER RAILWAY BRIDGE

▨ Given cells

MATERIAL	Concrete	Grade f''c =	50	wc (kg/m^3) =	2400	f_r (Mpa)=	3,052	Cover(mm) =	50
		f'_c (Mpa)=	40	E_c (Mpa)=	31,975				
		f'_{ci} (Mpa)=	24	E_{ci} (Mpa)=	24,768				
		go(N/Kg) =	10						

Post-tens	Strand	Grade=	270	f's (N/mm^2) =	1860	LR or SR =	LR
		Nominal Ø(mm) =	15,24	f'y (N/mm^2) =	1675		
		Cross section Area(mm^2) =	140,00	Ultimate load (KN)=	260,7		
		Weight (kg/m)=	1,102				

Mild steel	Grade=	S-400	f_y (N/mm^2)=	400	fsy (N/mm2) =	347,83
	Esi (N/mm^2) =	210000				
Mild steel	Transversal Ø =	10	Area (mm^2)=	71	weight(Kg/m)=	0,56

Prestressing system = Post-tensioned

Bridge	Bridge	Span length=	20	Dw (m) =	3,8	Gauge (m) =	1,435
Dimension		Bw(m) =	5,6	ABT1 (m) =	0,5	ABT2(m) =	0,5
	Sleepers	S_w (m) =	2,2	S_d (m)=	0,2	S_h (m) =	0,25
	Curb	Fw(m) =	0,9	F_h (m) =	0,5	t_{pt} (mm) =	50
		Bottom slab (A) (m) =	2,6	S_{Lp} spacing =	0,6	W_{pw} (Kg/m^3) =	900
		Jh (mm) =	160	d_b (mm) =	160	W_{bl}(kg/m^3) =	1900
		C/C support length (m)	20,5	Cx (m) =	0,60		

Analysis	Minimum depth recommended AASHTO (Hmin) 0,045L	1223	(mm)
	Minimum top slab thickness (T_{ts}) (S+3000)/17	329	(mm)
	Minimum web thickness (bw)	305	(mm)
	Minimum bottom slab thickness (T_{bs}) Max(1/16((GS*10^-3)-bw);150)	218	(mm)

Box-GIRDER

Cross Section Properties	Box-Girder					
	Yt (mm) =	444	Zt (mm^3) =	1,09E+09	I_{cgc} (mm^4) =	4,83548E+11
	Yb (mm)=	779	Zb (mm^3) =	6,21E+08	Ac (m^2) =	2,888

Dead Load	Weight of the Girder per linear meter	69,313	kN/m

Diaphram	Dt (m) =	0,5	qdiaphragms	13,301	kN

Super Imposed Load	Weight of rail guard rail fastenings (qrail)	3	kN/m
	Weight of sleepers (qsleepers)	4,400	kN/m
	Weight of ballast (qballast)	17,632	kN/m
	Weigt of curb (qcurb)	21,6	kN/m
	Weight of water proofing (qwater proofing)	1,755	kN/m
	Additionnal weight (q additional)	0,25	kN/m
	Catenarie (q catenaries)	0,02	kN/m

Total superimposed dead Load	DSI	48,657	kN/m

Total Dead Load	Dead load of girder + Super imposed load	117,970	kN/m

Live Load	Live Load Distribution width(m)=	2,36	Vmax (KN)=	1737	kN
	(Length of tie + d_b)		Mmax (KN-m) at mid =	7599,25	kN-m
			Peq LL =	144,6615	kN/m

Impact Load	For L ≤ 4m 60%	Impact factor	impact load (KN-m)	
	For 4< L ≤ 39m 125/($L^{(1/2)}$)%	0,28	2097,991936	kN-m
	For L > 39m 20%		39,93797856	kN/m

Box-GIRDER

ALLOWABLE STRESSES	Concrete	Transfer	Compression (fct) =	22,00	N/mm^2
			Tension (ftt) =	3,05	N/mm^2
		Service	Compression (fcs) =	16,00	N/mm^2
			Tension (fts) =	0	N/mm^2
	Steel		Initial stress in tendons (fpi) =	1374	N/mm^2
			The effective prestress (fpe) =	1302	N/mm^2

Maximum Moments	Maximum Unfactored moments M_{DL} =	6197	kN-m
	Maximum Unfactored moments due to Total load =	8440	kN-m
	Maximum factored moments due to DL =	8676	kN-m
	Maximum factored moments due to Total Load =	15192	kN-m
	Maximum moments at end =	0	kN-m

Maximum tensile Stress	f_{top} =	-2,64	N/mm^2
	f_{bottom} =	13,59	N/mm^2

Maximum eccentricty	e_{max}=	683 mm	------
	$e_{anchorages}$ =	0 mm	

Loss	Pre-tensioned	η	22,00%	0,78
	Post-tensioned	η	18,00%	0,82

Minimum Pre-stressing Force	Pmin or Peff =	9401	kN
	P =	11093	kN
	Number of strands =	61	strands
	Area of prestressing steel =	8520	mm^2

Checking the stress mid span	Concrete	Tansfer	Top fiber =	3,052	N/mm^2	SAFE
			Bottom fiber =	3,611	N/mm^2	SAFE
		Service				
			Top fiber =	5,113	N/mm^2	SAFE
			Bottom fiber =	0,000	N/mm^2	SAFE

Checking stresses at the end	Concrete at transfer	Top fiber =	3	SAFE
		Bottom fiber =	3	SAFE

Number of Duct	Total strand Area (mm2) =	8520
	Ø =	100
	Duct Area (mm2) =	7853,98
	Number of duct (mm2) =	4

Check for Ultimate Strength

φ =	0,95	A^*s =	8520	y^* =	0,28
f^*su =	1817	As =	142	d =	1127,15
f\`c =	40	A\`s =	0	dt =	1168
f\`s=	1860	P^* =	0,0013	P^*_w=	0,01239
fsy =	1675	p =	0,000022	P_w=	0,000199
b =	5600	p\`=	0	$P\`_w$=	0
b\`=	610	β_1 =	0,76	A_{sr} =	40102818,46

Tension

Ø (mm) =	10	Number =	2	Area =	71	Length	20650

Compression

Ø (mm) =	0	Number =	0	Area =	0	Length	0

CASE 1 Rectangular section

Case 2 Flange section

a= 82,5 < 329 =Tts

CASE 1

a=	82,54

Reinforcement indices

0,062 < 0,275 =0,36β1 OK

Design flexural strength

φM_n = 1,6E+10 > 15 191,60 = Md OK

Cracking moment

φ Mn = 1,6E+10 > 6085,795 = φM_{cr} OK

TensIon reinforcement quantity

Nominal mass (kg/m) 0,56

Quantity (Kg) =	23,128

Compression reinforcement quantity

Nominal mass (kg/m) 0,56

Quantity (Kg) =	0,000
Total (Kg) =	23,128

Box-GIRDER

Design for Shear

Vi =	5491303	Mmax =	13982401379	d =	1127,15
Vd =	1209195	Mcr =	-1851844691	φ=	0,90
Vp =	2591258	fpe=	5,11		
Vu =	6700498	fd =	9,98		
Vc =	109195329				

Vu/φ =	7444997,74	<	54597665	= 0,5Vc	(N)
Vu/φ =	7444997,74	<	109195329	= Vc	(N)
Vs =	-101750331,33	<	1435017	=0,66*(f`c)$^{0,5}$b`d	(N)
Vs =	-101750331,33	<	721857	=0,332*(f`c)$^{0,5}$b`d	(N)
Av(m^2/m)=	0,000260	>	3,02518E-07	=Avmin (m^2/m)	

CASE 1

Ø (mm) =	10	Number =	172	Area =	71	s(mm)=	235

Required web steel (m^2/m) 3,02518E-07 < 0,000071 Provided web steel (m^2)

9,07555E-06

9,0756E-06

OK

As provided = 0,000071 mm2 ; Use #172 Ø 10 mm rebars stirrups for 2 webs

C/C spacing = 235 Length (mm) = 2098 No = 170

Nominal mass (kg/m) 0,56 Quantity (Kg) = 202,079

CASE 1 : No Web Steel is required (hint: use minimum)

CASE 2 : Use minimum required web steel

CASE 3 : Enlarge Section

CASE 4 : Select web steel , MIN[min req (s)<0,75d< 600mm]

CASE 5 : Select web steel, MIN[min req (s)<0,375d< 300mm]

Box-GIRDER

Design bottom slab

longitudinal

0,50% 1645 mm^2/m

S = 120,973 mm

As = 1645 mm^2/m

Ø (mm) =	16	Number =	10	Area =	199	Length	20650

Required As = 1645 < 1990 =Provided As OK

Nominal mass (kg/m) 1,552

Quantity (Kg) =	320,488

As provided= 1645 mm2	Use #10 Ø16 mm Rebars
C/C Spacing = 121 mm	Length = 20650 mm

Transversal

0,40% 1316 mm^2/m

S = 196 mm

As = 658 mm^2/m

Ø (mm) =	13	Number =	105	Area =	129	Length	2,295

Required As = 658 < 13587,7 =Provided As OK

Nominal mass (kg/m) 0,994

Quantity (Kg) =	240,284

As provided= 658 mm2	Use #105 Ø13 mm Rebars
C/C Spacing = 196 mm	Length = 2,295 mm

Check Bearing Stress

f_{br} = 25 A = 750000 N° of anchorages = 2

Ag = 750000

f_{br} = 25 < 54 OK

Actual bearing stress = 7,40 < 25 OK

Reinf to resist bursting

Max prestressing force acting of each end anchorages = 5546,57

Bursting force = 3741,21 fsy = 347,83

Required reinforcement (mm^2) = 10756 legs= 4

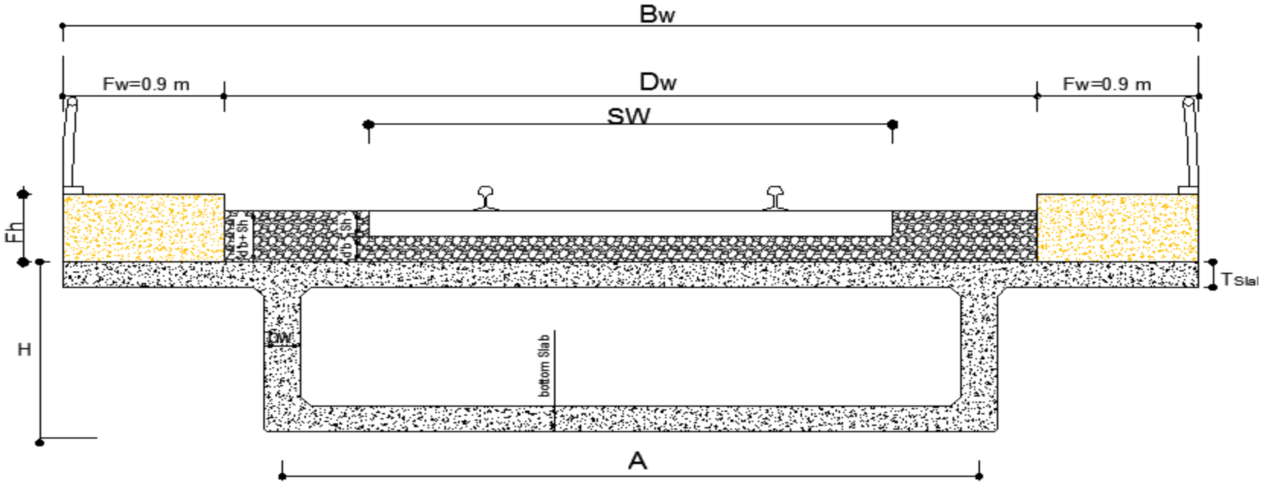
Ø (mm) =	10	Number =	4	Area =	71	length=	844

Nominal mass (kg/m) 0,56

Quantity (Kg) =	1,891

4 bars of 10 mm of diameter will be use around each anchorages device to resist bursting.

Section Properties



Box-Girder : Section Properties

	b	h	Area	y	A*y
A1	5,6	0,329	1,8424	1,0585	1,9501804
2 A2	0,305	0,676	0,41236	0,556	0,22927216
A3	2,905	0,218	0,63329	0,109	0,06902861
			2,88805		2,24848117

Yt (mm) = 444

Yb (mm) = 779

	b	h	Area	Ix	d	Ad^2	Icgc
A1	5,6	0,329	1,8424	0,016619	0,27995	0,14440	0,16101
2*A2	0,305	0,676	0,41236	0,015703	0,22255	0,02042	0,03613
A3	2,905	0,218	0,63329	0,002508	0,66955	0,28390	0,28641
				0,03483			0,48355

Zt (mm^3) = 1087961119

Zb (mm^3) = 621090912,7

ANALYSIS OF LONGITUDINAL GIRDER

Girder dead load

X	Mqsw
0	0,000
2,05	1310,799
4,1	2330,310
6,15	3058,532
8,2	3495,465
10,25	3641,109
12,3	3495,465
14,35	3058,532
16,4	2330,310
18,45	1310,799
20,5	0,000
Max	**3641,109**

X	V_{qsw}
0	710,460
2,05	568,368
4,1	426,276
6,15	284,184
8,2	142,092
10,25	0,000
12,3	-142,092
14,35	-284,184
16,4	-426,276
18,45	-568,368
20,5	-710,460
Max	**710,4603**

Super Imposed dead load

X	Mq_{SI}
0	0
2,05	920,1646913
4,1	1635,84834
6,15	2147,050946
8,2	2453,77251
10,25	2556,013031
12,3	2453,77251
14,35	2147,050946
16,4	1635,84834
18,45	920,1646913
20,5	0
Max	**2556,013**

X	Vq_{SI}
0	498,734
2,05	448,861
4,1	398,987
6,15	349,114
8,2	299,241
10,25	249,367
12,3	199,494
14,35	149,620
16,4	99,747
18,45	49,873
20,5	0,000
Max	**498,734**

Total Dead Load (M_{TG} + M_{SI})

X	Mpermanent
0	0,000
2,05	2230,964
4,1	3966,158
6,15	5205,583
8,2	5949,237
10,25	6197,122
12,3	5949,237
14,35	5205,583
16,4	3966,158
18,45	2230,964
20,5	0,000

X	Vpermanent
0	1209,195
2,05	967,356
4,1	725,517
6,15	483,678
8,2	241,839
10,25	0,000
12,3	-241,839
14,35	-483,678
16,4	-725,517
18,45	-967,356
20,5	-1209,195

Max	**6197,122**

Max	**1209,19455**

Impact Load

X	M_I
0	0,000
2,05	755,277
4,1	1342,715
6,15	1762,313
8,2	2014,072
10,25	2097,992
12,3	2014,072
14,35	1762,313
16,4	1342,715
18,45	755,277
20,5	0,000
Max	**2097,992**

X	V_I
0	409,364
2,05	327,491
4,1	245,619
6,15	163,746
8,2	81,873
10,25	0,000
12,3	-81,873
14,35	-163,746
16,4	-245,619
18,45	-327,491
20,5	-409,364
Max	**409,364**

Box-GIRDER

		Moments		Shears	
D =	Mpermanent	6197,122	kN-m	1209,1946	kN
L =	M_{LL}	144,662	kN-m	1737	kN
I =	M_I	2097,992	kN-m	409,364	kN

Group loading combinations-Service load design

		Moment	**Shear**
I	D + L + I + CF + E + B + SF =	8439,776	3355,320
II	D + E + B + SF + W =	6197,122	1209,195
III	Group I + 0.5W + WL + LF + F =	8439,776	3355,320
IV	Group I + OF =	8439,776	3355,320
V	Group II + OF =	6197,122	1209,195
VI	Group III + OF =	8439,776	3355,320
VII	Group I + ICE =	8439,776	3355,320
VIII	Group II + ICE =	6197,122	1209,195
	Unfactored Max =	**8439,776**	**3355,320**

Group loading combinations-Load Factor Design (AREMA Table 8-2-5)

		Moment	**Shear**
I	1.4 (D + 5/3 (L + I) + CF + E + B + SF) =	13908,829	6700,498
IA	1.8 (D + L + I + CF + E + B + SF)=	15191,596	6039,576
II	1.4 (D + E + B + SF + W)=	8675,971	1692,872
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)=	11815,686	4697,448
IV	1.4 (D + L + I + CF + E + B + SF + OF)=	11815,686	4697,448
V	Group II + 1.4 (OF)=	8675,971	1692,872
VI	Group III + 1.4 (OF)=	11815,686	4697,448
VII	1.0 (D + E + B + EQ)=	6197,122	1209,195
VIII	1.4 (D + L + I + E + B + SF + ICE)=	11815,686	4697,448
IX	1.2 (D + E + B + SF + W + ICE)=	7436,546	1451,033
	Factored Max =	**15191,596**	**6700,498**

Design Moment & shear (Unfactored)

M_{UNF} =	**8 439,78**	kN-m
V_{UNF} =	**3 355,32**	kN

Design Moment & shear (Factored)

M_{FT} =	**15 191,60**	kN-m
V_{FT} =	**6 700,50**	kN

LIMITING ZONE

CABLE PATH			Stress in the bottom fiber at transfer	Stress in the top fiber at transfer	Stress in the bottom fiber at service load	Stress in the top fiber at service load
L1a	Min Moment	Max moment	e<	e<	e>	e>
0,00	0	0	-1017	-676	215	1475
2,05	2230,964	3038,319186	-1218	-877	-108	1152
4,10	3966,158	5401,45633	-1374	-1034	-360	900
6,15	5205,583	7089,411433	-1486	-1145	-539	721
8,20	5949,237	8102,184495	-1553	-1212	-647	613
10,25	6197,122	8439,775516	-1575	-1235	-683	577
12,30	5949,237	8102,184495	-1553	-1212	-647	613
14,35	5205,583	7089,411433	-1486	-1145	-539	721
16,40	3966,158	5401,45633	-1374	-1034	-360	900
18,45	2230,964	3038,319186	-1218	-877	-108	1152
20,50	0	0	-1017	-676	215	1475

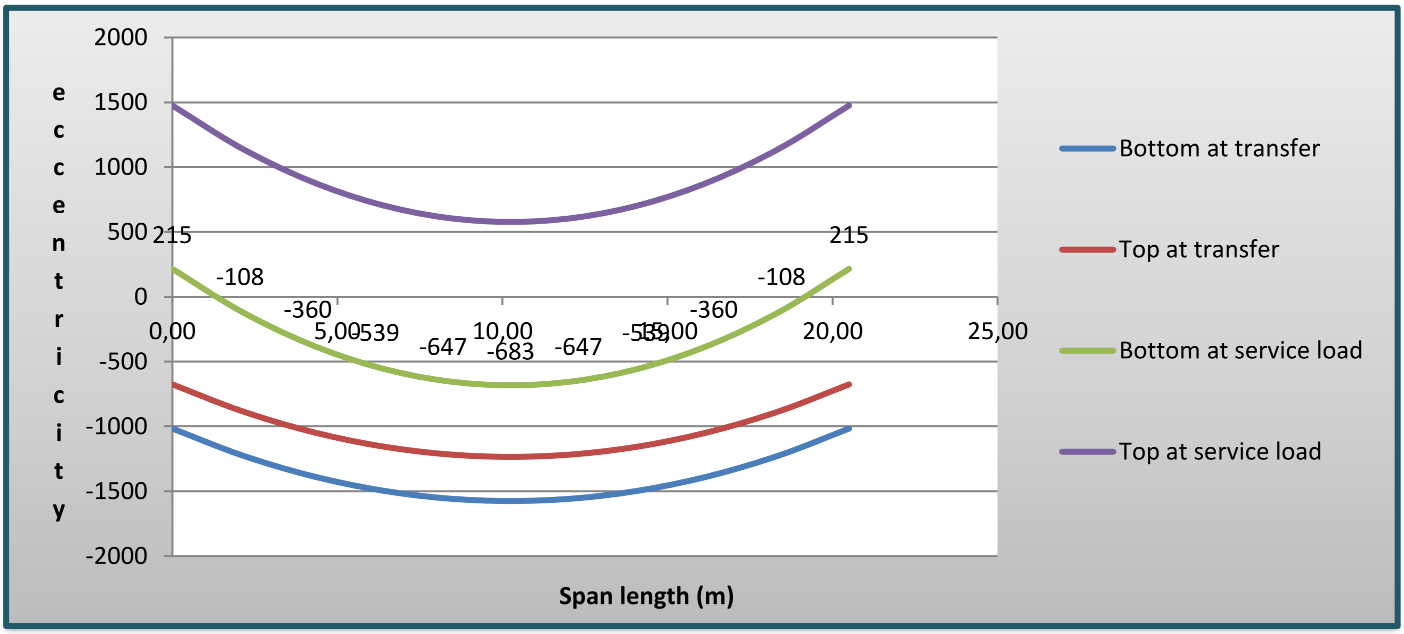


Table 7.1 : Unit Prices for materials

20 m span

N°	Materials	Unit	Unit Price (Birr)	QUANTITY	TOTAL AMOUNT Box-Girder
1	Concrete	m^3	4000	58,34	233 354,44
2	Steel bearing	kg	400	1341,30	536 521,53
3	Reinforcing bars	kg	40	787,87	31 514,81
4	Formwork	m^2	400	242,30	96 921,60
5	Cable Duct	m	350	80,00	28 000,00
7	Steel bearing	Pcs	25000	4,00	100 000,00
8	Anchorage Device Pre-tension	-	5000	2,00	10 000,00

TOTAL (Birr)	1 036 312,38

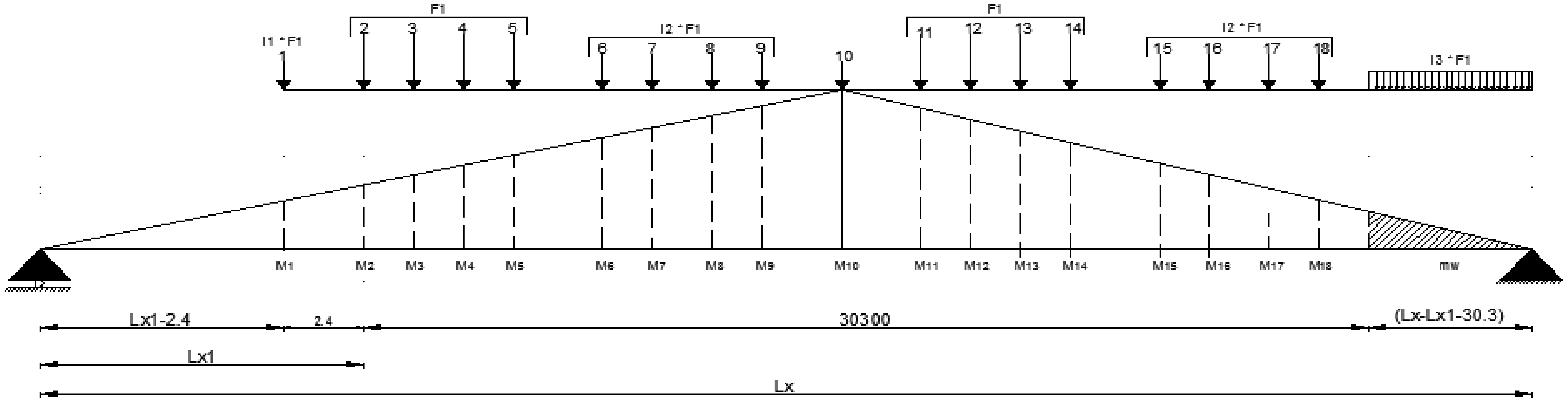
Pre-stressed Box-girder railway bridge is economical for the given span length

Appendix D

EXCEL DESIGN OUTPUTS OF INFLUENCE LINE FOR DYNAMICS LOADS

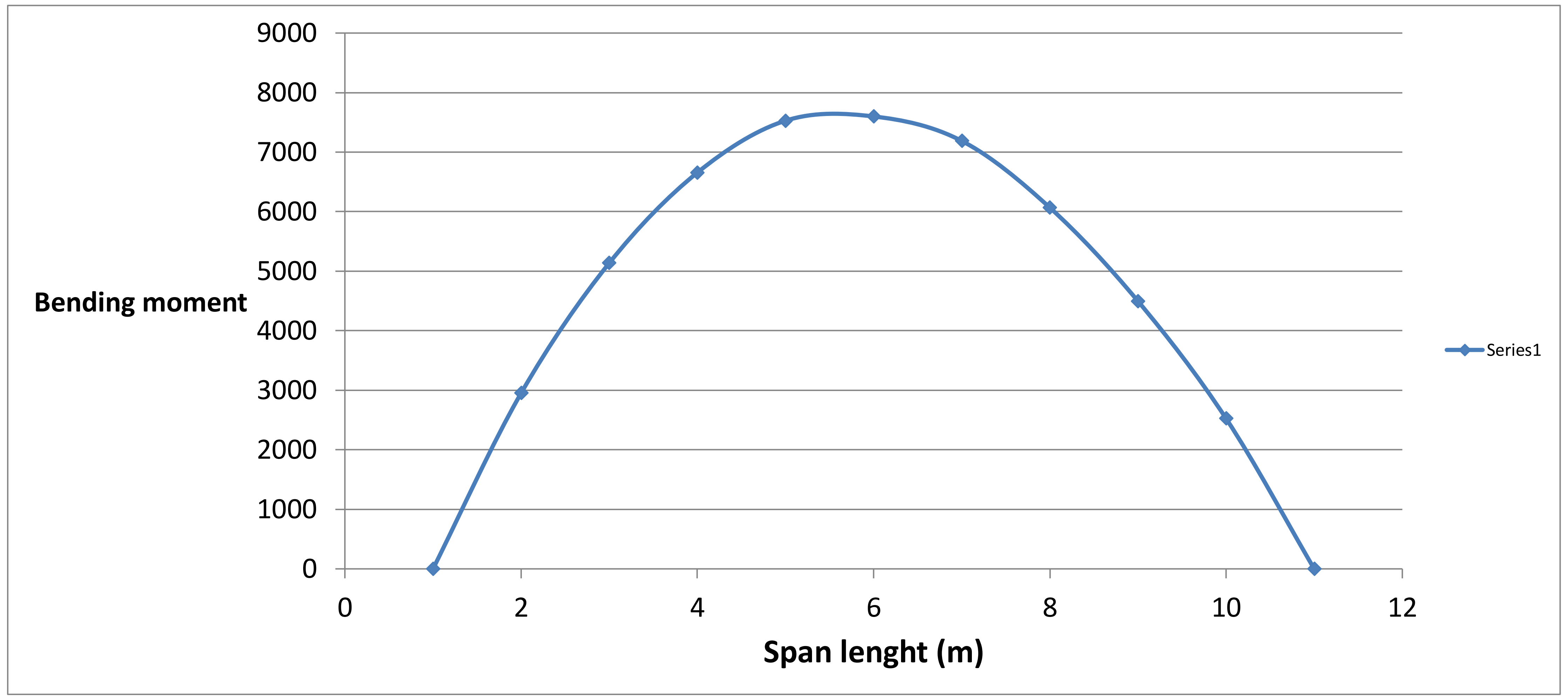
Moment

F3=	180 KN	I1 =	0,50
F1 =	360 KN	I2 =	0,64
F2=	230 KN	I3 =	0,33
F4=	120 KN/m		
Span L1	20,5		



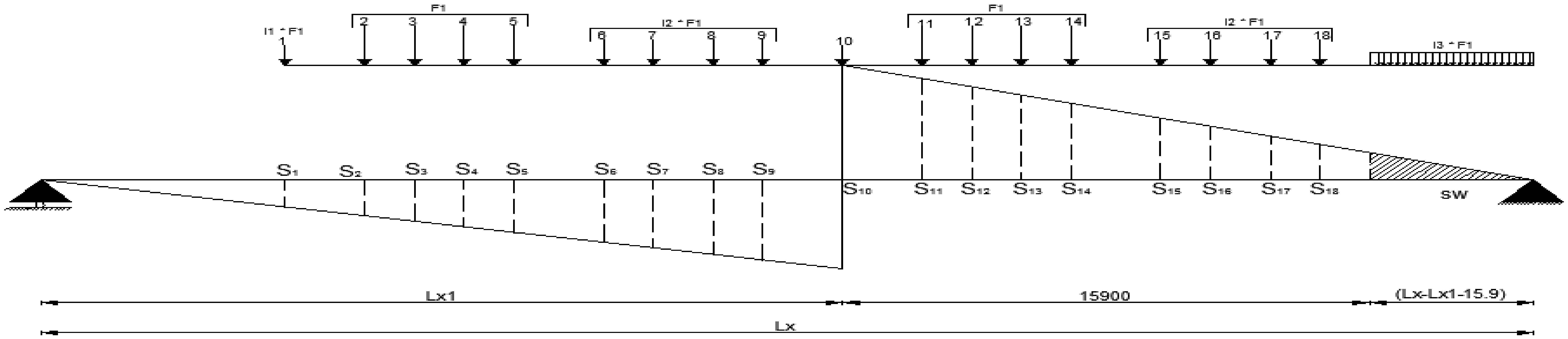
Wheel number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel distance (m)		2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	2,4	2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	1,5	Lw	
Distance cumulative from X		16,8	14,4	12,9	11,4	9,9	7,2	5,7	3,9	2,4	0	2,4	3,9	5,4	6,9	9,6	11,1	12,9	14,4	15,9	Mmax (KN-m)
Influence line coefficient for bending moment (m_x) for X=0,0*L =	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Influence line coefficient for bending moment (m_x) for X=0,1*L =	2,05	0	0	0	0	0	0	0	0	0	37,823	32,903	29,828	26,753	23,678	18,143	15,0675	11,378	8,3025	6,6651	2951,72
Influence line coefficient for bending moment (m_x) for X=0,2*L =	4,1	0	0	0	0	0	0	0	3,28	27,88	67,24	57,4	51,25	45,1	38,95	27,88	21,73	14,35	8,2	0,5125	5136,6
Influence line coefficient for bending moment (m_x) for X=0,3*L =	6,15	0	0	0	0	0	0	6,4575	32,288	53,813	88,253	73,493	64,268	55,043	45,818	29,213	19,9875	8,9175	0	0	6655,8
Influence line coefficient for bending moment (m_x) for X=0,4*L =	8,2	0	0	0	0	0	12,3	30,75	52,89	71,34	100,86	81,18	68,88	56,58	44,28	22,14	9,84	0	0	0	7527,6
Influence line coefficient for bending moment (m_x) for X=0,5*L =	10,25	0	0	0	0	3,5875	31,263	46,638	65,088	80,463	105,06	80,463	65,088	49,713	34,338	6,6625	0	0	0	0	7599,25
Influence line coefficient for bending moment (m_x) for X=0,6*L =	12,3	0	0	0	7,38	19,68	41,82	54,12	68,88	81,18	100,86	71,34	52,89	34,44	15,99	0	0	0	0	0	7188
Influence line coefficient for bending moment (m_x) for X=0,7*L =	14,35	0	0	8,9175	18,143	27,368	43,973	53,198	64,268	73,493	88,253	53,813	32,288	10,763	0	0	0	0	0	0	6067,5
Influence line coefficient for bending moment (m_x) for X=0,8*L =	16,4	0	8,2	14,35	20,5	26,65	37,72	43,87	51,25	57,4	67,24	27,88	3,28	0	0	0	0	0	0	0	4496
Influence line coefficient for bending moment (m_x) for X=0,9*L =	18,45	3,3825	8,3025	11,378	14,453	17,528	23,063	26,138	29,828	32,903	37,823	0	0	0	0	0	0	0	0	0	2524,8
Influence line coefficient for bending moment (m_x) for X= L =	20,5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M_{LL} = 7599,25



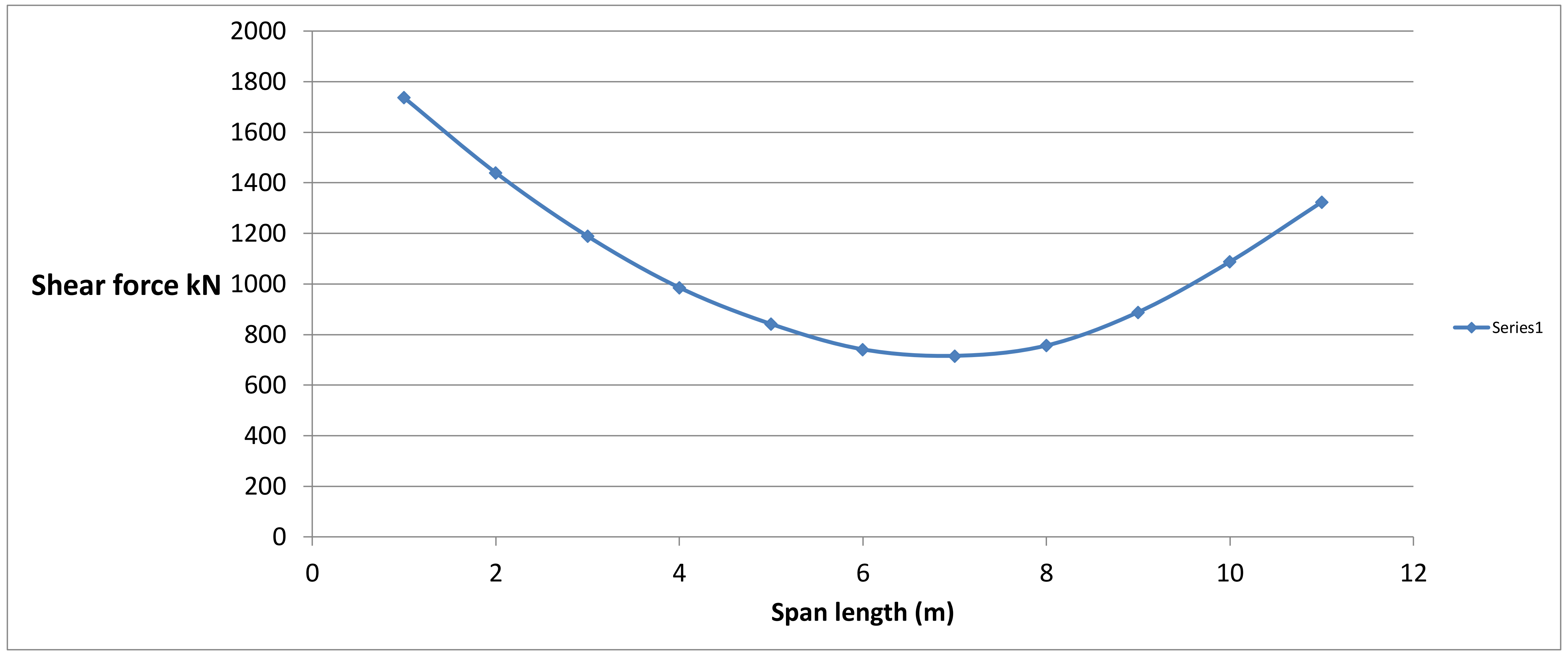
INFLENCE LINE FOR MOVING LOAD

SHEAR



Wheel number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel distance (m)		2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	2,4	2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	1,5	Lw	
Distance cumulative from X		16,8	14,4	12,9	11,4	9,9	7,2	5,7	3,9	2,4	0	2,4	3,9	5,4	6,9	9,6	11,1	12,9	14,4	15,9	Vmax (KN)
Influence line coefficient for shear force (S_x) for X=0,0*L =	0	0	0	0	0	0	0	0	0	0	20,5	18,1	16,6	15,1	13,6	10,9	9,4	7,6	6,1	10,58	1736,8
Influence line coefficient for shear force (S_x) for X=0,1*L =	2,05	0	0	0	0	0	0	0	0	0	18,45	16,05	14,55	13,05	11,55	8,85	7,35	5,55	4,05	3,2512	1439,9
Influence line coefficient for shear force (S_x) for X=0,2*L =	4,1	0	0	0	0	0	0	0	0,2	1,7	16,4	14	12,5	11	9,5	6,8	5,3	3,5	2	0,125	1188,9
Influence line coefficient for shear force (S_x) for X=0,3*L =	6,15	0	0	0	0	0	0	0,45	2,25	3,75	14,35	11,95	10,45	8,95	7,45	4,75	3,25	1,45	0	0	985,76
Influence line coefficient for shear force (S_x) for X=0,4*L =	8,2	0	0	0	0	0	1	2,5	4,3	5,8	12,3	9,9	8,4	6,9	5,4	2,7	1,2	0	0	0	841,71
Influence line coefficient for shear force (S_x) for X=0,5*L =	10,25	0	0	0	0	0,35	3,05	4,55	6,35	7,85	10,25	7,85	6,35	4,85	3,35	0,65	0	0	0	0	741,39
Influence line coefficient for shear force (S_x) for X=0,6*L =	12,3	0	0	0	0,9	2,4	5,1	6,6	8,4	9,9	8,2	5,8	4,3	2,8	1,3	0	0	0	0	0	715,9
Influence line coefficient for shear force (S_x) for X=0,7*L =	14,35	0	0	1,45	2,95	4,45	7,15	8,65	10,45	11,95	6,15	3,75	2,25	0,75	0	0	0	0	0	0	756,54
Influence line coefficient for shear force (S_x) for X=0,8*L =	16,4	0	2	3,5	5	6,5	9,2	10,7	12,5	14	4,1	1,7	0,2	0	0	0	0	0	0	0	888,49
Influence line coefficient for shear force (S_x) for X=0,9*L =	18,45	1,65	4,05	5,55	7,05	8,55	11,25	12,75	14,55	16,05	2,05	0	0	0	0	0	0	0	0	0	1087,6
Influence line coefficient for shear force (S_x) for X= L =	20,5	3,7	6,1	7,6	9,1	10,6	13,3	14,8	16,6	18,1	0	0	0	0	0	0	0	0	0	0	1323,6

V_{LL} = 1736,761



V_{LL} =	1736,76	kN
M_{LL} =	7599,25	kN/m