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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ECONOMICAL ANALYSIS AND DESIGN FOR THE SELECTION OF A PRE-STRESSED T–AND BOX GIRDER RAILWAY BRIDGES

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Railway Stream

BY

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ABSTRACT

A bridge is an important part of a transport network since it involves a lot of money, effort and required during railway construction project.

Girder bridges are by the far the most common of all bridges types nowadays and using pre-stressed concrete instead reinforced concrete is a way to obtain a lighter structure self-weight particularly when designing a railway bridges because of their heavier loads in counter part of the highway bridges.

The aim of this thesis is to develop a useful excel sheets for analysis and design and finally perform the selection of an economical types of bridges whether a pre-stressed T- or Box-girder taking construction cost as basis of comparisons.
The designed bridges have the lengths between 20 and 30 meters because the expected demarcation point is located in this range and the AREMA recommendations have been considered.

There will be some limitations in the calculations for example post-tensioned system is used, the bridges are supposed to have straight track and are considered simple supported. The developed sheets are illustrated using particular examples of both T- and Box-Girder bridges.

The conducted comparative study based on the prevailing rates of construction cost as main comparator in order to identify the economical span range of the both superstructures girders revealed that pre-stressed T-girder is economical up to 24m and that of the Box-girder is economical beyond 24m.

Keywords: Bridges, T-girder, Box-girder, Excel, Pre-stressed, Railway

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LIST OF SYMBOLS

A = gross area of Section

A_c = Total concrete area (m2)

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 = 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 reinforcementv

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

S_b = noncomposite section modulus for the extreme bottom fiber

S_t = noncomposite section modulus for the extreme top fiber

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 from centroidal axis of gross section to the extreme top fiber

y_b = distance from centroidal axis of gross section to the extreme bottom fiber

C° = degree celsius

ϕ = Strength Reduction Factor

α = total angular change of prestressing steel profile in radians from jacking end to point x

β_1 = factor for concrete strength

μ = friction curvature coefficient

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
LFD :	Load Factor Design

Chapter 1: Introduction

1.1 Background

In Djibouti the two first railway bridges was constructed on Holl-Holl and Chebele valley in 1917 by French engineers, recently news railway bridges with standard gauge have replaced the old ones with metric gauge.

Ethiopia and Djibouti especially Ethiopia has a important among of water channel, an valley and others several obstacle which makes the design of transport project sometimes dear because of building bridges, tunnels to overcome those obstacle.

Analysis and design of prestressed concrete bridges was always a complex task for structural designers, because designing a prestressed concrete structure requires a high degree of precision from designer in the fact that over-prestressing or under-prestressing a structure may reduce it is safety whereas over-reinforced a reinforced concrete structure may not affect the safety but the economies.

A thesis submitted to the school of graduate studies of Addis Ababa University by Abiy Alemu [3], a Computer Program for Comparative Study of the Analysis and Design of T-and Box-Girder Bridges has demarcated the economical span range of the two types bridges using in his case reinforced concrete for a highway bridges. Accounting the prevailing component cost for construction of bridges.

Moreover, the economical span range for the two types of bridges is not clearly demarcated as referred to many literatures [1, 7, 9, 11, 12, 13] there is a total void of span range for the two types of bridges particularly when using prestressed concrete for railway bridges.

According to Robert Benaim is book [13], in countries with labour costs that are lower than those in UK prestressing concrete becomes relatively cheaper in comparison with reinforced concrete. In view of saving money prestressed concrete should be considered rather than reinforced for medium and large span length particularly for heavier bridges in this case railway bridges.

One of the decisions that should be made at the early stage of design phase is determination of the span length, then taking span length as the first parameter to take into consideration for the selection the types of superstructures, a comparative study using excel sheets in order to select and identify the economical span range of T- and box girder railway bridges on the basis of construction cost only. The type of bridge that will be selected should also fulfill the fundamental design objectives safety, serviceability and aesthetics in addition to cost-effectiveness.

1.2 Objectives

The aim of this thesis is to develop a useful excel sheets for analysis and design of the superstructure of pre-stressed T-and Box-girder railway bridges following the American standard AREMA, thus to demarcate the economical span with a more precise manner using cost as basis of comparison.

The best appropriate type of bridge used or to be used can be selected with the aid of the developed excel sheets.

- Analysis and design T- and Box-girder Railway bridge following AREMA specifications.
- Develop excel calculation sheets.
- Perform the selection using cost as the basis of selection.
- This excel sheets can be enlarged for extensions research according to need.

1.3 Applications and limitations

1.3.1 Applications

- The developed excel sheet will be applicable for analysis and design of T- and Box-girder pre-stressed railway bridges for any span length and also perform the selection of the bridge type either pre-stressed T- or Box-girder railway bridge on the basis of economy and cost.
- The outputs of this excel sheets are well organized and will be easier to understand, it will be benefit for bridge designer especially in pre-stressed field.
- Furthermore, the department may use this for forthcoming research in line with upgrading the excel sheets to handle many other aspects of the bridge design thus enlarges the scope.

1.3.2 Limitations

The following are envisaged as the major constraints of the study.

- It takes into consideration the pre-stressed post-tensioned concrete as material, not pre-tensioned, reinforced concrete, steel, those can be the object of further research.
- The cost takes into consideration of the members per unit length comprising the individual costs of concrete, high tensile steel pre-stressing material and supplementary reinforcement, end anchorages, formwork, ducts and bearing.
- Only the superstructure cost of the bridge is considered for economical analysis since it is assumed that the substructures are almost the same for both T- and box-girder railway bridges having the same span length.
- This calculation excel sheets is developed for simply supported span railway bridges, and for single track.
- The live load is assumed to be the same for both structures with the same span length.
- The computation will proceed a span at time.

1.4 Structure of the thesis

This thesis is organized into six chapters:

The first chapter is an introduction to the research an include some background, the objectives of this thesis, the applications and the limitations.

The second chapter include a literature survey on the economies of the structures, advantages and disadvantage of pre-stressed concrete, the types and classification of bridges, selection of bridges and materials.

The third chapter contains analysis and design of pre-stressed post-tensioned T-and box girder railway bridges.

The fourth chapter focused on the excel sheet and reveal the different steps followed to obtain the results.

The fifth chapter present an illustrative design examples of the developed excel sheets, further summarizes the results of the outputs, cost analysis for different spans and comparisons carried are also included.

The sixth chapter contains the conclusions and recommendations that were reached in this study in line with the thesis objectives and findings.

Finally the appendices provide additional data and detailed results

Chapter 2: Literature survey

2.1 General

One of the reasons that explain the increase of the railway projects in Africa is because railroads become the only efficient way to transport large amounts of supplies and commercial products. The important infrastructure construction in a railroad projects are, railway track, railway bridges, railway tunnels, railway station, the railway bridges is the main subject in this research.

A bridge is a structure providing passage over an obstacle, the obstacle may be a road, railway line, river, a valley, and the passage may be for a highway, railway traffic, pedestrian, and a canal or pipe line. Bridge can carry a single, double or multiple tracks in conformity with the project. Bridge engineering is one of the fascinating fields in civil engineering calling for expertise in many areas: structural analysis and design, geotechniques, traffic projection, surveying, runoff calculation and methods of construction, when it project to carry a railway traffic this makes her heavier bridges, more difficult to design, and more dear.

2.2 Advantage and disadvantage of pre-stressed concrete

Advantage

- The use of high strength concrete and steel in prestressed members results in lighter and slender members than is possible with RC members.
- In fully prestressed members the member is free from tensile stresses under working loads, thus whole of the section is effective.
- In prestressed members, dead loads may be counter-balanced by eccentric pre-stressing.
- Prestressed concrete member possess better resistance to shear forces due to effect of compressive stresses presence or eccentric cable profile.
- Use of high strength concrete and freedom from cracks, contribute to improve durability under aggressive environmental conditions.
- Long span structures are possible so that saving in weight is significant & thus it will be economic.
- Factory products are possible.
- Prestressed members are tested before use.

- Prestressed concrete structure deflects appreciably before ultimate failure, thus giving ample warning before collapse.
- Fatigue strength is better due to small variations in pre-stressing steel, recommended to dynamically loaded structures.[11]

Disadvantages

- The availability of experienced engineers and builders is rare.
- Initial equipment cost is very high.
- Pre-stressed sections are brittle
- Pre-stressed concrete sections are less fire resistant.

2.3 Differences between Pre-stressed Concrete and Reinforced Concrete

- In prestressed concrete member steel plays active role. The stress in steel prevails whether external load is there or not. But in RC steel plays a passive role. The stress in steel in RC 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 RC the stress in steel is variable with the lever arm.
- Prestressed concrete has more shear resistance, whereas shear resistance of RC 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 RC deflections are more.
- Prestress concrete is more durable as high grade of concrete is used which are more dense in nature. RC. 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%.
- RC. is uneconomical for long span because in RC. dimension of sections are large requiring more concrete & steel. Moreover as self-weight increases more reactions acted on columns & footings, which requires higher sizes.[11]

2.4 Economies of the structures

One of the most important constructions in railway construction project or any infrastructural development of a country is a bridge, it very costly structures compared to others this one properly constructed will serve the cause successfully without any impediment for a longer period and at the economical cost.

The first essential aspect of economical design is an appreciation of the possibilities of materials may lead to substantial economies whether to choice a prestressed concrete girder for a less initial cost or steel girder if we consider the life cycle cost.

The cost of prestressed concrete structures is obviously affected by the prices of high strength concrete, high strength steel, formwork and anchorages devices and reinforcement steel.

Upon the relation between these prices, the economical properties of the quantities of high strength concrete and steel depend.

There are possibly other factors to be taken into account in any particular case, such as the use of available materials in the country.

For example in the United kingdom economy generally results from the use of simple formwork even if this requires more concrete compared with a design requiring more complex and more expensive formwork. Formwork is obviously cheaper if angles are right angles, if surface are plane and if there is some repetition of use unless structurally necessary or essential to durability. [6]

Some of the factors which may have to be considered are whether to use a strand, wires or high strength bars, then to use a strand with different wires, two, three or seven, whether a few wires of large diameter of high strength can replace a large number of wires of smaller diameter, whether using a full or partially prestressed, whether to use a post-tensioned system or pre-tensioned, whether a segmental or span by span or other construction, whether to use a cast in place or precast structure, the methods of construction that will be follow takes an important place on the economy part.

Precast concrete construction usually reduces considerably the amount of formwork and the moulds can generally be used very many more times than can site formwork. In some cases however the loss of structural rigidity due to the absence of monolithic construction may offset the economy otherwise resulting from precast construction. To obtain the economic advantage of pre-casting and the structural advantage of in situ casting it is often convenient to combine both types of construction in the same structure means composite section. And precast members should be rather light in view of cost of transport and handling, thus combined sections are always more economical.

Whether the bridge is open deck or ballast deck, the dead load for open deck structures can be significantly less than for ballast deck structures, but transfer more dynamic effect of live load into the bridge than ballast deck however open is considered more economical.

Whether to limit the depth of the girder or not, it can be stated that prestressed concrete is especially competitive when of limited depth. [6] Therefore it is seem that greater economy will be obtained by using prestressed concrete in cases of limited depth.

However it is rather doubtful whether the costs of the prestressed steel could be appreciably reduced by increasing the depth of the girder while the cost of the wires or strand and that for tensioning, including anchorage would be reduced that for the concrete and formwork would be increased.

Whether the extra cost of an expensive type of construction is warranted by the improvement in facilities or whether the initial cost of a construction of high quality with little or no maintenance cost is more economical than less costly construction combined with the expense of maintenance. There is also a wider aspect of economy, such as whether the anticipated life and use of a proposed structure warrant the use of a higher or lower factor of safety than is usual.

In many cases the most economical design can be determined only by comparing the approximate costs of different designs. This is particularly true in borderline cases and is practically the only way of determining when a prestressed T-girder railway bridges superstructures ceases to be economical than one with Box-girder railway bridges. The combination to obtain the best design parameters is very large and is important to choice thoroughly.

It is not easy to explore all those way in this work due to lack of time, therefore we take into consideration the following parameters only.

- Concrete material.
- Post-tensioned
- Cast-in-place T-and Box-girder Railway Bridge.
- Ballasted deck.
- Constant depth

2.5 Design standards

The design of railway Bridge, like many other civil engineering projects, is dependent on certain standards and criteria. Design of railway bridges in a modern transportation system would require a set of rigorous design specifications to ensure the safety and overall quality of the constructed project. Ethiopia railway cooperation did not establish yet a standard to follow for railway structures analysis and design. For this reasons American standard is the one use for this research

Early railroads developed independent specifications governing the design loadings, allowable strains, and quality of material, fabrication, and construction of their own bridges. There was a proliferation of specifications written by individual railroads, suppliers, and engineers. One of the earliest general specifications is titled Specification for Iron Railway Bridges and Viaducts, by Clarke, Reeves and Company (Phoenix Bridge Company). By 1899 private railroads joined efforts in forming AREMA. Many portions of those original individual railroad specifications were incorporated into the first manual titled Manual of Recommended Practice for Railway Engineering and Maintenance of Way published in 1905. In 1911 the Association dropped "Maintenance of Way" from its name and became the American Railway Engineering Association (AREA); however, in 1997 the name reverted back to the original name with the consolidation of several railroad associations. The Manual is not deemed a specification but rather a recommended practice. Certain provisions naturally are standards by necessity for the interchange of rail traffic, such as track gauge, track geometrics, clearances, basic bridge loading, and locations for applying loadings. Individual rail-roads may, and often do, impose more stringent design requirements or provisions due to differing conditions peculiar to that railroad or region of the country, but basically all railroads subscribe to the provisions of the Manual. [14]

2.6 Types and classification of bridges

2.6.1 Types of bridges

For bridges construction we must consider a number of factors. Different bridge styles distribute stresses different ways, initial cost, future maintenance cost, construction time, locations of the railway bridges, we must take the length and width of the bridge, local environmental conditions and building materials into account to decide what type of bridge to build. Using the wrong type can result in disaster.

There are six main types of bridges:

- Arch
- Beam
- Cable stayed
- Cantilevered
- Suspension
- Truss

2.6.2 Classification of bridges

Bridges are grouped per the nature and characteristic feature of their superstructures, which in turn classified based on:

- Material: Steel, concrete, wood, hybrid, stone/brick.
- Usage: Railway, Highway, Pedestrian.
- Span: Short, Medium, Long
- Structural form: Slab, girder, truss, Arch, Suspension, and Cable stayed.

Structural forms refer to the load resisting mechanism of a bridge. Only certain types of structural forms a suitable and economically viable for certain span ranges. [5]

2.6.3 Span length

- The effective span lengths of simply supported beams shall be the distance center to center of bearings.
- The span length of continuous or restrained floor slabs and beams shall be the distance center to center of supports.

T-Girder

There is a gap for the economical span range for prestressed railway bridges per standard, but some information gathered in several books are shown below.

T-girder is generally more economical for spans of 12 to 18m. [14]

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. [14]

Prestressed concrete T-beams with depths from 1.2 to 1.7m are applicable for spans from 32.3 to 41.5m for highway bridges. [14]

The design of standard T-beams have been worked out considering both pre-stressing systems: Post-tensioning and pre-tensioning. The standard T-beams have been designed precast in segments with subsequent Post-tensioning for the span lengths of 24.33 to 42m. [14]

Ethiopia Bridge design manual recommend prestressed T-girder for span exceeding 20m.

In accordance with Australian standard prestressed concrete T-girder are selected for a span length ranging from 19 to 37m.

India standard indicated that pre-stressed concrete T-girder is suitable for spans in the range of 20-40m.

Box-Girder

The box girder bridge is a common structural form in both steel and concrete with a higher flexural capacity and torsional rigidity. The multicellular box girder may be thought of as a slab girder bridge with a bottom slab that encloses the sections and it is often used in cast in place prestressed concrete bridges. [11]

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.

Precast pre-stressed box girder sections are used frequently for simple spans of over 30m and are particularly suitable for widening bridges to control deflections. [14]

Cast-in-place prestressed concrete Box-girder is used mostly for spans of 30-180m with helping with a suspension or cable staying in need. [14]

Composite box girders are generally used in moderate medium span bridges 30-60m. [14]

Australian standard provide a span ranging between 35-50m to pre stressed concrete box girder. Single or multicellular box girders are preferred for larger spans of the order of 30-70m [11].

2.7 Selection of bridges

In general all the factors are related to the economy, safety, and aesthetics.

- Geometric conditions of the sites.
- Subsurface conditions of the sites.
- Functional requirements.
- Aesthetics.
- Economics and ease of maintenance.
- Construction and erection consideration.
- Legal considerations.

The important and the various factors need to be considered for bridge types selection is generally based on initial cost, maintenance costs, construction time, aesthetics and durability of the bridge. A bridge type selection is developed combining the knowledge base of design manuals and design specifications. According to those different specifications the most important factors affecting bridge superstructure types is the span length. Occasionally the selection is complicated by other consideration such as [14]:

- The deflection limit.
- Life cycle cost.
- Traffic maintenance during construction stages.
- Construction scheduling and worker safety.
- Feasibility of false work layout.
- Passage of flood debris.
- Seismicity at the site.
- Suitability for future widening.
- Commitments made to officials and individuals of the community.

2.8 Bridges deck

Railroad bridges typically are designed as either open deck or ballast deck structures.

2.8.1 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. [1]

2.8.2 Ballast deck

Ballast deck bridges have the track structure supported on ballast which is carried by the structures offer advantages is 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. [1]

2.9 Materials

Prestressed concrete structures, using high-strength materials to improve serviceability and durability, are an attractive alternative for medium and long-span bridges, and have been used worldwide since the 1950s [14]. It is basically concrete in which internal stresses of a suitable magnitude and distributions are introduced so that the stresses resulting from external loads are counteracted to a desired degree. In reinforced concrete members, the prestress is commonly introduced by tensioning the steel reinforcement. [11] The primary materials needed for the design of cast-in-place post-tensioned are: concrete, pre-stressing steel, and mild reinforcing.

2.9.1 Concrete

Concrete particularly high-strength concrete, is a major constituent of all prestressed concrete elements. Hence it is strength and long term endurance has to be achieved through proper quality control and quality assurance at the production stage. [9]

Strength and endurance are two major qualities that are particularly important in prestressed concrete structures long term detrimental effect can rapidly reduce the pre-stressing forces and could result in unexpected failure. Hence measures have to be taken to ensure strict quality control and quality assurance at the various stages of production and construction as well as maintenance. [9] A concrete with high compressive strength at a reasonably early age and comparatively higher tensile strength as compared with ordinary RC member, low shrinkage, minimum creep characteristics and high young`s modulus are necessary.

Compressive strength (f’c) is the characteristic that best gives an overall picture of the quality of a concrete. The minimum f’c for pre-stressed concrete should be 30 MPa [1]

2.9.2 Pre-stressing Steel

The high-tensile steel bars commonly used for prestressed concrete generally consists of strand, wires and bars and shall applied conforming to one of the following specifications:

- Standard Specification for Steel Strand Uncoated Seven-Wire for Pre-stressed Concrete (ASTM A416).
- Standard Specification for Uncoated Stress-Relieved Steel Wire for Pre-stressed Concrete (ASTM A421).
- Standard Specification for Uncoated High-Strength Steel Bar for Pre-stressing Concrete (ASTM A722).[1]

For bridge construction, wire systems are generally not used [12].

2.9.3 Reinforcing steel

Non prestressed reinforcement generally consists of deformed bars or welded wire reinforcement. [12]

Pre-stressing tendons, structural steel, steel pipe and tubing shall conform to one of the ASTM specifications. [1]

Design shall not be based on a yield strength fy in excess of 420MPa. [1]

Chapter 3: Analysis & Design of post-tensioned T- and Box-girder

The analysis use is simple influence line for moving loads and volumetric calculation for dead loads. This analysis method gives fairly accurate results for practical designs with reasonable computational time.

3.1 Loadings, Material properties, Design assumptions

3.1.1 Loadings

Bridge designer has to predict all loads are expected to be applied to the bridge during the transfer and it is service life.

Such as load and forces are:

- Dead load (D).
- Live Load (L).
- Impact load (I).
- Centrifugal force (CF).
- Earth pressure (E).
- Buoyancy (B).
- Wind load on structures (w).
- Wind load on live load (WL).
- Longitudinal force due to friction resistance at expansion bearings (F).
- Earthquake seismic load (EQ).
- Ice Pressure (ICE).
- Other forces (rib, shortening, shrinkage, temperature and /or settlement of supports) (OF).

When designing railway bridge structures, the above loads must be considered. In addition to the (D) of the structures itself, there are the usual (L) (Cooper E80-series) from the rolling machine. To these are added the dynamic components of the traffic such as impact, centrifugal, lateral and longitudinal forces. Then there are the environmental considerations such as wind, snow, thermal, and seismic loads. Finally, because railway structures must perform under heavier loads, have longer service life, and dissimilar maintenance constraints compared to their highway counterparts, other factors, including fatigue and maintenance issue, tend to influence railway structures design more than roadway structures.

DEAD LOAD (D)

The dead load shall be the weight of the bridge including the deck and track, together with any other fixed loads, and shall consist of the estimated weight of different member plus that to the track, ballast, fill, and other portions of the structures supported thereby. The unit weight of material comprising the dead load shall be assumed as follows.

Table 3.1: Materials and weight

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)

The live load representing the weight of trains is specified in terms of E-80 loading the AREMA specifications require that bridges be designed for a train composed of two engines followed by a line of railroad cars, the wheels of the engines are represented by concentrated loads and the railroad cars by a uniformly distributed load. Where the live load is specific equipment, complete data shall be obtained, including the spacing of axles and the static load on each axle.

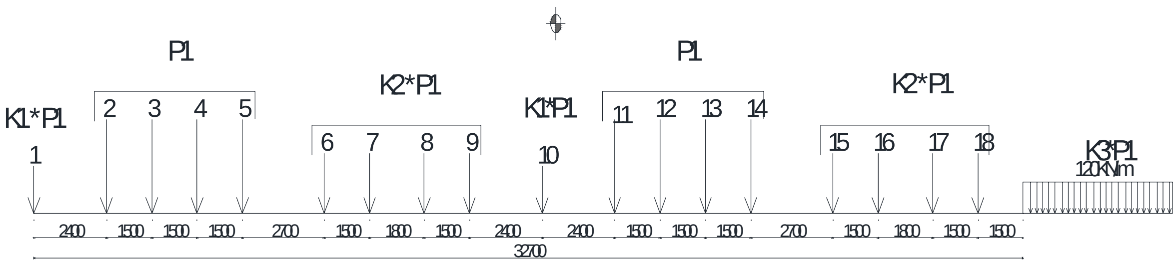


Figure 3-1: E-Cooper series loading

Where

K1 = P3/P1 = 180/360 = 0.5

K2= P2/P1 = 230/360 =0.64

K3= P4/P1= 120/360 = 0.33

IMPACT LOAD (I)

Impact load, due to the sum of vertical effects and rocking effect created by passage of locomotives and train loads, shall be determined by taking a percentage of the live load

For $L \leq 4$ meters $\quad I = 60\%$ 3-1

For $4 < L \leq 39$ meters $\quad I = \frac{125}{\sqrt{L}}\%$ 3-2

For $L > 39$ meters $\quad I = 20\%$ 3-3

CENTRIFUGAL FORCE (CF)

Centrifugal force is the force a train moving along a curve exerts on a constraining object track and supporting structure and acts outwardly away from the center of . In the process, both a horizontal force and an overturning moment are produced. For concrete structures box girders, for example, the structure is typically stiff enough in the transverse direction that the horizontal force is not significant. For all types, the bearings and substructure must be able to resist the centrifugal horizontal force.

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

$C = 0.000452S^2D$ 3-4

$E_a = 0.0068S^2D - 75$ 3-5

$S = \frac{(E_a - 75)^{0.5}}{(0.0068D)^{0.5}}$ 3-6

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)

If the track is in a curve centrifugal forces shall be taken into account according to the above equations. In this thesis the track will be considered straight, so centrifugal force is not considered.

EARTH PRESSURE (E)

Earth pressure forces to be applied to the structure shall be determined in accordance with the provisions of Arema for Retaining Walls, Abutments and Piers.

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.

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)

$$L_{F1} = 200 + 17.5L \qquad 3\text{-}7$$

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

Longitudinal traction force (kN)

$$L_{F2} = 200\sqrt{L} \quad 3\text{-}8$$

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

$$L_F = Max(L_{F1}; L_{F2}) \quad 3\text{-}9$$

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. Referring Seismic Design for Railway Structures chapter for additional guidance.

STREAM FLOW PRESSURE (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:

$$P_{avg} = K(V_{avg})^2 \quad 3\text{-}10$$

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 3.2: Temperature ranges

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

3.1.2 Material properties

According to recommendations and specifications of AREMA, PCI (Pre-stressed concrete institute) and PTI (Post-tensioning institute), the materials used in designing railway bridges for these excel sheets are as follows:

- The minimum required compressive strength f'c for pre-stressed concrete should be 30 MPa where f'c=0.8fc but in this design 50MPa is use.
- The density of concrete is taken as 2400 kg/m^3 for the computation of the modulus of elasticity of elasticity of concrete Ec, and 25kN/m^3 for dead load computations.
- Modulus of elasticity E_c for concrete may be taken as $0.043(wc)^{1.5}$ $(f`c)^{0.5}$ in MPa.
- Modulus of elasticity of non-pre-stressed steel reinforcement is equal to 200 GPa.
- Grade 270 is used in pre-stressed low relaxation seven wire strands steel with a minimum ultimate strength of 1890 MPa.

3.1.3 Design Assumptions

The following assumptions will be taken into consideration during the design.

- The abutments are considered to be the same for both T and Box-girder railway bridges, hence the design of abutments and it is associated costs are not taken into account for cost analysis.
- The bridges has a 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 T and Box-girder.
- The bearings are assumed to be the same for the same span length.

3.2 Design specifications

3.2.1 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 T-girder Railway Bridge will be taken for Box-girder.

3.2.2 Load factors and load combinations

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 3.3: 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 3.4: 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.

3.2.3 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)

$$Min[(915 + db); 1524] \qquad 3\text{-}11$$

Lateral deck distribution width (mm)

$$Min[(S_w + db); 4300] \qquad 3\text{-}12$$

S_w = Sleepers length

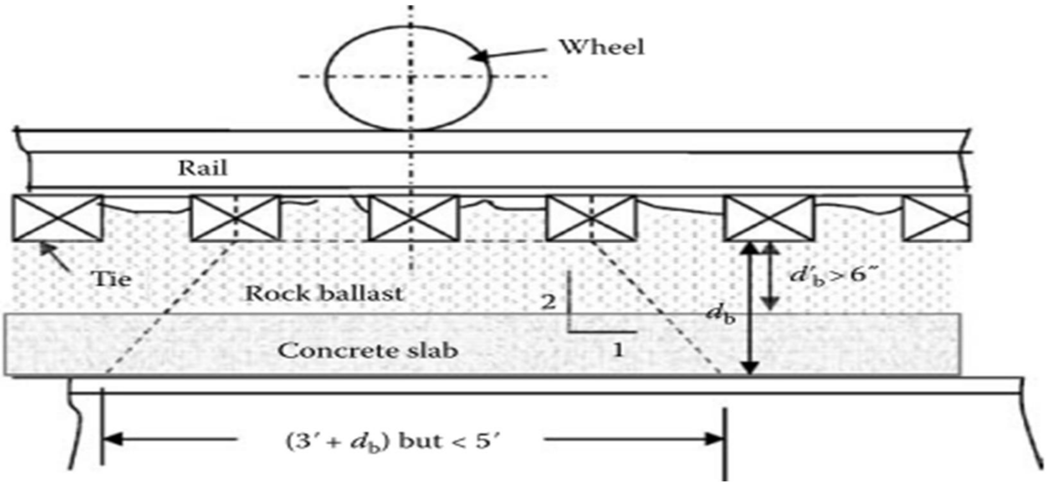


Figure 3-2: Longitudinal distribution of wheel load according to AREMA Manual

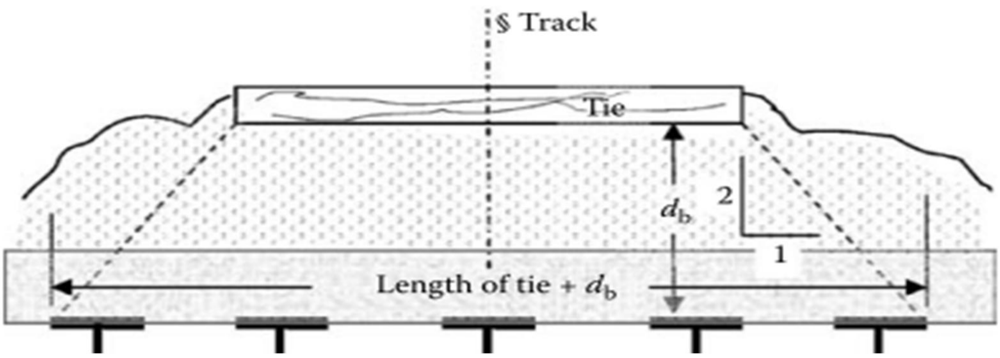


Figure 3-3: Lateral distribution of track load according to AREMA Manual

3.3 General Analysis and Design of T-girder

3.3.1 Analysis procedure

Superstructure depth

AASHTO LRFD Table 2.5.2.6.3-1 present the following traditional minimum depth for constant depth cast-in-place prestressed concrete box girder superstructures as a function of span length. For simple span girders the traditional minimum depth is:

$$D = 0.045 \times L \qquad 3\text{-}13$$

In terms of span-to-depth ratio, this equation becomes:

$$\left(\frac{L}{D} = 22\right)$$

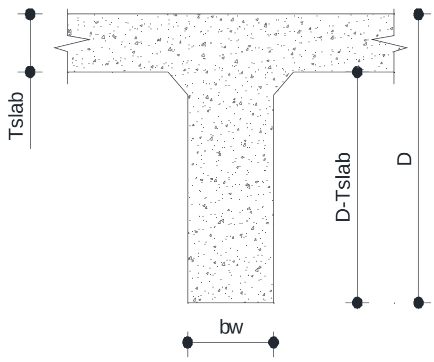


Figure 3-4: Half Cross section of T-girder

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:

$$T_{slab} = Max\left[\frac{(S+2750)}{15}\ ;150\right] \qquad 3\text{-}14$$

Where

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

S = Clear span between girders (mm)

The minimum Web Thickness

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

b_w = 12inches = 305 mm

Section properties

Having the minimum dimensions of the section we can calculate the section properties of the section.

$I_T = \sum I_x + \sum A_x d_x^2$ 3-15

$S_t = I_T / Y_t$ 3-16

$S_b = I_T / Y_b$ 3-17

Where

A_x= 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 T- section (mm^4)

S_t = Section modulus of top fibre of girder (mm^3)

S_b = Section modulus of top fibre of girder (mm^3)

Permanent Load

Permanent load are 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 girder railway bridge
- Superimposed permanent load

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

$q\left(\frac{kN}{m}\right) = \gamma\left(\frac{kN}{m^3}\right) \times A(m^2)$ 3-18

Self-weight of the girder

$q_{sw} = q_{Tslab} + q_{girders} + q_{diaphragms} = q_{TG} + q_{diaphragms}$

Superimposed permanent load

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

Where

γ = Unit weight of the different components

q_{sw} = Total permanent load from self-weight of the girder (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. [1]

Diaphragms shall be provided in accordance with following points, except that adequate bracing of the compression flange shall be provided by a cast-in-place deck.

- 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 Slab +girders +diaphragms

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

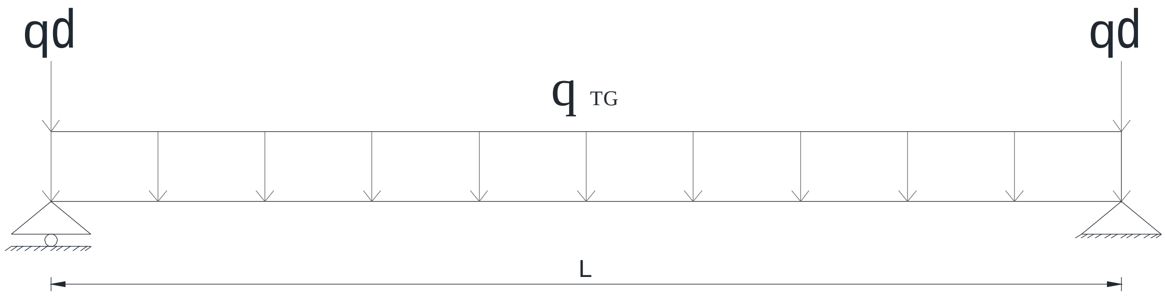


Figure 3-5: Self weight of the girder Placement (L ≤ 24m)

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

For 0 ≤ X ≤ L

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

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

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

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

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

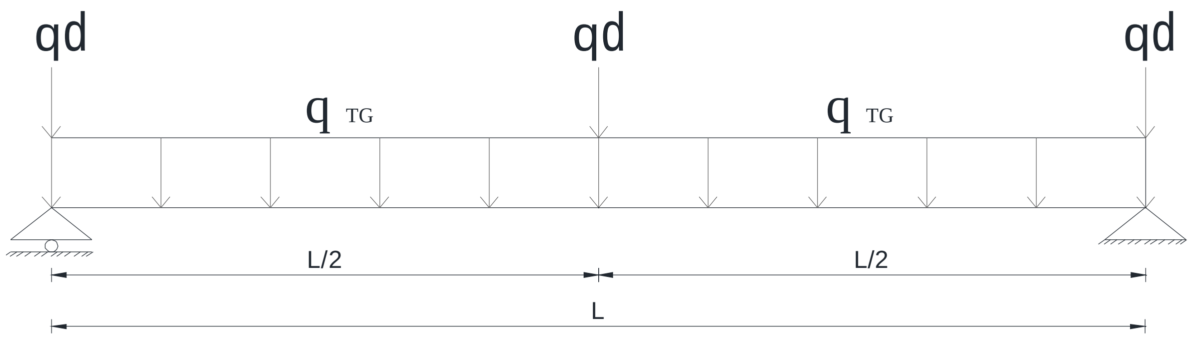


Figure 3-6: Self weight of the girder Placement (L ≤ 24m)

For 0 ≤ X ≤ L/2

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

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

For X > L/2

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

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

Where
M_{qsw} = Moment due to Permanent load from slab + girder + diaphragms (kN-m)
V_{qsw} = Shear force due Permanent load from slab + girder + diaphragms (kN)

Super imposed dead load

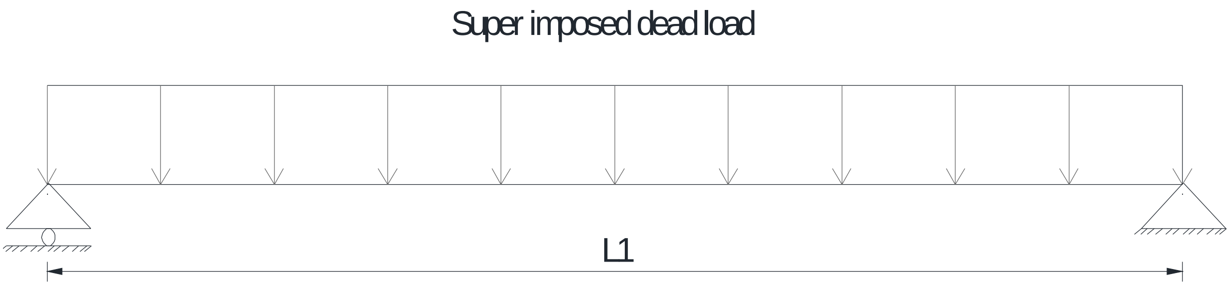


Figure 3-7: Super imposed dead load placement for girder

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 from Slab +girders +diaphragms + superimposed DL

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

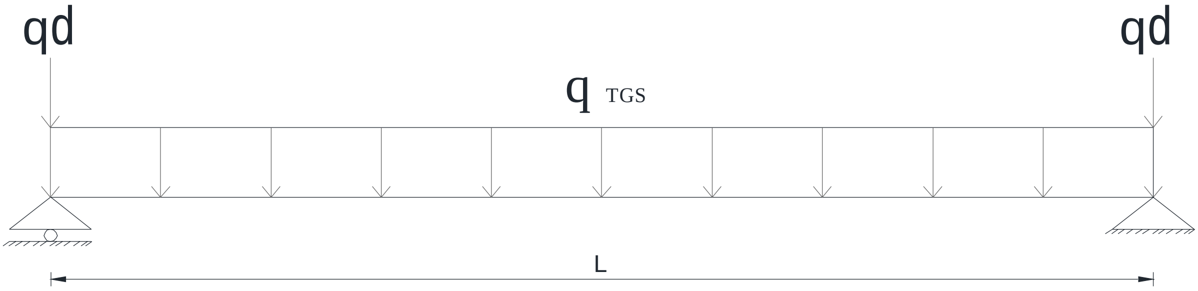


Figure 3-8: Permanent load placement (L≤24m)

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 $0 \leq X \leq L$

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

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

Where $q_{TGS} = q_{Tslab} + q_{girders} + q_{super\ imposed\ load}\ (q_{SI})$

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

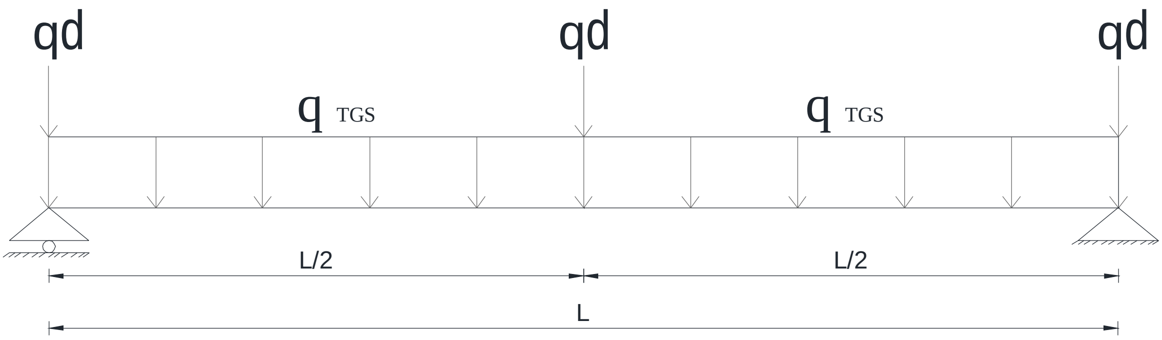


Figure 3-9: Permanent load placement (L>24m)

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

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

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

For $X > L/2$

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

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

Where

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

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

Impact load

Load from impact q_I is determined by taking a percentage of the live load accordingly to the span length following the equation 3-1, 3-2, 3-3

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 P3 = 180kN, followed by a four consecutive wheels load with the same spacing and magnitude respectively of 1.5m, P1= 360kN. Then another four consecutive wheels load with the same magnitude of P2= 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 P4= 120kN/m.

Those entire characteristics are represented on the figure below cooper loading.

K1=P3/P1= 180/360=0.5

K2=P2/P1= 230/360=0.6389

K3=P4/P1= 120/360=0.33

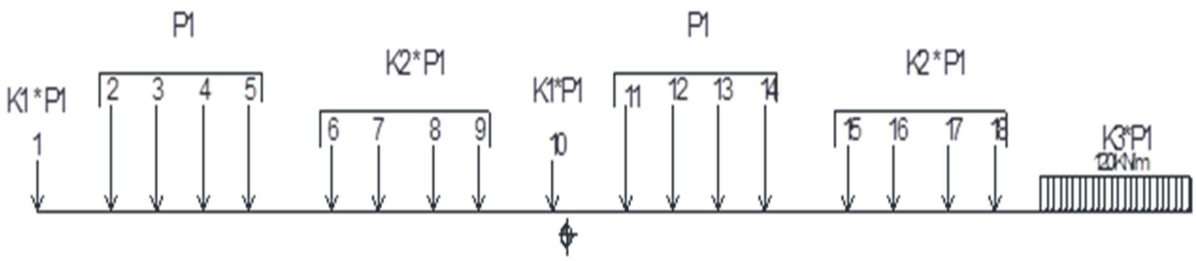


Figure 3-10: Train moving to the left (wheel load in kN)

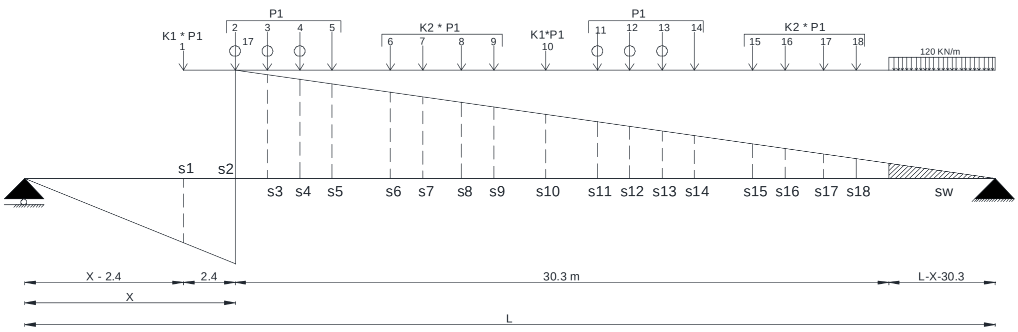


Figure 3-11: Influence Line for Shear Force at (X) Distance from End Support case 1.

Case:1 Influence line coefficient for shear force

S_1= (X-2.4)/L
S_2= (L-X)/L
S_3= (L-X-1.5)/L
S_4= (L-X-3)/L
S_5= (L-X-5.7)/L
S_6= (L-X-7.2)/L
S_7= (L-X-9)/L
S_8= (L-X-10.5)/L
S_9= (L-X-12.9)/L
S_{10}= (L-X-15.3)/L
S_{11}= (L-X-16.8)/L
S_{12}= (L-X-18.3)/L
S_{13}= (L-X-19.8)/L
S_{14}= (L-X-22.5)/L
S_{15}= (L-X-24)/L
S_{16}= (L-X-25.8)/L
S_{17}= (L-X-27.3)/L
S_{18}= (L-X-28.8)/L
S_w= $(Lw)^2/2L$= $(L-X-30.3)^2/2L$

V1 (X) = P1 [K1(S1)+(S2+ S3+ S4+ S5)+K2*(S6+ S7+ S8+ S9)+K1*(S10)+(S11+ S12+ S13+ S14)+K2*(S15+ S16+ S17+ S18)+K3*(Sw)
V1(X) = P1 [K1((X-2.4)/L)+((L-X)/L)+((L-X-1.5)/L)+((L-X-3)/L)+((L-X-5.7)/L))+K2*(((L-X-7.2)/L)+ ((L-X-9)/L)+((L-X-10.5)/L)+((L-X-12.9)/L))+K1*((L-X-15.3)/L)+(((L-X-16.8)/L)+((L-X-18.3)/L)+((L-X-19.8)/L)+ ((L-X-22.5)/L))+K2*(((L-X-24)/L)+((L-X-25.8)/L)+((L-X-27.3)/L)+((L-X-28.8)/L))+K3*($(L-X-30.3)^2/2L$)

V (X)= Shear force at a distance (X) due to train load moving to the left

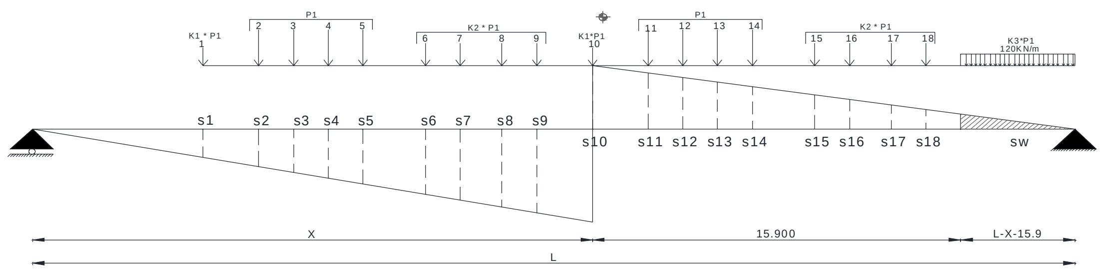


Figure 3-12: Influence Line for Shear Force at (X) Distance from End Support case 2

Case:2 Influence line coefficient for shear force

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

V2 (X) = P1 [K1(S1)+(S2+ S3+ S4+ S5)+K2*(S6+ S7+ S8+ S9)+K1*(S10)+(S11+ S12+ S13+ S14)+K2*(S15+ S16+ S17+ S18)+K3*(Sw)

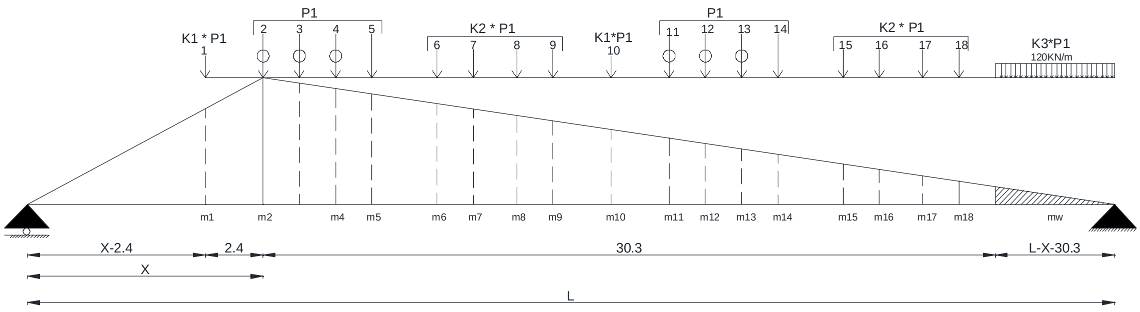


Figure 3-13: Influence Line for Bending Moment at (X) Distance from End Support (moving Left) case 1.

Case: 1 Influence line coefficients for bending moment

m1 = (X–2.4)*(L–X)/L
m2 = (X)*(L–X)/L
m3 = (X)*(L–X -1.5)/L
m4 = (X)*(L–X-3)/L
m5 = (X)*(L–X-5.7)/L
m6= (X)*(L–X-7.2)/L
m7 = (X)*(L–X-9)/L
m8 = (X)*(L–X-10.5)/L
m9 = (X)*(L–X-12.5)/L
m10 = (X)*(L–X-15.3)/L
m11 = (X)*(L–X-16.8)/L
m12 = (X)*(L–X-18.3)/L
m13 = (X)*(L–X-19.8)/L
m14 = (X)*(L–X-22.5)/L
m15 = (X)*(L–X-24)/L
m16 = (X)*(L–X-25.8)/L
m17 = (X)*(L–X-27.3)/L
m18 = (X)*(L–X-28.8)/L
mw = (X)*$(Lw)^2$/2L= (X)*$(L-X-30.3)^2$/2L

M1 (X) = P1 [K1*(m1)+((m2)+ (m3)+ (m4)+ (m5))+K2*((m6)+ (m7)+ (m8)+ (m9))+K1*(m10)+((m11)+ (m12)+ (m13)+ (m14))+ K2*((m15)+ (m16)+ (m17)+ (m18))+K3*(mw)]

M1 (X) = P1 [K1*((X–2.4)*(L–X)/L)+(((X)*(L–X)/L)+ ((X)*(L–X-1.5)/L)+ ((X)*(L–X-3)/L)+ ((X)*(L–X-5.7)/L))+K2*(((X)*(L–X-7.2)/L)+ ((X)*(L–X-9)/L)+ ((X)*(L–X-10.5)/L)+ ((X)*(L–X-12.5)/L))+K1*((X)*(L–X-15.3)/L)+(((X)*(L–X-16.8)/L)+ ((X)*(L–X-18.3)/L)+ ((X)*(L–X-19.8)/L)+ ((X)*(L–X-22.5)/L))+ K2*(((X)*(L–X-24)/L)+ ((X)*(L–X-25.8)/L)+ ((X)*(L–X-27.3)/L)+ ((X)*(L–X-28.8)/L))+K3*((X)*$(L-X-30.3)^2$/2L)]

M1 (X) = Bending moment at a distance(X) due to train load moving to the left case 1.

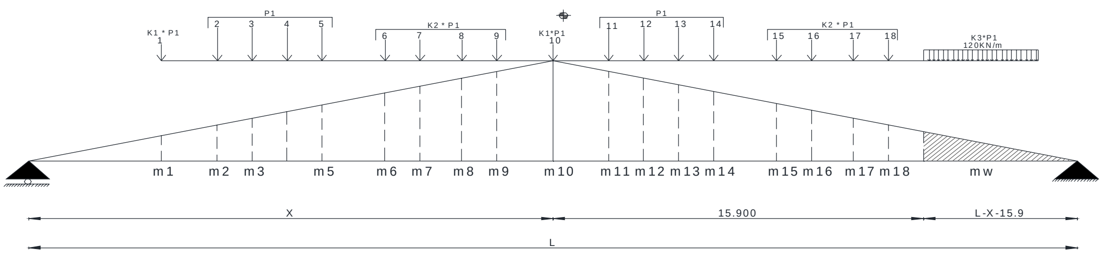


Figure 3-14: Influence Line for Bending Moment at (X) Distance from End Support (moving Left) case 2.

Case: 2 Influence line coefficients for bending moment

m1 = (X–16.8)*(L–X)/L
m2 = (X-14.4)*(L–X)/L
m3 = (X-12.9)*(L–X -1.5)/L
m4 = (X-11.4)*(L–X-3)/L
m5 = (X-9.9)*(L–X-5.7)/L
m6= (X-7.2)*(L–X-7.2)/L
m7 = (X-5.7)*(L–X-9)/L
m8 = (X-3.9)*(L–X-10.5)/L
m9 = (X-2.4)*(L–X-12.5)/L
m10 = (X)*(L–X)/L
m11 = (X)*(L–X-2.4)/L
m12 = (X)*(L–X-3.9)/L
m13 = (X)*(L–X-5.4)/L
m14 = (X)*(L–X-6.9)/L
m15 = (X)*(L–X-9.6)/L
m16 = (X)*(L–X-11.1)/L
m17 = (X)*(L–X-12.9)/L
m18 = (X)*(L–X-14.4)/L
mw = $(X)*(Lw)^2/2L= (X)*(L-X-15.9)^2/2L$

M2 (X) = P1 [K1*(m1)+((m2)+ (m3)+ (m4)+ (m5))+K2*((m6)+ (m7)+ (m8)+ (m9))+K1*(m10)+((m11)+ (m12)+ (m13)+ (m14))+ K2*((m15)+ (m16)+ (m17)+ (m18))+K3*(mw)]

M2 (X) = Bending moment at a distance(X) due to train load moving to the left case 2.

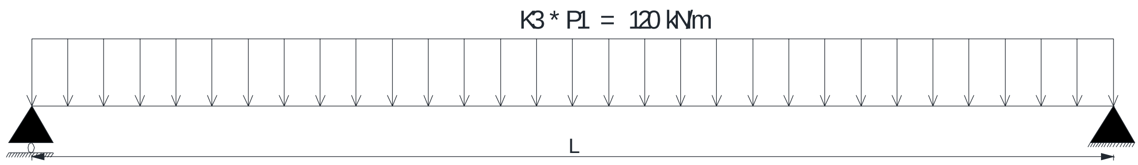


Figure 3-15: Influence Line for bending moment and shear force for moving uniform load case 3.

$M3 = \frac{PL^2}{8}$ (kN/m)

$V3 = \frac{PL}{2}$ (kN)

$$M_{LL} = Max[M_1;\ M_2;\ M_3] \quad 3\text{-}19$$

$$V_{LL} = Max[V_1;\ V_2;\ V_3] \quad 3\text{-}20$$

If the value in bracket is negative, then it is taken as zero. It implies that the wheel is out of the span. In above derivation of concentrated and distributed live load due to train, (X) is the location of the wheel 2 for train moving left for the case 1 and the wheel 10 for the case 2.

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. Among the above calculated results of live load effects, the maximum effects are the one which govern the design. The maximum values for different will be taken in consideration for design.

Permissible linear stress

Concrete

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

Pre-tensioned	0.6 f'ci	3-21
Post-tensioned	0.55 f'ci	3-22

  - Extreme fiber stress in tension

Members with bonded auxiliary reinforcement:	$0.623\sqrt{f'ci}$	3-23
Members without bonded auxiliary reinforcement:	$0.25\sqrt{f'ci}$	3-24

- Stress in concrete at service loads

Compression	0.40 f'c	3-25
Tension	0	3-26

- Cracking stress

For normal weight concrete $0.623\sqrt{f'ci}$ 3-27

For sand lightweight concrete $0.523\sqrt{f'ci}$ 3-28

- Anchorage bearing stress

Post-tensioned anchorage at service load Max 21 MPa 3-29

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) 3-30

  - 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) 3-31

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

    Max (0.82f*y; 0.75f's) 3-32

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

    Max (0.82f*y; 0.70f's) 3-33

    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.

3.3.2 Design procedures

Design of pre-stressed members shall be based on strength (Load factor design) and on behaviour at service load conditions at all stages that may be critical during the life of the structure from the time pre-stressing is first applied. Stress concentrations due to pre-stressing, and effects of temperature, creep and shrinkage shall be considered in design.

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

$$M_{unfactored} = Max[Group(I, II, III, IV, V, VI, VII, VIII)] \quad 3\text{-}34$$
$$V_{unfactored} = Max[Group(I, II, III, IV, V, VI, VII, VIII)] \quad 3\text{-}35$$

Maximum factored Moment & Shear

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

$$M_{factored} = Max[Group(I, IA, II, III, IV, V, VI, VII, VIII, IX)] \quad 3\text{-}36$$
$$V_{factored} = Max[Group(I, IA, II, III, IV, V, VI, VII, VIII, IX)] \quad 3\text{-}37$$

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 others 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 3.5: Types of Losses of Pre-stress

Pre-tensioning	**Post-tensioning**
1) Elastic shortening of concrete	1) Elastic shortening of concrete
2) Creep of concrete	2) Creep of concrete
3) Shrinkage of concrete	3) Shrinkage of concrete
4) Relaxation of tendons stress	4) Relaxation of tendons stress
	5) Friction
	6) 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 3.6: 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

$$f_t \geq f_{tt} - \left[\frac{M_{permanent}}{S_t}\right] \qquad 3\text{-}38$$

$$f_b \leq f_{ct} + \left[\frac{M_{permanent}}{S_b}\right] \qquad 3\text{-}39$$

- At working load

$$f_t \geq \left[\frac{f_{cs}}{\eta}\right] - \left[\frac{M_{permanent}+M_{LL}+M_I}{\eta * S_t}\right] \qquad 3\text{-}40$$

$$f_b \geq \left[\frac{f_{ts}}{\eta}\right] + \left[\frac{M_{permanent}+M_{LL}+M_I}{\eta * S_b}\right] \qquad 3\text{-}41$$

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.

$$f_t = \frac{P}{A} - \frac{Pe}{S_t} \qquad 3\text{-}42$$

and

$$f_b = \frac{P}{A} + \frac{Pe}{S_b} \qquad 3\text{-}43$$

Eliminating e from the equation

$$P = \frac{[A(f_b * S_b + f_t * S_t)]}{[(S_t + S_b)]} \qquad 3\text{-}44$$

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

$$e_{max} = \frac{[S_t * S_b(f_b - f_t)]}{[A * (f_t * S_t + f_b * S_b)]} \quad 3\text{-}45$$

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

$$e_1 \leq \left(\frac{-S_t f_{tt}}{P}\right) + \left(\frac{S_t}{A}\right) + \left(\frac{M_{permanent}}{P}\right) \quad 3\text{-}46$$

$$e_2 \leq \left(\frac{S_b f_{ct}}{P}\right) - \left(\frac{S_b}{A}\right) + \left(\frac{M_{permanent}}{P}\right) \quad 3\text{-}47$$

$$e_3 \geq \left(\frac{-S_t f_{cs}}{\eta P}\right) + \left(\frac{S_t}{A}\right) + \left(\frac{M_{permanent} + M_{LL} + M_I}{\eta P}\right) \quad 3\text{-}48$$

$$e_4 \geq \left(\frac{S_b f_{ts}}{\eta P}\right) - \left(\frac{S_b}{A}\right) + \left(\frac{M_{permanent} + M_{LL} + M_I}{\eta P}\right) \quad 3\text{-}49$$

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)

$$\frac{M_{permanent}}{S}$$

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

$$\frac{M_{LL}}{S}$$

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

$$\frac{M_I}{S}$$

Actual resultant fiber stress at span (e = e_{max})

- Stress in concrete at transfer

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent}}{S_t}\right) \quad \text{3-50}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent}}{S_b}\right) \quad \text{3-51}$$

- Stress in concrete at service load

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{S_t}\right) \quad \text{3-52}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent}+M_{LL}+M_I}{S_b}\right) \quad \text{3-53}$$

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

- Stress in concrete at transfer

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent}}{S_t}\right) \quad \text{3-54}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent}}{S_b}\right) \quad \text{3-55}$$

- Stress in concrete at service load

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent}+M_{LL}+M_I}{S_t}\right) \quad \text{3-56}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent}+M_{LL}+M_I}{S_b}\right) \quad \text{3-57}$$

As there are no moments at the ends of a simple supported beam due to the external loads or to the beams own weight, the bending stress due to different moments is zero there and the equations reduces to

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) \tag{3-58}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) \tag{3-59}$$

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 finishes, 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.

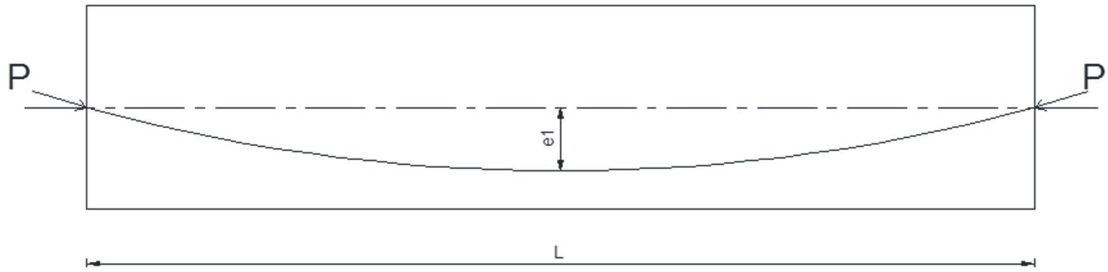


Figure 3-16: Parabolic tendons central anchors

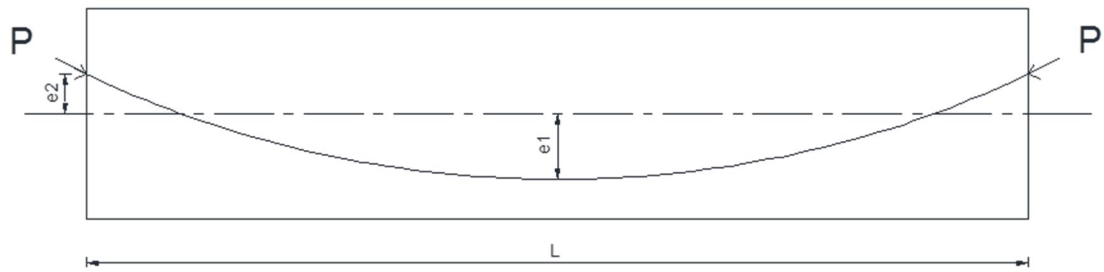


Figure 3-17: Parabolic tendons eccentric anchors

Flexural strength

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. [2]

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:

$$\varphi Mn = \varphi\left[A^{*}s * f^{*}su * \left\{1 - 0.6\left(\frac{P*fsu}{f'c}\right)\right\}\right] \quad 3\text{-}60$$

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:

$$\varphi Mn = \varphi\left[A^{*}s * f^{*}su * d\left\{1 - 0.6\left(\left(\frac{P^{*}f^{*}su}{f'c}\right) + \left(\left(\frac{dt}{d}\right)\left(\frac{pfsy}{f'c}\right)\right)\right)\right\} + As * fsy * dt\left\{1 - 0.6\left(\left(\frac{d}{dt}\right)\left(\frac{P^{*}f^{*}su}{f'c}\right) + \left(\frac{pfsy}{f'c}\right)\right)\right\}\right] \quad 3\text{-}61$$

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:

$$\varphi Mn = \varphi\left[Asr * f^{*}su * d\left\{1 - 0.6\left(\frac{Asr*f^{*}su}{b`df'c}\right)\right\} + 0.85f`c(b - b')(t)(d - 0.5t)\right] \quad 3\text{-}62$$

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:

$$\varphi Mn = \varphi\left[Asr * f^{*}su * d\left\{1 - 0.6\left(\frac{Asr*f^{*}su}{b'df'c}\right)\right\} + As\, fsy(dt - d) + 0.85f'c(b - b')(t)(d - 0.5t)\right] \quad 3\text{-}63$$

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

$$f^{*}su = f's\left[1 - \left(\frac{\gamma^{*}}{\beta_1}\right)\left(\frac{P^{*}f's}{f'c}\right)\right] \quad 3\text{-}64$$

With non-prestressed tension reinforcement included:

$$f^{*}su = f's\left\{1 - \left(\frac{\gamma^{*}}{\beta_1}\right)\left[\left(\frac{P^{*}f's}{f'c}\right) + \frac{d_t}{d}\left(\frac{Pf_{sy}}{f'c}\right)\right]\right\} \quad 3\text{-}65$$

- Unbonded Members

f*su = fse + 100

Where γ*:

0.28 for low relaxation steel

0.40 for stress-relieved-steel

0.55 for bars

Ductility limits

- **Maximum pre-stressing steel**

For rectangular sections

$$\frac{P^{*}f^{*}su}{f'c} \quad 3\text{-}66$$

For flanged sections

$$\frac{A_{sr}f^{*}su}{b'df'c} \quad 3\text{-}67$$

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

$$\varphi Mn = \varphi[(0.36\beta_1 - 0.08\beta_1^2)f'cbd^2] \quad 3\text{-}68$$

For flanged section

$$\varphi Mn = \varphi[(0.36\beta_1 - 0.08\beta_1^2)f'cbd^2 + 0.85f'c(b-b')t(d-0.5t)] \quad 3\text{-}69$$

- **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:

$$M^*cr = (f_r + f_{pe})S_c - M_{d/nc}\left(\frac{S_c}{S_b} - 1\right) \quad 3\text{-}70$$

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:

$$\left(\frac{Pf_{sy}}{f'c}\right)\frac{d_t}{d} + \left(\frac{P^*f^*su}{f'c}\right) - \left(\frac{P'f'_y}{f'_s}\right) \leq 0.36\beta_1 \quad 3\text{-}71$$

For flanged sections:

$$\left(\frac{A_sf_{sy}}{b'df'c}\right) + \left(\frac{A_{sr}f^*su}{b'df'c}\right) - \left(\frac{A'_sf'_y}{b'df'c}\right) \leq 0.36\beta_1 \quad 3\text{-}72$$

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.

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.[1]

Members subject to shear shall be designed so that:

$$Vu \leq \varphi\ (Vc + Vs)$$

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.

$$Vc = Min\ [\ Vci\ ;\ Vcw\ ] \qquad 3\text{-}73$$

Flexure-shear strength V_{ci}:

$$V_{ci} = Max\left[\left(5 * 10^4 * \sqrt{f'c}b'd + V_d + \left(\frac{V_i * M_{cr}}{M_{max}}\right)\right); 220\sqrt{f'cb'd}\right] \quad 3\text{-}74$$

Where

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

$$M_{cr} = \left(\frac{I}{y_t}\right)\left(0.5\sqrt{f'e} + f_{pe} - f_d\right) \quad 3\text{-}75$$

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}:

$$V_{cw} = 10 * 10^5\left[\left(0.29\sqrt{f'c} + 0.3f_{pe}\right)b'd\right] + V_p \quad 3\text{-}76$$

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 [1].

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

$$V_s = Min\left[\left(\frac{Av f_{sy} d}{s}\right); 0.66\sqrt{f'c}b'd\right] \quad 3\text{-}77$$

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.

The minimum area of web reinforcement shall be:

$$A_v = \frac{(0.345 b_w s)}{f_{sy}} \quad 3\text{-}78$$

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 cast-in-place T-beam and box girder flanges shall conform to the provisions in [1] except that the minimum reinforcement in bottom flanges shall be 0.3 percent of the flange section [1].

Bearing strength

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

$$f_b = Min\left[\left(0.7f'ci\sqrt{\frac{A}{Ag}}\right); 2.25f'ci\right] \quad 3\text{-}79$$

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

3.4 General Analysis and Design of Box-girder

3.4.1 Analysis procedure

Structural Depth

For pre-stressed box-girder simple supported spans, the minimum structural depth including deck shall be determined as follow [2]

D = 0.045 L

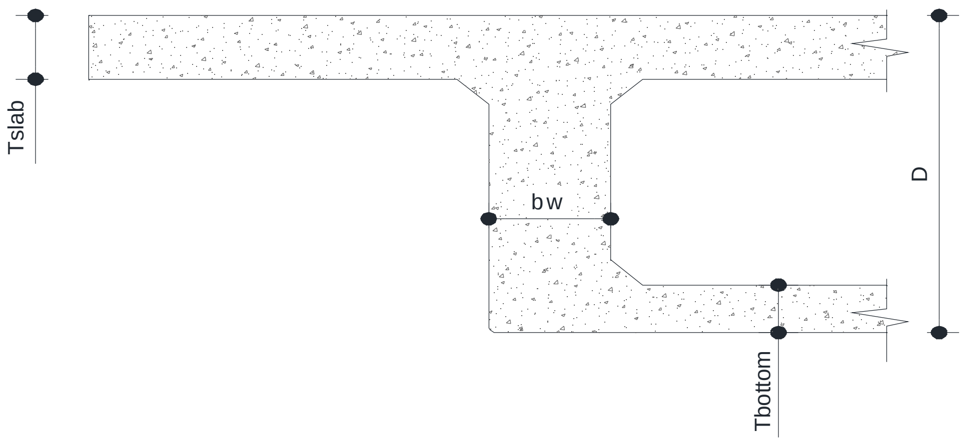


Figure 3-18: Box-Girder bridges cross section

The minimum top slab thickness shall be

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

$$t_{slab} = Max\left[\frac{(S+3000)}{17}; 150\right] \quad \text{3-80}$$

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.[1]

$$t_{bottom} = Max\left[\frac{(S-bw)}{16}; 150\right] \quad \text{3-81}$$

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

$I_T = \sum I_x + \sum A_x d_x^2$
$S_t = I_T / Y_t$
$S_b = I_T / Y_b$

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)
S_t = Section modulus of top fiber of girder (mm^3)
S_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 (3-1)
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 $0 \leq X \leq L$

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

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

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 $0 \leq X \leq L/2$

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

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

For $X > L/2$

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

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

Where

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

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

Super imposed dead load

The bending moment and shear force due to superimposed dead load at every distance L_{1a} 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 $0 \leq X \leq L$

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

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

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

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

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

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

For $X > L/2$

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

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

Where

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

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

Impact load

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

Variable load

The influence lines for shear force and moment for moving load of Box-Girder railway bridge is similar to that of T-Girder bridge. Moreover, the influence line coefficients, the equation for shear force and 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 T-girder will be the same as we use the same material for both sections.

3.4.2 Design procedures

Maximum unfactored Moment & Shear

The maximum value for the different group loading combination in table (3-3)

$$M_{unfactored} = Max[Group(I, II, III, IV, V, VI, VII, VIII)] \quad 3\text{-}82$$

$$V_{unfactored} = Max[Group(I, II, III, IV, V, VI, VII, VIII)] \quad 3\text{-}83$$

Maximum factored Moment & Shear

The maximum value for the different group loading combination in table (3-4)

$$M_{factored} = Max[Group(I, IA, II, III, IV, V, VI, VII, VIII, IX)] \quad 3\text{-}84$$

$$V_{factored} = Max[Group(I, IA, II, III, IV, V, VI, VII, VIII, IX)] \quad 3\text{-}85$$

Loss of pre-stress

The similar estimation loss in T-girder will be taken in to consideration for Box-girder.

Minimum pre-stressed force

$$P = \frac{[A(f_{inf} * S_b + f_{sup} * S_t)]}{[(S_t + S_b)]} \quad 3\text{-}86$$

Maximum eccentricity

$$e_{max} = \frac{[S_t * S_b(f_{inf} - f_{sup})]}{[A * (f_{sup} * S_t + f_{inf} * S_b)]} \quad 3\text{-}87$$

Actual resultant fiber stress at span ($e = e_{max}$)

- Stress in concrete at transfer

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent}}{S_t}\right) \quad 3\text{-}88$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent}}{S_b}\right) \quad 3\text{-}89$$

- Stress in concrete at service load

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) + \left(\frac{M_{permanent} + M_{LL} + M_I}{S_t}\right) \tag{3-90}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) - \left(\frac{M_{permanent} + M_{LL} + M_I}{S_b}\right) \tag{3-91}$$

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

At top

$$f_t = \left(\frac{P}{A}\right) - \left(\frac{Pe}{S_t}\right) \tag{3-92}$$

At bottom

$$f_b = \left(\frac{P}{A}\right) + \left(\frac{Pe}{S_b}\right) \tag{3-93}$$

Deflection

Since the stress limits calculated in steps 12 and 13 are within the tolerable range under the action of all the loadings considered, there is no need to check for the deflections.

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 T-girder is carried for Box- girder to obtain the design moment and shear.

Ductility limits

Similarly as did for T-girder Bridge the same procedure is carried out for computing the maximum pre-stressing steel and the minimum reinforcement.

Design for shear

The objective of design is to provide ultimate resistance for shear, greater than the shear demand under ultimate load (Vu). For simply supported pre-stressed beams, the maximum shear near the support is given by the beam theory.

A similar Procedure that as has been made for T-girder bridge is also considered here.

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 T-girder.

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.[1]

Chapter 4: Demarcating T- and Box-girder Railway Bridges

4.1 Basics of demarcation

There are more than one feasible alternative to obtain the best solution, the minimum material content does not necessarily give the best alternative and even that not involves less cost or more revenue. Factor such as constructability, aesthetics, maintainability, repeatability, simplicity, speed of construction must all be taken in to account.

However cost comparison will be a helpful parameter for selecting bridges type of the same material. In this point of view the materials used for the construction of the bridges such as total volume of concrete, total amount of pre-stressed tendons and reinforcing bars, area of formwork, and cables duct or their associated costs are the only costs taken in to consideration using the unit prices assumed to estimate the construction cost. The cost of materials (construction cost) will be used in the spreadsheet, to select and identify the economical span range for the two types of bridges.

4.2 Design steps for T-and Box-girder railway Bridges

4.2.1 Design steps for Prestressed T-Girder railway bridges

- **Input Data**

- Material

  Concrete: Grade, concrete, density

  Pre-stressed steel: Grade, nominal diameter, cross section area, weight, ultimate load.

  Mild steel: Grade, tensile strength, yields strength, modulus of elasticity.

- Bridge dimensions

  Clear span, track gauge, track width, total width, curb width, curb depth, support width, sleepers width, sleepers depth and height, water proofing thickness and girder spacing.

- Cost

  One meter cubic concrete, one kilograms of pre-stressed steel, one kilograms of reinforced bars, one meter square of formwork, one meter of cable duct, one piece of anchorages devices, one piece of steel bearings.

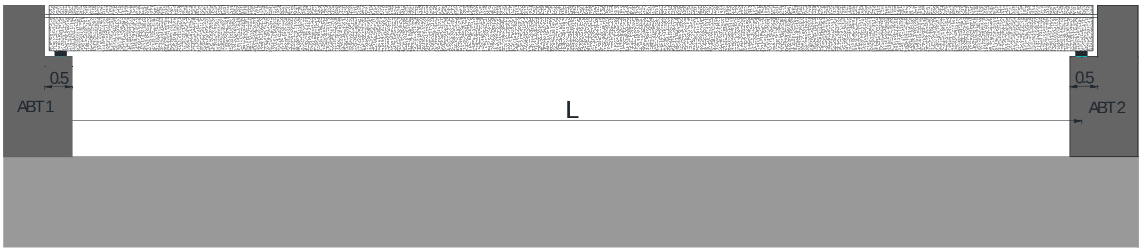


Figure 4-1: Longitudinal Section of a T-and Box- Girder Bridge

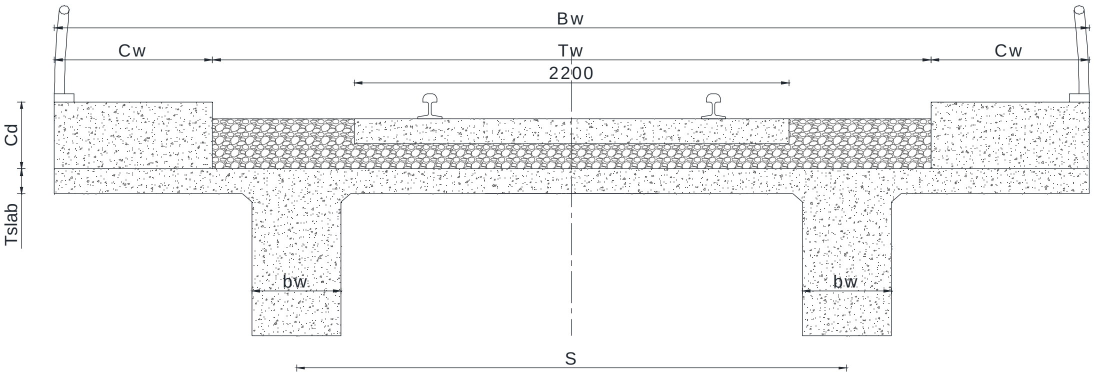


Figure 4-2: T-Girder bridges cross section

- **Analysis**

Given the above input data of material and dimension of the bridges going to be designed, the following will be computed.

- The thickness of top slab is computed using equation (3-14).
- The dead load force effects are obtained by analyzing the volumetric calculation for top slab, and all superimposed load and diffusion area through the ballast.
- The critical live load force effects are obtained by varying the position of the wheel load 2,10 following three cases as shown in the figures (3-11) (3-12) (3-13) (3-14) (3-15).
- The maximum load combinations are taken into consideration from table (3-3), (3-4) for service and ultimate limit design respectively.
- The maximum value for the different group loading combination in table (3-3). and table (3-4) are obtained from equations (3-35),(3-36),(3-37) and (3-38).

- **Design**

- The Pre-stressed force are calculated on the basis of equations (3-44) and corresponding eccentricity is given by the equations (3-45)

- The unfactored and factored design shear force and bending moments are determined respectively from equation (3-34),(3-35),(3-36) and (3-37).
- The actual fiber stress are computed on the basis of equation (3-50),(3-51),(3-52) and (3-53) for both extreme fiber top and bottom at transfer and at service load respectively.
- Design for shear will be investigated and equation (3-78) will be used for minimum shear reinforcement.
- Ultimate limit states are investigated to check the sufficiency of the section. If the section is adequate, then the section is sufficient and the design is completed.
- If the stress condition are not satisfy must increment the girder geometry until satisfaction.

▪ **Total amount of materials**

Approved the adequacy of the section for different limit states, the total amount of materials used are obtained by determining all pre-stressed tendons, volume of concrete, all reinforcing bars, area of formwork, and all cables ducts used, anchorages devices are also added in cost.

After calculating all the required materials, the total cost of the superstructures is obtained by applying current material prices for the purpose of cost-analysis.

The detail of the above steps is shown in a flow chart (Annexe I)

4.2.2 Design steps for Prestressed Box-Girder railway bridges

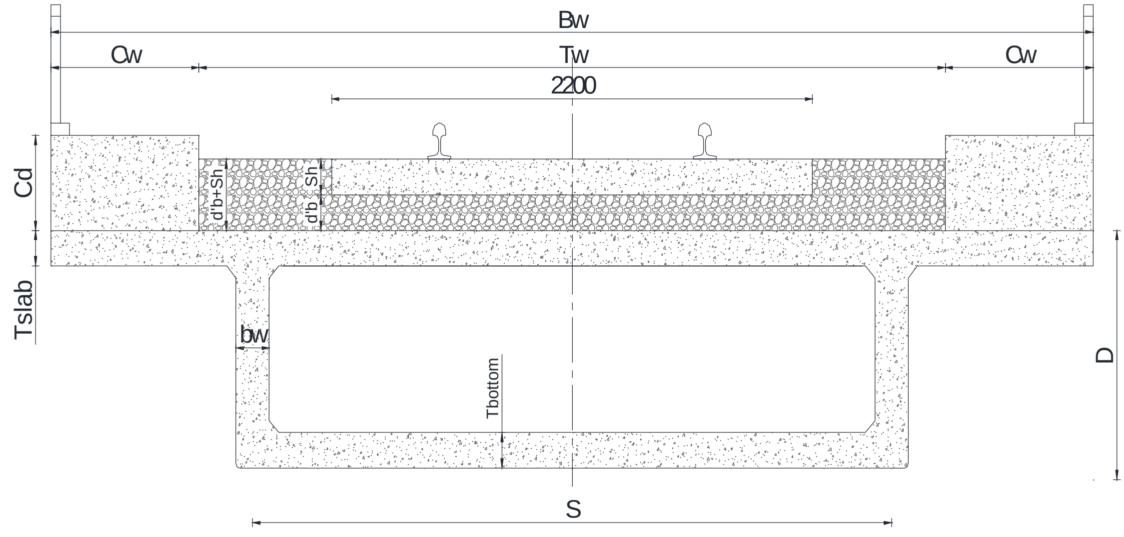


Figure 4-3: Box-Girder bridges cross section

- **Input data**

- Material

  Concrete: Grade, concrete, density

  Pre-stressed steel: Grade, nominal diameter, cross section area, weight, ultimate load.

  Mild steel: Grade, tensile strength, yields strength, modulus of elasticity.

- Bridge dimensions

  Clear span, track gauge, track width, total width, curb width, curb depth, support width, sleepers width, sleepers depth and height, water proofing thickness and girder spacing.

- Cost

  One meter cubic concrete, one kilogram of pre-stressed steel , one kilogram of reinforced bars, one meter square of formwork, one meter of cable duct, one piece of anchorages devices, one piece of steel bearings.

- **Analysis**

- The thickness of top slab and bottom slab are computed using equation (3-80) (3-81) respectively.
- The dead load force effects are obtained by analyzing the volumetric calculation for top slab, and all superimposed load and diffusion area through the ballast.

- The critical live load force effects are obtained by varying the position of wheels following three cases as shown in the figure (3-11) (3-12) (3-13) (3-14) (3-15).
- The maximum load combinations are taken into consideration from table (3-3), (3-4) for service and ultimate limit design respectively.
- The maximum value for the different group loading combination in table (3-3). and table (3-4) and are obtained from equations (3-82),(3-83),(3-84) and (3-85)..

- **Design**

- The Pre-stressed force are calculated on the basis of equations (3-86) and corresponding eccentricity is given by the equations (3-87)
- The unfactored and factored design shear force and bending moments are determined respectively from equation (3-82),(3-83),(3-84) and (3-85).
- The actual fiber stress are computed on the basis of equation (3-88),(3-89),(3-90) and (3-91) for both extreme fiber top and bottom at transfer and at service load respectively.
- Design for shear will be investigated and equation (3-78) will be used for minimum shear reinforcement.
- Ultimate limit states are investigated to check the sufficiency of the section. If the section is adequate, then the section is sufficient and the design is completed.
- If the stress condition are not satisfy must increment the girder geometry until satisfaction.
-

The detail of the above steps is shown in a flow chart (Annexe I)

Chapter 5: Results and discussions

5.1 Demonstrative design Examples

Demonstrative design examples showing the whole process of computation involved in the excel cells, for 30 m span T and Box-girder railway bridges, are attached in the Annexe (II), (III) respectively. The summary of necessary outputs obtained for different spans of pre-stressed T - and Box-girder bridges are presented in Table (5-1) and Table (5-2), respectively.

Table 5.1: Summary of Outputs for T-Girder railway bridges.

Span length (m)	T-Girder dimensions				Pres-stressed					Reinforcements						Reinf Quantity (Kg)	Volume of concrete (m^3)	Area of formwork (m^2)
	Depth (mm)	Top slab thick (mm)	Web width (mm)	Girder spacing (m)	Pres-stressed force (KN)	Tendons		Cable duct (m)	Anchorages steel plates	Mild Reinf Longitudinal Direction		Web Reinf		Bursting Reinf		Grade 420		
20	1440	200	450	2,6	23820	Ø=	15,24	100	2	Ø=	16	Ø=	13	Ø=	13	682,64	45,17	168,3
						N=	131	6				c/c=	289	c/c=				
						L=	20	20		L=	20650	L=	3140	L=	1200			
						kg	2880	120		N=	6	N=	140		45			
24	1720	210	450	2,6	27628	Ø=	15,24	100	2	Ø=	16	Ø=	13	Ø=	13	919,93	61,45	228,3
						N=	152	6				c/c=	289	c/c=				
						L=	24	24		L=	24650	L=	3680	L=	1470			
						kg	4009	144		N=	6	N=	168		52			
30	2160	210	450	2,6	33267	Ø=	15,24	100	2	Ø=	16	Ø=	13	Ø=	13	1346,88	88,81	338,2
						N=	183	8				c/c=	289	c/c=				
						L=	30	30		L=	30650	L=	4560	L=	1910			
						kg	6034	240		N=	6	N=	208		63			

Table 5.2: Summary of Outputs for Box-Girder railway bridges.

Span length (m)	Box-Girder dimensions: Depth (mm)	Box-Girder dimensions: Top slab thick (mm)	Box-Girder dimensions: Bottom slab thick (mm)	Box-Girder dimensions: Web width (mm)	Box-Girder dimensions: Girder spacing (m)	Pres-stressed: Pres-stressed force (KN)	Pres-stressed: Tendons		Pres-stressed: Cable duct (m)		Pres-stressed: Anchorages steel plates	Reinforcements: Mild Reinf: Bottom slab: Transversal Direction		Reinforcements: Mild Reinf: Bottom slab: Longitudinal		Reinforcements: Mild Reinf: Longitudinal Direction		Reinforcements: Web Reinf		Reinforcements: Bursting Reinf		Reinf Quantity (Kg) Grade 420	Volume of concrete (m^3)	Area of formwork (m^2)
							Ø=	15,24	Ø=	100		Ø=	16	Ø=	16	Ø=	16	Ø=	13	Ø=	13			
20	1050	210	150	350	2,6	24611	N=	135	N=	6	2	c/c=	474	c/c=	189,52	c/c=		c/c=	372			803,28531	42,45	190,5
							L=	20	L=	20		L=	2,25	L=	20650	L=	20650	L=	2140	L=	800			
							kg	2976		120								No =	108	No=	46			
							Ø=	15,24	Ø=	100		Ø=	16	Ø=	16	Ø=	16	Ø=	13	Ø=	13			
24	1250	230	150	350	2,6	27324	N=	150	N=	6	2	c/c=	433	c/c=	173,04	c/c=		c/c=	372			1031,1195	56,71	248,7
							L=	24	L=	24		L=	2,25	L=	24650	L=	24650	L=	2500	L=	980			
							kg	3965		144								No =	130	No=	51			
							Ø=	15,24	Ø=	100		Ø=	16	Ø=	16	Ø=	16	Ø=	13	Ø=	13			
30	1555	240	150	350	2,6	30681	N=	168	N=	8	2	c/c=	415	c/c=	165,83	c/c=		c/c=	372			1494,7664	78,84	348,1
							L=	30	L=	30		L=	2,25	L=	30650	L=	30650	L=	3090	L=	1275			
							kg	5565		240								No =	162	No=	58			

5.2 Results

In order to identify the economical span of the two superstructures types T-and Box-girder prestressed railway bridges. The quantity of material obtained from the output and the corresponding material prices are used for the cost-analysis.

Table 5.3: Unit Prices for materials

N°	Materials	Unit	Unit Price (Birr)
1	Concrete	m^3	4100
2	Pre-stressed tendons	kg	370
3	Reinforcing bars	kg	46
4	Formwork	m^2	450
5	Post-tensioned galvanized metal duct	m	150
6	Elastomeric bearing	pcs	25000

Table 5.4: Total Costs for different span lengths

N°	Span length (m)	Total superstructures Cost (Birr)	
		T-Girder	Box-Girder
1	20	1 475 946,88	1 515 755,27
2	21	1 593 738,76	1 612 863,29
3	22	1 736 435,40	1 748 963,52
4	23	1 864 586,68	1 871 840,26
5	24	2 001 829,57	1 980 337,94
6	25	2 142 435,34	2 093 049,50
7	26	2 290 060,26	2 213 616,04
8	27	2 441 274,22	2 327 045,07
9	28	2 609 458,16	2 454 164,99
10	29	2 776 695,71	2 607 748,14
11	30	2 946 672,77	2 743 533,73

Figure (5-1) shows relationships between span length and total cost of superstructure for span length of a bridge ranging from 20 to 30m.

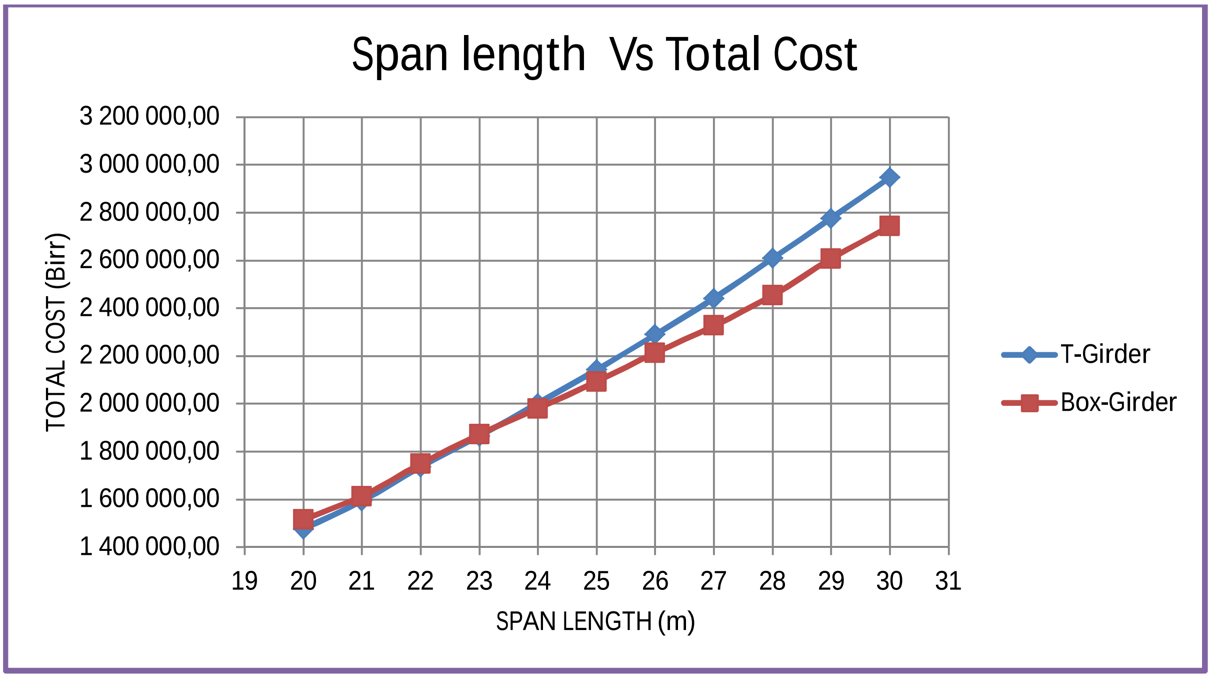


Figure 5-1: Span Length Vs Total Cost

5.3 Discussions

- The results showed in table(5-1) and (5-2) revealed that a larger volume of concrete govern the total cost of T-girder railway bridge compare to his equivalent Box-girder railway bridge, which is related to a large structural depth of T-girder.
- But on the other hand it revealed that the formwork area of pre-stressed box-girder railway bridge is a little larger than this of T-girder, surely due to known formwork type.
- As we have seen the result in table (5-4) we can easily understand that the demarcated span length is 24m.
- The cost can very due to inflation or other factor during time for this purpose this result can vary during time also.

Chapter 6: Conclusions and Recommendations

6.1 Conclusions

In this thesis different pre-stressed T- and Box-girder railway bridge with different span length were studied and a excel sheets has been developed, in order to compare those two superstructure. Analysis and design of those structure have been carried out step by step in the excel sheets. The comparisons have been done regarding their construction cost only.

The demarcate span length is 24m, pre-stressed box-girder railway bridge is economical with a spans length over 24m and span length less than 24m T-girder is more economical.

Box-girder has a larger formwork area due to his complex shape than t-girder.
According the result presented in Table (5-1) & (5-2) we remark that the T bridges is governed by a large volume of concrete, which is higher than those in box-girder bridge due to larger structural depth of girders.

Larger volumes of concrete result from a heavier structure, thus this explain the higher number of prestressed strands for T-girder than Box-girder Bridge.
As the observation of the table (5-4) it found that the two bridges superstructure total cost are very closer, therefore taking into account all cost of bridges may be better for a more exact selection.

Regional factors such as availability of construction material fabrication, location, transportation, method of construction, maintenance, repair or replacement shall be considered. Lower first cost does not necessarily lead to lowest total cost.

6.2 Recommendations

Based on the conclusion of this study the following recommendations are made:

- The economical span that demarcates the two types of bridges (T- & Box- Girder bridges) is 24m. Accordingly, T-Girder bridge is economical up to 24m and that of the Box-Girder bridge is economical beyond 24m.

- This excel can be use for academic purpose, more investigation will be need for a real construction project.

- Further researches should be done on the comparative study of continuous reinforced and prestressed concrete bridges.

- It is important that additional research needs to be done on basis of life cycle cost instead of its initial cost of construction.

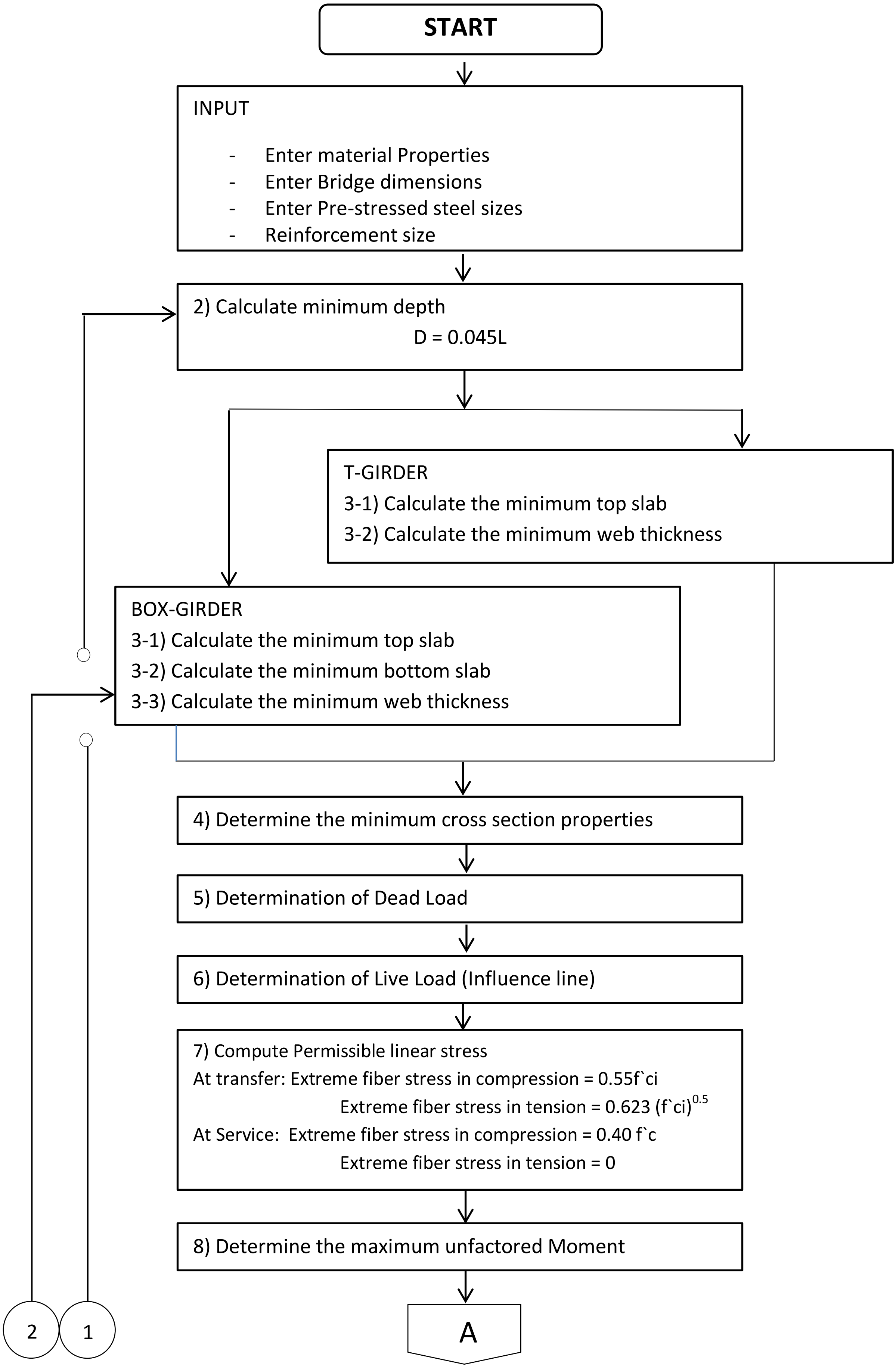
Reference

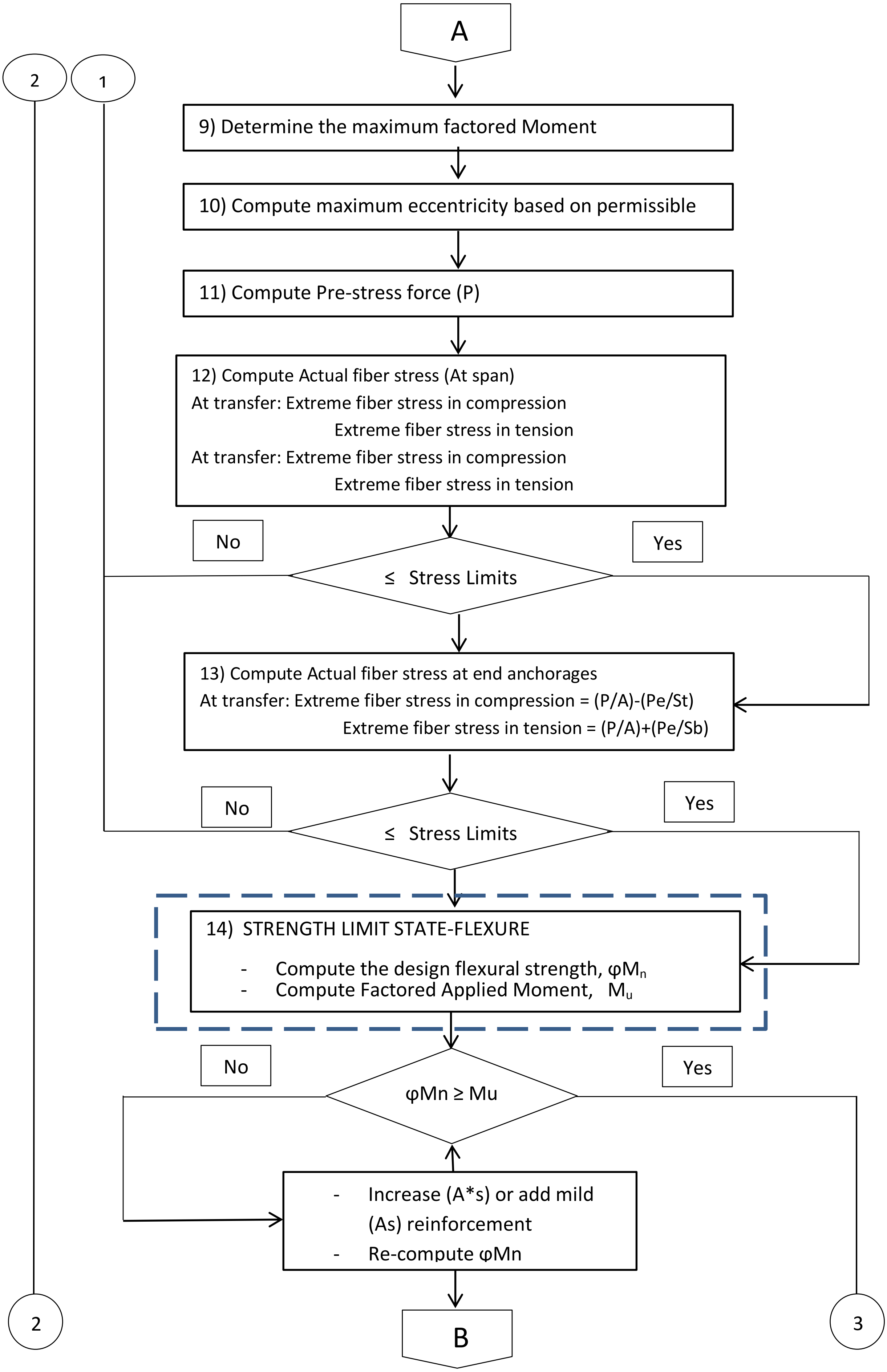
1. AREMA American Railway Engineering and Maintenance-of -way Association 2010.
2. AASHTO LRFD Bridge design specifications 2012.
3. Abiy Alemu, Master`s thesis, Computer program for comparative study of the analysis and Design of T and Box-girder bridges AAU 2013.
4. Abrham Gebre, Master`s thesis, Computer program for comparative study of the analysis and Design of slab and T-girder bridges AAU 2006.
5. Asnake Adamu, (Lecture Notes, Civil Engineering department AAiT).
6. Charles E. Reynolds & James C. Steedman, Reinforced concrete Designer`s handbook tenth Edition.
7. Covern john, Post-tensioned Box-girder Design manual 2015.
8. Dereje Gudeta, Economical span combination and section for a three span continuous post-tensioned concrete railway bridges 2015.
9. Edward G.Nawy, Prestressed concrete a fundamental approach Fifth edition.
10. John F.Unsworth, Design of modern steel railway bridges 2010 press.
11. N Krishna Raju, Prestressed concrete fourth edition.
12. PCI Precast/Prestressed concrete Institute, Bridge Design Manual third edition 2011.
13. Robert Benaim, The design of prestressed concrete bridges concepts and principles.
14. Wai-Fah Chen and Lian duan, Bridge Engineering handbook 2000 CRC press.

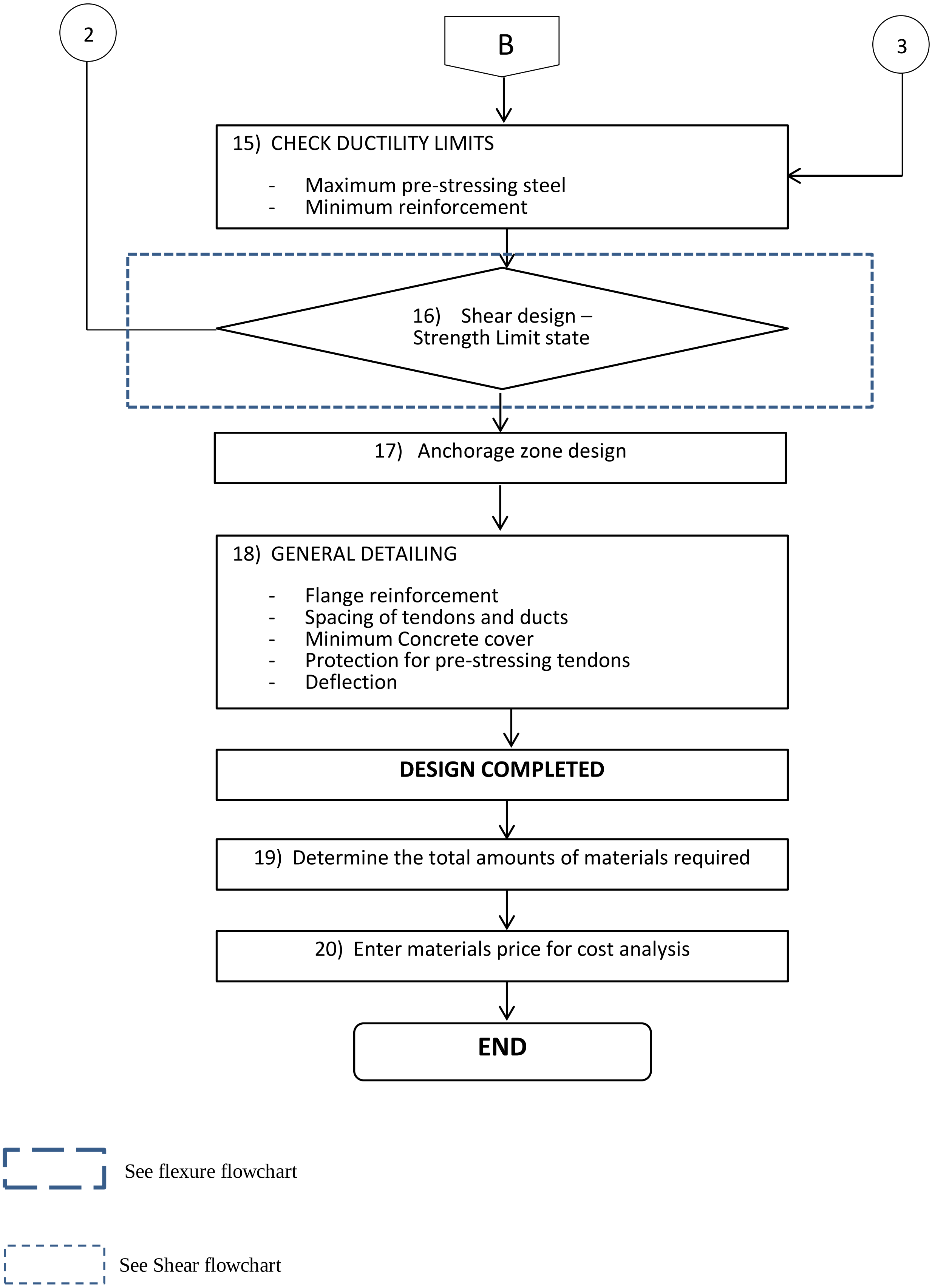
ANNEXE I

FLOWCHARTS

FLOW CHART FOR PRE-STRESSED T- AND BOX-GIRDER BRIDGE

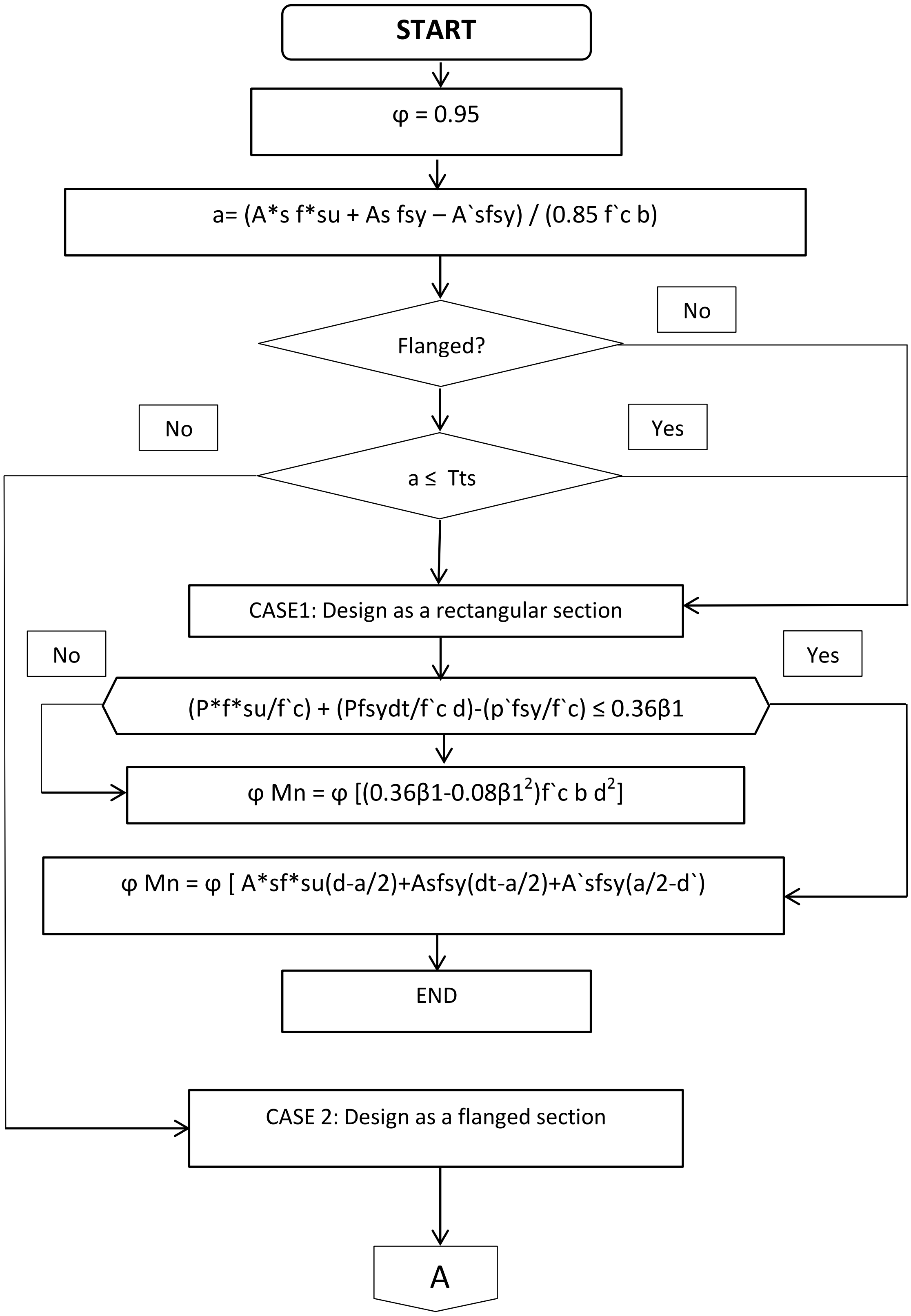


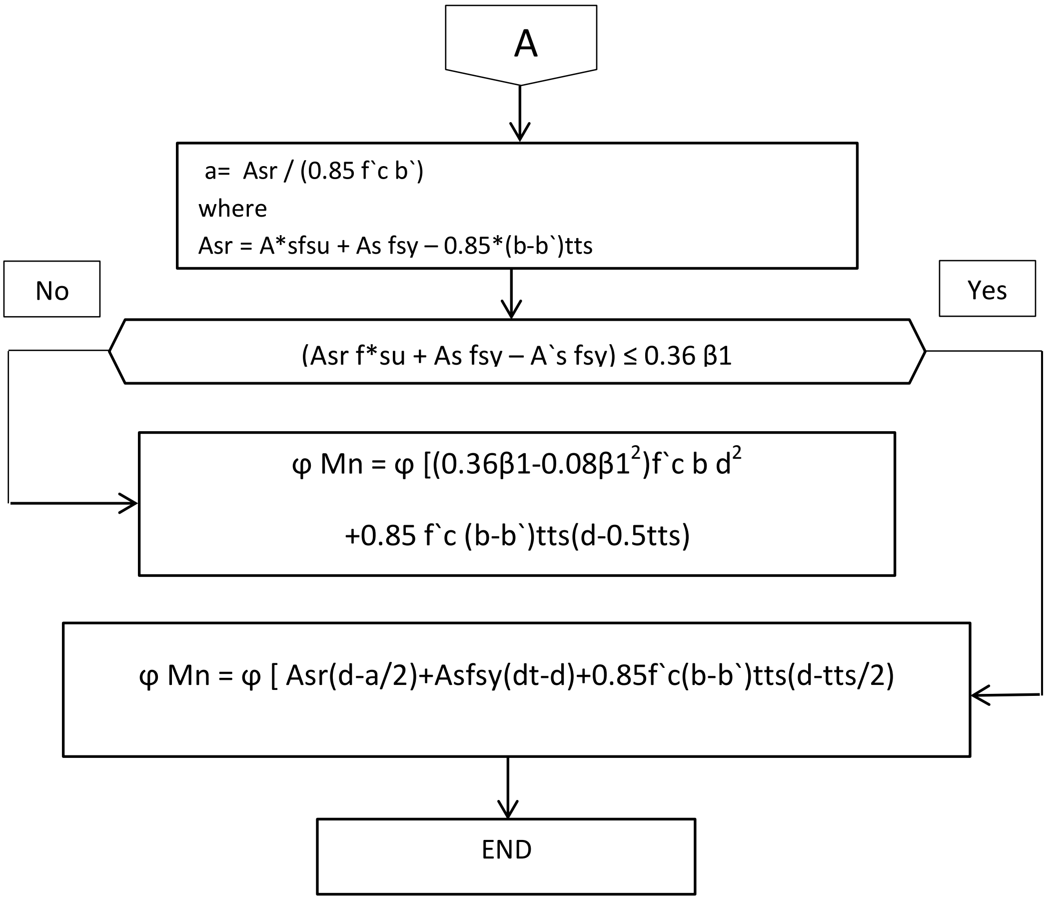

A
2
1
9) Determine the maximum factored Moment
10) Compute maximum eccentricity based on permissible
11) Compute Pre-stress force (P)
12) Compute Actual fiber stress (At span)
At transfer: Extreme fiber stress in compression
Extreme fiber stress in tension
At transfer: Extreme fiber stress in compression
Extreme fiber stress in tension
No
Yes
≤ Stress Limits
13) Compute Actual fiber stress at end anchorages
At transfer: Extreme fiber stress in compression = (P/A)-(Pe/St)
Extreme fiber stress in tension = (P/A)+(Pe/Sb)
No
Yes
≤ Stress Limits
14) STRENGTH LIMIT STATE-FLEXURE
- Compute the design flexural strength, φM_n
- Compute Factored Applied Moment, M_u
No
Yes
φMn ≥ Mu
- Increase (A*s) or add mild (As) reinforcement
- Re-compute φMn
2
B
3



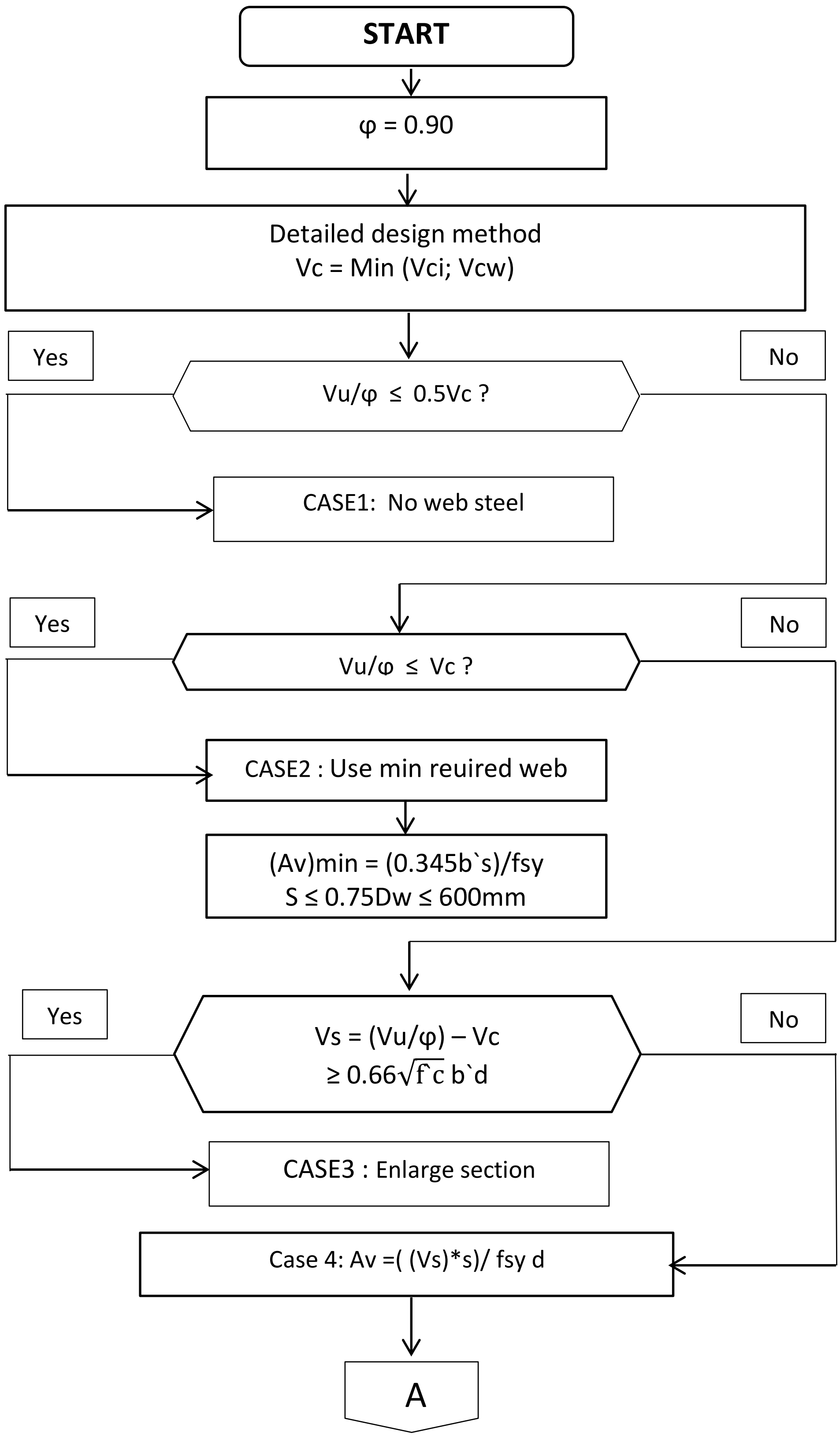
2
B
3
15) CHECK DUCTILITY LIMITS
- Maximum pre-stressing steel
- Minimum reinforcement
16) Shear design – Strength Limit state
17) Anchorage zone design
18) GENERAL DETAILING
- Flange reinforcement
- Spacing of tendons and ducts
- Minimum Concrete cover
- Protection for pre-stressing tendons
- Deflection
DESIGN COMPLETED
19) Determine the total amounts of materials required
20) Enter materials price for cost analysis
END
See flexure flowchart
See Shear flowchart

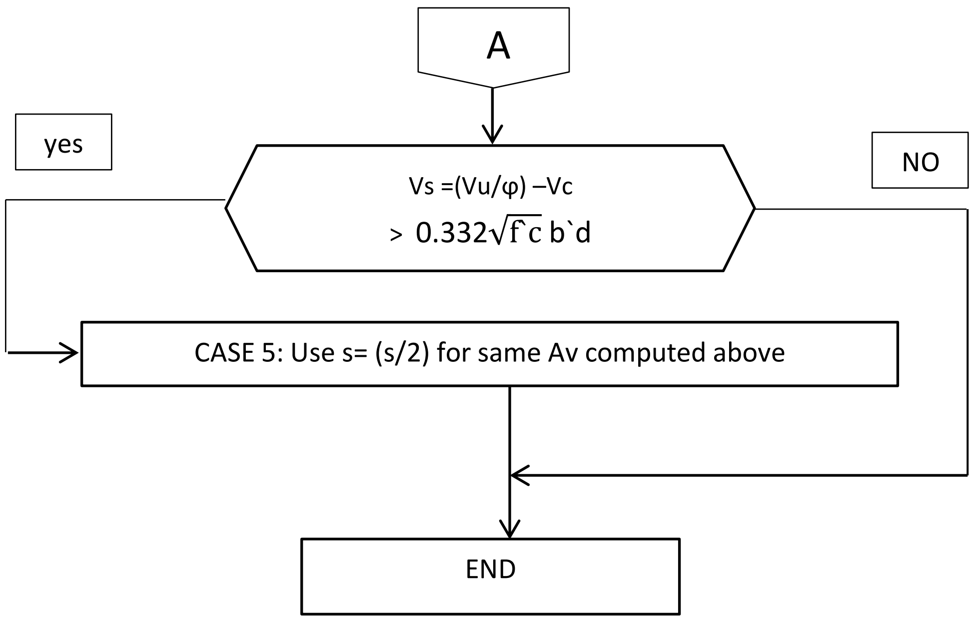
Flowchart for flexural analysis of rectangular and flanged pres-stressed section based on cgs profile depth




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 [(0.36β1-0.08β1²)f`c b d²
+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

ANNEXE II

Excel design outputs of T-Girder

T-GIRDER

ECONOMICAL ANALYSIS AND DESIGN FOR THE SELECTION OF PRE-STRESSED T- AND BOX-GIRDER	Author : Khaire Mohamed	Advisor : Dr.Abrham Gebre
	Internal examiner :	External examiner:
Location : Addis Ababa Institut of Technologie (AAiT)	Date : june 2016	

POST-TENSIONED ANALYSIS & DESIGN FOR T-GIRDER RAILWAY BRIDGE

Input cells

MATERIALS

Concrete	Grade =	50	wc (kg/m^3) =	2500	f_r (Mpa)=	3,052	Cover(mm) =	40
	f'_c (Mpa)=	40	E_c (Mpa)=	33,994				
	f'_{ci} (Mpa)=	24	E_{ci} (Mpa)=	26,332				
	go(N/Kg) =	10						

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 Dimension

Bridge	Clear span =	30	Tw (m) =	4,2	Gauge (m) =	1,435
	Bw(m) =	5,6	WLs (m) =	0,5	WRs(m) =	0,5
Sleepers	S_w (m) =	2,6	S_d (m)=	0,2	S_h (m) =	0,25
Curb	Cw(m) =	0,7	C_d (m) =	0,4	t_{pt} (mm) =	50
	B-G spacing (S) (m) =	2,6	S_{Lp} spacing =	0,6	W_{pw} (Kg/m^3) =	900
	d'$_b$ (mm) =	160	d_b (mm) =	370	W_{bl}(kg/m^3) =	1900
	C/C support length (m)	30,5	Cx (m) =	0,58		

Analysis

Minimum depth recommended AASHTO (D) 0,045 L	2160	(mm)
Minimum top slab thickness (T_{ts}) (S+2,75)/15	210	(mm)
Minimum web thickness (b_{min})	450	(mm)

T-GIRDER

Cross Section Properties	TG						
	Yt (mm) =	752	St (mm^3) =	1,838E+09	I_{cgc} (mm^4) =	1,38176E+12	
	Yb (mm)=	1408	Sb (mm^3) =	981139304	Ac (m^2) =	2,931	
Dead Load	Weight of the Girder per linear meter			73,275		kN/m	
Diaphrams	Dt (m) =	0,5	DP	14,125		kN	
Super Imposed Load	Weight of rail guard rail fastenings (W_{RL})			3		kN/m	
	Weight of sleepers (W_{SL})			5,417		kN/m	
	Weight of ballast (W_{bl})			18,848		kN/m	
	Weigt of curb (W_{cb})			14		kN/m	
	Weight of water proofing (W_{wp})			1,935		kN/m	
	Additionnal weight (Wa)			0,25		kN/m	
	Catenarie			0,02		kN/m	
Total super imposed dead Load		DSI		43,470		kN/m	
Total Dead Load	Dead load of girder + Super imposed load			116,745		kN/m	
Live Load	Live Load Distribution width(m)=	2,97		Vmax (KN)=	2679	kN	
	(Length of tie + $d_{b)}$			Mmax (KN-m) at mid =	17538,50	kN-m	
				Peq LL =	150,8283	kN/m	
Impact Load	For L ≤ 4m 60%			Impact factor	impact load (KN-m)		
	For 4< L ≤ 39m 125/($L^{(1/2)}$)%			0,23	3969,653	kN-m	
	For L > 39m 20%				34,13838	kN/m	

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} =	13683	kN-m
	Maximum Unfactored moments due to Total load =	35191	kN-m
	Maximum factored moments due to DL =	19156	kN-m
	Maximum factored moments due to Total Load =	69342	kN-m
	Maximum moments at end =	0	kN-m

Maximum tensile Stress	f_{top} =	-4,39	N/mm^2
	f_{bottom} =	35,87	N/mm^2

Maximum eccentricty	e_{max}=	914 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 =	28192	kN
	P =	33267	kN
	Number of strands =	183	strands
	Area of prestressing steel =	25550	mm^2

Checking the stress mid span	Concrete	Tansfer	Top fiber =	3,052	N/mm^2	SAFE
			Bottom fiber =	21,92	N/mm^2	SAFE
		Service	Top fiber =	14,752	N/mm^2	SAFE
			Bottom fiber =	0,000	N/mm^2	SAFE

Checking stresses at the end	Concrete at transfer	Top fiber =	10	SAFE
		Bottom fiber =	10	SAFE

Number of Duct	Total strand Area (mm2) =	25550	If the span length is greater than 40 m a duct with inner diameter of 110 mm is used otherwise 100mm of diameter is used
	Ø =	100	
	Duct Area (mm2) =	7853,98	
	Number of duct (mm2) =	8	

Check for Ultimate Strength

φ =	0,95	A*s =	25550	y* =	0,28
f*su =	1770	As =	1194	d =	1665,19
f`c =	40	A`s =	0	dt =	2112
f`s=	1860	P* =	0,0027	P^*_w=	0,01705
fsy =	1675	p =	0,000101	P_w=	0,000628
b =	5600	p`=	0	$P`_w$=	0
b`=	900	β_1 =	0,76	A_{sr} =	13654215,84

Tension

Ø (mm) =	16	Number =	6	Area =	199	Length	30650

Compression

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

CASE 1 Rectangular section
Case 2 Flange section

a= 248,0 > 210 =Tts

CASE 2

a=	446,22

Reinforcement indices

403,083 > 0,275 =0,36β1 /erreinforced

Design flexural strength

φM_n = 7,003E+10 > 69 341,78 = Md OK

Cracking moment

φ Mn = 7,003E+10 > 20962,391 = φM_{cr} OK

Tenson reinforcement quantity

Nominal mass (kg/m) 1,552

Quantity (Kg) =	285,413

Compression reinforcement quantity

Nominal mass (kg/m) 1,552

Quantity (Kg) =	0,000
Total (Kg) =	285,413

Design
for
Shear

Vi =	8179799	Mmax =	67554358016	d =	1728,00
Vd =	1787419	Mcr =	7295518610	φ=	0,90
Vp =	7770808	fpe=	14,75		
Vu =	9967218	fd =	13,95		
Vc =	248569505				

Vu/φ =	11074686,27	<	124284753	= 0,5Vc	(N)
Vu/φ =	11074686,27	<	248569505	= Vc	(N)
Vs =	-237494818,92	<	3245863	$=0,66*(f`c)^{0,5}b`d$	(N)
Vs =	-237494818,92	<	1632767	$=0,332*(f`c)^{0,5}b`d$	(N)
$Av(m^2/m)$=	0,000395	>	4,46339E-07	=Avmin (m^2/m)	

CASE 1

Ø (mm) =	13	Number =	208	Area =	129	s(mm)=	289

Required web steel (m^2/m)	4,46339E-07	<	0,000129	Provided web steel (m^2)

OK

As provided = 0,000129 mm2 ; Use #208 Ø 13 mm rebars stirrups for 2 webs
C/C spacing = 289 Length (mm) = 4560 No = 208

Nominal mass (kg/m) 0,994 | Quantity (Kg) = 942,789

CASE 1 : No Web Steel is required
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]

Check Bearing Stress	f_{br} =	25	A =	750000	N° of anchorages =	2	
			Ag =	750000			
	f_{br} =	25	<	54	OK		
	Actual bearing stress =	22,18	<	25	OK		

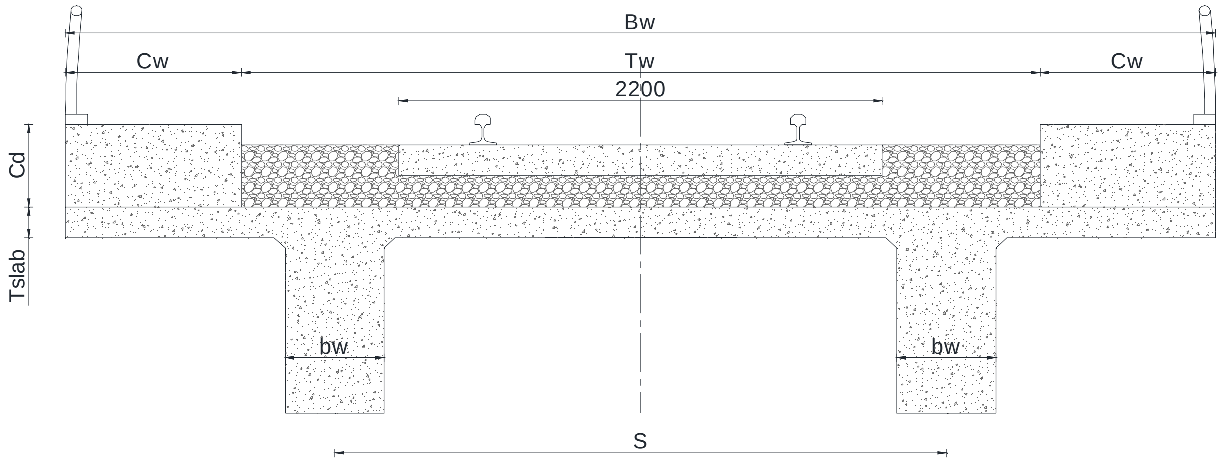
Reinf to resist bursting	Max prestressing force acting of each end anchorages =			16633,36
	Bursting force =	11219,35	fsy =	347,83
	Required reinforcement (mm^2) =	32255	legs=	4

Ø (mm) =	13	Number =	63	Area =	129	length=	1910

Nominal mass (kg/m)	0,994	Quantity (Kg) =	118,678

63 bars of 13 mm of diameter will be use around each anchorages device to resist bursting.

Section Properties



	b	h	Area	y	A*y
A1	5,6	0,21	1,176	2,055	2,41668
2 A2	0,45	1,95	1,755	0,975	1,711125
			2,931		4,127805

Yt (mm) =	752
Yb (mm) =	1408

	b	h	Area	Ix	d	Ad^2	Icgc
A1	5,6	0,21	1,176	0,0043	0,64667	0,49179	0,49611
2*A2	0,45	1,95	1,755	0,5561	0,43333	0,32954	0,88566
				0,5604			1,38176

St (mm^3) =	1838250929
Sb (mm^3) =	981139304

ANALYSIS OF LONGITUDINAL GIRDER

Dead Load force effect

Girder dead load MTG = DG + DP

X	M_{TG}
0	0,000
3,05	3088,924
6,1	5496,207
9,15	7221,849
12,2	8265,851
15,25	8628,212
18,3	8265,851
21,35	7221,849
24,4	5496,207
27,45	3088,924
30,5	0,000
Max	8628,212

X	V_{TG}
0	1124,506
3,05	901,0175
6,1	677,5288
9,15	454,04
12,2	230,5513
15,25	7,0625
18,3	-230,5513
21,35	-454,04
24,4	-677,5288
27,45	-901,0175
30,5	-1124,506
Max	1124,506

Super Imposed dead load

X	M_{SI}
0	0
3,05	1819,694584
6,1	3235,012593
9,15	4245,954029
12,2	4852,51889
15,25	5054,707177
18,3	4852,51889
21,35	4245,954029
24,4	3235,012593
27,45	1819,694584
30,5	0
Max	5054,707

X	V_{SI}
0	662,912
3,05	596,621
6,1	530,330
9,15	464,039
12,2	397,747
15,25	331,456
18,3	265,165
21,35	198,874
24,4	132,582
27,45	66,291
30,5	0,000
Max	662,912

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

X	M_{DL}
0	0,000
3,05	4908,618
6,1	8731,219
9,15	11467,803
12,2	13118,370
15,25	13682,919
18,3	13118,370
21,35	11467,803
24,4	8731,219
27,45	4908,618
30,5	0,000
Max	13682,919

X	V_{DL}
0	1787,419
3,05	1431,347
6,1	1075,276
9,15	719,205
12,2	363,1337
15,25	7,0625
18,3	-363,1337
21,35	-719,205
24,4	-1075,276
27,45	-1431,347
30,5	-1787,419
Max	1787,419

Impact Load

X	M_I
0	0,000
3,05	1429,075
6,1	2540,578
9,15	3334,508
12,2	3810,867
15,25	3969,653
18,3	3810,867
21,35	3334,508
24,4	2540,578
27,45	1429,075
30,5	0,000
Max	3969,653

X	V_I
0	520,610
3,05	416,488
6,1	312,366
9,15	208,244
12,2	104,122
15,25	0,000
18,3	-104,122
21,35	-208,244
24,4	-312,366
27,45	-416,488
30,5	-520,610
Max	520,610

T-GIRDER

		Moments		Shears	
D =	$M_{DL} = M_{TG} + M_{SI}$	13682,919	KN-m	1787,4187	KN
L =	M_{LL}	17538,500	KN-m	2678,60	KN
I =	M_{I}	3969,653	KN-m	520,610	KN

Group loading combinations-Service load design

		Moment	Shear
I	D + L + I + CF + E + B + SF =	35191,072	4986,632
II	D + E + B + SF + W =	13682,919	1787,419
III	Group I + 0.5W + WL + LF + F =	35191,072	4986,632
IV	Group I + OF =	35191,072	4986,632
V	Group II + OF =	13682,919	1787,419
VI	Group III + OF =	35191,072	4986,632
VII	Group I + ICE =	35191,072	4986,632
VIII	Group II + ICE =	13682,919	1787,419
	Unfactored Max =	35191,072	4986,632

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) =	69341,777	9967,218
IA	1.8 (D + L + I + CF + E + B + SF)=	63343,929	8975,938
II	1.4 (D + E + B + SF + W)=	19156,086	2502,386
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)=	49267,501	6981,285
IV	1.4 (D + L + I + CF + E + B + SF + OF)=	49267,501	6981,285
V	Group II + 1.4 (OF)=	19156,086	2502,386
VI	Group III + 1.4 (OF)=	49267,501	6981,285
VII	1.0 (D + E + B + EQ)=	13682,919	1787,419
VIII	1.4 (D + L + I + E + B + SF + ICE)=	49267,501	6981,285
IX	1.2 (D + E + B + SF + W + ICE)=	16419,503	2144,902
	Factored Max =	69341,777	9967,218

Design Moment & shear (Unfactored)

M_{UNF} =	35 191,07	kN-m

V_{UNF} =	4 986,63	kN

Design Moment & shear (Factored)

M_{FT} =	69 341,78	kN-m

V_{FT} =	9 967,22	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	-314	-796	335	416
3,05	4887,078	12668,7859	-461	-943	-115	-33
6,10	8688,138	22522,286	-575	-1057	-464	-383
9,15	11403,181	29560,5004	-657	-1139	-714	-632
12,20	13032,207	33783,429	-706	-1188	-864	-782
15,25	13575,216	35191,0719	-722	-1204	-914	-832
18,30	13032,207	33783,429	-706	-1188	-864	-782
21,35	11403,181	29560,5004	-657	-1139	-714	-632
24,40	8688,138	22522,286	-575	-1057	-464	-383
27,45	4887,078	12668,7859	-461	-943	-115	-33
30,50	0	0	-314	-796	335	416

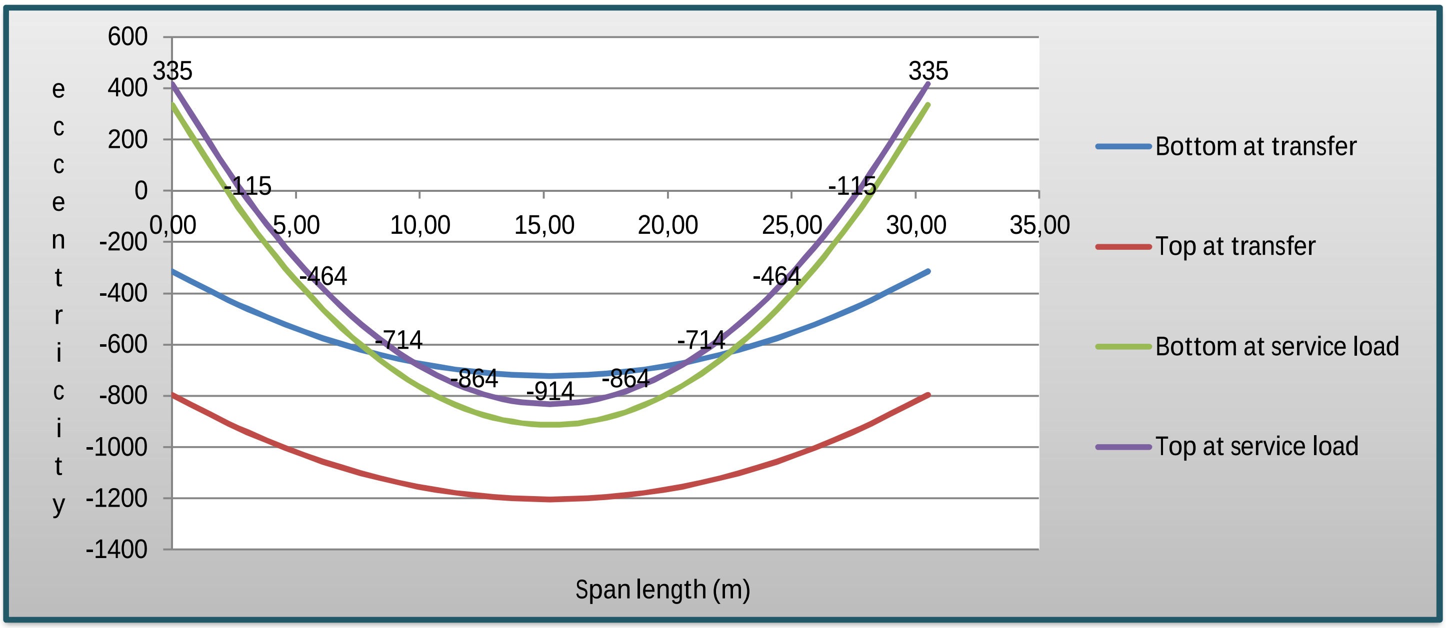


Table 7.1 : Unit Prices for materials

30m span

N°	Materials	Unit	Unit Price (Birr)	QUANTITY	TOTAL AMOUNT T-Girder
1	Concrete	m^3	4100	88,81	364 118,13
2	Pre-stressed tendons	kg	370	6 033,56	2 232 418,74
3	Reinforcing bars	kg	46	1 346,88	61 956,48
4	Formwork	m^2	450	338,18	152 179,43
5	Cable Duct	m	150	240,00	36 000,00
7	Steel bearings	Pcs	25000	4,00	100 000,00

TOTAL (Birr)	2 946 672,77

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

ANNEXE III

Excel design outputs of Box-Girder

Box-GIRDER

ECONOMICAL ANALYSIS AND DESIGN FOR THE SELECTION OF PRE-STRESSED T- AND BOX-GIRDER	Author : Khaire Mohamed	Advisor : Dr.Abrham Gebre
	Internal examiner :	External examiner:
Location : Addis Ababa Institut of Technologie (AAiT)	Date : June 2016	

POST-TENSIONED ANALYSIS & DESIGN FOR BOX-GIRDER RAILWAY BRIDGE

Input cells

MATERIALS

Concrete					
Grade =	50	wc (kg/m^3) =	2500	f_r (Mpa)=	3,052
f'_c (Mpa)=	40	E_c (Mpa)=	33,994	Cover(mm) =	40
f'_{ci} (Mpa)=	24	E_{ci} (Mpa)=	26,332		
go(N/Kg) =	10				

Strand					
Grade=	270	f's (N/mm^2) =	1860	LR or SR =	LR
Nominal Ø(mm) =	15,24	fy (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 Dimension

Bridge						
	Clear span =	30	tw (m) =	4,2	Gauge (m) =	1,435
	Bw (m) =	5,6	WLs (m) =	0,5	WRs(m) =	0,5
Sleepers	S_w (m) =	2,6	S_d (m)=	0,2	S_h (m) =	0,25
Curb	Cw(m) =	0,7	C_d (m) =	0,4	t_{pt} (mm) =	50
	B-G spacing (S) (mm) =	2,6	S_{Lp} spacing =	0,6	W_{pw} (Kg/m^3) =	900
	d'_b (mm) =	160	d_b (mm) =	400	W_{bl}(kg/m^3) =	1900
	C/C support length (m)	30,5	Cx (m) =	0,63		

Analysis

Minimum depth recommended AASHTO (D) 0,045 L	1555	(mm)
Minimum top slab thickness (T_{ts}) (S+3)/17	240	(mm)
Minimum web thickness (bw)	350	(mm)
Minimum bottom slab thickness (T_{bs}) Max(1/16((GS*10^-3)-bw);150)	150	(mm)

Box-GIRDER

Cross Section Properties	Box-Girder						
	Yt (mm) =	571	St (mm^3) =	1,383E+09	I_{cgc} (mm^4) =	7,90098E+11	
	Yb (mm)=	984	Sb (mm^3) =	803317050	Ac (m^2) =	2,602	

Dead Load	Weight of the Girder per linear meter	65,050	kN/m
Diaphrams	Dt (m) = 0,5 qdiaphragms	16,075	kN

Super Imposed Load	Weight of rail guard rail fastenings (qrail)	3	kN/m
	Weight of sleepers (qsleepers)	5,417	kN/m
	Weight of ballast (qballast)	18,848	kN/m
	Weigt of curb (qcurb)	14	kN/m
	Weight of water proofing (qwater proofing)	1,935	kN/m
	Additionnal weight (q additional)	0,25	kN/m
	Catenarie (q catenaries)	0,02	kN/m

Total super imposed dead Load	DSI	43,470	kN/m
Total Dead Load	Dead load of girder + Super imposed load	108,520	kN/m

Live Load	Live Load Distribution width(m)=	3,00	Vmax (KN)=	2679	kN
	(Length of tie + d_b)		Mmax (KN-m) at mid =	17538,50	kN-m
			Peq LL =	150,8283	kN/m

Impact Load	For L ≤ 4m 60%	Impact factor	impact load (KN-m)	
	For 4< L ≤ 39m 125/($L^{(1/2)}$)%	0,23	3969,653	kN-m
	For L > 39m 20%		34,13838	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} =	12741	kN-m
	Maximum Unfactored moments due to Total load =	30365	kN-m
	Maximum factored moments due to DL =	17838	kN-m
	Maximum factored moments due to Total Load =	58959	kN-m
	Maximum moments at end =	0	kN-m

Maximum tensile Stress	f_{top} =	-6,16	N/mm^2
	f_{bottom} =	37,80	N/mm^2

Maximum eccentricty	e_{max}=	859 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 =	26001	kN
	P =	30681	kN
	Number of strands =	168	strands
	Area of prestressing steel =	23564	mm^2

Checking the stress mid span	Concrete	Tansfer	Top fiber =	3,052	N/mm^2	SAFE
			Bottom fiber =	21,938	N/mm^2	SAFE
		Service	Top fiber =	15,798	N/mm^2	SAFE
			Bottom fiber =	0,000	N/mm^2	SAFE

Checking stresses at the end	Concrete at	Top fiber =	10	SAFE
	transfer	Bottom fiber =	10	SAFE

Number of Duct	Total strand Area (mm2) =	23564	If the span length is greater than 40 m a duct with inner diameter of 110 mm is used otherwise 100mm of diameter is used
	Ø =	100	
	Duct Area (mm2) =	7853,98	
	Number of duct (mm2) =	8	

Check for Ultimate Strength

φ =	0,95	A*s =	23564	y* =	0,28
f*su =	1761	As =	1592	d =	1430,56
f`c =	40	A`s =	0	dt =	1507
f`s=	1860	P* =	0,0029	P^*_w=	0,02353
fsy =	1675	p =	0,000189	P_w=	0,001509
b =	5600	p`=	0	$P`_w$=	0
b`=	700	β_1 =	0,76	A_{sr} =	4182547,13

Tension

Ø (mm) =	16	Number =	8	Area =	199	Length	30650

Compression

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

CASE 1 Rectangular section
Case 2 Flange section

a= 232,0 < 240 =Tts

CASE 1

a=	231,97

Reinforcement indices

0,138 < 0,275 =0,36β1 OK

Design flexural strength

φM_n = 5,535E+10 > 58 958,85 = Md OK

Cracking moment

φ Mn = 5,535E+10 > 18171,56 = φM_{cr} OK

Tension reinforcement quantity

Nominal mass (kg/m) 1,552

Quantity (Kg) =	380,550

Compression reinforcement quantity

Nominal mass (kg/m) 1,552

Quantity (Kg) =	0,000

Total (Kg) =	380,550

Design for Shear

Vi =	8130016	Mmax =	57295885255	d =	1430,56
Vd =	1662962	Mcr =	4285807254	φ=	0,90
Vp =	7166781	fpe=	15,80		
Vu =	9792979	fd =	15,86		
Vc =	160605252				

Vu/φ =	10881087,65	<	80302626	= 0,5Vc	(N)
Vu/φ =	10881087,65	<	160605252	= Vc	(N)
Vs =	-149724164,18	<	2090011	$=0,66*(f`c)^{0,5}b`d$	(N)
Vs =	-149724164,18	<	1051339	$=0,332*(f`c)^{0,5}b`d$	(N)
Av(m^2/m)=	0,000301	>	3,47152E-07	=Avmin (m^2/m)	

CASE 1

Ø (mm) =	13	Number =	162	Area =	129	s(mm)=	372

1,04146E-05	Required web steel (m^2/m)	3,47152E-07	<	0,000129	Provided web steel (m^2)

1,04146E-05

OK

As provided = 0,000129 mm2 ; Use #162 Ø 13 mm rebars stirrups for 2 webs

C/C spacing = 372 Length (mm) = 3090 No = 161

Nominal mass (kg/m) 0,994

Quantity (Kg) =	497,577

CASE 1 : No Web Steel is required (hint: use min also)
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]

Design bottom slab

longitudinal

0,50%	1200	mm^2/m
S =	165,83333	mm
As =	1200	mm^2/m

Ø (mm) =	16	Number =	6	Area =	199	Length	30650

Required As =	1200	>	1194	=Provided As	NOT OK, so increase number
Nominal mass (kg/m)		1,552		Quantity (Kg) =	285,413

As provided= 1200 mm2	Use #6 Ø16 mm Rebars
C/C Spacing = 165,8 mm	Length = 30650 mm

Transversal

0,40%	960	mm^2/m
S =	415	mm
As =	480	mm^2/m

Ø (mm) =	16	Number =	74	Area =	199	Length	2,250

Required As =	480	<	14712	=Provided As	OK
Nominal mass (kg/m)		1,552		Quantity (Kg) =	258,162

As provided= 480 mm2	Use #74 Ø16 mm Rebars
C/C Spacing = 415 mm	Length = 2,25 mm

Check Bearing Stress

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

Ag = 750000

f_{br} = 25 < 54 OK

Actual bearing stress = 20,45 < 25 OK

Reinf to resist bursting

Max prestressing force acting of each end anchorages = 15340,45

Bursting force = 10347,26 fsy = 347,83

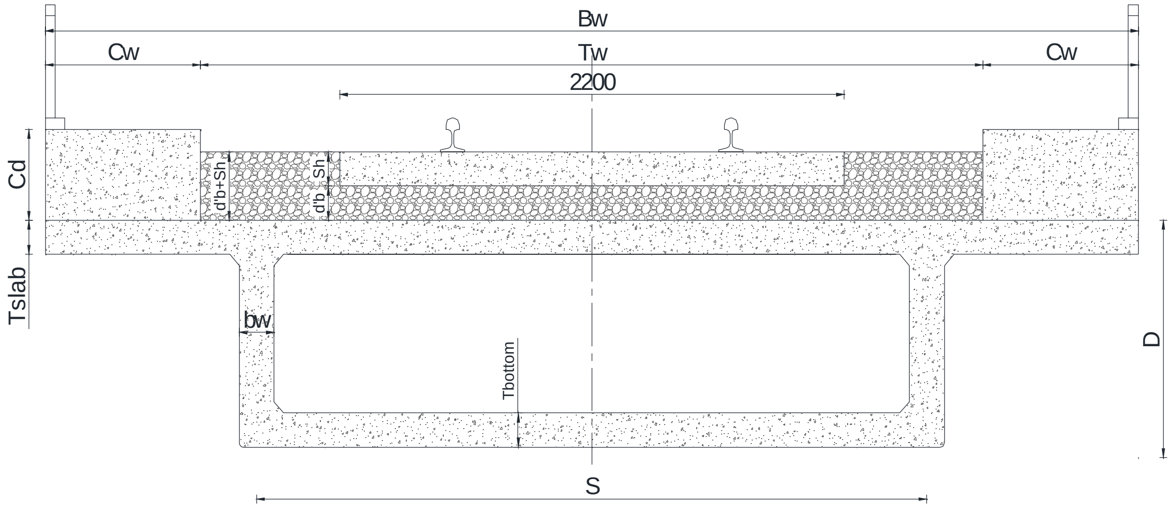
Required reinforcement (mm^2) = 29748 legs= 4

Ø (mm) =	13	Number =	58	Area =	129	length=	1275

Nominal mass (kg/m) 0,994 Quantity (Kg) = 73,064

58 bars of 13 mm of diameter will be use around each anchorages device to resist bursting.

Section Properties



	b	h	Area	y	A*y
A1	5,6	0,24	1,344	1,435	1,92864
2 A2	0,35	1,165	0,8155	0,7325	0,59735375
A3	2,95	0,15	0,4425	0,075	0,0331875
			2,602		2,55918125

Yt (mm) =	571
Yb (mm) =	984

	b	h	Area	Ix	d	Ad^2	Icgc
A1	5,6	0,24	1,344	0,006451	0,45146	0,27392	0,28038
2*A2	0,35	1,165	0,8155	0,092235	0,25104	0,05140	0,14363
A3	2,95	0,15	0,4425	0,00083	0,90854	0,36526	0,36609
				0,099516			0,79010

St (mm^3) =	1382604199
Sb (mm^3) =	803317050,4

ANALYSIS OF LONGITUDINAL GIRDER

Girder dead load

X	Mqsw
0	0,000
3,05	2747,589
6,1	4890,050
9,15	6427,383
12,2	7359,589
15,25	7686,667
18,3	7359,589
21,35	6427,383
24,4	4890,050
27,45	2747,589
30,5	0,000
Max	7686,667

X	V_{qsw}
0	1000,050
3,05	801,648
6,1	603,245
9,15	404,843
12,2	206,440
15,25	8,038
18,3	-206,440
21,35	-404,843
24,4	-603,245
27,45	-801,648
30,5	-1000,050
Max	1000,05

Super Imposed dead load

X	Mq_{SI}
0	0
3,05	1819,694584
6,1	3235,012593
9,15	4245,954029
12,2	4852,51889
15,25	5054,707177
18,3	4852,51889
21,35	4245,954029
24,4	3235,012593
27,45	1819,694584
30,5	0
Max	5054,707

X	Vq_{SI}
0	662,912
3,05	596,621
6,1	530,330
9,15	464,039
12,2	397,747
15,25	331,456
18,3	265,165
21,35	198,874
24,4	132,582
27,45	66,291
30,5	0,000
Max	662,912

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

X	Mpermanent
0	0,000
3,05	4567,283
6,1	8125,062
9,15	10673,337
12,2	12212,108
15,25	12741,374
18,3	12212,108
21,35	10673,337
24,4	8125,062
27,45	4567,283
30,5	0,000
Max	12741,374

X	Vpermanent
0	1662,962
3,05	1331,977
6,1	1000,992
9,15	670,007
12,2	339,022
15,25	8,038
18,3	-339,022
21,35	-670,007
24,4	-1000,992
27,45	-1331,977
30,5	-1662,962
Max	1662,96242

Impact Load

X	M_I
0	0,000
3,05	1429,075
6,1	2540,578
9,15	3334,508
12,2	3810,867
15,25	3969,653
18,3	3810,867
21,35	3334,508
24,4	2540,578
27,45	1429,075
30,5	0,000
Max	3969,653

X	V_I
0	520,610
3,05	416,488
6,1	312,366
9,15	208,244
12,2	104,122
15,25	0,000
18,3	-104,122
21,35	-208,244
24,4	-312,366
27,45	-416,488
30,5	-520,610
Max	520,610

		Moments		Shears	
D =	Mpermanent	12741,374	kN-m	1662,9624	kN
L =	M_{LL}	13653,600	kN-m	2679	kN
I =	M_{I}	3969,653	kN-m	520,610	kN

Group loading combinations-Service load design

		Moment	Shear
I	D + L + I + CF + E + B + SF =	30364,627	4862,176
II	D + E + B + SF + W =	12741,374	1662,962
III	Group I + 0.5W + WL + LF + F =	30364,627	4862,176
IV	Group I + OF =	30364,627	4862,176
V	Group II + OF =	12741,374	1662,962
VI	Group III + OF =	30364,627	4862,176
VII	Group I + ICE =	30364,627	4862,176
VIII	Group II + ICE =	12741,374	1662,962
	Unfactored Max =	30364,627	4862,176

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) =	58958,848	9792,979
IA	1.8 (D + L + I + CF + E + B + SF)=	54656,329	8751,917
II	1.4 (D + E + B + SF + W)=	17837,924	2328,147
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)=	42510,478	6807,046
IV	1.4 (D + L + I + CF + E + B + SF + OF)=	42510,478	6807,046
V	Group II + 1.4 (OF)=	17837,924	2328,147
VI	Group III + 1.4 (OF)=	42510,478	6807,046
VII	1.0 (D + E + B + EQ)=	12741,374	1662,962
VIII	1.4 (D + L + I + E + B + SF + ICE)=	42510,478	6807,046
IX	1.2 (D + E + B + SF + W + ICE)=	15289,649	1995,555
	Factored Max =	58958,848	9792,979

Design Moment & shear (Unfactored)

M_{UNF} =	30 364,63	kN-m
V_{UNF} =	4 862,18	kN

Design Moment & shear (Factored)

M_{FT} =	58 958,85	kN-m
V_{FT} =	9 792,98	kN

Box-GIRDER

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	-267	-669	309	319
3,05	4542,769	10931,26584	-415	-817	-112	-101
6,10	8076,034	19433,36148	-531	-932	-439	-428
9,15	10599,794	25506,28695	-613	-1014	-672	-662
12,20	12114,050	29150,04223	-662	-1064	-812	-802
15,25	12618,802	30364,62732	-679	-1080	-859	-848
18,30	12114,050	29150,04223	-662	-1064	-812	-802
21,35	10599,794	25506,28695	-613	-1014	-672	-662
24,40	8076,034	19433,36148	-531	-932	-439	-428
27,45	4542,769	10931,26584	-415	-817	-112	-101
30,50	0	0	-267	-669	309	319

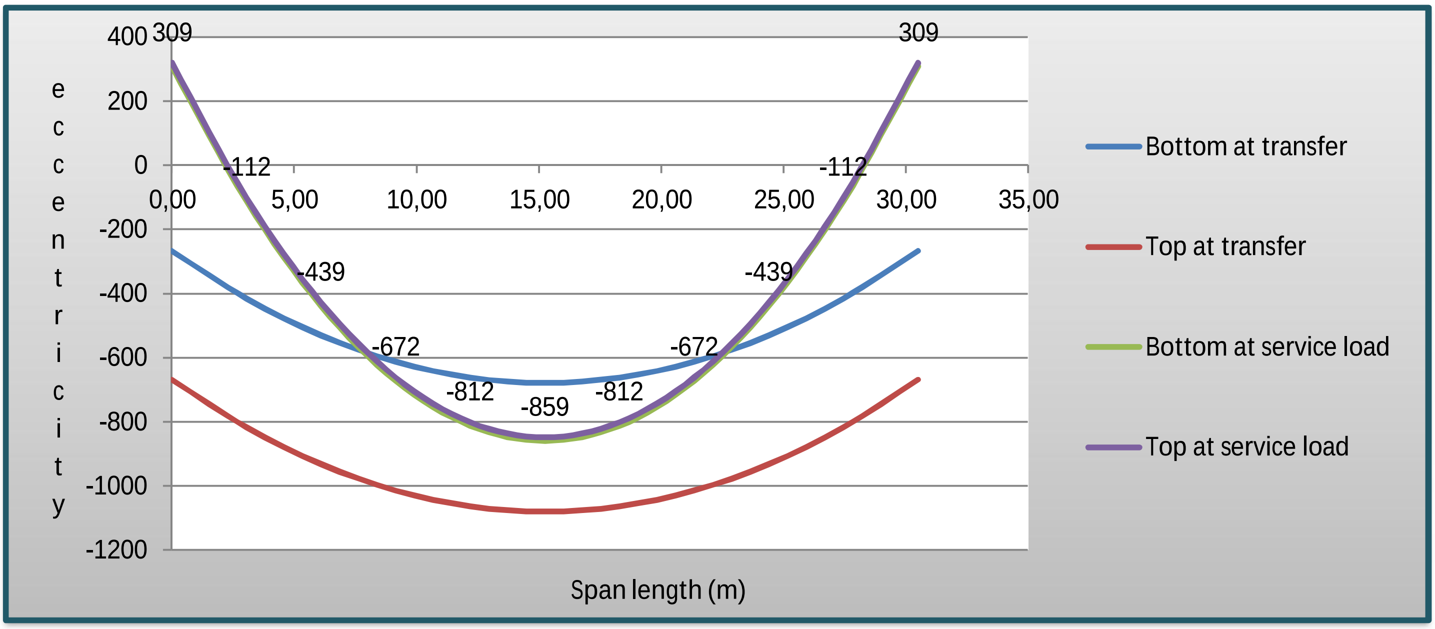


Table 7.1 : Unit Prices for materials

30 m span

N°	Materials	Unit	Unit Price (Birr)	QUANTITY	TOTAL AMOUNT Box-Girder
1	Concrete	m^3	4100	78,84	323 246,46
2	Pre-stressed tendons	kg	370	5564,57	2 058 892,24
3	Reinforcing bars	kg	46	1494,77	68 759,25
4	Formwork	m^2	450	348,08	156 635,78
5	Cable Duct	m	150	240,00	36 000,00
7	Steel bearings	Pcs	25000	4,00	100 000,00

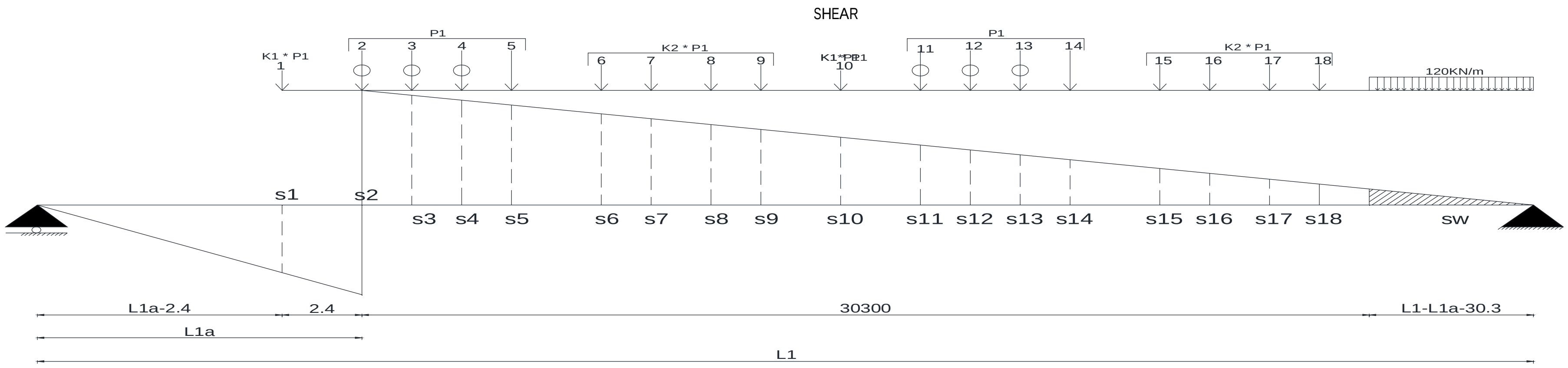
TOTAL (Birr)	2 743 533,73

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

ANNEXE IV

Influence lines for moving loads

INFLENCE LINE FOR MOVING LOAD

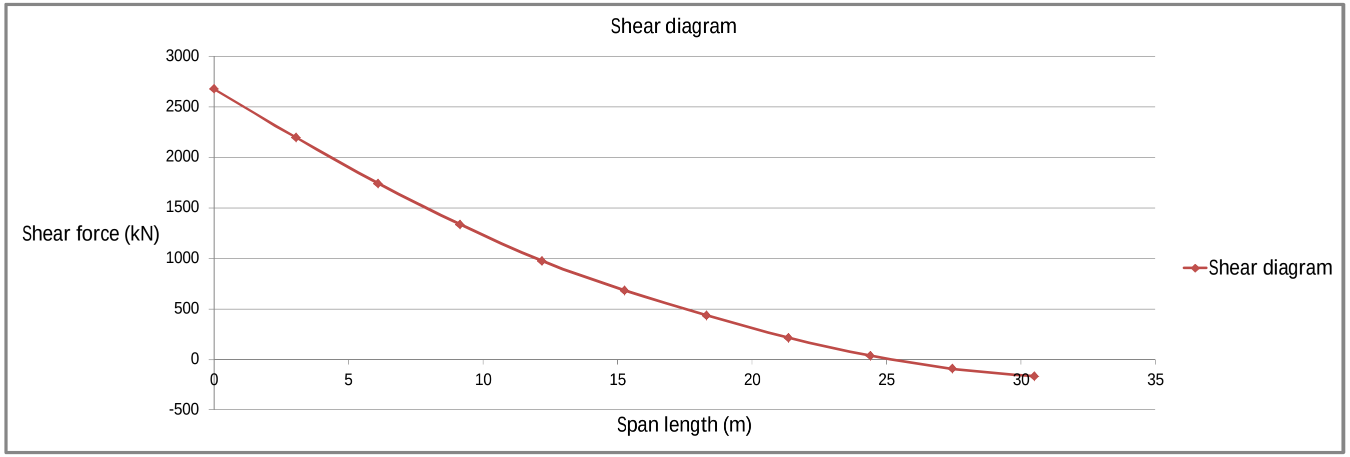


P3 = 180 KN K1 = 0,50
P1 = 360 KN K2 = 0,64
P2 = 230 KN K3 = 0,33
P4 = 120 KN/m

Span L1 30,5

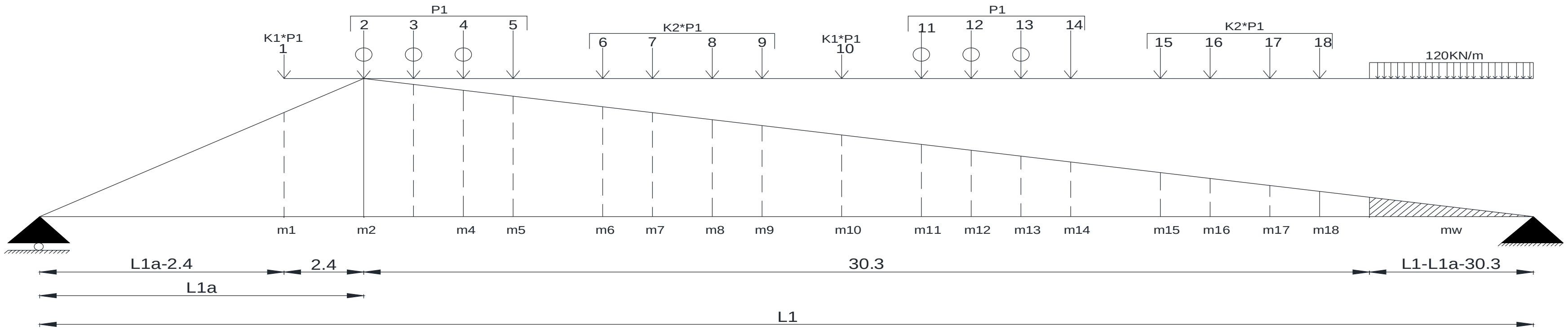
Wheel number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel distance (m)		2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	2,4	2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	1,5	Lw	
Distance cumulative from X		2,4	0	1,5	3	5,7	7,2	9	10,5	12,9	15,3	16,8	18,3	19,8	22,5	24	25,8	27,3	28,8	30,3	Vmax (KN)
Influence line coefficient for shear force (S_x) for X = 0,0*L =	0	0	30,5	29	27,5	24,8	23,3	21,5	20	17,6	15,2	13,7	12,2	10,7	8	6,5	4,7	3,2	1,7	0,02	2678,6
Influence line coefficient for shear force (S_x) for X = 0,1*L =	3,05	0,65	27,45	25,95	24,45	21,75	20,25	18,45	16,95	14,55	12,15	10,65	9,15	7,65	4,95	3,45	1,65	0,15	0	0	2194,87
Influence line coefficient for shear force (S_x) for X = 0,2*L =	6,1	3,7	24,4	22,9	21,4	18,7	17,2	15,4	13,9	11,5	9,1	7,6	6,1	4,6	1,9	0,4	0	0	0	0	1742,3
Influence line coefficient for shear force (S_x) for X = 0,3*L =	9,15	6,75	21,35	19,85	18,35	15,65	14,15	12,35	10,85	8,45	6,05	4,55	3,05	1,55	0	0	0	0	0	0	1336,85
Influence line coefficient for shear force (S_x) for X = 0,4*L =	12,2	9,8	18,3	16,8	15,3	12,6	11,1	9,3	7,8	5,4	3	1,5	0	0	0	0	0	0	0	0	974,557
Influence line coefficient for shear force (S_x) for X = 0,5*L =	15,25	12,85	15,25	13,75	12,25	9,55	8,05	6,25	4,75	2,35	0	0	0	0	0	0	0	0	0	0	685,148
Influence line coefficient for shear force (S_x) for X = 0,6*L =	18,3	15,9	12,2	10,7	9,2	6,5	5	3,2	1,7	0	0	0	0	0	0	0	0	0	0	0	436,426
Influence line coefficient for shear force (S_x) for X = 0,7*L =	21,35	18,95	9,15	7,65	6,15	3,45	1,95	0,15	0	0	0	0	0	0	0	0	0	0	0	0	215,607
Influence line coefficient for shear force (S_x) for X = 0,8*L =	24,4	22	6,1	4,6	3,1	0,4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	37,7705
Influence line coefficient for shear force (S_x) for X = 0,9*L =	27,45	25,05	3,05	1,55	0,05	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-92,951
Influence line coefficient for shear force (S_x) for X = L =	30,5	28,1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-165,84

V_{LL} = 2678,6033



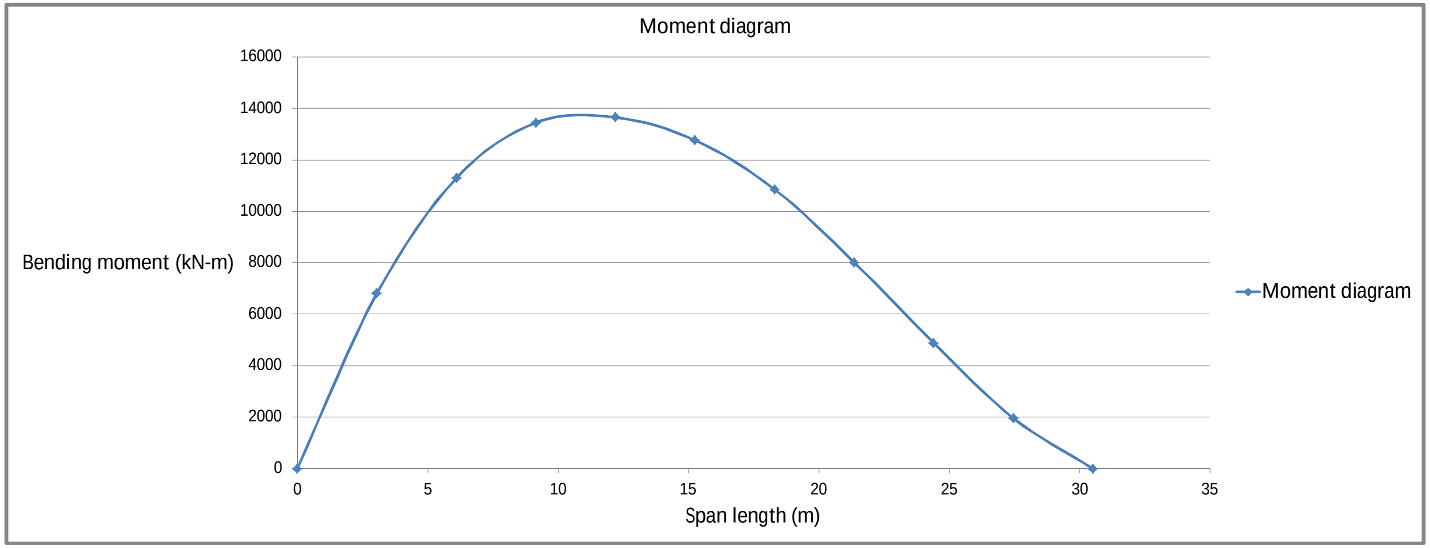
INFLENCE LINE FOR MOVING LOAD

MOMENT

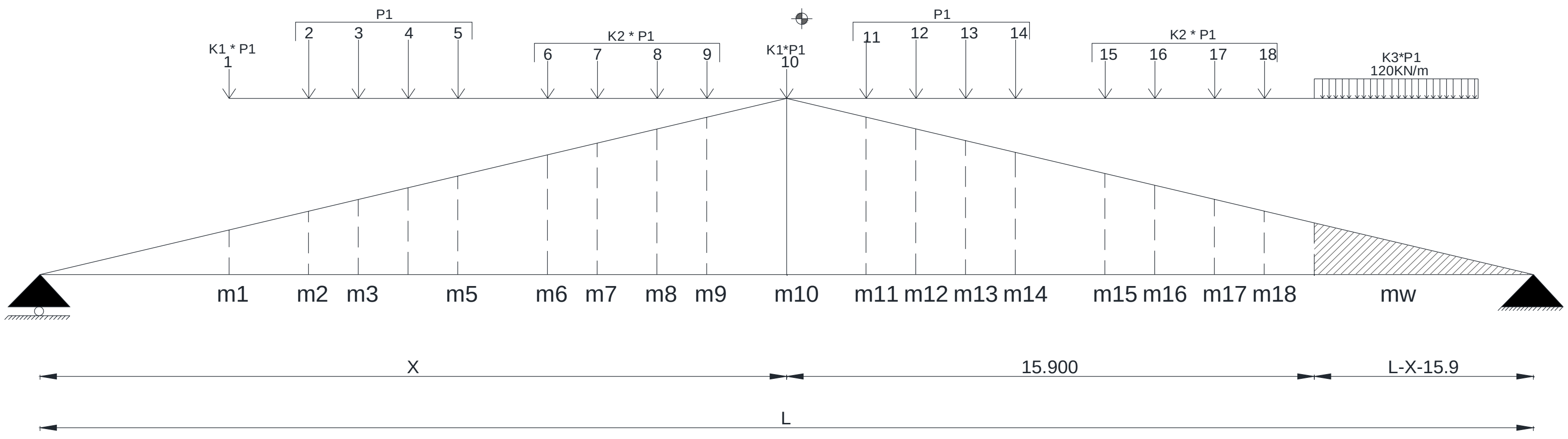


Wheel number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel distance (m)		2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	2,4	2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	1,5	Lw	
Distance cumulative from X		2,4	0	1,5	3	5,7	7,2	9	10,5	12,9	15,3	16,8	18,3	19,8	22,5	24	25,8	27,3	28,8	30,3	Mmax (KN-m)
Influence line coefficient 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 =	3,05	17,843	83,723	79,148	74,573	66,338	61,763	56,273	51,698	44,378	37,058	32,483	27,908	23,333	15,098	10,523	5,0325	0,4575	0	0	6811,35
Influence line coefficient for bending moment (m_x) for X=0,2*L =	6,1	90,28	148,84	139,69	130,54	114,07	104,92	93,94	84,79	70,15	55,51	46,36	37,21	28,06	11,59	2,44	0	0	0	0	11294
Influence line coefficient for bending moment (m_x) for X=0,3*L =	9,15	144,11	195,35	181,63	167,9	143,2	129,47	113	99,278	77,318	55,358	41,633	27,908	14,183	0	0	0	0	0	0	13447,2
Influence line coefficient for bending moment (m_x) for X=0,4*L =	12,2	179,34	223,26	204,96	186,66	153,72	135,42	113,46	95,16	65,88	36,6	18,3	0	0	0	0	0	0	0	0	13653,6
Influence line coefficient for bending moment (m_x) for X=0,5*L =	15,25	195,96	232,56	209,69	186,81	145,64	122,76	95,313	72,438	35,838	0	0	0	0	0	0	0	0	0	0	12761,5
Influence line coefficient for bending moment (m_x) for X=0,6*L =	18,3	193,98	223,26	195,81	168,36	118,95	91,5	58,56	31,11	0	0	0	0	0	0	0	0	0	0	0	10848,6
Influence line coefficient for bending moment (m_x) for X=0,7*L =	21,35	173,39	195,35	163,33	131,3	73,658	41,633	3,2025	0	0	0	0	0	0	0	0	0	0	0	0	8014,2
Influence line coefficient for bending moment (m_x) for X=0,8*L =	24,4	134,2	148,84	112,24	75,64	9,76	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4881,6
Influence line coefficient for bending moment (m_x) for X=0,9*L =	27,45	76,403	83,723	42,548	1,3725	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1957,5
Influence line coefficient for bending moment (m_x) for X= L =	30,5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M_{LL} = 13653,6

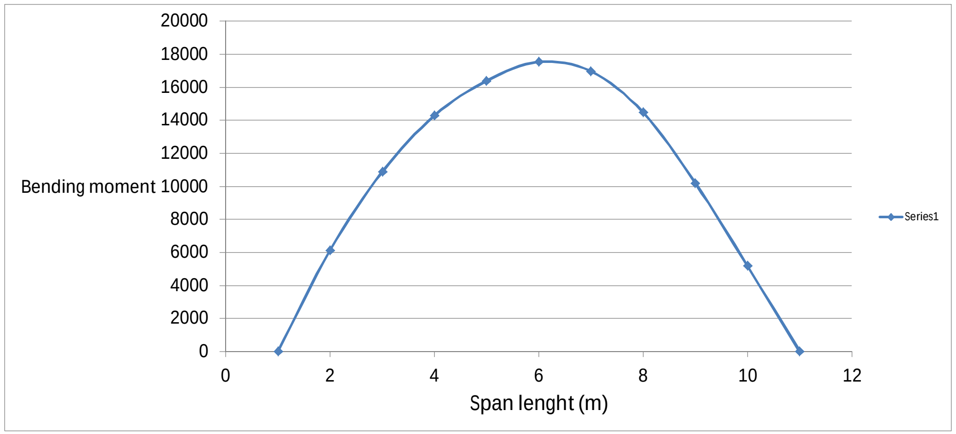


INFLENCE LINE FOR MOVING LOAD

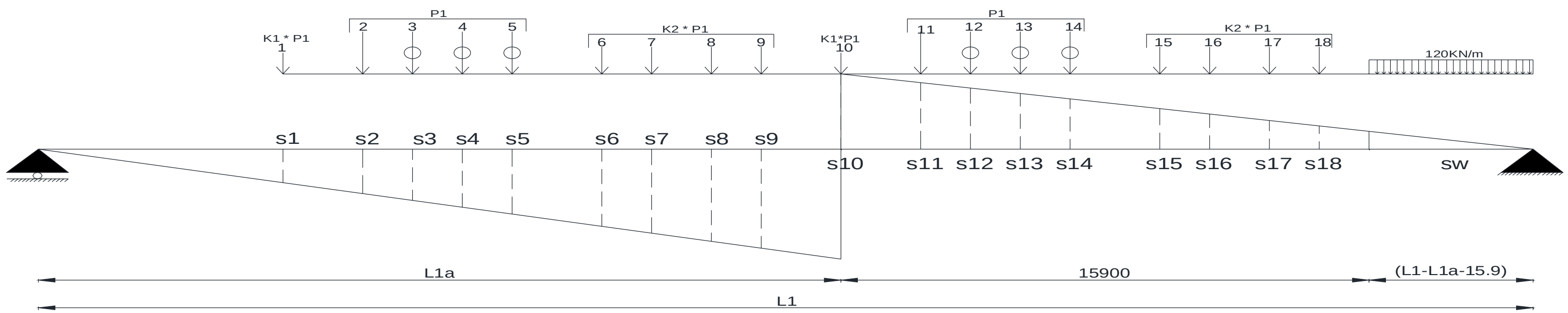


		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel number		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	Ww	
Wheel distance (m)		2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	2,4	2,4	1,5	1,5	1,5	2,7	1,5	1,8	1,5	1,5	Lw	
Distance cumulative from X		16,8	14,4	12,9	11,4	9,9	7,2	5,7	3,9	2,4	0	2,4	3,9	5,4	6,9	9,6	11,1	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 =	3,05	0	0	0	0	0	0	0	0	17,843	83,723	76,403	71,828	67,253	62,678	54,443	49,8675	44,378	39,803	203,44	6133,67
Influence line coefficient for bending moment (m_x) for X=0,2*L =	6,1	0	0	0	0	0	0	9,76	53,68	90,28	148,84	134,2	125,05	115,9	106,75	90,28	81,13	70,15	61	220,36	10874,2
Influence line coefficient for bending moment (m_x) for X=0,3*L =	9,15	0	0	0	0	0	41,633	73,658	112,09	144,11	195,35	173,39	159,67	145,94	132,22	107,51	93,7875	77,318	63,593	135,89	14283,9
Influence line coefficient for bending moment (m_x) for X=0,4*L =	12,2	0	0	0	14,64	42,09	91,5	118,95	151,89	179,34	223,26	193,98	175,68	157,38	139,08	106,14	87,84	65,88	47,58	35,136	16391
Influence line coefficient for bending moment (m_x) for X=0,5*L =	15,25	0	12,963	35,838	58,713	81,588	122,76	145,64	173,09	195,96	232,56	195,96	173,09	150,21	127,34	86,163	63,2875	35,838	12,963	0	17538,5
Influence line coefficient for bending moment (m_x) for X=0,6*L =	18,3	18,3	47,58	65,88	84,18	102,48	135,42	153,72	175,68	193,98	223,26	179,34	151,89	124,44	96,99	47,58	20,13	0	0	0	16969,8
Influence line coefficient for bending moment (m_x) for X=0,7*L =	21,35	41,633	63,593	77,318	91,043	104,77	129,47	143,2	159,67	173,39	195,35	144,11	112,09	80,063	48,038	0	0	0	0	0	14476,8
Influence line coefficient for bending moment (m_x) for X=0,8*L =	24,4	46,36	61	70,15	79,3	88,45	104,92	114,07	125,05	134,2	148,84	90,28	53,68	17,08	0	0	0	0	0	0	10187,2
Influence line coefficient for bending moment (m_x) for X=0,9*L =	27,45	32,483	39,803	44,378	48,953	53,528	61,763	66,338	71,828	76,403	83,723	17,843	0	0	0	0	0	0	0	0	5183,4
Influence line coefficient for bending moment (m_x) for X= L =	30,5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

M_{LL} = 17538,5

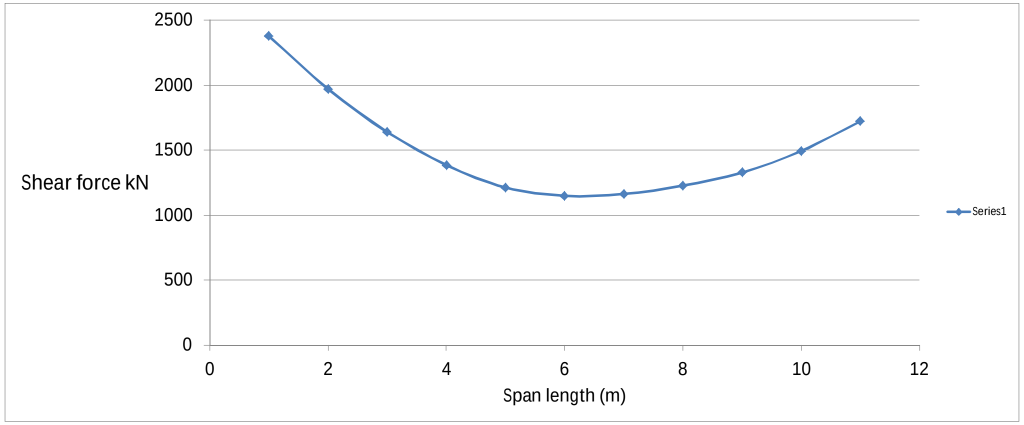


INFLENCE LINE FOR MOVING LOAD



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	30,5	28,1	26,6	25,1	23,6	20,9	19,4	17,6	16,1	106,58	2377,8
Influence line coefficient for shear force (S_x) for X=0,1*L =	3,05	0	0	0	0	0	0	0	0	0,65	27,45	25,05	23,55	22,05	20,55	17,85	16,35	14,55	13,05	66,701	1971,8
Influence line coefficient for shear force (S_x) for X=0,2*L =	6,1	0	0	0	0	0	0	0,4	2,2	3,7	24,4	22	20,5	19	17,5	14,8	13,3	11,5	10	36,125	1640,1
Influence line coefficient for shear force (S_x) for X=0,3*L =	9,15	0	0	0	0	0	1,95	3,45	5,25	6,75	21,35	18,95	17,45	15,95	14,45	11,75	10,25	8,45	6,95	14,851	1386,1
Influence line coefficient for shear force (S_x) for X=0,4*L =	12,2	0	0	0	0,8	2,3	5	6,5	8,3	9,8	18,3	15,9	14,4	12,9	11,4	8,7	7,2	5,4	3,9	2,88	1213,6
Influence line coefficient for shear force (S_x) for X=0,5*L =	15,25	0	0,85	2,35	3,85	5,35	8,05	9,55	11,35	12,85	15,25	12,85	11,35	9,85	8,35	5,65	4,15	2,35	0,85	0	1150,1
Influence line coefficient for shear force (S_x) for X=0,6*L =	18,3	1,5	3,9	5,4	6,9	8,4	11,1	12,6	14,4	15,9	12,2	9,8	8,3	6,8	5,3	2,6	1,1	0	0	0	1162,8
Influence line coefficient for shear force (S_x) for X=0,7*L =	21,35	4,55	6,95	8,45	9,95	11,45	14,15	15,65	17,45	18,95	9,15	6,75	5,25	3,75	2,25	0	0	0	0	0	1226,9
Influence line coefficient for shear force (S_x) for X=0,8*L =	24,4	7,6	10	11,5	13	14,5	17,2	18,7	20,5	22	6,1	3,7	2,2	0,7	0	0	0	0	0	0	1328,3
Influence line coefficient for shear force (S_x) for X=0,9*L =	27,45	10,65	13,05	14,55	16,05	17,55	20,25	21,75	23,55	25,05	3,05	0,65	0	0	0	0	0	0	0	0	1494,1
Influence line coefficient for shear force (S_x) for X= L =	30,5	13,7	16,1	17,6	19,1	20,6	23,3	24,8	26,6	28,1	0	0	0	0	0	0	0	0	0	0	1722,4

V_{LL} = 2377,823



V_{LL} =	2678,60	kN
M_{LL} =	17538,50	kN/m