

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



**ECONOMICAL CONCRETE GRADE FOR
DESIGN OF VARIABLE SPAN CAST-IN-
SITU PRESTRESSED BOX GIRDER
RAILWAY BRIDGE**

A Thesis in Railway Civil Engineering

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October 2017

Addis Ababa

A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge**' presented by **Henok Tadesse**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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I certify that research work titled “Economical concrete grade for design of variable span cast-in-situ prestressed box girder Railway Bridge” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ACKNOWLEDGMENT

I respect and thank Dr. Asnake Adamu, for providing me an opportunity to do on the research title and giving me all support and guidance which made me complete the thesis duly.

I would not forget to remember Beniam Befkadu and Teklit kiflay, of Ethiopian construction Works Corporation for their encouragement and more over for their timely support and guidance till the completion of the study.

ABSTRACT

Elevated rail transportation has the advantage that with limited construction time a more open and lighter space below the structure can be realized, which is crucial to keep cities accessible. This study is interested in one of the elevated railway superstructure type, cast-in-situ posttensioned box girder, which gives flexibility in tendon arrangement and span length. Moreover, it makes statical verification of preliminary design requirement and on the basis of which the main design quantities estimated, then comparative study undertaken to know the benefit or loss behind the concrete grade used for considered span lengths of 30m, 40m, 50m, and 60m investigated. Concrete grade used for girder governs the prestressing layout, the amount of prestress in bridge girder, stress limit flexibility; and Generally, any specific comparisons between different concrete grades, used for each span length box girder possible depth given.

For variable span lengths considered in this study, with the same possible box girder depth provision for each span and for considered, 30MPa, 40MPa and 50MPa, cylindrical compressive strength concrete grade, by making the girder concrete strength higher, saving in prestress tendon used amount and a bit more noticeable benefit from non prestress reinforcement reduction can be enhanced for each span length particularly in context design to Euro code.

Also, for variable span lengths and concrete grade considered, as the span get longer, using higher strength concrete for each span:

- More reduced depth can be provided (10cm, 15cm, 15cm and 20cm, total box girder depth decrement can be enhanced for span length of 30m, 40m, 50m and 60m, respectively by varying the concrete grade of the girder from 30MPa to 50MPa cylindrical compressive strength);
- More saving in prestress tendon amount and non-prestress reinforcement quantity can be made by keeping the possible box girder depth the same for each span.

Finally, based on cost assumptions made, as the span get longer, using higher strength concrete more reduced depth can be provided with comparative cost and with the same possible box girder depth provision for each span length, varying the girder concrete from 30MPa to 50MPa makes the longer span box girders, more cost effective or saving in cost/span ratio become significant.

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CHAPTER 1 INTRODUCTION

1.1 Background

Railway transportation development is becoming a major policy for developing countries mainly due to; limited use of space in urban areas compared to large transport capacity, better reliability, safety and low energy consumption.

To keep grade requirement of train and especially in urban area there is a space constraint it is seen more and more that the infrastructure elevated is most essential. Furthermore, there are fewer conflicts between traffic movements, which reduce the number of accidents, making it a safer way to travel and also an elevated Rail track system has the advantage that it is cheaper than an underground Rail track system in most case and the construction time is much shorter. Considering this from a practical and economical point of view, Provision of Railway Bridge is often the most suitable solution in urban area, for grade requirement and topographic compensation in rural railway lines.

One of the design objectives of such Railway bridge is to minimize the cost and this study is focused on Cast-in-Situ Prestressed Box Girder Railway Bridge super structure preliminary design which is suitable for taking advantage of any tendon profile and flexibility in span longer than suitable precast girder beam. Moreover, there can be a gained profit in term of cost on this super structure by applying different concrete grades.

This study is interested in varying the concrete grade for the following main reasons

- Better controllable parameter (concrete mixes to meet specific requirement can readably prepare),
- Structural parameter can reliably predict by tests,
- Quite feasible to move a range of concrete grade without requiring excessive amount of labor or cement,
- Most economical option than increasing section size or adding reinforcement to enhance the required structural capacity in most case,
- Prestressed concrete utilizes strength of materials effectively and the concrete remains in compression under service loads,

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- The 28-day compressive strength is the primary parameter, which affects a number of properties of hardened concrete such as tensile strength, shear strength, and modulus of elasticity.

However; there are limitations for varying concrete grade and to make use efficiently these includes:

- More stringent quality control of work specification, mix design and special care in material selection,
- Reducing water-cement ratio without affecting their workability in higher strength concrete production; admixture added which in turn affect their cost,
- Field conditions proportioning, mixture, vibration, moisture conditions are not the same as those in curing room and the 28-day strength may not be achieved,
- Weaker grade concrete are less brittle than the stronger one.

Throughout the study C-30, stand for concrete having 30MPa cylinder compressive strength, and similar designations interpolated in the same way.

1.2 Objective and Scope of the study

1.2.1 Objective

One of the design objectives of Railway Bridge is to minimize the cost. This study is interested in whether there can be gained profit in term of cost on the super structure by applying different concrete grade. More specific, the study would like to know, if Cast-in-Situ Prestressed Box Girder Railway Bridge super structure made of C-30, C-40 and C-50 concrete grades, what preliminary structural design parameters result. Besides, the question what is economical interpretation in terms of cost behind the concrete grade used for variable span investigated.

Hence, the main objectives of the study include:

- Determine the preliminary design parameters and structural verification of cast-in-situ prestress box girder superstructure for Railway Bridge of variable span length of 30m, 40m, 50m and 60m; made of 30MPa, 40MPa and 50MPa cylindrical strength concrete grade,
- For each considered span length compare these designs with each other in terms of preliminary design quantity and assumed cost,

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- And, finally for considered span length, economical interpretation in terms of preliminary design quantity and assumed cost behind the concrete grade used investigated.

1.2.2 Scope

In this study simple span cast-in-situ prestress box railway girder with span length of 30m, 40m, 50m and 60m, preliminary design where carried out using, C-30, C-40 and C-50, Cylindrical compressive strength concrete grades.

Research scope limited to applying concrete grade from 30MPa to 50MPa cylindrical compressive strength, for the railway girder as it is having a small impact on the substructure.

The width of the top flange is generally defined by railway Carriage width; from cast in situ build ability requirement the practical minimum thickness of the web is 350mm and a slab thickness of 200mm, considered in preliminary design and analysis.

Then, for considered span length, construction cost for each design box girder using different class of concrete elements quantified and choice between these designs based on the following costs will be made,

- the quantity of material
- the unit cost of material
- the unit cost of construction

1.3 Organization of the thesis

Chapter 1 has presented the essential of railway bridge construction thereby introducing research objectives and scope of the study.

In Chapter 2, over view of common slab girder type and list of their construction consideration, also addressing Properties of some materials to be used for Prestressed Concrete and selected previous studies on high strength concrete mix proportion presented. These proportions serve as a basis for cost comparisons in the later chapters. Selected Previous study related to this research title is also discussed.

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Sub-Chapter 2.5 discusses the basic requirements of the railway bridge design consideration introduction and for the purposes of this thesis undertaking, standard types of loadings and loading combinations to be considered for railway bridges design which are extracted from standard documents are defined.

Chapter 3 presents all the procedural sample calculation adopted for the design of the bridge superstructure in such a manner that the objectives primarily set at the beginning of this study are achieved. Chapter 4 reports the summary of the thesis findings and results against the objectives set forth. Finally, Chapter 5 presents conclusions and recommendations of the study in line with the thesis objectives and findings.

CHAPTER 2 LITERATURE REVIEW

2.1 Common Slab-Girder Bridge types overview

2.1.1 General

Selection of bridge type consideration include a number of factor in general, these factor are related to function, economy, safety, construction experience, traffic control, soil conditions, seismicity, and aesthetics. Among the type of bridge truss bridge has two major advantages [9]:

- Axial loads are the primary member forces
- The overall depth is greater than equivalent solid-web girder due to open-web system

Both these factors lead to economy in material and reduced dead weight and the increase depth also lead to reduced deflections, that is, more rigid structure. Girder type of railway bridges are primarily loaded in shear and flexure. This action is relatively inefficient when compared to axial compression in arches and to tensile forces in suspension structures because, only extreme fibers portion of the cross section fully stressed, it is difficult to obtain an efficient distribution of material in a girder cross section [9]. But from total economic perspective slab-girder Bridge provides an economical and long-lasting solution for the vast majority of bridges found in our country. In order to choose the desired slab and girder bridge type only the most common Slab and girder bridge types are described in here. Together with information about the construction methods give a better understanding of bridge types and its possibilities.

2.1.2 Slab and Girder Bridge

A, Reinforced Concrete

Slab

A reinforced concrete slab is the most economical bridge superstructure for spans of up to approximately 12 meters. They generally require more reinforcing steel and structural concrete than girder-type bridges of the same span. However, the design details and formworks are easier and less expensive. The slab is neat and has a nice appearance.

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Common spans range from 5 to 13 meters, with structural-depth-to-span ratios of 0.06 for simple spans and 0.045 for continuous Spans.

T-beam

T-beams are generally used for clear spans up to about (20m), longer if continuity exists. They however require more complicated formwork, particularly for inclined bridges. The spacing of girders in a T-beam bridge depends on the overall width of the bridge, the slab thickness and the cost of the formwork. The underside appearance of the multiple ribs is not attractive, but if the bridge is over small waterway rather than travelled roadway, there is less objection [9].

Cast in place reinforced concrete box girder

Non-prestressed reinforced concrete box girder bridges are adaptable for use in many locations. These bridges are used for spans of 15 to 35m and are often more economical than steel girders and precast concrete girders. Form work is simpler than for skewed T-beam, but it is still complicated. Appearance is good from all directions. Utilities, pipes, and conduits are concealed. The High torsional resistances make them desirable on curved alignment.

B, Prestressed concrete

Precast concrete box beam

Can be used for span from 10 to 50m these bridges are most suitable for locations where use of false work is impractical or too expensive but longer girders are heavy, and firm ground is needed to store the girder and provide support for the lifting cranes. The construction time is much shorter than that needed for cast in place T-beam and its high structural efficiency which minimizes the prestress force required. Once the span becomes too large where transportation and handling may present a problem the box girder becomes a precast segmental girder. Girder longer than 35m may have to be brought to the site in segments and then multiple precast box girder elements are connected to each other by mean of post-tensioning. The selection between cast-in-situ and precast segmental is dependent of project features, site conditions, environmental and public constraints, construction time and available equipment.

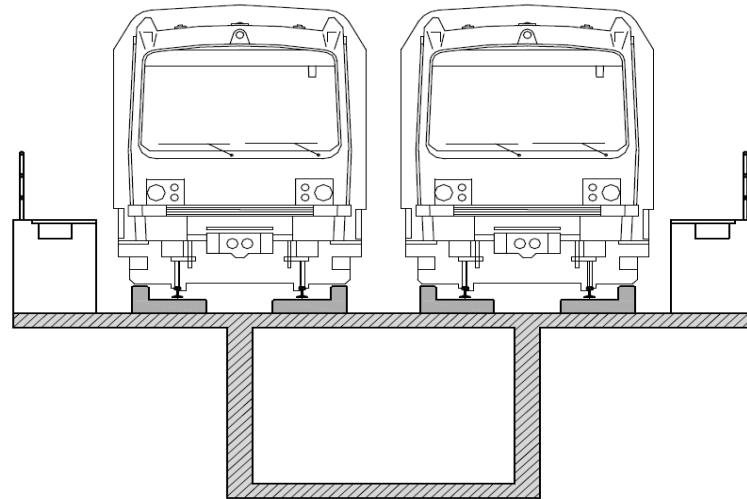


Figure 2-1: Cross-section superstructure with pre-cast box Girder Bridge

Precast concrete I-beam

Can be used for span from 10 to 50m and are competitive with steel girders. They have many of the same characteristics as precast concrete box Beam Bridge. The I-beam may be laterally unstable until incorporated into the structure and should be braced until the diaphragms are cast [9].

Slab

Cast-in-situ prestressed concrete slabs using high-strength materials are more expensive than reinforced concrete slabs. A precast prestressed slab is economical when many spans are involved hole. Common spans range from 6 to 15 meters.

Cast in place posttensioned concrete box girder

Cast-in place concrete box girders are especially suited to curved alignment because of their superior torsional resistance and the ability to keep the cross section constant as it follows the curves. With the use of posttensioning, clear spans of (43m) are common [9]. Prestress concrete box Girder Bridge affords many advantages in terms of safety, appearance, maintenance, and economy. Because longer spans up to 180m can be constructed economically the number of piers can be reduced and shoulder obstacles eliminated for safety travel at over passes. Appearance from all direction is neat and simple with greater slenderness than conventional reinforced concrete box-girder bridges. Because of the prestress the dead-load deflections are minimized, Maintenance is very low, except that bearing and transverse deck joint details require attention. Cast in situ

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concrete bridges may not be the first choices if speed of construction is primary importance. Also, if formwork cannot be suitably supported, such as in congested urban setting where traffic must be maintained, the design of special false work to provide a construction plate form may be a disadvantage.

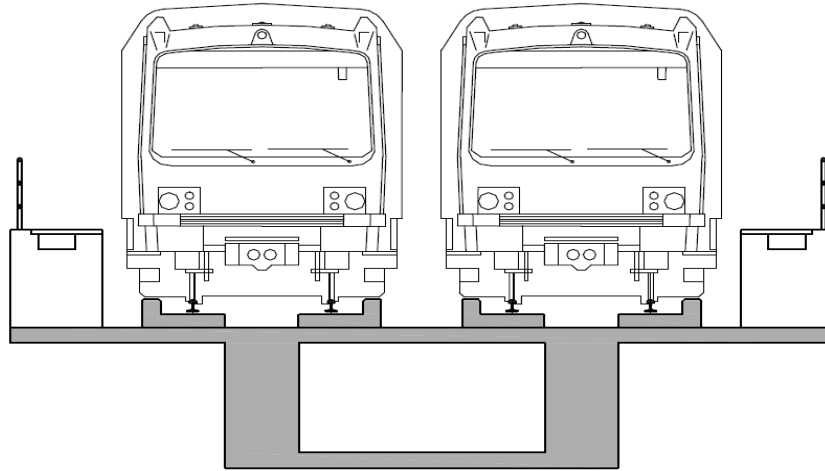


Figure 2-2: Cross-section superstructure with cast in-situ box Girder Bridge



Figure 2-3: Anchored end span bridge located in north-central Illinois

2.2 Background of Construction methods

There are different methods to construct a bridge. Determining the method of construction and bridge type requires taking into account the required span length and the existing site constraints. Furthermore, the construction schedule, the contractor's equipment or the size of the project may also be determining factors to prefer one method over another. The methods of construction discussed here concern construction methods for concrete slab and Girder bridges:

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Cast-in-situ construction

It is a method where the concrete is poured into the formwork at the construction site. The concrete is delivered by trucks or is produced at the site. This way there is no need of complicated transport of large bridge elements and by continuous pouring a large monolith structure can be constructed with the desired geometrical shape.

One of the main disadvantages of box girder Bridge is the difficulty to cast in-situ due to the inaccessibility of the bottom slab and need to extract internal shutter [15]. Either the box has to be design so that the entire cross section may be cast in one continuous pour, or the cross section has to be cast in stages. The most common method of building cast in-situ box girder superstructure is to cast the cross section in stage, either the bottom slab cast first with the web and top slab cast in second phase or the webs and bottom slab constitute the first phase, completed by the top slab. Where cast-in-situ construction is used for longer span bridges, the required false work becomes more sophisticated.

Balanced cantilever

The construction of a balanced cantilever method begins at the bridge piers. From there cast-in-situ segments of 3 to 5-meter length are poured into the special formwork. The formwork moves in tandem with each segment. This way the structure stays in balance (notice the name of this method). The completed segments are used as erection platform and launching base for the subsequent segments. It is a flexible and efficient method which gives little disturbance to its surroundings. However, it is a relative slow building method. By utilizing precast segments this restriction can be improved. This construction method is chosen where a bridge has few spans which range from 50 to 250 meters.

Precast construction

Precast segmental decks

Precast segmental deck construction is used for long bridges where the deck depth is difficult for cast-in-situ construction. For this system generally box girder segments are used. The segments are connected either by internal or external prestressing. The construction has a repetitive character which can be very efficient in case of a tight organized time schedule. The balanced or free cantilever method about a pier is a preferred choice.

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Precast beams

Precast beam decks are generally used for short span bridges ranging from 5 to 50 meters. The beams are placed by a crane which has a rate of construction of approximately four beams a day. A cast-in-situ slab top deck is normally used and it is important to examine the transportation towards the construction site.

Span-by-span construction

Span-by-span is a relatively new construction technique. It is considered to be the most economic and rapid method of construction available for long bridges and viaducts with individual spans up to 60 meters. This system makes use of precast segments which are continuously placed from one abutment to the other. The segments can be positioned by a temporary staying mast system or by a launching truss system. Once all the segments are in position the segments are connected by longitudinal prestressing. Finally deck joints are cast and Closed and prestressing ducts are grouted. When the span is completed the launching truss moves to the next span where the construction cycle starts again until the bridge is completed.

Incremental launching

For bridge decks greater than 250 meters in length, the incremental launching method can be considered. With this method of construction, the bridge deck is built in sections by pushing the structure outwards from an abutment towards the pier. The sections are cast contiguously, one after another, and are then stressed together. The superstructure is launched over temporary sliding bearings on the piers until the bridge is completed.

Table 2-1: Range of application of some prestressed box girder bridge type

Span (m)	Bridge Type
30-91	Cast-in-situ post tensioned box girder
30-91	Precast-balanced cantilever segmental, constant depth
61-183	Precast-balanced cantilever segmental, variable depth
61-305	Cast-in-situ cantilever segmental

2.3 Prestress concrete material

2.3.1 Concrete

2.3.1.1 General

Concrete is a versatile building material mainly due to shape flexibility to conform to almost any alignment and profile, or they can be looks a like precast girders and box beams manufactured in nearby plant, in addition the constitute raw materials of concrete (cement, fine aggregate, coarse aggregate and water) are found in most area of the world. In many countries without well-developed steel industry, reinforced concrete is naturally crucial building material.

Prestressed concrete utilizes high strength materials efficiently by pretensioning high strength steel so that the concrete remains in compression under service loads activated while the surrounding concrete is compressed.

“Prestressing operation result in a self-equilibrating internal stress system which accomplishes tensile stress in the steel and compressive stress in the concrete that significantly improves the system response to induced service loads (Collins and Mitchell, 1997)”

Prestress concrete some limitations are its low superstructure ductility, the need for higher concrete compressive strengths, and larger member sizes to accommodate ducts inside the girders.

In most case abridge engineer will select a particular class of concrete from a series of predesigned mixes, usually on the basis of desired 28-day compressive strength, which is the primary parameter, affecting properties of hardened concrete such as tensile strength, shear strength, modulus of elasticity and if strength evaluation of a concrete bridge has been in service for a number of years required, the compressive strength of a core sample is a good indication of the quality and durability of the concrete in bridge [9].

In this thesis, concrete classification assumption based on compressive strength:

- Normal strength concrete ($f_{cu} < 40MPa$)
- High strength concrete ($40MPa \leq f_{cu} \leq 60MPa$)

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- High performance concrete ($f_{cu} > 60MPa$)

Where, (f_{cu}) Stands for compressive cubic strength of concrete and only high strength concrete ($40MPa \leq f_{cu} \leq 60MPa$) literatures survey that subject matter with the research objective undertaken here.

2.3.1.2 High strength concrete

2.3.1.2.1 Introduction

High strength concrete contains stronger aggregates, high Portland cement content and lower water cement ratio than normal strength concrete. With the development of concrete technology in the past, when effort had been made to produce very high strength concrete with compressive strength approaching 200Mpa attention was given to providing well-graded mixture of coarse and fine aggregate so that the spaces between the maximum aggregate sizes would be filled with smaller particles of gravel or crushed stone, which in turn would have their spaces filled with fine aggregate or sand.

In the recent year appreciable improvement has been made in the placing, vibrating, and finishing of concrete. This improvement has resulted in lower water cement ratios and if desired, Strength increase can be made by further lowering water-cement ratio, adding admixture, and selecting good clean and solid aggregates. Therefore, it is now become normal to use concrete having high compressive strength; and the actual concrete strengths used by designer for a particular job will depend on the size of the loads and the quality of the aggregate available.

In China 13 major highway or railway bridges at strategic locations across the country have been constructed using concrete with cubic compressive strength varying 50MPa to 80MPa, mostly at 60MPa.

In Ethiopia high strength concrete use and application is limited and construction specification of concrete mostly specify to, 25MPa and 30MPa cubic strength concrete grades, although the majority of bridge construction use concrete specified to 30MPa.

Specification of concrete should be set based on performance or prescription. Some time performance and prescription in combination used. And concrete specification need to be

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scientific and drafted by professional material engineers. But the existing practice in Ethiopia will not lead to produce expected quality of concrete.

Some of Challenges in producing high strength concrete in Ethiopia [1]

- Lack of competence in righting specification due to limited guidelines and research outputs
- Nonstandard fine and coarse aggregates
- Labor intensive and old concrete production technology
- No authorized detailed guideline in high strength production in Ethiopia. Confusion, misunderstanding in selection and abuse of different international mixing standard, because these standards developed for their own specific prevailing conditions.
- Cement producer do not have sufficient knowledge of concrete production.

These challenges should be overcome and exploit the opportunity and get the full benefit of high strength concrete which includes, improve quality of concrete, increased hardening rates of slender structure; lead to complete the project on time, longer span bridge can be build and other advantages; for realization of the future Ethiopian construction development.

2.3.1.2.2 Concrete mix preparation considerations

The production of high strength concrete requires special attention to water cement ratio, gradation and mineralogy of the aggregates, composition and fineness of cement, and curing. Among many property of concrete that can specify for targeted purpose, the properties most usually specified are:

- Workability of fresh concrete
- The compressive strength at specified age
- The durability, by means of specifying the minimum cement content and/or the maximum free-water/cement ratio and, in some cases, requiring the use of selected types of materials

The mix design process must take account of those factors or parameters that have major on the characteristics of the concrete. For instance, water-cement ratio by weight is the single most important strength parameter in concrete mix.

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For each class of concrete, a minimum amount of cement specified. By increasing the cement above this minimum, it is possible to increase the water content and still obtain the same water cement ratio. This increase of water content results excess water, which is not needed for the chemical reaction with cement and for wetting the surface of the aggregate, eventually evaporates and causes excessive shrinkage and less durable concrete. Also upper limit on the denominator of water content of the mixture should be places, which may produce problems in workability and placement of mixture in the forms.

In practical concrete mixture, the overall properties of the principal components are mostly controlled by the requirements:

- When freshly mixed, the mass be workable and place able of mix in the form.
- Hardened concrete mass possesses strength and durability adequate to the purpose it is intended.
- Cost of the final product is a minimum content with acceptable quality.

The design of concrete mixes to meet the above or specific requirements, most countries have developed their own mixing standard; British and American standards, are the commonly used mix proportioning standards in Ethiopia and their basic steps listed here under.

2.3.1.2.2.1 British (DOE) Method [5]

DOE method provides the principle to drive in an attempt way to produce a concrete mix proportion having the required workability and strength from data of material property. Typical data are given in the manual but where appropriate information available related to local materials, this can be used instead [5].

The manual has also information to adjust the mix proportions and to use these for actual production or to prepare a revised trail mix.

Here are the basic stages in DOE method: -

Stage 1: - Selection of water cement ratio from curves.

Strength of concrete better related to water cement ratio and DOE method as a start point strength of concrete does not depend on the absorption characteristics of aggregate being

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In a saturated surface-dry condition assumed.

Due to variability of concrete strengths the mix concrete has “margin” difference to the specified strength of concrete. This margin is based on variability of concrete strength data expressed as a standard deviation, or substantial margin is applied if fewer number of site result obtain.

$$f_m = f_c + KS$$

Where $f_m =$ the target mean strength

$f_c =$ Specified characteristic strength

$S =$ the standard deviations

$K =$ a value appropriate to percentage defective permitted below the Characteristic strength, for instance, 5% defective level specified in BS5328, $K = 1.64$

Next, to select the appropriate compressive strength free water cement ratio curve start at water cement ratio 0.5 this value are given in table format; according to specified age, cement strength class and the aggregate to be used. This strength value then plotted on (compressive strength and free-water cement ratio relation curve), a curve drawn from this point parallel to printed curves until it intercepts horizontal line passing through ordinate representing the target mean strength. The corresponding value for water cement ratio can then be read from abscissa.

Alternative equation to water – cement ratio curve of compressive strength and free-water cement ratio relation curve:

$$f = K_1 * K_2^{w/c}$$

Where $f =$ the strength of concrete

$w/c =$ Water cement ratio

$$K_1 = [58.11 * \ln f_{0.5} - 60.47]$$

$$K_2 = \left[\frac{f_{0.5}}{540.76} - \frac{1}{6371.84} \right]$$

$f_{0.5} =$ Strength value corresponding to water cement ratio of 0.5

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Stage 2: - Estimate water content based on slump, maximum size aggregate and Shape of Aggregate

Determination of free water content from (free-water content required to give various level of workability table) depending on the type (crushed or uncrushed) and maximum size of aggregate to give a concrete of the specified slump.

Stage 3: - Calculate cement content.

$$W/C^* = \min(W/C_{curve}, W/C_{durability})$$

$$C = \text{Max}\{W/(W/C^*), C_{durability}\}$$

Where: W/C^* = minimum water cement ratio

W/C_{curve} = Water cement ratio curve value

$W/C_{durability}$ = Water cement ratio from durability requirement

$C_{durability}$ = Minimum cement content from durability requirement

W = Free water content

Stage 4: - Wet density of concrete and total aggregate content.

Fully compacted density of concrete obtained from (Estimated wet density of fully compacted concrete figure) depending on free water content and various specific gravity of combined aggregate in the saturated surface dry condition. If no information available regarding the relative density of aggregate, DOE standard [5], recommend a value of 2.6 for uncrushed aggregate and 2.7 for crushed aggregate.

$$\text{Total aggregate content} = D - C - W$$

(Saturated and surface dry)

Where D = the wet density of concrete (kg / m^3)

C = the cement content (kg / m^3)

W = the free water content (kg / m^3)

Stage 5: - Determination of fine aggregate content based on maximum size of Aggregate, Water cement ratio and grading zone of sand from curves.

(Recommended proportions of fine aggregate according to percentage passing a 600 μm sieve Figure) plot adoption of proportion of fine aggregate based on workability requirement, the grading of the fine aggregate which taken into account by the percentage

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of fine aggregate passing the $600\mu\text{m}$ test sieve and the free-water cement ratio. The best proportion of fine aggregate depend on many factor however, proportion give satisfactory concrete in the first trail mix which can then be adjusted for the exact conditions prevailing. Next the fine and coarse aggregate contents, is made using the following calculation:

*Fine aggregate content = total aggregate content * proportion of fine*

Coarse aggregate content = total aggregate content – Fine aggregate content

2.3.1.2.2 American Concrete Association (ACI) Absolute Volume method of concrete mix design [3]

Two basic type of information required to perform mix design:

- Property of material in the mix, and
- Mix design specifications (desired mix properties)

The procedures for concrete mix design presented here are taken from “Standard Practice for Selecting Proportions for Normal, Heavy weight and Mass Concrete (ACI)” published by the American Concrete Institute [3].

Step 1: - Estimation of mixing water and air content

Mass of water required to make 1 m^3 of mix is selected depending on:

- Air entrainment requirement
- Using the design range of slump. If slump is not specified, a value appropriate for the work given in the manual.
- The nominal maximum size of coarse aggregate.

Large nominal maximum sizes of well graded aggregates have less void space than smaller size aggregate; and they require less mortar per unit volume of concrete.

Other unaccounted factor such as: aggregate texture and shape, mixing water requirements may be somewhat above or below the tabulated values, but they are sufficiently accurate for the first estimate.

Next the volume of air in 1 m^3 of mix selected from the same table using the air entrainment requirement and nominal maximum size of coarse aggregate.

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Step 2:- Selection of water to cementitious ($W/(C + F)$) for the average 28 day Cylinder Compressive strength

With typical materials, the water cement ratio given on (Relationships between Water-Cement Ratio and Compressive Strength of Concrete table) to produce the average strength, f_{ck} , which is based on 28-day tests of specimens cured under standard laboratory conditions.

Step 3: - Calculate cement content from water content and water cement ratio

$$\text{Cement content} = \text{water content} / \text{water cement ratio}$$

However, the specification includes a separate minimum limit on cement in addition to requirements for strength and durability; the mixture must be based on whichever criterion leads to the larger amount of cement.

Step 4:- Estimate the coarse aggregate content for maximum size of aggregate and Fineness modulus of sand

The quantity of coarse aggregate is estimated by taking the volume from (Volume of Coarse Aggregate per unit of Volume of Concrete table), which is based on aggregates in dry-rodded conditions and multiplying it by the dry-rodded unit mass of the aggregate.

Step 5:- Estimate of Fine aggregate content

If the weight of the concrete per unit volume is assumed, or (first estimate of Mass of Fresh Concrete table) used to make a first estimate of the unit weight of concrete. The required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients.

Step 6: - Adjustment for Aggregate Moisture

The aggregate quantities actually to be weighed out for the concrete must allow for moisture in the aggregates. Generally, the aggregates will be moist and their dry weights should be increased by the percentage of water they contain, both absorbed and surface.

The mixing water added to the batch must be reduced by an amount equal to the free moisture contributed by the aggregate, i.e. total moisture minus absorption.

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2.3.1.2.3 Selected research reports on high strength concrete production

The following selected research that was conducted at Addis Ababa university institute of technology on high strength concrete production and the results reports as follow:

Tigist Workeneh (2002) [16] made investigation on the potential of locally available materials for production of high strength structural concrete. Material properties used in the investigation and test results in the scope of this study interested in are the following:

“Mugher” Ordinary Portland cements used with fine aggregates (river sand) mean loose unit Wight of 1500g/cm^3 , which blended to meet the grading requirements of the Ethiopian standard and crushed stones Coarse aggregate, with maximum size 20mm and specific gravity of 2.7. After the coarse aggregates were screened, they were blended to meet the grading requirement for coarse aggregate of Ethiopian standard.

The researcher found difficult to produce high strength concrete mixes without using chemical admixture, for water cement ratio is lowered below 0.4 and the super plasticizer Conplast SP430, which complies with ASTM C494 as type A and Type F, used with rate of 0.7 to 2.00 liters/100kg of cementitious materials according to manufacturer manual.

Mix proportions are made and Fine aggregate characterized by the percentage passing the $600\mu\text{m}$ test sieve, and 46% passing this sieve test. The expected slump of mixes is from 0 to 10mm since water to cement ratio is below 0.4 and based on the BS 882:1983 recommendation, 27% fine and 73% of coarse aggregate proportion is used throughout the investigation.

Table 2-2: Test results of concrete mix [16]

Cement type	Water cement ratio	Mix raw material type	Mix proportion quantity in (kg / m^3)	28 th day average strength (MPa)	Specified Cubic strength in (MPa)
“Mugher” Ordinary Portland cement	0.38	Cement	550	70.9	57.8
		Fine aggregate	443.10		
		Coarse aggregate	1197.9		
		Admixture	0.7		
		Water	209		
	0.38	Cement	400	78.2	65.0
		Fine aggregate	498.96		
		Coarse aggregate	1349.04		
		Admixture (%)	1.4		
		Water	152		

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Adisu Fentaw (2014) [2] study on the “use of Derba ordinary Portland cements for structural concrete production”, first the property of raw material he used for concrete mix proportion listed then, the results obtained only in the scope this research interest are presented in table format.

“Derba” ordinary Portland cement which complies with the requirement of Ethiopian standards used with Natural/river sand as fine aggregate for concrete mix production. This sand washed, sieved and it was graded to meet the Ethiopian Standard Requirement and manual recommendations.

Table 2-3: Physical property of fine and coarse aggregate [2]

Test description		Fine aggregate	Coarse aggregate
Silt Content		1.8 %	-
Moisture Content		0.7 %	1.14 %
Absorption Capacity		3.0 %	0.875 %
Unit Weight		-	1596 kg / m ³
Specific Gravity	Bulk	2.684	2.74
	Bulk (SSD)	2.695	2.76
	Apparent	2.72	2.81
Fineness Modulus		3.0 %	1.11%

In the production of trail concrete mix, for 40MPa, 50MPa and 60MPa cubic strength concrete grade; a DOE [5] concrete mix design method was followed.

Table 2-4: Test results of concrete mix [2]

Cement type	Water cement ratio	Mix raw material type	Mix proportion quantity in (kg / m ³)	28 th day average strength in (MPa)	Specified Cubic strength in (MPa)
“Derba” Ordinary Portland cement	0.47	Cement	360	55	42
		Fine aggregate	532		
		Coarse aggregate	1367		
		Water	170		
	0.42	Cement	405	64.6	51.5
		Fine aggregate	482		
		Coarse aggregate	1373		
		Water	170		

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Cement type	Water cement ratio	Mix raw material type	Mix proportion quantity in (kg / m^3)	28 th day average strength in (MPa)	Specified Cubic strength in (MPa)
“Derba” Ordinary Portland cement	0.35	cement	485	75.2	62
		Fine aggregate	426		
		Coarse aggregate	1348		
		Water	170		

These proportions serve as a basis for cost comparisons in the later chapters, hence the specified cubic strength from test result are close to C-40, C-50 and C-60 concrete grade cubic strength requirement.

2.3.2 Prestressing Steel

The most common prestressing steel is seven wire stands, which is available in stress-relieved strand and low relaxation strand with ultimate strength varying between 1570MPa and 1860MPa. Most common strand nominal size are 13mm nominal size (12.5mm or 12.9mm actual size) made of wire that are 4mm in diameter and 15mm nominal size (15.2mm or 15.7mm actual size) made of wire that are 5mm in diameter [15].

During manufacture of the strands high carbon steel rod is drawn through increase the strength of the wire to over 1700Mpa. Six wires are then wrapped around one central wire in helical manner to form a strand. The cold drawing and twisting of wires creates locked in or residual stress in the strands. These residual stresses cause the stress-strain response to be more round and to exhibit an apparently lower yield stress. The apparent yield stress can be raised by heating the strands to about 350°C and allowing them to cool slowly, through process called stress relieving [9].

The most common post-tensioned tendon sizes along with their section area to suit the standard anchor blocks available listed below:

- $A_p = 2850mm^2$ Cross-sectional area per prestressing tendon when 19 strand with $150mm^2$ section area used.
- $A_p = 1900mm^2$ Cross-sectional area per prestressing tendon when 19 strand with $100mm^2$ section area used.
- $A_p = 1200mm^2$ Cross-sectional area per prestressing tendon when 12 strand with $100mm^2$ section area used.

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$$A_p = 2700mm^2$$

Cross-sectional area per prestressing tendon when 27 strand with $100mm^2$ section area used.

Wire tendon made up of between 5mm and 7mm wire and high strength deformed bars ranging from 16mm up to 75mm in diameter are also used for prestressing steel. The bars deformations are often like raised screw threads so that devices for posttensioning and anchorage bars can attach to their ends. The ultimate strength of the bars is about 1000Mpa.

2.3.3 Anchorage and Jack stressing

The tendon anchor consists of steel fabrications or cast iron forgings which are sized to apply acceptable pressure to the concrete. A stressed anchor is known as “live”, while unstressed anchors are known as “dead”. Each strand is locked to a live anchor by hardened steel wedge that fit into conical holes. A strand of dead anchors may also be locked by wedges that are pushed home before the tendon is stressed, or by metal sleeves that are, swaged on to individual strands [15].

Another type called Buried anchors are advantageous by exploiting the bond of the concrete to the steel tendons, or by looping the individual wires or strands at unstressed end of tendons, saving in anchorage hardware and in labor may be achieved.

Jacks are used for stressing the tendons, for smaller tendons, the jacks, weighing up to 250 kg sufficient and can easily maneuvered in to position with readily available lifting equipment, while for the larger tendons special lifting frames or cranes are required to move up to 2,000kg weight jacks.

A tendon may be stressed from one end only, or from both ends. Single-end stressing is preferable, as it minimizes the cost of labor and reduces the number of jacking required on site. The decision on whether to stress from one end or two depends principally on the comparison of friction losses for one end jacking and cost for two end jacking. When tendons are less than about 50m long, they are normally single-end stressed. However, much longer cables may be single-end stressed, for instance when their alignment is very flat and friction losses are low, or when it is not practical to fit a jack to one end.

2.4 Previous Studies

R.J.A.kenter (2010) has conducted a research on behalf of engineering office of Rotterdam Public Works. The objective was to determine the dimensions and normative structural verifications of the elevated metro railway structure having an elevation of 15 meters and a span of 45-meter precast segmental box girder with external prestressing tendons made of conventional concrete, Ultra High Performance Concrete or Fiber Reinforced Polymers and to compare these designs with each other. The Result showed that the dead load difference between the three designed railway girders is quite large 102.02kN/m for concrete box girder, 69.4kN/m for UHPC box girder and 34.48kN/m for FRP sandwich girder and applying UHPC or FRP instead of conventional concrete for the railway girder has a small impact on the substructure therefore; choice between the three designs based on the construction/fabrication, costs and aesthetics of the railway girder made. The direct construction costs for the elevated metro structure with a concrete box girder are about €450,000 per span of 45 meters.

The study concluded that, when the unit price of UHPC is lower than € 450/m³, the UHPC box girder becomes a serious competitor of the conventional concrete box girder from a financial point of view depending on the market price of UHPC and the application of UHPC results in amore slender railway girder. This can also be a reason to choose for the UHPC box girder instead of the concrete box girder. For the FRP railway girder holds that FRP is currently far too expensive to compete with the (UHP) concrete box girder.

2.5 Bridge Design Consideration

2.5.1 Introduction

After the selection of possible alternative bridge types that satisfy the function and aesthetic requirement, the requirement of constructability, safety and stability come into play. The engineer should verify by calculation that all applicable specifications, design, and construction requirements are satisfied.

In engineering design, the general principle behind assuring safety is that the resistance provided by each component in a structure exceeds the applied loads induced demand.

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The evaluation of inequality (resistance greater than or equal to effect of the loads) done for specific loading and its resistance for the same condition considered. In other words, inequality evaluated for each specific loading condition that links a resistance and the effect of loads.

Different design procedures developing throughout the time, among these procedures allowable Stress design is one of these methods in the past when majority of bridge were open web trusses or arches made of metallic structure material which stress-strain curve behave linearly up to a relatively well defined yield point and fraction of this yield point stress used to specify load effect stress.

Major short coming of allowable resistance

- Resistance of material is only based on elastic behavior of material
- Provision of safety factor by limiting allowable stress instead of a reasonable measure of strength or reliability in terms of probability of failure.
- Safety factor is applied only to resistance. Loads are considered without variation

Because of these limitations Allowable stress method replaced by limit state design. Another type of design procedure is Load and resistance factor design (LRFD), in this procedure both load and resistance factors are which determined using statistics and a pre-selected probability of failure.

Advantage of load resistance factor design includes:

- Integrate both load and resistance factor to account uncertainty
- Factors developed which achieve fairly uniform safety levels can be used without involving probability or statistical analysis.

In AASHTO standard specification load and resistance factor design method adopted.

2.5.2 Railway Bridge design consideration

2.5.2.1 Introduction

The principles of designing railroad structures are similar to those for structures carrying highways. However, structure carrying railways have much heavier loading than those subjected to highway loadings due to increased live load to dead load ratio, and impact required for railways. Also significant torsional moment induced due to this heavier loading.

Almost all railroad structures are simple spans for the following main reason: [6]

- For most load combinations maximum negative moment for continuous beam is nearly equal and in some combinations even greater than the simple beam positive moment because of unfavorable live load placements in the spans.
- Continuity introduces complications and saving is not realized by its use.
- Continuous span make difficulty to replace in emergencies, since constructability and maintainability without interruption to traffic are crucial for Railroad Bridge.

Deck design in railroad Bridge is not critical as in Highway Bridge, except to have support system for rails. Also some bridges, particularly in transit applications, use direct fixation of rails to girder or other structural support.

The general futures of design, loadings, allowable stresses, etc., for railway structures in America are controlled by the specifications of the American Railway Engineering and Maintenance of Way Association (AREMA) [4]. Based on this manual different railroad company vary somewhat in their interpretation and application of specifications as stated in AREMA manual for Railway Engineering.

The AREMA manual provides for design of railroad structures using Allowable Stress Design (ASD), and Load Factor Design (LFD) methods. The AREMA manual covers all phase of design, construction, maintenance and operation. Chapter 8, concrete structures and Foundations, governs the design and construction of plain and reinforced concrete members, rigid concrete structures, retaining walls, pile foundations, substructure of railway structures, etc. AREMA manual recommends that design be based on Cooper E80 Live loading as shown in the figure 2-4.

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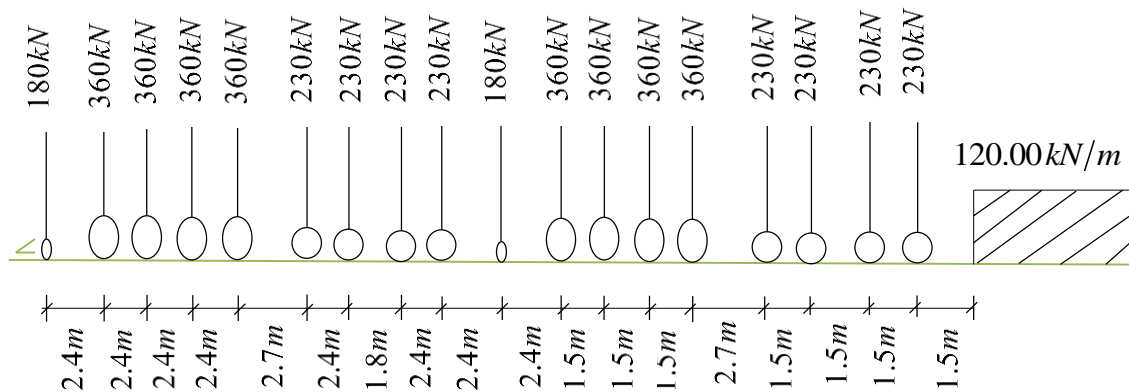


Figure 2-4: Cooper E 80(EM 360) Axle Load Diagram

For this research undertaken Loads, Load factors and Load combinations values are taken from “Standard, 04.01.02.01 for Railways of 1435mm gauges, Bridges and culverts Main requirements, Volume 2” prepared by Railway System Standard Development Consultancy Service, in collaboration with Ethiopian Railways Corporation [14].

2.5.2.2 Railway Bridge Loading

2.5.2.2.1 Dead Load

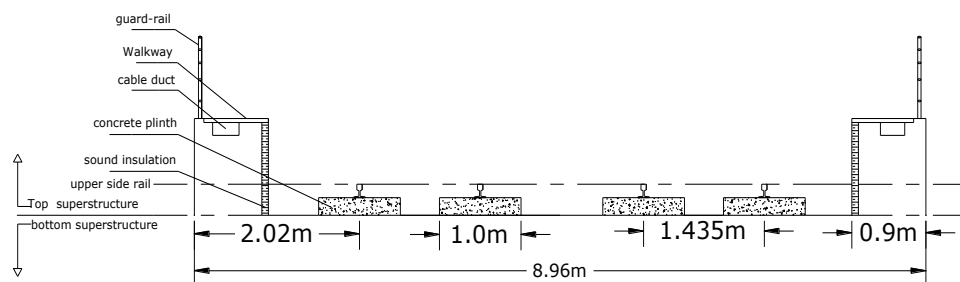


Figure 2-5: Cross Section top part superstructure

The characteristic vertical load of dead weight shall be determined according to the designed sizes of members and parts of the structure.

2.5.2.2.2 Live load

The designed characteristic vertical live load from rolling stock shall be specified as a combination of concentrated load $Q_{vk} = 220kN$ or $250kN$ and uniformly distributed load

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$q_{vk} = 92 \text{ kN/m}$ or 80 kN/m (Figure 2-6). The sections of uniformly distributed load of 80 kN/m may be of any length.

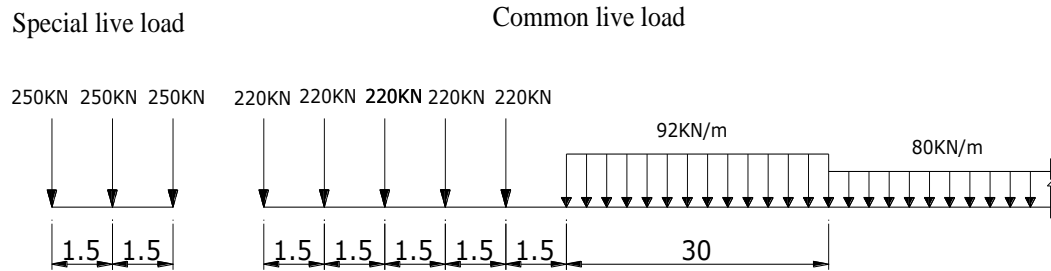


Figure 2-6: Standard Railway Live load (distance given in meter)

The design live load is located at bridge superstructure in the most unfavorable position that causes the maximum effort. For two tracks loaded case the full live load effect on both tracks used for analysis.

2.5.2.2.3 Centrifugal force

Characteristic horizontal transverse load from centrifugal force for bridges located on the curves shall be taken from each track or traffic lane. The value of the centrifugal force for $V \leq 120 \text{ km/h}$ shall be specified as follows:

$$\text{For concentrated live load } N(\text{kN}) : F = \frac{v^2}{127R} (f * N),$$

$$\text{For the distributed live loads } q(\text{kN}) : F = \frac{v^2}{127R} (f * q),$$

Where $v(\text{km/hr})$ is the design speed; $R(\text{m})$ stands for the radius of curve; f stands for the vertical live load reduction factor, calculated by:

$$f = 1.00 - \frac{v-120}{1000} \left(\frac{814}{v} + 1.75 \right) \left(1 - \sqrt{\frac{2.88}{L}} \right)$$

Where L stands for load length (m) of the curve part of the bridge and If $L \leq 2.88 \text{ m}$ or $V \leq 120 \text{ km/h}$, f is taken as 1.0; if $f > 1.0$ according to calculation, it is taken as 1.0; if $L > 150 \text{ m}$, make $L = 150 \text{ m}$ to calculate f . Centrifugal force shall be calculated at 2.0 m above the rail top in horizontal and outward acting direction.

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2.5.2.2.4 Braking force

Braking force or tractive force shall be calculated by 10% of the vertical static live load of the train. However, when calculating together with centrifugal force or vertical dynamic action of the train, the braking force or tractive force shall be calculated by 7% of the vertical static live load of the train. The braking force or tractive force shall be calculated at 2 m above acting on the rail top.

2.5.2.2.5 Wind load

Wind load acting on bridge structures and rolling stock should consider both average and pulsation components. Characteristic value of wind load W_n shall be determined as a sum of characteristic values of average W_m and pulsation W_p

Characteristic value of average W_m and pulsation W_p components of wind load shall be determined as per «Loads and actions» standard taking into account the results of meteorological observations. If the results of meteorological observations are not available then characteristic intensity of full transverse horizontal wind load shall be 0.98 kpa when the structures are not loaded with vertical live load and 0.59 kpa when there is vertical live load.

2.5.2.3 Load Factors

2.5.2.3.1 Dead load factor

Possible negative deviation from characteristic loads shall be considered by the safety factors γ_f . The safety factors γ_f for dead loads and forces are specified in Table 2-5.

Table 2-5: Safety factors for dead loads

Loads and forces	Safety factor γ_f
All loads and forces except listed below in this Table	1.1(0.9)
Weight of ballast deck bridge	1.3(0.9)
Soil horizontal pressure of the embankment weight: 1. onto the bridge piers (including abutments) 2. onto the pipe sections	1.4(0.7) 1.3(0.8)
Action of shrinkage and creep of the concrete	1.1(0.9)
Action of subsidence	1.5(0.5)

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Values γ_f indicated in Table 2-5 in brackets shall be specified when unfavorable loading combinations increase their total effect to the members of the structure.

2.5.2.3.2 Impact factor

Loads from rolling stock should be taken into account with the dynamic coefficient $1 + \mu$ for the reinforced concrete, concrete bridge superstructure and culvert, rigid frame bridge, if the earths fill thickness on the top, $h \geq 1m$ (calculated from the rail base), the vertical dynamic action is neglected. If $h < 1m$: calculated by the following formulae:

$$1 + \mu = 1 + \alpha \left(\frac{6}{30 + L} \right), \text{ Where } \alpha = 4(1 - h) \leq 2 \text{ and } L \text{ is bridge span length given in meter}$$

2.5.2.3.3 Characteristic load factor

Random deviation from characteristic loads shall be considered by the safety factors γ_f (Table 2-6)

Table 2-6: Safety factors for characteristic loads

Forces	Safety factor γ_f when designing			
	Structures of bridges depending on loading length			Links of pipes
	0	50	150 and more	
Vertical	1.30	1.15	1.10	1.30
Horizontal	1.20	1.10	1.10	1.20
Soil pressure of moving train on the sliding triangle	1.20 not depending on loading length			-

2.5.2.3.4 Summary of load factors for specific load combination

When calculating loads and actions, one shall apply safety factors for loads and dynamic Coefficients in accordance with Table 2-7.

Table 2-7: Safety factors for loads and dynamic coefficients

Limit state	Calculation type	Coefficients applied		Notes
		to all loads and actions apart from vertical live load	to vertical live load ($1 + \mu - 1.0$, if not specially specified)	
I (Ultimate)	a) all calculations except those given in points b, c, d.	γ_f	γ_f $1 + \mu$	
	b) fatigue calculation	$\gamma_f = 1.0$	$\gamma_f = 1.0$ $1 + 2/3 \mu$	

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	c) stability calculation	γ_f	γ_f	For idle stock $\gamma_f = 1.0$
	d) calculation for load combinations (including seismic load)	γ_f	γ_f	Seismic load safety factor is $\gamma_f = 1.0$
II (Service)	All calculations including those for crack openings in reinforced concrete	$\gamma_f = 1.0$	$\gamma_f = 1.0$	

2.5.2.4 Load combination

Bridge structures shall be designed for the loads and forces that are specified in accordance with (Table 2-8).

Table 2-8: Design loads

Number and type of load		Number of load										Notes
		1	2	3	4	5	6	7	8	9	10	
DEAD LOAD												
1	Dead weight of the structure											Accounted for together with all loads
2	Earth pressure from embankment weight and earth settlement											
LIVE LOAD												
3	Vertical loads									×		Not accounted for with construction Load. Design efforts from the general load combinations are Identified.
4	Earth pressure from rolling stock									×		
5	Horizontal longitudinal load from brake or traction force									×		
6	Horizontal transverse impacts of rolling stock					×		×	×	×	×	The calculation Applies the maximum transverse impact of the rolling stock
7	Horizontal transverse load from centrifugal force						×			×		The calculation applies the maximum transverse load
OTHERS												
8	Wind load						×				×	Determination of effort from wind load
9	Construction load			×	×	×	×	×			×	Searching for the design effort during construction period
10	Earthquake load						×		×	×		Identification of seismic impact

Note: Load combinations marked by «×» are excluded from calculations

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When designing the bridges, load combination coefficient $\eta \leq 1$ that considers decrease of probability for simultaneous occurrence of design loads should be used (Table 2-9).

Table 2-9: Load combination coefficients for live loads and actions

Numbers of loads that are the most unfavorable for the calculation	Numbers of load Combinations acting simultaneously with or separately from the most unfavorable ones	Coefficient η for various combinations of live loads and actions							
		3	4	5	6	7	8	9	10
3 and 4	7	1	1	-	-	1	-	-	-
	6	1	1	-	1	-	-	-	-
	5, 7, 8	0.8	0.8	0.7	-	0.8	0.5	-	-
	7-8	0.8	0.8	-	-	0.8	0.5	-	-
3 and 4	6	0.8	0.8	-	0.7	-	-	-	-
	5, 8	0.8	0.8	0.7	-	-	0.5	-	-
	8	0.8	0.8	-	-	-	0.5	-	-
5	3-4, 7-8	0.8	0.8	0.8	-	0.8	0.5	-	-
6	9, 12	0.7	0.7	-	0.8	-	-	-	-
7	5, 8	0.8	0.8	0.7	-	0.8	0.5	-	-
	8	0.8	0.8	-	-	0.8	0.5	-	-
8	3-4, 7	0.7	0.7	-	-	0.7	0.5	-	-
	3-5	0.7	0.7	0.7	-	-	0.5	-	-
	3-4, 7	0.7	0.7	-	-	0.7	0.5	-	-
	9	-	-	-	-	-	0.8	1	-
9	8	-	-	-	-	-	0.7	1	-
10	3-5, 7	0.7	0.7	0.7	-	0.7	-	-	0.8

2.5.2.5 Design requirements

2.5.2.5.1 General

All analysis cases are designed to satisfy ultimate limit state (ULS) and service limit state (SLS) requirements by specifying the amount of prestressing tendons, longitudinal reinforcement and stirrups. In carrying out the analysis CSI Bridge Software [7] implemented. The ULS design consists of flexural, shears and torsional strength requirements, while the SLS check includes stress, and deflection limitations.

In the analysis and design main consideration is given to longitudinal behavior and only transverse analysis moment for deck slab result given, based on the assumption that transverse reinforcement demands remain constant. For instance, in the deck slab most of

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reinforcements are minimum provisions; and this assumption effect on the comparative study is insignificant.

2.5.2.5.2 Structural slab design consideration

General structure of slab shall follow the rules in Table 2-10

Table 2-10: General structure of slab

Items	Ballast slab	Sidewalk slab
	Minimum diameter of forcing reinforcement (mm)	10
Maximum spacing of forcing reinforcement (mm)	200	200
Number of forcing reinforcement bars that stretching to the support points	Not less than 3 bars, 1/4 of the area of reinforcement within the slab	–
Minimum diameter of distributing reinforcement (mm)	8	6
Maximum spacing of distributing reinforcement (mm)	300	–

2.5.2.5.3 Ultimate Limit States

In ULS, the flexural resistance is a pure couple between compression in the concrete and tension in the longitudinal prestressing tendons. The analysis assumes that other non-prestressed reinforcements do not contribute to flexural strength and that cracked concrete has no tensile strength. Also, an equivalent rectangular concrete stress distribution is used to determine this section capacity.

Shear resistance comprises of three components: concrete, prestressing tendons, and stirrups. Concrete shear resistance depends on the cross-section while the prestressing shear resistance depends on the vertical component of prestressing force which is determined by flexural requirements. Therefore, the only independent variable that can increase shear strength is the amount of stirrups designed at ultimate state.

2.5.2.5.4 Serviceability Limit States

This section describes the serviceability limit states design requirements which comprise of stress and deflection limitations.

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2.5.2.5.4.1 Stress

For service limit state stress checks, at some interval of girder section and at different loading stage done. And partial prestressing is used for each concrete girder design.

2.5.2.5.4.2 Deflection

The maximum vertical deflection generated by the static live load (i.e. without the consideration of vertical dynamic force of train) shall comply with the following regulations [14]:

- For simply supported beam, the maximum vertical deflection shall not surpass $1/800$ of the span.
- For the side span of continuous beam, it shall not surpass $1/800$ of the span, and the mid span shall not exceed $1/700$ of the span

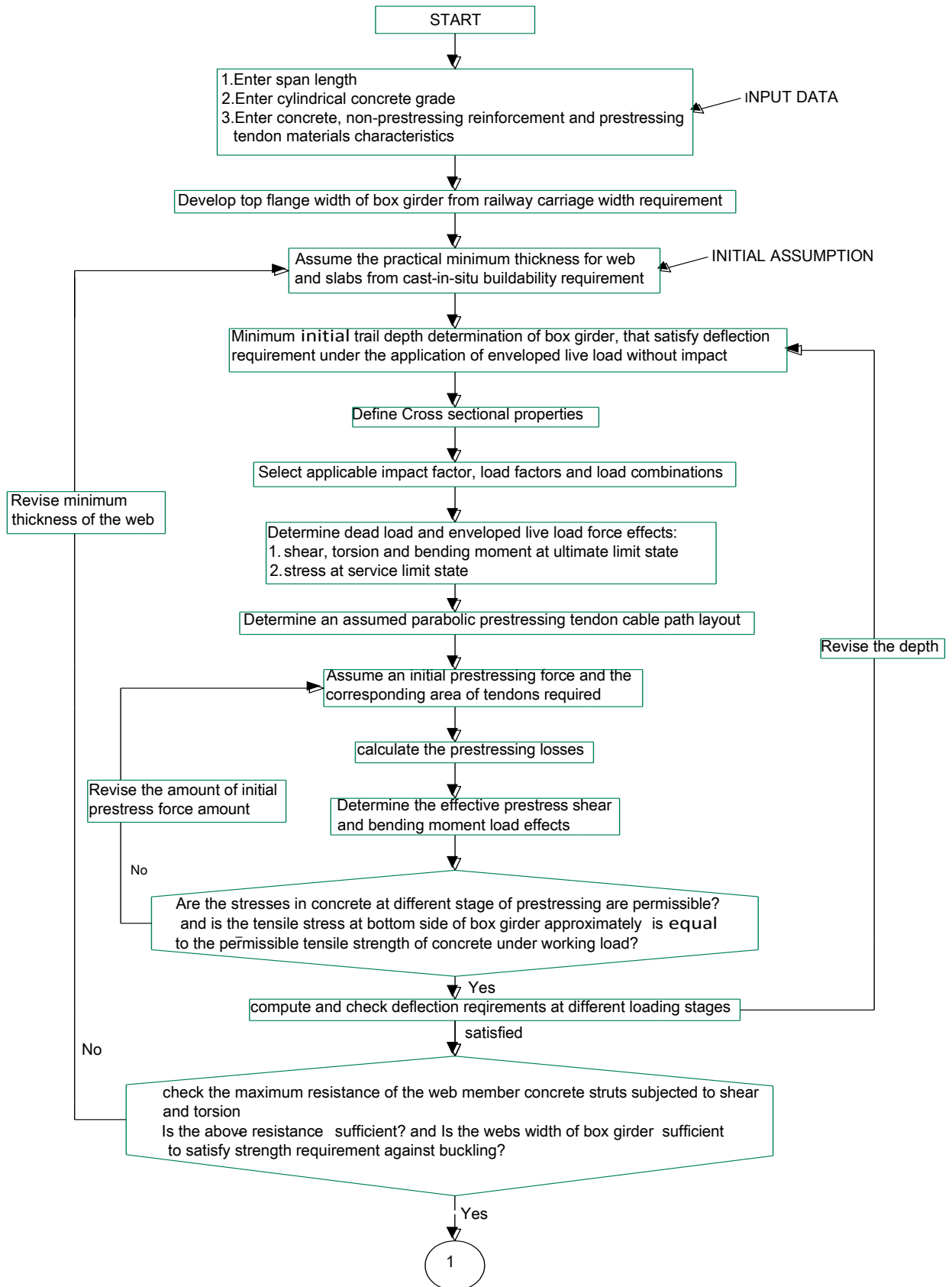
2.5.2.6 Preliminary design main assumptions

Main analysis assumptions include:

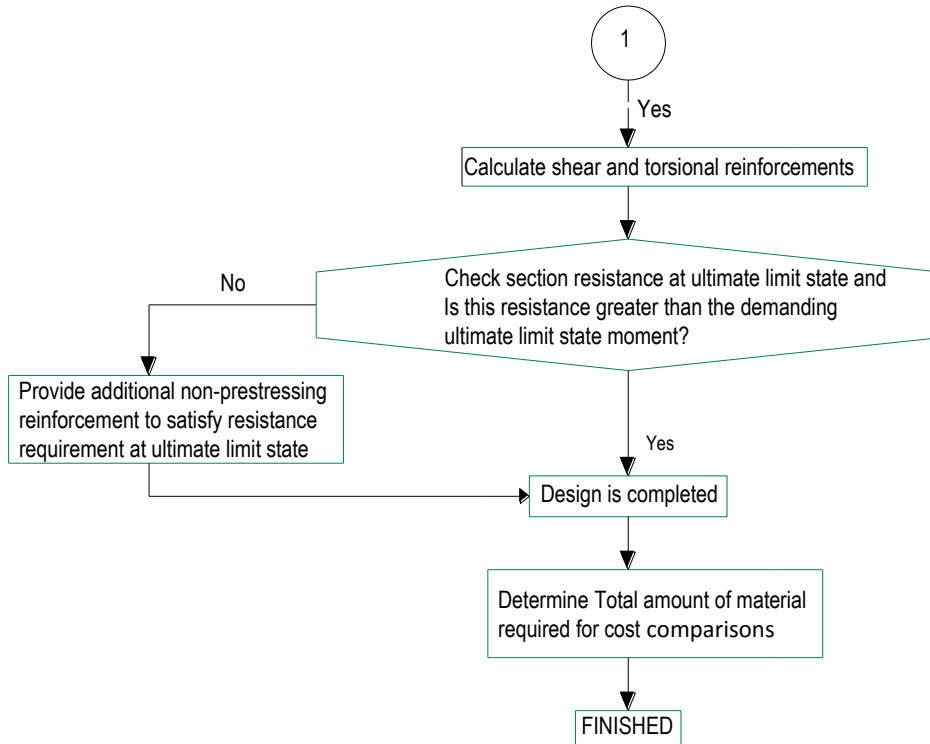
- Tendon eccentricity in duct is neglected. The center of gravity of tendons is assumed to be at the centroid of the duct.
- The full width of the web considered as effective for shear resistance calculation
- Substructure design is not considered in the analysis.
- The following loadings are not considered in preliminary design: Earth Pressure, Buoyancy, wind Force on the Structure, Wind Force on Live Load, Longitudinal Force, Earth quake, Stream Flow Pressure and ice Pressure.
- One end prestressing for all span length considered.

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2.5.2.7 Preliminary Design flow chart



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**CHAPTER 3 STATICAL CALCULATIONS FOR CONCRETE
BOX GIRDER BRIDGE**

3.1 Calculations Concrete Box Girder C-30

3.1.1 Introduction

This sub chapter presents the calculation of optimal depth box girder in concrete C-30 for sample span length of 30m, also analysis result other than this sample span length presented in sections of the latter chapter and appendix. First the material characteristics of the concrete and steel are described. Part 3.1.3 deals with the geometry and the structural schematization of the box girder and its characteristics. General load factor for specified sample span length, loads and load effects to which the sample box girder is subjected are treated in part 3.1.4.4. Next for sample calculation the layout of the prestressing tendons is shown and the stresses in the box girder due to external loading and prestressing are calculated. It also contains the calculations of the prestressing losses. Furthermore, this sample calculation describes the calculations on deflection, shear + torsion and the ultimate resistance moment of the sample box girder. The formulas and values used in the calculations are taken from [12] and other references, which are then stated in the text.

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3.1.2 Material Characteristics

3.1.2.1 Concrete C30

Density of Concrete	ρ_c	2500 kg/m ³
Partial factor for concrete	γ_c	1.5
Coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied	α_{cc}	0.85
Compressive cylinder strength of concrete at 28 days.	f_{ck}	30 N/mm ²
Modulus of elasticity of Concrete	E_c	3.2*10 ⁴ N/mm ²
Design Tensile strength of concrete	$f_{ctd} = f_{ctk,0.05} / \gamma_c$	1.33 N/mm ²

3.1.2.2 Reinforcing steel FeB 460

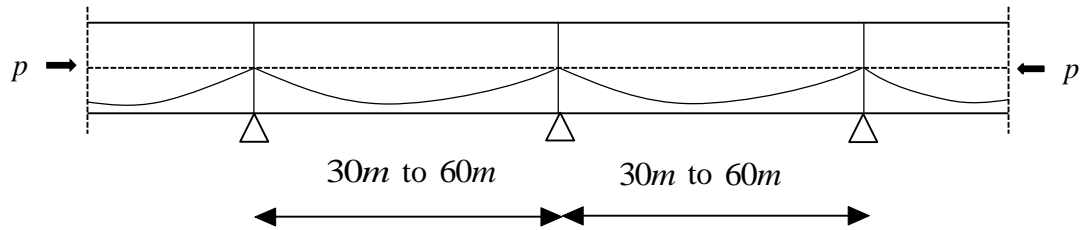
Density of reinforcing steel	ρ_s	7850 kg/m ³
Characteristic yield strength	f_{yk}	460 N/mm ²
Partial factor for reinforcing steel	γ_s	1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_s$	400 N/mm ²
Design value of modulus of elasticity of reinforcing steel	E_s	200,000 N/mm ²

3.1.2.3 Prestressing steel FeP 1860

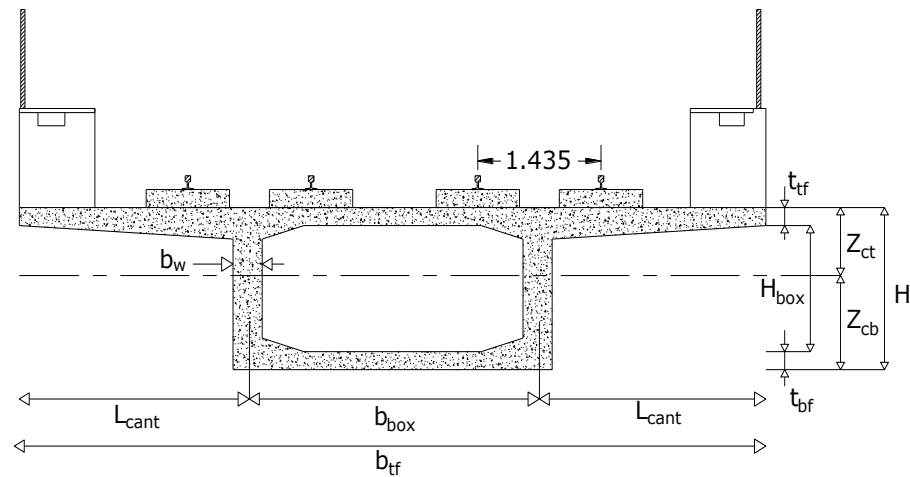
Density of prestressing steel	ρ_s	7850 kg/m ³
Characteristic tensile strength of prestressing steel	f_{pk}	1860 N/mm ²
Partial factor for prestressing steel	γ_s	1.15
Characteristic 0.1% proof-stress of prestressing steel	$f_{p0.1k}$	1600 N/mm ²
Design tensile strength of prestressing steel	$f_{pd} = f_{p0.1k} / \gamma_s$	1391 N/mm ²
Ultimate tensile strength of prestressing steel	f_{pk} / γ_s	1617 N/mm ²
Design value of modulus of elasticity of prestressing steel	E_p	195,000N/mm ²
Maximum tensile stress in the tendon (during tensioning)	min(0.8* f_{pk} or 0.9* $f_{p0.1k}$)	1440 N/mm ²
Maximum tensile stress in the tendon (Immediately after tensioning)	min(0.75* f_{pk} or 0.85* f_{pk})	1360 N/mm ²
Working stress after all losses	$\leq 0.6* f_{pk}$	1116 m ²

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3.1.3 Geometry box girder



Statically determinate box girders supported by columns



Cross-section of the box girder

3.1.3.1 General

Length span	L	Variable from 30m to 60m
Depth box girder	H	Design Variable
Center to center girder spacing	b_{box}	3.83m
Thickness top flange	t_{tf}	0.2m
Width web	b_w	0.35m minimum value
Thickness bottom flange	t_{bf}	0.2m
Cantilever length top flange	L_{cant}	2.74m
Width top flange	b_{tf}	8.96m

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3.1.3.2 Concrete cover

The concrete cover for this box girder made of C-30 is:

$$C_{nom} = C_{min} + \Delta C_{dev} = 50mm \text{ Where:}$$

$$\text{Minimum cover } C_{min} = \max \{ C_{min,b}; C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add}; 10mm \} = 45mm$$

$C_{min,b}$ Minimum cover due to bond requirement;

Minimum cover = diameter of bar = 16mm, $C_{min,dur}$ Minimum cover due to environmental conditions; It is assumed that exposure class XF1 and XF3 can be classified as exposure class XD1 for the determination of the concrete cover. With the recommended structural class is S4, exposure class XD1, a design working life of 100 years, class C-30, consideration for slab and an ensured special quality control of the concrete production. Therefor for slab class S4 $C_{min,dur} = 35 \text{ mm}$ and for girder S5 $C_{min,dur} = 40mm$ but, for consistency use the same cover for slab and girder.

$\Delta C_{dur,\gamma}$ Additive safety element recommended value is 0mm. $\Delta C_{dur,st}$ Reduction for use of stainless steel recommended value is 0 mm. $\Delta C_{dur,add}$ reduction for use of additional protection recommended value is 0 mm. ΔC_{dev} allowance for deviation recommended value is 10mm. for consistency the same cover for slab and girder assumed.

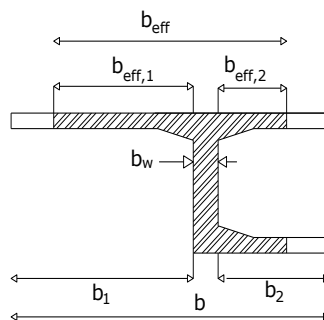
3.1.4 Sample calculation

3.1.4.1 Geometric property

Length span	L	30	m
Depth of girder	H	1.8	m
Width web	b_w	0.35	m

3.1.4.2 Effective width of flanges

The effective width of the flanges is based on the distance l_0 between pointes of zero moment, for simple beam l_0 is simply the span length 30 meters.



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$$b_{eff} \leq \left\{ \begin{array}{l} \sum b_{eff,i} + b_w \\ b \end{array} \right\} \quad \text{Where: } b_{eff,i} \leq \left\{ \begin{array}{l} 0.2b_i + 0.1l_0 \\ 0.2l_0 \\ b_i \end{array} \right\}$$

Effective width of flanges		Value	
Effective width cantilever length top flange	$b_{eff,1}$	2.56	<i>m</i>
Effective width inner top flange	$b_{eff,2}$	1.56	<i>m</i>
Effective width bottom flange	$b_{eff,3}$	1.56	<i>m</i>
Total effective flange width		Value	
Effective width top flange	$b_{eff,t}$	8.96	<i>m</i>
Effective width bottom flange	$b_{eff,b}$	3.83	<i>m</i>

3.1.4.3 Cross-sectional properties

Cross-sectional property of concrete

$$A_c = b_{tf} * t_{tf} + 2 * b_w * H_{box} + b_{box} * t_{bf}$$

Distance from bottom to centroid axis

$$Z_{cb} = \left(b_{tf} * t_{tf} * \left(H - t_{tf} / 2 \right) + 2 * b_w * H_{box} * \left(H_{box} / 2 + t_{bf} \right) + b_{box} * t_{bf} * \left(t_{tf} / 2 \right) \right) / A_c$$

Distance from top to centroid axis

$$Z_{ct} = H - Z_{cb}$$

Moment of inertia of concrete section

$$I_c = \frac{1}{12} * b_{eff,t} * t_{tf}^3 + b_{eff,t} * t_{tf} * \left(Z_{ct} - t_{tf} / 2 \right)^2 + 2 * \frac{1}{12} * b_w * H_{box}^3 + 2 * b_w * H_{box} * \left(Z_{cb} - H_{box} / 2 - t_{bf} \right)^2 + \frac{1}{12} * b_{eff,b} * t_{bf}^3 + b_{eff,b} * t_{bf} * \left(Z_{cb} - t_{bf} / 2 \right)^2$$

Section modulus bottom

$$W_b = I_c / Z_{cb}$$

Section modulus top

$$W_t = I_c / Z_{ct}$$

Perimeter concrete box girder

$$u = b_f + 2 * t_{tf} + 2 * L_{cant} + 2 * H_{box} + b_{box}$$

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Values cross-sectional properties box girder		Value	
Cross-sectional area of concrete	A_c	4.073	m^2
Distance from bottom to centroid axis	Z_{cb}	1.163	m
Distance from top to centroid axis	Z_{ct}	0.637	m
Second moment of area of the concrete section	I_c	1.75	m^4
Section modulus bottom	W_b	1.505	m^3
Section modulus top	W_t	2.748	m^3
Perimeter concrete box girder	u	21.27	m

3.1.4.4 Loads

3.1.4.4.1 General

Acceleration due to gravity	g	9.81	m/s^2
Dynamic factor	$1 + \mu = 1 + \alpha(6/30 + L)$	1.2	
Safety factor for Wight of deck, favorable	$\gamma_{p,fav}$	0.9	
Safety factor for Wight of deck, unfavorable	$\gamma_{p,unfav}$	1.3	
Safety factor for prestress force, favorable	$\gamma_{p,fav}$	1.1	
Safety factor for prestress force, unfavorable	$\gamma_{p,unfav}$	0.9	
Safety factor for variable actions, favorable	$\gamma_{f,fav}$	0.0	
Safety factor for vertical variable actions, unfavorable	$\gamma_{f,unfav}$	1.21	

3.1.4.4.2 Load schematization in longitudinal direction

Vertical loads

Permanent loads			
Dead load box girder	$g_{dead} = A_c * \rho_c * g$	101.82	kN/m
Concrete plinths		10.0	kN/m per track
Rail		0.97	kN/m per track
Cables		1.2	kN/m per cable duct
Walkway + guard-rail		2	kN/m per walkway
Sound insulation		1.3	kN/m per walkway
Concrete slope (drainage between walkways)		0.5	kN/m^2
Variable loads			
Common live load	Shown in (figure 2-6)		One and two track loaded envelop
Special live load	Shown in (figure 2-6)		One and Two track loaded envelop

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Maximum Bending moment due to standard live load

Station (m)	Common live load Moment one track loaded in (MNm)	Common live load Moment two track loaded in (MNm)	Special live load Moment one track loaded in (MNm)	Special live load Moment two track loaded in (MNm)	Enveloped standard live load Moment in (MNm)
0.0	0.00	0.00	0.00	0.00	0.00
1.0	1.99	3.97	0.82	1.64	3.97
2.0	3.72	7.44	1.58	3.16	7.44
3.0	5.34	10.68	2.27	4.54	10.68
4.0	6.73	13.46	2.89	5.77	13.46
5.0	7.97	15.93	3.44	6.88	15.93
6.0	8.99	17.99	3.91	7.82	17.99
7.0	9.85	19.70	4.33	8.65	19.70
8.0	10.60	21.20	4.68	9.36	21.20
9.0	11.15	22.30	4.97	9.95	22.30
10.0	11.61	23.22	5.19	10.38	23.22
11.0	11.91	23.83	5.42	10.85	23.83
12.0	12.08	24.16	5.59	11.18	24.16
13.0	12.17	24.35	5.71	11.38	24.35
14.0	12.23	24.46	5.77	11.55	24.46
15.0	12.19	24.39	5.79	11.58	24.39
16.0	12.23	24.46	5.77	11.55	24.46
17.0	12.17	24.35	5.71	11.42	24.35
18.0	12.08	24.16	5.59	11.18	24.16
19.0	11.91	23.83	5.42	10.85	23.83
20.0	11.61	23.22	5.19	10.38	23.22
21.0	11.15	22.30	4.97	9.95	22.30
22.0	10.60	21.20	4.68	9.37	21.20
23.0	9.85	19.70	4.33	8.65	19.70
24.0	8.99	17.99	3.91	7.82	17.99
25.0	7.97	15.93	3.44	6.88	15.93
26.0	6.73	13.45	2.88	5.77	13.45
27.0	5.34	10.68	2.27	4.54	10.68
28.0	3.72	7.44	1.58	3.16	7.44
29.0	1.99	3.97	0.82	1.64	3.97
30.0	0.00	0.00	0.00	0.00	0.00

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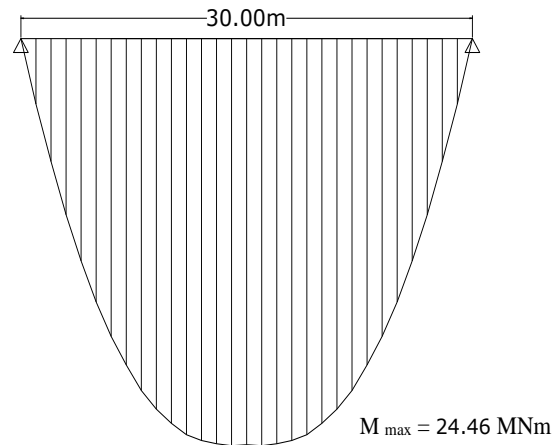


Figure 3-1: Moment Envelop line due to standard live load for 30m span

Service and ultimate state bending moment

Station (m)	Standard live load Enveloped moment in (MNm)	Utility and attachment moment in (MNm)	Box girder dead load Moment in (MNm)	External load Service limit state moment in (MNm)	External load Ultimate limit state moment in (MNm)
0.0	0.00	0.00	0.00	0.00	0.00
1.0	3.97	0.50	1.56	6.03	7.48
2.0	7.44	0.96	3.00	11.40	14.15
3.0	10.68	1.38	4.31	16.37	20.32
4.0	13.46	1.75	5.50	20.71	25.71
5.0	15.93	2.09	6.56	24.58	30.52
6.0	17.99	2.38	7.47	27.84	34.57
7.0	19.70	2.64	8.28	30.62	38.03
8.0	21.20	2.86	8.97	33.03	41.03
9.0	22.30	3.05	9.55	34.90	43.36
10.0	23.22	3.21	10.05	36.48	45.33
11.0	23.83	3.33	10.42	37.58	46.71
12.0	24.16	3.42	10.70	38.28	47.59
13.0	24.35	3.48	10.90	38.73	48.16
14.0	24.46	3.52	11.02	39.00	48.50
15.0	24.39	3.52	11.04	38.95	48.44
16.0	24.46	3.52	11.02	39.00	48.50
17.0	24.35	3.48	10.90	38.73	48.16
18.0	24.16	3.42	10.70	38.28	47.59
19.0	23.83	3.33	10.42	37.58	46.71
20.0	23.22	3.21	10.05	36.48	45.33
21.0	22.30	3.05	9.55	34.90	43.36
22.0	21.20	2.86	8.97	33.03	41.03
23.0	19.70	2.64	8.28	30.62	38.03
24.0	17.99	2.38	7.47	27.84	34.57
25.0	15.93	2.09	6.56	24.58	30.52
26.0	13.45	1.75	5.50	20.70	25.70
27.0	10.68	1.38	4.31	16.37	20.32
28.0	7.44	0.96	3.00	11.40	14.15
29.0	3.97	0.50	1.56	6.03	7.48
30.0	0.00	0.00	0.00	0.00	0.00

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Maximum moment Service stress

	SLS moment <i>MNm</i>	Top fiber stress (MPa)	Bottom fiber stress (MPa)
Box girder dead load	11.02	+4.3	-7.07
Utility and attachment dead load	3.52	+1.36	-2.26
Common live load one track loaded	12.23		
Common live load two track loaded	24.46		
Special rail live load one track loaded	5.80		
Special rail live load two track loaded	11.58		
Standard live load Envelop	24.46	+9.46	-15.68
Total	39.00	+15.12	-25.01

Maximum shear per girder due to Standard live load

Station (m)	Common live load Shear one track loaded in (kN)	Common live load Shear two track loaded in (kN)	Special live load Shear one track loaded in (kN)	Special live load Shear two track loaded in (kN)	Enveloped standard live load Shear in (kN)
0.0	1846.59	2021.73	757.36	852.78	2021.73
1.0	1846.59	2021.73	757.36	852.78	2021.73
2.0	1626.55	1876.38	695.54	820.67	1876.38
3.0	1433.63	1742.30	636.45	782.98	1742.30
4.0	1279.01	1619.49	586.82	744.14	1619.49
5.0	1157.28	1506.29	547.37	706.68	1506.29
6.0	1057.74	1395.01	515.27	669.64	1395.01
7.0	977.66	1291.98	490.62	636.22	1291.98
8.0	908.76	1191.17	470.45	604.42	1191.17
9.0	848.63	1094.28	453.23	573.77	1094.28
10.0	796.84	1007.33	437.32	543.27	1007.33
11.0	748.49	924.42	425.18	517.30	924.42
12.0	702.09	841.91	412.38	490.10	841.91
13.0	658.14	764.16	400.05	463.66	764.16
14.0	615.74	693.84	387.71	437.44	693.84
15.0	574.37	627.28	375.33	411.19	627.28
15.0	-574.37	-627.28	-375.33	-411.19	-627.28
16.0	-615.74	-693.84	-387.71	-437.44	-693.84
17.0	-658.14	-764.16	-400.05	-463.66	-764.16
18.0	-702.09	-841.91	-412.38	-490.10	-841.91
19.0	-748.49	-924.42	-425.18	-517.30	-924.42
20.0	-796.84	-1007.33	-437.32	-543.27	-1007.33
21.0	-848.63	-1094.28	-453.23	-573.77	-1094.28
22.0	-908.76	-1191.17	-470.45	-604.42	-1191.17
23.0	-977.66	-1291.98	-490.62	-636.22	-1291.98
24.0	-1057.74	-1395.01	-515.27	-669.64	-1395.01
25.0	-1157.28	-1506.29	-547.37	-706.68	-1506.29
26.0	-1279.01	-1619.49	-586.82	-744.14	-1619.49

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

27.0	-1433.62	-1742.28	-636.45	-782.98	-1742.28
28.0	-1626.55	-1876.38	-695.54	-820.67	-1876.38
29.0	-1846.59	-2021.73	-757.36	-852.78	-2021.73
30.0	-1846.59	-2021.73	-757.36	-852.78	-2021.73

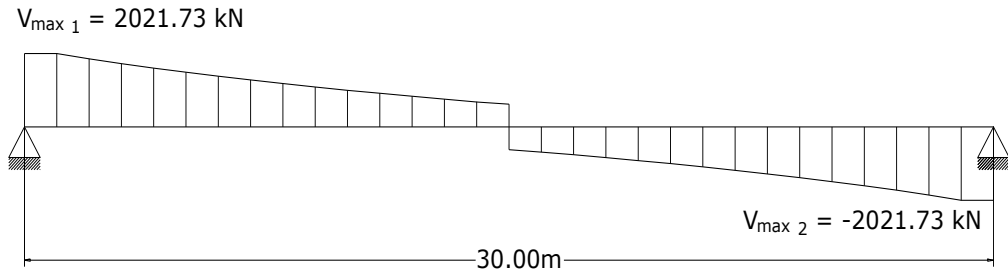


Figure 3-2: Shear Envelop line per girder due to Standard live load for 30m span

Ultimate limit state external load shear force per girder

Station (m)	Enveloped standard live load Shear in (kN)	Utility and attachment Shear in (kN)	Box girder dead load shear in (kN)	External load Ultimate limit state Shear in (kN)
0.0	2021.73	248.08	803.50	3813.35
1.0	2021.73	248.08	750.68	3744.68
2.0	1876.38	230.68	696.21	3475.38
3.0	1742.30	212.49	639.29	3215.50
4.0	1619.49	194.06	581.61	2967.95
5.0	1506.29	175.89	524.78	2733.48
6.0	1395.01	157.44	466.70	2499.34
7.0	1291.98	139.67	411.10	2279.30
8.0	1191.17	122.06	356.00	2062.79
9.0	1094.28	104.78	302.37	1853.37
10.0	1007.33	88.04	251.25	1659.95
11.0	924.42	71.81	200.13	1472.07
12.0	841.91	55.47	149.80	1285.56
13.0	764.16	39.38	99.91	1105.71
14.0	693.84	23.44	50.10	935.15
15.0	627.28	7.52	2.72	772.32
15.0	-627.28	-7.52	-2.72	-772.32
16.0	-693.84	-23.44	-50.10	-935.15
17.0	-764.16	-39.38	-99.91	-1105.71
18.0	-841.91	-55.47	-149.80	-1285.56
19.0	-924.42	-71.81	-200.13	-1472.07
20.0	-1007.33	-88.04	-251.25	-1659.95
21.0	-1094.28	-104.78	-302.37	-1853.37
22.0	-1191.17	-122.06	-356.00	-2062.79
23.0	-1291.98	-139.67	-411.10	-2279.30
24.0	-1395.01	-157.44	-466.70	-2499.34
25.0	-1506.29	-175.89	-524.78	-2733.48
26.0	-1619.49	-194.06	-581.61	-2967.95

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

27.0	-1742.28	-212.49	-639.29	-3215.47
28.0	-1876.38	-230.68	-696.21	-3475.38
29.0	-2021.73	-248.08	-750.68	-3744.68
30.0	-2021.73	-248.08	-803.50	-3813.35

Ultimate State Maximum Shear

	shear force (kN)	Load Factors	ULS shear force (kN)
Box girder dead load	803.50	1.3	1044.55
Utility and attachment dead load	248.08	1.3	322.50
Common live one track loaded	1846.59		
Common live two track loaded	2021.73		
Special railway live one track loaded	757.36		
Special railway live two track loaded	852.78		
Standard live load Envelop	2021.73	1.21	2446.3
Total	3073.31		3813.35

Maximum Torsion due to rail live load

Station (m)	Common live load torsion one track loaded (kNm) ±	Common live load torsion two track loaded (kNm) ±	Special live load torsion one track loaded (kNm) ±	Special live load torsion two track loaded (kNm) ±	Enveloped standard live load torsion (kNm) ±
0.0	3378.26	3387.93	1285.55	1297.18	3387.93
1.0	3378.26	3387.93	1285.55	1297.18	3387.93
2.0	3163.38	3203.90	1201.46	1247.59	3203.90
3.0	2959.47	3056.91	1125.35	1252.39	3056.91
4.0	2773.24	2941.66	1060.78	1293.35	2941.66
5.0	2607.53	2858.99	1008.13	1333.16	2858.99
6.0	2456.99	2807.26	964.60	1369.77	2807.26
7.0	2317.89	2771.52	929.58	1399.82	2771.52
8.0	2185.27	2747.99	899.73	1423.65	2747.99
9.0	2058.84	2722.43	873.70	1440.49	2722.43
10.0	1940.20	2705.14	849.72	1456.43	2705.14
11.0	1823.03	2685.75	829.69	1467.42	2685.75
12.0	1707.86	2670.97	809.82	1475.34	2670.97
13.0	1595.50	2660.03	790.55	1480.10	2660.03
14.0	1485.21	2652.12	771.17	1481.33	2652.12
15.0	1376.20	2646.32	750.80	1481.33	2646.32
16.0	1485.21	2652.12	771.17	1481.33	2652.12
17.0	1595.50	2660.03	790.55	1480.10	2660.03
18.0	1707.86	2670.97	809.82	1475.34	2670.97
19.0	1823.03	2685.75	829.69	1467.42	2685.75
20.0	1940.20	2705.14	849.72	1456.43	2705.14
21.0	2058.84	2722.43	873.70	1440.49	2722.43

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22.0	2185.27	2747.99	899.73	1423.65	2747.99
23.0	2317.89	2771.52	929.58	1399.82	2771.52
24.0	2456.99	2807.26	964.60	1369.77	2807.26
25.0	2607.53	2858.99	1008.13	1333.16	2858.99
26.0	2773.24	2941.66	1060.78	1293.35	2941.66
27.0	2959.47	3056.91	1125.35	1252.39	3056.91
28.0	3163.38	3203.90	1201.46	1247.59	3203.90
29.0	3378.26	3387.93	1285.55	1297.18	3387.93
30.0	3378.26	3387.93	1285.55	1297.18	3387.93

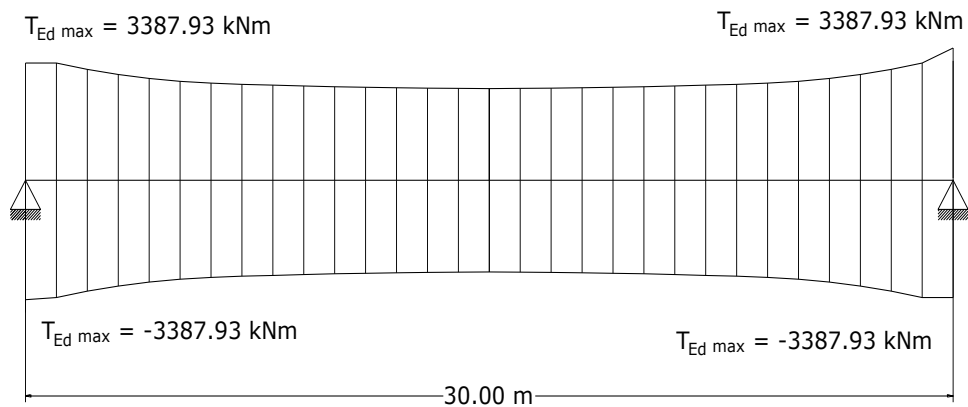


Figure 3-3: Torsion envelop line due to Standard live load for 30m span

Ultimate limit state external load Torsion moment

Station (m)	Enveloped standard live load Torsion (kNm)	External load Service limit state Torsion (SLS)(kNm)	External load Ultimate limit state Torsion (ULS)(kNm)
0.0	3387.93	3387.93	4099.40
1.0	3387.93	3387.93	4099.40
2.0	3203.90	3203.90	3876.72
3.0	3056.91	3056.91	3698.86
4.0	2941.66	2941.66	3559.41
5.0	2858.99	2858.99	3459.38
6.0	2807.26	2807.26	3396.78
7.0	2771.52	2771.52	3353.54
8.0	2747.99	2747.99	3325.07
9.0	2722.43	2722.43	3294.14
10.0	2705.14	2705.14	3273.22
11.0	2685.75	2685.75	3249.76
12.0	2670.97	2670.97	3231.87
13.0	2660.03	2660.03	3218.64
14.0	2652.12	2652.12	3209.07
15.0	2646.32	2646.32	3202.05
16.0	2652.12	2652.12	3209.07
17.0	2660.03	2660.03	3218.64
18.0	2670.97	2670.97	3231.87
19.0	2685.75	2685.75	3249.76

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20.0	2705.14	2705.14	3273.22
21.0	2722.43	2722.43	3294.14
22.0	2747.99	2747.99	3325.07
23.0	2771.52	2771.52	3353.54
24.0	2807.26	2807.26	3396.78
25.0	2858.99	2858.99	3459.38
26.0	2941.66	2941.66	3559.41
27.0	3056.91	3056.91	3698.86
28.0	3203.90	3203.90	3876.72
29.0	3387.93	3387.93	4099.40
30.0	3387.93	3387.93	4099.40

Ultimate state Maximum Torsion moment

	SLS Torsion moment <i>kNm</i>	Load Factors	ULS Torsion moment <i>(kNm)</i>
Common live one lane loaded	3378.26		
Common live two lane loaded	3387.93		
Special railway live one lane loaded	1285.55		
Special railway live two lane loaded	1481.33		
Standard live load Envelop	3387.93	1.21	4099.40
Total	3387.93		4099.40

3.1.4.4.3 Approximate Load schematization in transversal direction

Vertical load

Permanent loads			
Concrete plinths	(/2 plinths per track/1m (width plinth))	5	<i>kN/m</i>
Rail	(/ 2 rails per track)	0.485	<i>kN</i> per rail
Cables		1.2	<i>kN</i> per cable duct
Walkway + guard-rail	(/ 1 m (width walkway))	2	<i>kN/m</i>
Sound insulation		1.3	<i>kN</i> per walkway
Concrete slope (drainage between walkways)		0.5	<i>kN/m</i>
Variable loads			
Concentrated load due to Live (without dynamic factor)	Q_{mob}	250	<i>kN</i> per track
Concentrated load due to Live (with dynamic factor)	$Q_{mob} * (1 + \mu) / 2$	150	<i>kN</i> per rail

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

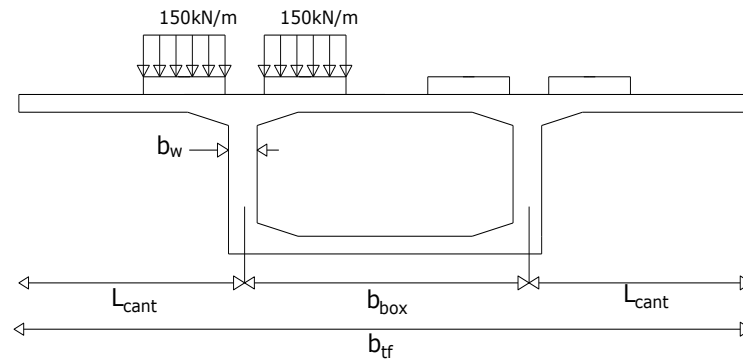


Figure 3-4: Rail live load on the left track of the deck for 30m span

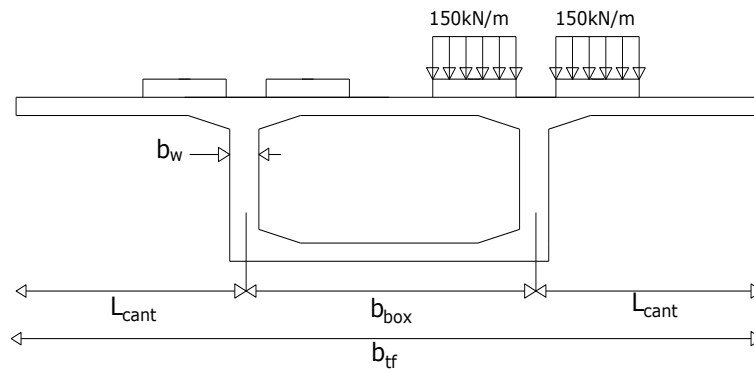


Figure 3-5: Rail live load on the right track of the deck for 30m span

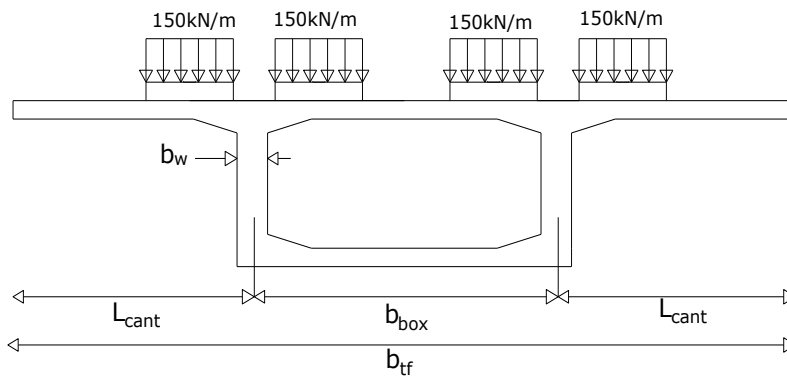


Figure 3-6: Rail load on the both track of the deck for 30m span

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

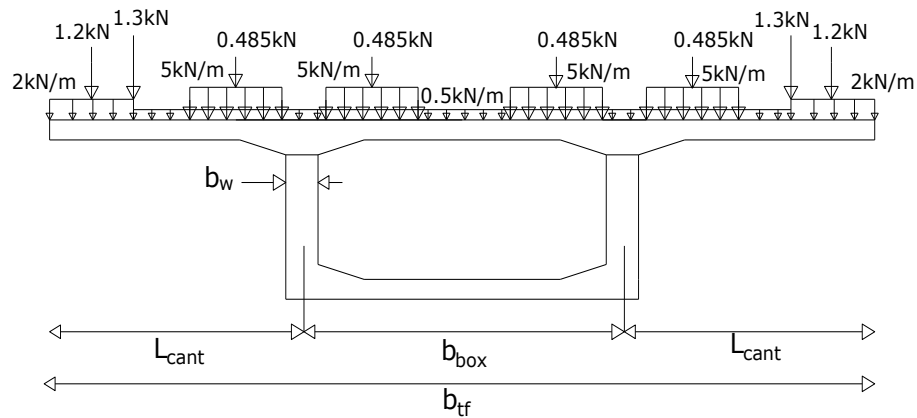


Figure 3-7: transversal utility and attachment loads for 30m span

The deck is schematized in the transversal direction as a floor of 1 meter wide with two supports (the webs). The width of 1 meter in longitudinal direction comes from [4], which says that for the calculation of the deck the wheel pressure in longitudinal direction of the track may be spread to two sides over a distance of 0.9 meter + twice the effective depth of the slab. For a more conservative calculation only the width of 1 meter is taken. The width of 1 meter in transverse direction comes from [11], which says that for calculation of the deck the wheel pressure in transverse direction of the track may be spread to two sides of concrete plinths with slope of 4:1 up to the level considered.

Top slab Transverse ultimate maximum moment

Distance from left edge of deck (m)	Left lane loaded ultimate Moment (kNm)	Right lane loaded ultimate Moment (kNm)	Two lane loaded ultimate Moment (kNm)	Enveloped maximum ultimate moment (kNm)
0.00	0.00	0.00	0.00	0.00
0.46	-0.94	-0.94	-0.94	-0.94
0.85	-3.87	-3.87	-3.87	-3.87
1.24	-8.08	-8.08	-8.08	-8.08
1.63	-14.51	-13.43	-14.51	-14.51
2.02	-43.30	-20.61	-43.30	-43.30
2.38	-96.41	-29.29	-96.41	-96.41
2.74	-170.27	-39.59	-170.27	-170.27
3.12	-44.12	-4.34	-42.70	-44.12
3.50	4.33	5.04	10.01	10.01
4.00	24.90	13.92	36.08	36.08
4.50	20.28	20.28	36.94	36.94
5.00	13.92	24.90	36.08	36.08
5.50	5.04	4.33	10.01	10.01
5.84	-4.34	-44.12	-42.70	-44.12
6.22	-39.59	-170.27	-170.27	-170.27
6.60	-29.29	-96.41	-96.41	-96.41

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6.94	-20.61	-43.30	-43.30	-43.30
7.33	-13.43	-14.51	-14.51	-14.51
7.72	-8.08	-8.08	-8.08	-8.08
8.11	-3.87	-3.87	-3.87	-3.87
8.51	-0.94	-0.94	-0.94	-0.94
8.96	0.00	0.00	0.00	0.00

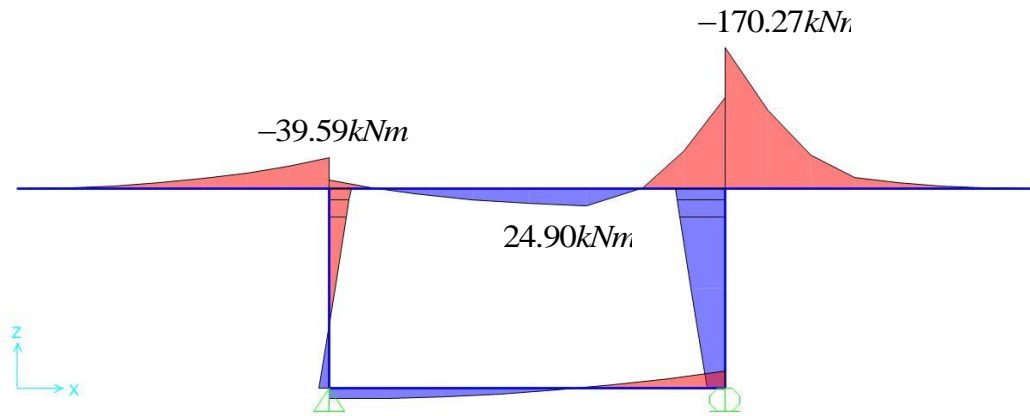


Figure 3-8: Rail live load on the right track ultimate moment diagram for 30m span

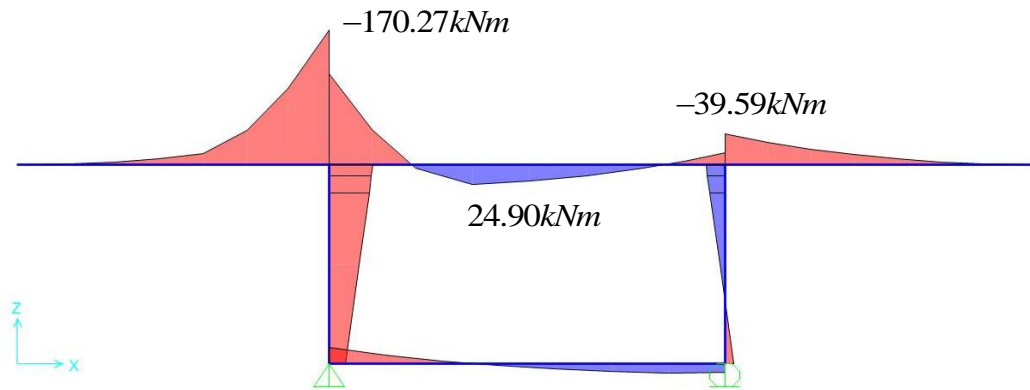


Figure 3-9: Rail live load on the left track ultimate moment diagram for 30m span

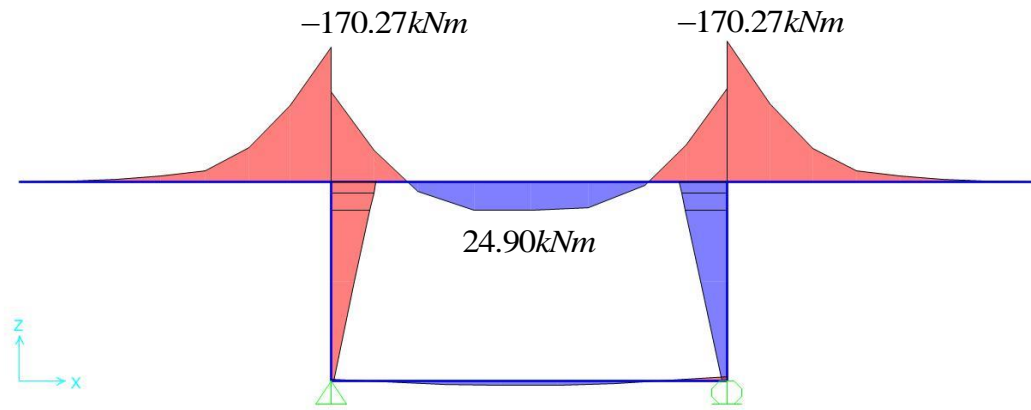
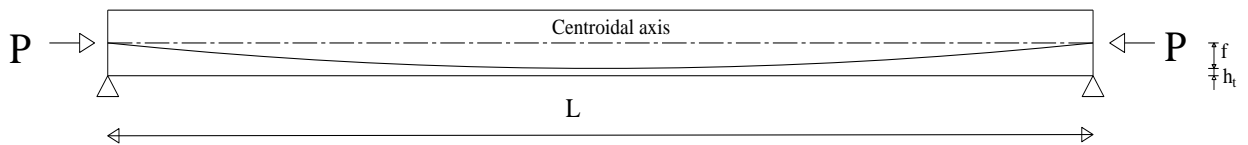


Figure 3-10: Rail live load on both track ultimate moment diagram for 30m span

3.1.4.5 Prestressing tendons

3.1.4.5.1 Layout prestressing tendons



The tendon vertical profile is parabolic with the following ordinate from center:

Tendon distance	0.0	2.5	5.0	7.5	10.0	12.5	15.0	17.5	20.0	22.5	25.0	27.5	30.0
Vertical ordinate	0.0	0.31	0.57	0.77	0.91	1.00	1.03	1.00	0.91	0.77	0.57	0.31	0.0

Distance between the center of the tendons and bottom side at mid-span, $h_t = 0.138m$ and

Tendon eccentricity at mid-span, $Z_{cp,m} = Z_{cb} - h_t = 1.025m$

3.1.4.5.2 Prestressing losses

3.1.4.5.2.1 Losses due to the instantaneous deformation of concrete

During tensioning the box girder will shorten. As the tendons are prestressed successively there arises an immediate prestressing loss which can be calculated for each tendon with the following formula:

$$\Delta p_{el} = A_p * E_p * \sum [j * \Delta \sigma_c(t) / E_{cm}]$$

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Where:

A_p	Cross-sectional area per prestressing tendon
$P = 38760kN$	Initial Prestress force
$\sigma_{pm0} = 1360N / mm^2$	Maximum initial tensile stress in the tendon
$E_p = 195,000 N/mm^2$	Modulus of elasticity of prestressing steel
$E_{cm} = 32000 N/mm^2$	Secant modulus of elasticity of concrete
$j = (n-1)/2n$	Is a coefficient where n is the number of identical Tendon successively prestressed

This prestressing loss taking into account the order in which the tendons are stressed can be compensated by slightly overstressing the tendons. The maximum overstress is needed in the first prestressed tendon as this tendon has the largest loss due the instantaneous deformation of concrete.

The required overstress $\sigma_{overstr}$ in the first prestressed tendon to compensate the losses due to instantaneous deformation of concrete can be calculated out of the formula below:

$$\Delta p_{el,1} = A_p * E_p * j * \Delta \sigma_{overstr} / E_{cm} = A_p * (\sigma_{overstr} - \Delta \sigma_{pm0})$$

Rearranging the above equation
$$\Delta \sigma_{overstr} = \frac{\sigma_{pm0}}{1 - E_p * \frac{j * A_p}{E_{cm} * A_c}} = 1362.61 N/mm^2$$

Total Area of tendon required $A_{ten,total} = P / \Delta \sigma_{pm0} = 28500 mm^2$

Ten tendons each tendon consists of 19 strands

With $150 mm^2$ Sectional area Provide $A_{ten,total}$

Number of tendon = $A_{ten,total} / A_p = 10$ number of tendon

For the first prestressed tendon $n = 10$

$$j = (n-1)/2n = 0.45$$

$\sigma_{p,max} = 1440 N/mm^2$	Maximum allowed tensile stress of The tendons during tensioning
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The stress caused by overstressing is far below this value and as also the concrete compressive stress during tensioning is limited to $\sigma_c \leq 0.6 * f_{ck}(t) = 28.3 N/mm^2$ assuming at the moment of transfer concrete has an age of 20days and moist cured. This small overstressing will not cause any problems for the structure. It can be concluded that the losses due to the instantaneous deformation of concrete can be compensated by overstressing the tendons. By overstressing the tendons the initial tensile stress in all the tendons after tensioning can be the maximum tensile stress $\sigma_{pm0} = 1362.61 N/mm^2$.

3.1.4.5.2.2 Losses due to friction and wobble

The force at any point due to friction and wobble effect in post-tensioned tendons is:

$$P_x = P_0 e^{-\mu(\sum \alpha + \omega x)}, \text{ Parameter assumed value as follows:}$$

Where ω = wobble coefficient	0.006	
μ = Friction coefficient	0.15	/m

3.1.4.5.2.3 Loss due to Anchor set

When Tendons are stressed by jacks that grip the tendon there is a loss of extension when load is transferred from the jack to the anchorage unit. The size of this loss depends principally on the type of tendon and anchor used. This prestressing loss also assumed to be compensated by overstressing the tendons.

3.1.4.5.2.4 Time dependent losses of prestress for post-tensioning

$$\Delta P_{c+s+r} = A_p \Delta \sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8 \Delta \sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} (1 + \frac{A_c}{I_c} Z_{cp}^2) [1 + 0.8 \varphi(t, t_0)]}$$

Where

Creep

$\varphi(\infty, t_0) = 1.8125$	Is the final creep coefficient according Figure 3.1b [12] (Outside conditions; C30; Class N; $t_0 = 30$ days; $h_0 = 2 * A_c / u = 382.96mm$)
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Shrinkage

$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 0.000204$	Is the estimated shrinkage strain in absolute value
---	---

Where

$$\varepsilon_{cd}(\infty) = k_h * \varepsilon_{cd,0} = 0.20419\%$$

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$$\varepsilon_{cd,0} = 0.28\%$$

Is the nominal unrestrained drying shrinkage value
According to Table 3.2 [12] (Relative humidity 80%;
C30)

$$k_h = 0.73$$

A coefficient depending on the notional size h_0
According to Table 3.3 [12]

$$\varepsilon_{ca}(\infty) = 2.5(f_{ck} - 10) * 10^{-6}$$

$$\varepsilon_{ca}(\infty) = 0.00005\%$$

Is the autogenous shrinkage

Relaxation

Relaxation class 2 (wire or strand):

$$\Delta\sigma_{pr} = (\sigma_{pi}) * 0.66 * \rho_{1000} * e^{9.1\mu} \left(\frac{t}{1000} \right)^{0.75(1-\mu)} * 10^{-5} = 60.93 \text{ N/mm}^2$$

Where:

$$\sigma_{pi} = \sigma_{pm0} = 1360 \text{ N/mm}^2$$

Relaxation class 2 therefore $\rho_{1000} = 2.5\%$

$$\mu = \sigma_{pi} / f_{pk} = 0.73$$

The long term (final) values of the relaxation losses may be estimated for a time equal to:
 $t = 500,000$ hours.

Concrete stress

$\sigma_{c,QP}$ Is the stress in the concrete adjacent to the tendons, due to self-weight,
Initial prestress and other permanent actions depending on the stage
Of construction considered. Stress at the Centroid axis at $t=0$ assumed.

This gives:

$$\sigma_{c,QP} = \frac{P_0}{A_c} = 9.516 \text{ N/mm}^2$$

Time dependent loss of prestress for post-tensioning at support

$$\Delta P_{c+s+r,s} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} \left(1 + \frac{A_c}{I_{c,s}} Z_{cp,s}^2\right) [1 + 0.8\varphi(t, t_0)]} = 4997.844 \text{ kN}$$

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Time dependent loss of prestress for post-tensioning at mid-span

$$\Delta P_{c+s+r,m} = nA_p \Delta \sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8 \Delta \sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_o) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} (1 + \frac{A_c}{I_{c,m}} Z_{cp,m}^2) [1 + 0.8 \varphi(\infty, t_o)]} = 4059.496 kN$$

$$Z_{cp,m} = 1024.9 mm$$

Is the distance between the center of gravity of the Concrete section and the tendons at mid span

$$Z_{cp,s} = 0$$

Is the distance between the center of gravity of the Concrete section and the tendons at support

Since external load effect is significant close to mid span; Loss at mid span value is adopted for further calculations.

3.1.4.5.3 Summary of Total prestressing losses

Tendon distance (m)	prestress force (kN) After setting	prestress force (kN) After long term losses
0.0	38760.00	34700.50
1.5	38371.06	34311.57
3.0	37948.15	33888.65
4.5	37530.17	33470.68
6.0	37117.04	33057.54
7.5	36708.66	32649.16
9.0	36304.93	32245.44
10.5	35905.78	31846.29
12.0	35511.12	31451.62
13.5	35120.86	31061.36
15.0	34734.91	30675.42
16.5	34353.21	30293.72
18.0	33975.68	29916.18
19.5	33602.23	29542.73
21.0	33232.79	29173.29
22.5	32867.29	28807.80
24.0	32505.67	28446.17
25.5	32147.85	28088.35
27.0	31793.76	27734.26
28.5	31443.34	27383.84
30.0	31127.82	27068.32

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3.1.4.6 Bending moments and shear force due to prestressing

The moment diagram and structural schematization due to prestressing is shown

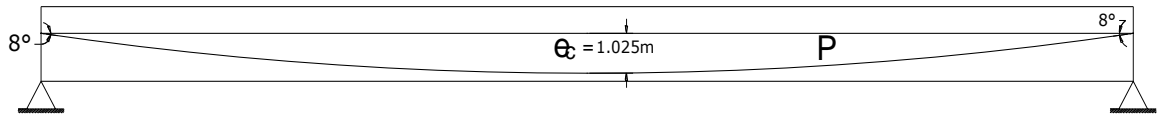


Figure 3-11: Parabolic prestress layout anchored at neutral axis for 30m span

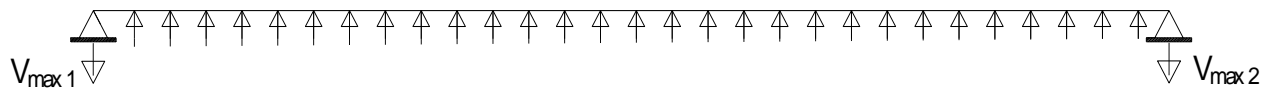


Figure 3-12: Equivalent load due to prestress action for 30m span

Effective Prestress force effect of shear and moment

Tendon distance (m)	Effective prestress Shear force (kN)	Effective prestress moment (MNm)
0.0	-3961.38	-0.02
1.0	-3961.38	-4.09
2.0	-3817.65	-8.01
3.0	-3591.13	-11.60
4.0	-3311.43	-14.87
5.0	-3007.23	-17.72
6.0	-2604.47	-19.99
7.0	-2306.79	-22.00
8.0	-2016.84	-23.64
9.0	-1734.82	-24.99
10.0	-1466.85	-26.16
11.0	-1124.98	-26.83
12.0	-872.15	-27.33
13.0	-629.01	-27.63
14.0	-391.87	-27.72
15.0	-158.47	-27.60
15.0	157.04	-27.60
16.0	380.64	-27.18
17.0	601.67	-26.63
18.0	821.94	-25.90
19.0	1044.50	-24.98
20.0	1335.69	-23.87
21.0	1555.53	-22.49
22.0	1778.81	-20.87
23.0	2001.93	-19.10
24.0	2225.86	-17.10

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

25.0	2518.72	-14.86
26.0	2728.51	-12.32
27.0	2913.79	-9.53
28.0	3052.34	-6.50
29.0	3121.21	-3.27
30.0	3121.21	-0.01

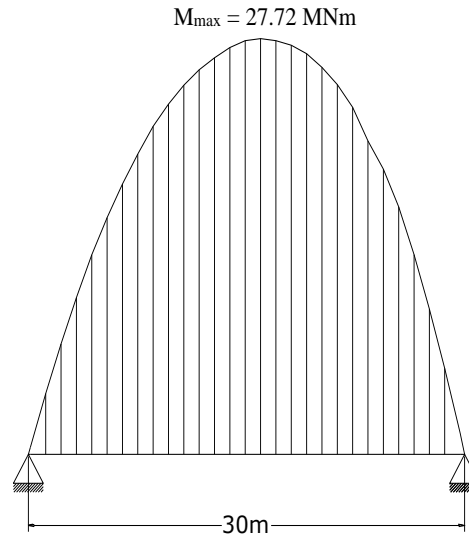


Figure 3-13: Bending moment diagram due to prestress action for 30m span

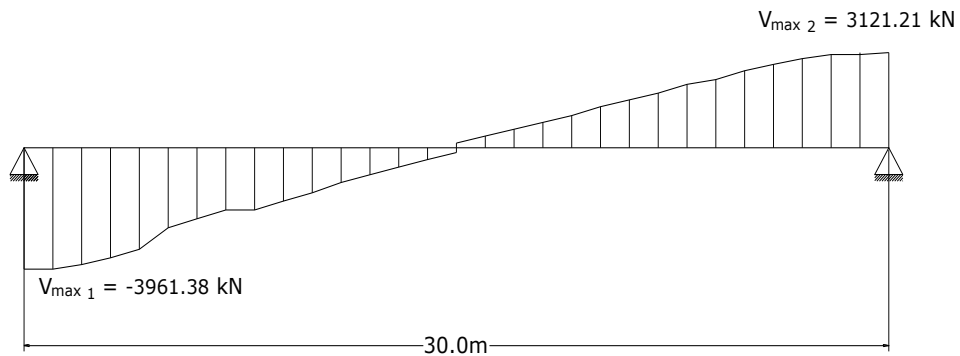


Figure 3-14: Shear force diagram due to prestress action for 30m span

3.1.4.7 Table of service stresses

Distance (m)	Bottom fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	-0.01	8.24	-0.87	0.00	0.00
	Cumulated		+8.22	+7.36	+7.36	+7.36
1	Partial	-1.02	11.18	-1.18	-0.33	-2.66
	Cumulated		+10.16	+8.98	+8.66	+5.99
2	Partial	-1.97	14.44	-1.38	-0.63	-4.89
	Cumulated		+12.47	+11.09	+10.46	+5.57

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3	Partial	-2.83	17.25	-1.85	-0.90	-7.00
	Cumulated		+14.42	+12.57	+11.66	+4.66
4	Partial	-3.60	19.52	-2.11	-1.15	-8.80
	Cumulated		+15.93	+13.82	+12.67	+3.87
5	Partial	-4.28	21.38	-2.33	-1.37	-10.40
	Cumulated		+17.09	+14.76	+13.39	+2.99
6	Partial	-4.85	22.98	-2.52	-1.55	-11.69
	Cumulated		+18.13	+15.61	+14.06	+2.37
7	Partial	-5.37	24.31	-2.69	-1.71	-12.77
	Cumulated		+18.94	+16.25	+14.54	+1.77
8	Partial	-5.80	25.42	-2.83	-1.85	-13.70
	Cumulated		+19.62	+16.79	+14.94	+1.23
9	Partial	-6.16	26.42	-2.96	-1.97	-14.39
	Cumulated		+20.26	+17.30	+15.33	+0.95
10	Partial	-6.48	27.07	-3.06	-2.07	-14.96
	Cumulated		+20.59	+17.53	+15.46	+0.49
11	Partial	-6.70	27.55	-3.14	-2.14	-15.32
	Cumulated		+20.85	+17.71	+15.57	+0.25
12	Partial	-6.87	27.89	-3.20	-2.19	-15.51
	Cumulated		+21.01	+17.81	+15.62	+0.11
13	Partial	-6.99	28.08	-3.25	-2.23	-15.62
	Cumulated		+21.08	+17.84	+15.60	-0.01
14	Partial	-7.06	28.12	-3.27	-2.25	-15.67
	Cumulated		+21.06	+17.78	+15.53	-0.14
15	Partial	-7.07	28.00	-3.29	-2.26	-15.63
	Cumulated		+20.92	+17.63	+15.38	-0.26
16	Partial	-7.06	27.73	-3.28	-2.25	-15.67
	Cumulated		+20.67	+17.39	+15.14	-0.54
17	Partial	-6.99	27.33	-3.26	-2.23	-15.62
	Cumulated		+20.34	+17.09	+14.85	-0.76
18	Partial	-6.87	26.80	-3.21	-2.19	-15.43
	Cumulated		+19.93	+16.72	+14.52	-0.91
19	Partial	-6.70	26.14	-3.16	-2.14	-15.32
	Cumulated		+19.44	+16.28	+14.14	-1.18
20	Partial	-6.48	25.33	-3.09	-2.07	-14.96
	Cumulated		+18.85	+15.77	+13.70	-1.26
21	Partial	-6.16	24.28	-2.98	-1.97	-14.39
	Cumulated		+18.12	+15.15	+13.18	-1.21
22	Partial	-5.80	23.10	-2.84	-1.85	-13.70
	Cumulated		+17.31	+14.47	+12.62	-1.08
23	Partial	-5.37	21.77	-2.09	-1.71	-12.77
	Cumulated		+16.41	+14.32	+12.60	-0.17
24	Partial	-4.85	20.29	-2.54	-1.55	-11.68
	Cumulated		+15.44	+12.90	+11.35	-0.33
25	Partial	-4.28	18.61	-2.35	-1.37	-10.40
	Cumulated		+14.33	+11.97	+10.61	+0.21
26	Partial	-3.60	16.62	-2.12	-1.15	-8.80

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

	Cumulated		+13.02	+10.91	+9.76	+0.96
27	Partial	-2.83	14.39	-1.84	-0.90	-7.00
	Cumulated		+11.56	+9.71	+8.81	+1.81
28	Partial	-1.97	11.85	-1.53	-0.63	-4.89
	Cumulated		+9.88	+8.35	+7.72	+2.83
29	Partial	-1.02	9.06	-1.18	-0.33	-2.60
	Cumulated		+8.04	+6.86	+6.53	+3.93
30	Partial	-0.01	6.67	-0.87	0.00	0.00
	Cumulated		+6.66	+5.80	+5.80	+5.80

Distance (m)	Top fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	0.01	8.21	-0.82	0.00	0.05
	Cumulated		+8.22	+7.40	+7.40	+7.45
1	Partial	0.59	6.90	-0.73	0.19	1.49
	Cumulated		+7.48	+6.75	+6.94	+8.43
2	Partial	1.12	5.41	-0.58	0.36	2.78
	Cumulated		+6.53	+5.95	+6.31	+9.09
3	Partial	1.62	3.95	-0.42	0.52	4.00
	Cumulated		+5.56	+5.14	+5.66	+9.65
4	Partial	2.06	2.59	-0.28	0.66	5.05
	Cumulated		+4.65	+4.37	+5.03	+10.08
5	Partial	2.47	1.33	-0.14	0.79	5.99
	Cumulated		+3.80	+3.66	+4.45	+10.44
6	Partial	2.83	0.25	-0.03	0.90	6.81
	Cumulated		+3.08	+3.05	+3.95	+10.76
7	Partial	3.15	-0.69	0.08	1.01	7.48
	Cumulated		+2.46	+2.54	+3.54	+11.02
8	Partial	3.42	-1.49	0.17	1.09	8.09
	Cumulated		+1.94	+2.10	+3.19	+11.28
9	Partial	3.66	-2.18	0.24	1.17	8.54
	Cumulated		+1.48	+1.73	+2.89	+11.44
10	Partial	3.85	-2.73	0.31	1.23	8.91
	Cumulated		+1.12	+1.43	+2.66	+11.56
11	Partial	4.01	-3.15	0.36	1.28	9.17
	Cumulated		+0.86	+1.22	+2.50	+11.67
12	Partial	4.13	-3.47	0.40	1.32	9.32
	Cumulated		+0.66	+1.06	+2.38	+11.70
13	Partial	4.21	-3.69	0.43	1.35	9.41
	Cumulated		+0.52	+0.95	+2.30	+11.71
14	Partial	4.26	-3.82	0.44	1.36	9.47
	Cumulated		+0.45	+0.89	+2.25	+11.72
15	Partial	4.28	-3.85	0.45	1.37	9.45
	Cumulated		+0.43	+0.88	+2.25	+11.70

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

16	Partial	4.28	-3.77	0.45	1.36	9.47
	Cumulated		+0.51	+0.96	+2.32	+11.79
17	Partial	4.26	-3.59	0.43	1.35	9.41
	Cumulated		+0.67	+1.10	+2.44	+11.85
18	Partial	4.21	-3.34	0.40	1.32	9.32
	Cumulated		+0.88	+1.28	+2.60	+11.92
19	Partial	4.01	-2.99	0.36	1.28	9.17
	Cumulated		+1.02	+1.38	+2.66	+11.83
20	Partial	3.85	-2.56	0.31	1.23	8.91
	Cumulated		+1.30	+1.61	+2.84	+11.74
21	Partial	3.66	-2.00	0.25	1.17	8.54
	Cumulated		+1.66	+1.90	+3.07	+11.61
22	Partial	3.42	-1.35	0.17	1.09	8.09
	Cumulated		+2.07	+2.24	+3.33	+11.42
23	Partial	3.15	-0.62	0.08	1.01	7.48
	Cumulated		+2.53	+2.61	+3.61	+11.10
24	Partial	2.83	0.21	-0.03	0.90	6.81
	Cumulated		+3.04	+3.02	+3.92	+10.73
25	Partial	2.47	1.14	-0.14	0.79	5.99
	Cumulated		+3.61	+3.47	+4.26	+10.25
26	Partial	2.06	2.19	-0.26	0.66	5.05
	Cumulated		+4.25	+3.99	+4.65	+9.70
27	Partial	1.62	3.32	-0.40	0.52	4.00
	Cumulated		+4.93	+4.53	+5.05	+9.04
28	Partial	1.12	4.50	-0.53	0.36	2.78
	Cumulated		+5.62	+5.09	+5.45	+8.23
29	Partial	0.59	5.66	-0.66	0.19	1.49
	Cumulated		+6.25	+5.59	+5.77	+7.26
30	Partial	0.01	6.69	-0.82	0.00	0.05
	Cumulated		+6.70	+5.88	+5.88	+5.93

Permissible stresses immediately and under working load Assuming at moment of transfer concrete has an age of 20days and a normal Portland cement, moist cured is being used.

$$f_{ck}(20) = [t/\alpha + \beta t] * [f_{ck}(28)] = 28.3Mpa \quad \text{Concrete strength at 20days from Specified 28day strength value}$$

Where, α and β are constants which are given as:

For normal Portland cement and

moist cured concrete $\alpha = 4$ and $\beta = 0.860$

steam cured concrete $\alpha = 1$ and $\beta = 0.965$

For high early strength cement and

moist cured concrete $\alpha = 2.3$ and $\beta = 0.920$

steam cured concrete $\alpha = 0.7$ and $\beta = 0.980$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

$\alpha = 4$	Constant for normal Portland cement And moist cured concrete
$\beta = 0.860$	Constant for normal Portland cement And moist cured concrete
$f_{ct} = 0.6 * f_{ck} (20) = 17 \text{Mpa}$	Permissible compressive stress at transfer
$f_{tt} = 0.21 f_{ck}^{2/3} = -2.03 \text{Mpa}$	Permissible tensile stress at transfer
$f_{cw} = f_{cd} = 17 \text{Mpa}$	Permissible compressive stress at working Condition
$f_{tw} = f_{td} = -1.33 \text{Mpa}$	Permissible tensile stress at working Condition

Tensile Stress check immediately after prestressing, top side

$$\sigma_{ct \max} = \frac{p_0}{A_c} - \frac{M_{m,p0}}{W_t} + \frac{M_{self}}{W_t} = +0.43 \text{MPa} > -2.03 \text{MPa} \rightarrow \text{Ok}$$

Compressive Stress check under working load, top side

$$\sigma_{ct \max} = \frac{p_\infty}{A_c} - \frac{M_{m,p\infty}}{W_t} + \frac{M_{self}}{W_t} + \frac{M_{utility}}{W_t} + \frac{M_{live}}{W_t} = 11.92 \text{MPa} < 17 \text{MPa} \rightarrow \text{Ok}$$

Tensile Stress check under working load, bottom side

$$\sigma_{cb \max} = \frac{p_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{self}}{W_t} - \frac{M_{utility}}{W_t} - \frac{M_{live}}{W_t} = -1.26 \text{MPa} > -1.33 \text{MPa} \rightarrow \text{Ok}$$

3.1.4.8 Deflection

The deflection due to dead weight is determined with the formula:

$$w = \frac{5}{384} \frac{qL^4}{EI_c}$$

Where:

$L = 30 \text{m}$	Length span
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Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

$I_c = 1.75m^4$ Moment of inertia of concrete section

$E = E_{C,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)} = 11377.8 N/mm^2$ Effective modulus of elasticity of concrete
For deflection at $t = \infty$ without variable load

$E = E_{cm} = 32000 N/m^2$ Secant modulus of elasticity of concrete for
Additional deflection under mobile load

The deflections and unity checks at mid-span for different Loads are:

Time	Loads	Deflection w	Maximum allowed Deflection w_{max}	Unity check w/w_{max}
At $t = 0$	$g_{boxdead} - q_{pr0}$	$-28mm$	$L/250 = 120mm$	0.23
At $t = \infty$ without variable load	$g_{boxdead} + g_{utility} - q_{pr\infty}$	$-44.8mm$		
Additional deflection under mobile load without impact	$q_{var-impact}$	+ $37.25mm$	$L/800 = 37.5mm$	0.99
Additional deflection under mobile load with impact	$q_{var+impact}$	$+45mm$		
At $t = \infty$ fully loaded	$g_{boxdead} + g_{utility}$ $+q_{var+impact} - q_{pr\infty}$	$+0.2mm$		

An upward deflection has a negative sign and a downward deflection has a positive sign. As the unity checks show, the design satisfies with respect to deflection. The normative deflection is the additional deflection under mobile load without impact.

3.1.4.9 Shear + torsion

3.1.4.9.1 Shear

$\rho_{w,min} = A_{sw} / (Sb_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk}$ Minimum shear reinforcement ratio

Where:

S Is the spacing of shear reinforcement

α Is the angle between Shear reinforcement and Longitudinal axis

$S_{max} = 0.75d(1 + \cot \alpha)$ Maximum longitudinal spacing of links

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

$$V_{Rd,c} = \frac{I_c * b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_1 \sigma_{cp} f_{ctd}}$$

The shear resistance of concrete in regions
Uncracked in bending.

$$\alpha_1 = 1$$

For Post tension tendon

S

First moment of area above and about the
Centroid axis

$$V_{Rd,c} = \left[C_{Rd,c} K (100 \rho_l f_{ck})^{1/3} + K_1 \sigma_{cp} \right] b_w d$$

The shear resistance of concrete in regions
Cracked in bending

Where:

$$K_1 = 0.15$$

$$V_{Rd,c} = (V_{\min} + K_1 \sigma_{cp}) b_w d$$

The shear resistance of concrete without any
Reinforcement

$$C_{Rd,c} = 0.18 / \gamma_c$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$$V_{\min} = 0.035 K^{3/2} * f_{ck}^{1/2}$$

Values of shear properties per girder		Value	
Live and dead load factor for one girder	$LDFV$	0.5	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	1.71	m
Inner liver arm	Z	1.44	m
Tensile strength of concrete	f_{ctd}	1.33	MPa
Second moment of area of the girder concrete section about horizontal axis	I_c	0.875	m^4
Yield strength of shear reinforcement	f_{yd}	400	MPa
Angle of the inclined struts in tress model	θ	45	degree
First moment of area above and about the horizontal centroid axis	S	0.604	m^3
Design strength of concrete	f_{cd}	17	Mpa
Prestress Concrete stress at centroid axis	σ_{cp}	3.4	Mpa
Reinforcement ratio of tensile longitudinal reinforcement	ρ_l	0.02	
Maximum shear force imitated by crushing of compression strut.	$V_{Rd,max}$	2714.34	kN
Minimum shear reinforcement ratio	ρ_{\min}	0.000953	
Minimum area of shear reinforcement	$A_{s,\min}$	333	(mm^2 / m)
Maximum spacing	S_{\max}	1.28	m

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Girder section distance in <i>m</i>	V_{Ed} in <i>kN</i>	$V_{Rd,c}$ in <i>kN</i>	$V_{Ed}/V_{Rd,c}$	$V_{Ed}/V_{Rd,max}$
0	1984.02	1286.36	1.54	0.73
1	1915.35	1286.36	1.49	0.71
2	1738.26	1286.36	1.35	0.64
3	1591.65	1286.36	1.24	0.59
4	1471.08	1286.36	1.14	0.54
5	1381.43	1286.36	1.07	0.51
6	1312.76	619.29	2.12	0.48
7	1224.45	619.29	1.98	0.45
8	1138.77	619.29	1.84	0.42
9	1058.49	619.29	1.71	0.39
10	1020.92	619.29	1.65	0.38
11	952.46	619.29	1.54	0.35
12	886.39	619.29	1.43	0.33
13	822.68	619.29	1.33	0.30
14	760.69	619.29	1.23	0.28
15	-750.03	619.29	1.21	0.28
16	-761.33	619.29	1.23	0.28
17	-830.03	619.29	1.34	0.31
18	-906.39	619.29	1.46	0.33
19	-988.63	619.29	1.60	0.36
20	-1057.29	619.29	1.71	0.39
21	-1139.17	619.29	1.84	0.42
22	-1245.89	619.29	2.01	0.46
23	-1361.64	619.29	2.20	0.50
24	-1483.08	619.29	2.39	0.55
25	-1589.41	619.29	2.57	0.59
26	-1733.05	619.29	2.80	0.64
27	-1896.31	619.29	3.06	0.70
28	-2082.65	619.29	3.36	0.77
29	-2293.43	1286.36	1.78	0.84
30	-2362.10	1286.36	1.84	0.87

For members with vertical shear reinforcement, the shear resistance V_{Rd} is the smaller value

of: $V_{Rd,s} = \frac{A_{sw}}{S} Z f_{ywd} \cot \theta$ f_{ywd} is the design yield strength of the shear

And Reinforcement

$$V_{Rd,max} = \alpha_c b_w Z v f_{cd} / (\cot \theta + \tan \theta)$$

Where:

$$\alpha_c = (1 + \sigma_{cp} / f_{cd}) \quad \text{For } 0 \leq \sigma_{cp} \leq 0.25 f_{cd}$$

$$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$$

S Is the spacing of shear reinforcements

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Girder section distance in (m)	V_{Ed} in (kN)	required A_{sw} in (mm^2 / m)	Type of bar	Center to center Spacing
0	1984.02	3444	Double $\Phi 16$	12cm C/C
1	1915.35	3325	Double $\Phi 16$	12cm C/C
2	1738.26	3018	Double $\Phi 16$	13cm C/C
3	1591.65	2763	Double $\Phi 16$	14cm C/C
4	1471.08	2554	Double $\Phi 16$	16cm C/C
5	1381.43	2398	Double $\Phi 16$	17cm C/C
6	1312.76	2279	Double $\Phi 16$	18cm C/C
7	1224.45	2126	Double $\Phi 16$	19cm C/C
8	1138.77	1977	Double $\Phi 16$	20cm C/C
9	1058.49	1838	Double $\Phi 16$	22cm C/C
10	1020.92	1772	Double $\Phi 16$	23cm C/C
11	952.46	1654	Double $\Phi 16$	24cm C/C
12	886.39	1539	Double $\Phi 16$	26cm C/C
13	822.68	1428	Double $\Phi 16$	28cm C/C
14	760.69	1321	Double $\Phi 16$	30cm C/C
15	-750.03	1302	Double $\Phi 16$	31cm C/C
16	-761.33	1322	Double $\Phi 16$	30cm C/C
17	-830.03	1441	Double $\Phi 16$	28cm C/C
18	-906.39	1574	Double $\Phi 16$	25cm C/C
19	-988.63	1716	Double $\Phi 16$	23cm C/C
20	-1057.29	1836	Double $\Phi 16$	22cm C/C
21	-1139.17	1978	Double $\Phi 16$	20cm C/C
22	-1245.89	2163	Double $\Phi 16$	18cm C/C
23	-1361.64	2364	Double $\Phi 16$	17cm C/C
24	-1483.08	2575	Double $\Phi 16$	16cm C/C
25	-1589.41	2759	Double $\Phi 16$	15cm C/C
26	-1733.05	3009	Double $\Phi 16$	13cm C/C
27	-1896.31	3292	Double $\Phi 16$	12cm C/C
28	-2082.65	3616	Double $\Phi 16$	11cm C/C
29	-2293.43	3982	Double $\Phi 16$	10cm C/C
30	-2362.10	4101	Double $\Phi 16$	10cm C/C

3.1.4.9.2 Additional longitudinal reinforcement due to shear

$$\Delta f_{td} = 0.5V_{Ed} * (\cot \theta - \cot \alpha)$$

Additional tensile force in the longitudinal
Reinforcement due to shear

Girder section distance in (m)	V_{Ed} in (kN)	required A_{st} in (mm^2)	Number of Bar and type
0	1984.02	2480	12 $\Phi 16$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

1	1915.35	2394	12Φ16
2	1738.26	2173	11Φ16
3	1591.65	1990	10Φ16
4	1471.08	1839	9Φ16
5	1381.43	1727	9Φ16
6	1312.76	1641	8Φ16
7	1224.45	1531	8Φ16
8	1138.77	1423	7Φ16
9	1058.49	1323	7Φ16
10	1020.92	1276	6Φ16
11	952.46	1191	6Φ16
12	886.39	1108	6Φ16
13	822.68	1028	5Φ16
14	760.69	951	5Φ16
15	-750.03	938	5Φ16
16	-761.33	952	5Φ16
17	-830.03	1038	5Φ16
18	-906.39	1133	6Φ16
19	-988.63	1236	6Φ16
20	-1057.29	1322	7Φ16
21	-1139.17	1424	7Φ16
22	-1245.89	1557	8Φ16
23	-1361.64	1702	9Φ16
24	-1483.08	1854	9Φ16
25	-1589.41	1987	10Φ16
26	-1733.05	2166	11Φ16
27	-1896.31	2370	12Φ16
28	-2082.65	2603	13Φ16
29	-2293.43	2867	15Φ16
30	-2362.10	2953	15Φ16

3.1.4.9.3 Torsion

Values of Torsion properties		Value	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	1.71	m
Inner liver arm	Z	1.44	m
Tensile strength of concrete	f_{ctd}	1.33	Mpa
Area enclosed by shear flow path	A_k	5.568	m^2

The maximum resistance of the member subjected to shear and torsion limited by the capacity of the concrete struts. In order not to this resistance the following condition should be satisfied:

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$$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max} \leq 1.0$$

Where:

$$T_{Rd,max} = 2v\alpha_c f_{cd} A_K t_{ef,i} \sin \theta \cos \theta \quad \text{The design torsional resistance moment}$$

$$\alpha_c = (1 + \sigma_{cp} / f_{cd}) \quad 0 \leq \sigma_{cp} \leq 0.25 f_{cd}$$

$$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$$

$$t_{ef,i} = \frac{A}{u} \quad \text{Effective thickness}$$

$$A \quad \text{Total area of the cross-section}$$

u Is the outer circumference of the Cross-section

T_{Ed} The design torsional moment

$$V_{Rd,max} = \alpha_c b_w z v f_{cd} / (\cot \theta + \tan \theta) \quad \text{Maximum design shear resistance}$$

The required cross-sectional area of the longitudinal reinforcement for torsion $\sum A_{Sl}$ calculated from:

$$\frac{\sum A_{Sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta$$

Where

u_k Is the perimeter of the area A_k

f_{yd} Is the design yield stress of the longitudinal reinforcement A_{Sl}

θ Is the angle of compression struts

And, the required cross-sectional area of the transverse reinforcement for torsion A_{St} calculated from:

$$\frac{A_{St} f_{yd}}{S} = \frac{T_{Ed}}{2A_k} \cot \theta$$

Where:

S Is the spacing of transverse reinforcement for torsion

Girder section distance in (m)	T_{Ed} in (kNm)	$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max}$	A_{St} required in (mm^2 / m)	A_{Sl} required in (mm^2 / m)
0	4122.99	0.93	926	926

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1	4122.99	0.90	926	926
2	3902.68	0.83	876	876
3	3740.59	0.76	840	840
4	3599.89	0.71	808	808
5	3492.66	0.67	784	784
6	3421.11	0.65	768	768
7	3371.07	0.61	757	757
8	3341.96	0.58	750	750
9	3300.99	0.55	741	741
10	3278.05	0.53	736	736
11	3250.51	0.51	730	730
12	3233.08	0.48	726	726
13	3219.56	0.46	723	723
14	3211.73	0.43	721	721
15	3204.38	0.43	719	719
16	3211.73	0.43	721	721
17	3219.56	0.46	723	723
18	3233.08	0.49	726	726
19	3250.51	0.52	730	730
20	3278.05	0.54	736	736
21	3300.99	0.58	741	741
22	3341.96	0.62	750	750
23	3371.07	0.66	757	757
24	3421.11	0.71	768	768
25	3492.66	0.75	784	784
26	3599.89	0.81	808	808
27	3740.59	0.88	840	840
28	3902.68	0.95	876	876
29	4122.99	1.04	926	926
30	4122.99	1.07	926	926

$$\rho_{w,\min} = A_{t,\min} / (Sb_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk}$$

Minimum torsion reinforcement ratio

$$A_{t,\min} = 333 \text{mm}^2 / \text{m}$$

Minimum area of torsion reinforcement

$$S_{\max} = 35 \text{cm} \leq \begin{cases} u/8 \\ b_w \\ S_{\max} = 0.75d(1 + \cot \alpha) \end{cases}$$

Longitudinal spacing of torsional link

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Girder section distance in (m)	A_{sr} required for each inner and outer link leg (mm^2 / m)	Bar and center to center spacing	A_{st} required for each inner and outer layer (mm^2 / m)	Number of Bar and type
0	926	Double $\Phi 16$ C/C 22cm	926	5 $\Phi 16$
1	926	Double $\Phi 16$ C/C 22cm	926	5 $\Phi 16$
2	876	Double $\Phi 16$ C/C 23cm	876	4 $\Phi 16$
3	840	Double $\Phi 16$ C/C 24cm	840	4 $\Phi 16$
4	808	Double $\Phi 16$ C/C 25cm	808	4 $\Phi 16$
5	784	Double $\Phi 16$ C/C 26cm	784	4 $\Phi 16$
6	768	Double $\Phi 16$ C/C 26cm	768	4 $\Phi 16$
7	757	Double $\Phi 16$ C/C 26cm	757	4 $\Phi 16$
8	750	Double $\Phi 16$ C/C 27cm	750	4 $\Phi 16$
9	741	Double $\Phi 16$ C/C 27cm	741	4 $\Phi 16$
10	736	Double $\Phi 16$ C/C 27cm	736	4 $\Phi 16$
11	730	Double $\Phi 16$ C/C 27cm	730	4 $\Phi 16$
12	726	Double $\Phi 16$ C/C 28cm	726	4 $\Phi 16$
13	723	Double $\Phi 16$ C/C 28cm	723	4 $\Phi 16$
14	721	Double $\Phi 16$ C/C 28cm	721	4 $\Phi 16$
15	719	Double $\Phi 16$ C/C 28cm	719	4 $\Phi 16$
16	721	Double $\Phi 16$ C/C 28cm	721	4 $\Phi 16$
17	723	Double $\Phi 16$ C/C 28cm	723	4 $\Phi 16$
18	726	Double $\Phi 16$ C/C 28cm	726	4 $\Phi 16$
19	730	Double $\Phi 16$ C/C 27cm	730	4 $\Phi 16$
20	736	Double $\Phi 16$ C/C 27cm	736	4 $\Phi 16$
21	741	Double $\Phi 16$ C/C 27cm	741	4 $\Phi 16$
22	750	Double $\Phi 16$ C/C 27cm	750	4 $\Phi 16$
23	757	Double $\Phi 16$ C/C 26cm	757	4 $\Phi 16$
24	768	Double $\Phi 16$ C/C 26cm	768	4 $\Phi 16$
25	784	Double $\Phi 16$ C/C 26cm	784	4 $\Phi 16$
26	808	Double $\Phi 16$ C/C 25cm	808	4 $\Phi 16$
27	840	Double $\Phi 16$ C/C 24cm	840	4 $\Phi 16$
28	876	Double $\Phi 16$ C/C 23cm	876	4 $\Phi 16$
29	926	Double $\Phi 16$ C/C 22cm	926	5 $\Phi 16$
30	926	Double $\Phi 16$ C/C 22cm	926	5 $\Phi 16$

3.1.4.10 Ultimate moment resistance

Values of box girder properties		Value	
Area of the top slab	$A_{slab,top}$	2.53	m^2
Equivalent thickness of top slab	$t_{f,equ}$	0.283	m
Sum of web width	Σb_w	0.7	m

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Area of the bottom slab	$A_{slab,bott}$	0.95	m^2
Equivalent thickness of bottom slab	$t_{bf, equ}$	0.248	m
Width of top slab	b_{tf}	8.96	m

$A_{ps} = 28500mm^2$ Total Area of tendon

$f_{ps} \leq f_{pd}$ Prestressing steel stress

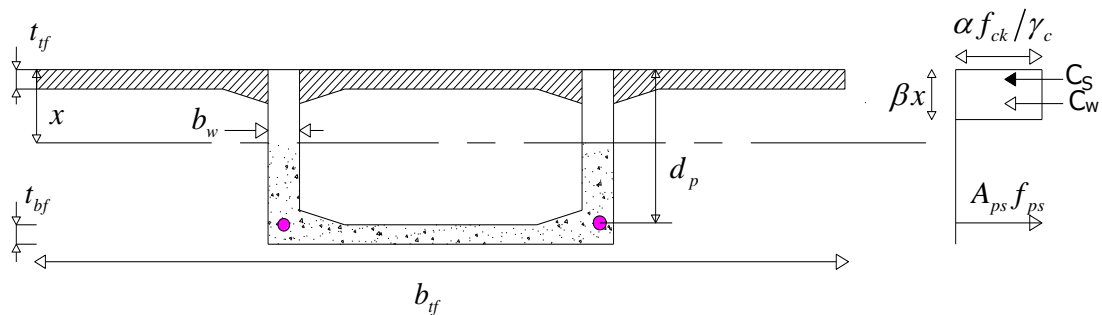
x Distance between neutral axis and compressive face

$\alpha = 0.85$ Coefficient taking account of long term effects on the Compressive strength and unfavorable effects resulting From the way the load is applied.

$\beta = 0.8$ Effective height of the compression zone

$\epsilon_{cu3} = 0.0035$ Ultimate compressive strain in the concrete

$f_{pd} = f_{pk} / \gamma_s = 1617 N/mm^2$ Design value for prestressing steel stress



Girder section distance in (m)	Tendon ultimate force (kN)	Distance between neutral axis and compressive face (x) in (m)	Compressive block depth below top fiber to neutral axis (βx) in (m)
0	43231.82	0.36	0.29
1	43862.52	0.43	0.35
2	44414.09	0.49	0.39
3	44978.09	0.55	0.44
4	45442.45	0.60	0.48
5	45800.68	0.64	0.51
6	46084.50	0.67	0.54
7	46084.50	0.67	0.54

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8	46084.50	0.67	0.54
9	46084.50	0.67	0.54
10	46084.50	0.67	0.54
11	46084.50	0.67	0.54
12	46084.50	0.67	0.54
13	46084.50	0.67	0.54
14	46084.50	0.67	0.54
15	46084.50	0.67	0.54
16	46084.50	0.67	0.54
17	46084.50	0.67	0.54
18	46084.50	0.67	0.54
19	46084.50	0.67	0.54
20	46084.50	0.67	0.54
21	46084.50	0.67	0.54
22	46084.50	0.67	0.54
23	46084.50	0.67	0.54
24	46084.50	0.67	0.54
25	45800.68	0.64	0.51
26	45442.45	0.60	0.48
27	44978.09	0.55	0.44
28	44414.09	0.49	0.39
29	43862.52	0.43	0.35
30	43231.82	0.36	0.29

$$C_s = (\alpha * f_{ck} / \gamma_c) * (b_f - \sum b_w) * h_{f, equ} \quad \text{Compressive force in flange}$$

$$C_w = (\alpha f_{ck} / \gamma_c) * \beta x * \sum b_w \quad \text{Compressive force in web}$$

$$M_{Rs} = C_s * (d_p - 0.5 * t_{ff}) \quad \text{Resistance moment due to flange compressive force}$$

$$M_{Rw} = C_w * (d_p - 0.5 * \beta x) \quad \text{Resistance moment due to web compressive force}$$

$$M_{RT} = M_{Rs} + M_{Rw} \quad \text{Total positive resistance moment of section}$$

Box Girder section distance in (m)	Positive moment resistance for the girder (M_{RT}) in (kNm)	Positive demand moment for the girder (M_{DE}) in (kNm)	Ratio (M_{DE} / M_{RT})
0	21412.12	0.00	0.00
1	27404.72	7508.38	0.27
2	33081.59	14341.80	0.43
3	38482.01	20664.80	0.54
4	43496.07	26250.30	0.60
5	48080.51	31407.70	0.65
6	52263.59	35932.19	0.69

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7	55832.23	39871.95	0.71
8	58981.03	43422.63	0.74
9	61709.99	46273.47	0.75
10	64019.11	48639.69	0.76
11	65908.39	50598.81	0.77
12	67377.83	51859.18	0.77
13	68427.43	52689.46	0.77
14	69057.19	52986.17	0.77
15	69267.11	52603.12	0.76
16	69057.19	53381.02	0.77
17	68427.43	52807.81	0.77
18	67377.83	51872.69	0.77
19	65908.39	50510.35	0.77
20	64019.11	49023.81	0.77
21	61709.99	46535.22	0.75
22	58981.03	43655.84	0.74
23	55832.23	39974.72	0.72
24	52263.59	35932.19	0.69
25	48080.51	31723.64	0.66
26	43496.07	26449.45	0.61
27	38482.01	20748.71	0.54
28	33081.59	14352.08	0.43
29	27404.72	7508.38	0.27
30	21412.12	0.00	0.00

The unity checks show that at all girder section positive moment resistance is greater than the demand external moment; no ultimate state additional reinforcement is required.

3.1.4.11 Buckling webs

Buckling verification is needed for the webs of the largest box girder depth. The buckling strength requirement of the webs should meet:

$$F_k \geq \alpha_{cr} * F_d \quad \text{Where:}$$

$$F_k = \frac{\pi^2 EI}{l_0^2} \quad \text{Euler buckling force}$$

$$\alpha_{cr} = 10 \quad \text{Force amplifier to reach the elastic critical buckling}$$

$$F_d = V_{Ed, \max} + T_{ed, \max} / z_{webs} \quad \text{Buckling force in one web}$$

$$= 6739.27 \text{ kN}$$

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$$Z_{webs} = 3.83m$$

Lever arm of the webs

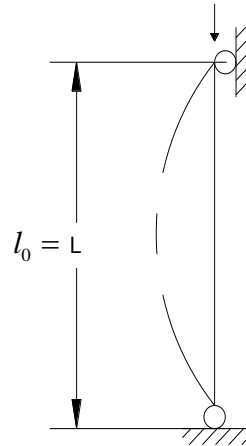


Figure 3-15: Buckling modes and corresponding effective lengths of isolated member

As the webs and flanges are relatively slender full rotation stiffness is not likely to occur to determine this rotation stiffness a more extensive calculation is necessary. To be able to make a simple verification of buckling it is therefore chosen to schematize the webs as buckling mode shown above among different modes.

$$l_0 = 3600mm$$

The effective buckling length equals the length of the webs

$$I \geq \frac{l_0^2 * \alpha_{cr} * F_d}{\pi^2 * E_c} = 2765464794mm^4 \quad \text{Required moment of inertia of the webs}$$

$$I = \frac{1}{12} * L_{webs} * t_w^3 \quad \text{Formula for the moment of inertia of the web}$$

For this calculation it is chosen to take effective length of the webs as 1 meter. This is chosen as in the local schematization of the deck the local rail point load is distributed over 1 meter in the longitudinal direction of the box girder.

$$L_{webs} = 1000mm$$

Effective length of the webs

$$t_{w,req} = \sqrt[3]{\frac{12I}{L_{webs}}} = 321.35mm$$

Minimum required width of the webs

$$b_w = 350mm > b_{w,req} = 321.35mm \rightarrow Ok$$

Verification of buckling of the webs

3.2 Sample Calculations Result of Concrete Box Girder C-40

3.2.1 Introduction

This sub chapter presents the calculations result of concrete box girder made of C-40 for sample span length of 40m also results other than this span length presented in the later chapter and appendix. For the extensive calculation of the box girder reference is made to Appendix A. The formulas and values used in the calculations are taken from [12] and other references, which are then stated in the text.

Note: - equation terms defined in previous chapter are also valid and are not repeated.

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3.2.2 External Load effects

3.2.2.1 Load schematization in longitudinal direction

3.2.2.1.1 Vertical loads

Maximum moment Service stress

	SLS moment (<i>MNm</i>)	Top fiber stress (<i>MPa</i>)	Bottom fiber stress (<i>MPa</i>)
Box girder dead load	21.52	+5.63	-8.85
Utility and attachment dead load	6.24	+1.63	-2.588
Common live load one track loaded	20.27		
Common live load two track loaded	40.55		
Special rail live load one track loaded	7.54		
Special rail live load two track loaded	15.09		
Standard live load Envelop	40.55	+10.63	-17.00
Total	68.31	+17.89	-28.44

Ultimate limit state maximum shear force per girder

	SLS shear force <i>kN</i>	Load Factors	ULS shear force <i>kN</i>
Box girder dead load	1186.17	1.3	1542.02
Utility and attachment dead load	334.62	1.3	435.00
Common live load one track loaded	2343.21		
Common live load two track loaded	2552.83		
Special railway live load one track loaded	761.84		
Special railway live load two track loaded	858.29		
Standard live load Envelop	2552.83	1.18	3012.34
Total	4073.61		4989.35

Ultimate limit state Maximum Torsional moment

	SLS Torsion moment (<i>kNm</i>)	Load Factors	ULS Torsion moment (<i>kNm</i>)
Common live load one track loaded	4244.63		
Common live load two track loaded	4259.02		
Special railway live one track loaded	1282.27		
Special railway live two track loaded	1474.34		

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Standard live load Envelop	4259.02	1.18	5025.64
Total	4259.02		5025.64

3.2.2.2 Approximate Load schematization in transversal direction

3.2.2.2.1 Vertical load

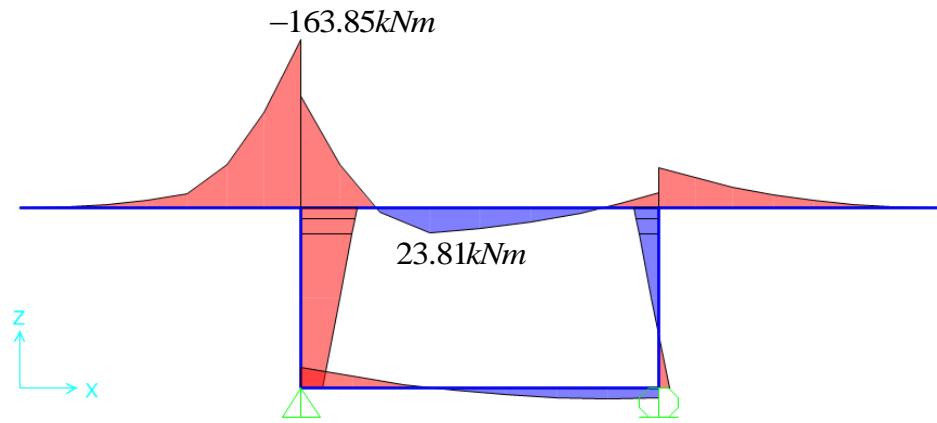


Figure 3-16: Rail live load on the left track ultimate moment diagram for 40m span

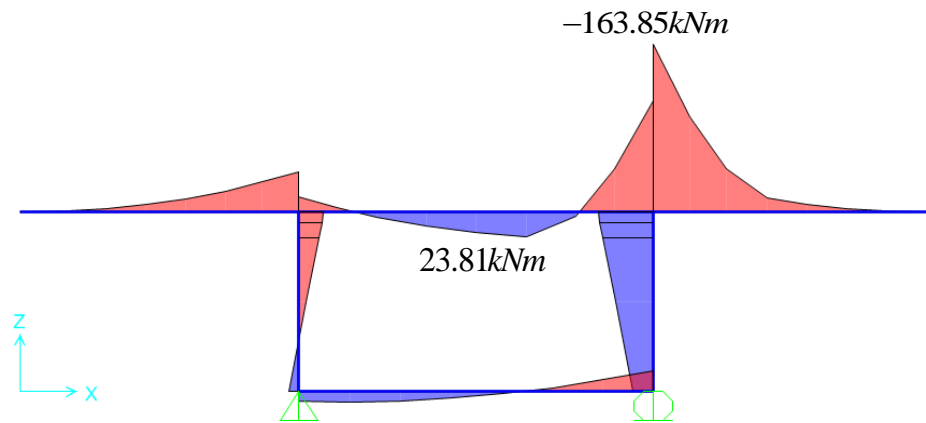


Figure 3-17: Rail live load on the right track ultimate moment diagram for 40m span

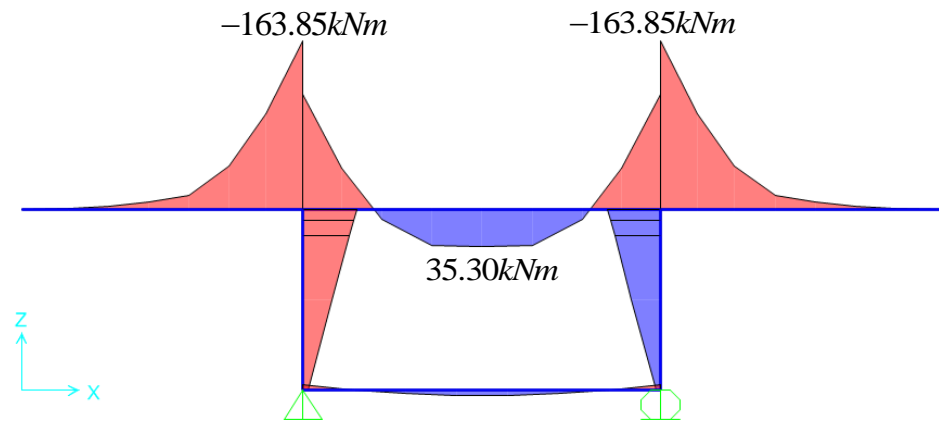
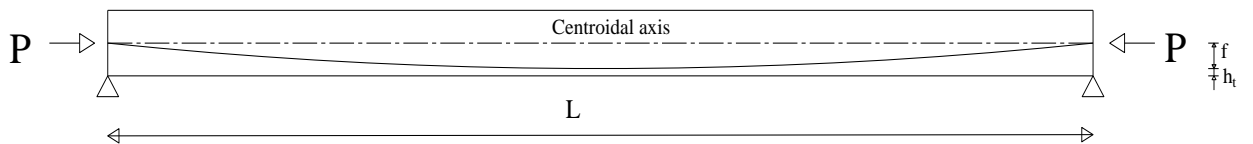


Figure 3-18: Rail live load on two tracks ultimate moment diagram for 40m span

3.2.3 Prestressing tendons

3.2.3.1 Layout prestressing tendons



The tendon vertical profile is parabolic with the following ordinate from center:

Tendon distance	0.0	3.33	6.7	10.0	13.33	16.7	20.0	23.33	26.7	30.0	33.33	36.7	40.0
Vertical ordinate	0.0	0.42	0.76	1.02	1.21	1.32	1.36	1.32	1.21	1.02	0.76	0.42	0.0

Distance between the center of the tendons and bottom side at mid-span $h_t = 0.17m$ and

Tendon eccentricity at mid-span $Z_{cp,m} = Z_{cb} - h_t = 1.36m$

3.2.3.2 Prestressing values

$$P = 51408kN$$

Initial Prestress force

$$A_p = 2700mm^2$$

Cross-sectional area per prestressing tendon

$$\text{Total Area of tendon required } A_{ten,total} = P / \sigma_{pm0} \quad 37800mm^2$$

$$\text{Number of tendon} = A_{ten,total} / A_p$$

14 number of tendon

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3.2.3.3 Prestressing losses

Summary of Total prestressing losses

Tendon distance (m)	prestress force (kN) Prior to setting	prestress force (kN) After long term losses
0.0	51408.00	46356.86
2.0	50738.60	45687.45
4.0	50028.35	44977.21
6.0	49328.64	44277.49
8.0	48639.21	43588.06
10.0	47959.84	42908.69
12.0	47290.30	42239.16
14.0	46630.39	41579.25
16.0	45979.89	40928.75
18.0	45338.60	40287.45
20.0	44706.31	39655.17
22.0	44082.85	39031.71
24.0	43468.01	38416.87
26.0	42861.63	37810.49
28.0	42263.52	37212.37
30.0	41673.51	36622.36
32.0	41091.43	36040.29
34.0	40517.12	35465.98
36.0	39950.43	34899.29
38.0	39391.21	34340.06
40.0	38878.28	33827.14

3.2.4 Service stresses check

Tensile Stress check immediately after prestressing, top side

$$\sigma_{ct\max} = \frac{P_0}{A_c} - \frac{M_{m,p0}}{W_t} + \frac{M_{self}}{W_t} = +1.36Mpa > -2.45Mpa \rightarrow Ok$$

Compressive Stress check under working load, top side

$$\sigma_{ct\max} = \frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_t} + \frac{M_{self}}{W_t} + \frac{M_{utility}}{W_t} + \frac{M_{live}}{W_t} = 14.19Mpa < 22.67Mpa \rightarrow Ok$$

Tensile Stress check under working load, bottom side

$$\sigma_{cb\max} = \frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{self}}{W_t} - \frac{M_{utility}}{W_t} - \frac{M_{live}}{W_t} = -1.63Mpa > -1.67Mpa \rightarrow Ok$$

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3.2.5 Deflection Check

The maximum deflections and unity checks for different Loads are:

Time	Loads	Deflection w	Maximum allowed Deflection w_{max}	Unity check w/w_{max}
At $t=0$	$g_{boxdead} - q_{pt0}$	$-43mm$	$L/250 = 160mm$	0.27
At $t=\infty$ without variable load	$g_{boxdead} + g_{utility} - q_{pt\infty}$	$-64.54mm$		
Additional deflection under mobile load without impact	$q_{var-impact}$	+ $49.65mm$	$L/800 = 50mm$	0.99
Additional deflection under mobile load with impact	$q_{var+impact}$	+ $58mm$		
At $t=\infty$ fully loaded	$g_{boxdead} + g_{utility}$ + $q_{var+impact} - q_{pt\infty}$	$-6.54mm$		

An upward deflection has a negative sign and a downward deflection has a positive sign. As the unity checks show, the design satisfies with respect to deflection. The normative deflection is the additional deflection under mobile load without impact.

3.2.6 Shear + torsion

3.2.6.1 Shear

Minimum shear reinforcement ratio	ρ_{min}	0.0011	
Minimum area of shear reinforcement	$A_{s,min}$	385	(mm^2 / m)
Maximum spacing	S_{max}	1.71	m

For members with vertical shear reinforcement, the shear resistance V_{Rd} is the smaller value of: $V_{Rd,s} = \frac{A_{sw}}{S} Z f_{ywd} \cot \theta$

And

$$V_{Rd,max} = \alpha_c b_w Z f_{cd} / (\cot \theta + \tan \theta)$$

Girder section distance in (m)	V_{Ed} in (kN)	required A_{sw} in (mm^2 / m)	Type of bar	center to center spacing
0	2511.26	3270	Double $\Phi 16$	12cm C/C
1	2435.77	3172	Double $\Phi 16$	13cm C/C

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2	2270.32	2956	Double Φ 16	14cm C/C
3	2095.01	2728	Double Φ 16	15cm C/C
4	1975.84	2573	Double Φ 16	16cm C/C
5	1876.80	2444	Double Φ 16	16cm C/C
6	1786.24	2326	Double Φ 16	17cm C/C
7	1699.91	2213	Double Φ 16	18cm C/C
8	1617.54	2106	Double Φ 16	19cm C/C
9	1539.22	2004	Double Φ 16	20cm C/C
10	1471.72	1916	Double Φ 16	21cm C/C
11	1403.69	1828	Double Φ 16	22cm C/C
12	1333.40	1736	Double Φ 16	23cm C/C
13	1269.52	1653	Double Φ 16	24cm C/C
14	1205.65	1570	Double Φ 16	26cm C/C
15	1143.59	1489	Double Φ 16	27cm C/C
16	1083.18	1410	Double Φ 16	28cm C/C
17	1024.24	1334	Double Φ 16	30cm C/C
18	966.90	385	Double Φ 16	104cm C/C
19	908.54	385	Double Φ 16	104cm C/C
20	-874.53	385	Double Φ 16	104cm C/C
21	-909.01	385	Double Φ 16	104cm C/C
22	-971.23	385	Double Φ 16	104cm C/C
23	-1035.23	1348	Double Φ 16	30cm C/C
24	-1104.30	1438	Double Φ 16	28cm C/C
25	-1181.75	1539	Double Φ 16	26cm C/C
26	-1265.98	1648	Double Φ 16	24cm C/C
27	-1352.09	1761	Double Φ 16	23cm C/C
28	-1450.95	1889	Double Φ 16	21cm C/C
29	-1550.65	2019	Double Φ 16	20cm C/C
30	-1651.97	2151	Double Φ 16	19cm C/C
31	-1772.10	2307	Double Φ 16	17cm C/C
32	-1894.60	2467	Double Φ 16	16cm C/C
33	-2025.01	2637	Double Φ 16	15cm C/C
34	-2162.53	2816	Double Φ 16	14cm C/C
35	-2308.37	3006	Double Φ 16	13cm C/C
36	-2464.19	3209	Double Φ 16	12cm C/C
37	-2638.85	3436	Double Φ 16	12cm C/C
38	-2860.48	3725	Double Φ 16	11cm C/C
39	-3069.04	3996	Double Φ 16	10cm C/C
40	-3144.52	4094	Double Φ 16	10cm C/C

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3.2.6.1.1 Additional longitudinal reinforcement due to shear

Girder section distance in (m)	V_{Ed} in (kN)	required A_{s_l} in (mm^2)	Number of Bar and type
0	2511.26	3139	16Φ16
1	2435.77	3045	15Φ16
2	2270.32	2838	14Φ16
3	2095.01	2619	13Φ16
4	1975.84	2470	12Φ16
5	1876.80	2346	12Φ16
6	1786.24	2233	11Φ16
7	1699.91	2125	11Φ16
8	1617.54	2022	10Φ16
9	1539.22	1924	10Φ16
10	1471.72	1840	9Φ16
11	1403.69	1755	9Φ16
12	1333.40	1667	8Φ16
13	1269.52	1587	8Φ16
14	1205.65	1507	8Φ16
15	1143.59	1429	7Φ16
16	1083.18	1354	7Φ16
17	1024.24	1280	7Φ16
18	966.90	0	-
19	908.54	0	-
20	-874.53	0	-
21	-909.01	0	-
22	-971.23	0	-
23	-1035.23	1294	7Φ16
24	-1104.30	1380	7Φ16
25	-1181.75	1477	8Φ16
26	-1265.98	1582	8Φ16
27	-1352.09	1690	9Φ16
28	-1450.95	1814	9Φ16
29	-1550.65	1938	10Φ16
30	-1651.97	2065	10Φ16
31	-1772.10	2215	11Φ16
32	-1894.60	2368	12Φ16
33	-2025.01	2531	13Φ16
34	-2162.53	2703	14Φ16
35	-2308.37	2885	15Φ16
36	-2464.19	3080	16Φ16
37	-2638.85	3299	17Φ16
38	-2860.48	3576	18Φ16
39	-3069.04	3836	19Φ16

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40	-3144.52	3931	20Φ16
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3.2.6.2 Torsion

$$\rho_{w,\min} = A_{t,\min} / (Sb_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk} \quad \text{Minimum torsion reinforcement ratio}$$

$$A_{t,\min} = 385 \text{ mm}^2 / \text{m} \quad \text{Minimum area of torsion reinforcement}$$

$$S_{\max} = 35 \text{ cm} \leq \begin{cases} u/8 \\ b_w \\ S_{\max} = 0.75d(1 + \cot \alpha) \end{cases} \quad \text{Longitudinal spacing of torsional link}$$

Girder section distance in (m)	A _{st} required for each inner and outer link leg (mm ² / m)	Bar and center to center spacing	A _{sl} required for each inner and outer layer (mm ² / m)	Number of Bar and type
0	824	Double Φ12 C/C 27cm	824	4Φ16
1	822	Double Φ12 C/C 27cm	822	4Φ16
2	792	Double Φ12 C/C 28cm	792	4Φ16
3	764	Double Φ12 C/C 29cm	764	4Φ16
4	741	Double Φ12 C/C 30cm	741	4Φ16
5	721	Double Φ12 C/C 30cm	721	4Φ16
6	707	Double Φ12 C/C 31cm	707	4Φ16
7	697	Double Φ12 C/C 31cm	697	4Φ16
8	688	Double Φ12 C/C 32cm	688	4Φ16
9	680	Double Φ12 C/C 32cm	680	4Φ16
10	673	Double Φ12 C/C 33cm	673	4Φ16
11	666	Double Φ12 C/C 33cm	666	4Φ16
12	663	Double Φ12 C/C 33cm	663	4Φ16
13	656	Double Φ12 C/C 33cm	656	4Φ16
14	653	Double Φ12 C/C 34cm	653	4Φ16
15	650	Double Φ12 C/C 34cm	650	4Φ16
16	647	Double Φ12 C/C 34cm	647	4Φ16
17	645	Double Φ12 C/C 34cm	645	4Φ16
18	644	Double Φ12 C/C 34cm	644	4Φ16
19	642	Double Φ12 C/C 34cm	642	4Φ16
20	642	Double Φ12 C/C 34cm	642	4Φ16
21	642	Double Φ12 C/C 34cm	642	4Φ16
22	644	Double Φ12 C/C 34cm	644	4Φ16
23	645	Double Φ12 C/C 34cm	645	4Φ16
24	647	Double Φ12 C/C 34cm	647	4Φ16

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25	650	Double $\Phi 12$ C/C 34cm	650	4 $\Phi 16$
26	653	Double $\Phi 12$ C/C 34cm	653	4 $\Phi 16$
27	656	Double $\Phi 12$ C/C 33cm	656	4 $\Phi 16$
28	663	Double $\Phi 12$ C/C 33cm	663	4 $\Phi 16$
29	666	Double $\Phi 12$ C/C 33cm	666	4 $\Phi 16$
30	673	Double $\Phi 12$ C/C 33cm	673	4 $\Phi 16$
31	680	Double $\Phi 12$ C/C 32cm	680	4 $\Phi 16$
32	688	Double $\Phi 12$ C/C 32cm	688	4 $\Phi 16$
33	697	Double $\Phi 12$ C/C 31cm	697	4 $\Phi 16$
34	707	Double $\Phi 12$ C/C 31cm	707	4 $\Phi 16$
35	721	Double $\Phi 12$ C/C 30cm	721	4 $\Phi 16$
36	741	Double $\Phi 12$ C/C 30cm	741	4 $\Phi 16$
37	764	Double $\Phi 12$ C/C 29cm	764	4 $\Phi 16$
38	792	Double $\Phi 12$ C/C 28cm	792	4 $\Phi 16$
39	822	Double $\Phi 12$ C/C 27cm	822	4 $\Phi 16$
40	824	Double $\Phi 12$ C/C 27cm	824	4 $\Phi 16$

3.2.7 Ultimate moment resistance check

Box Girder section distance in (m)	Positive moment resistance for girder (M_{RT}) in (kNm)	Positive demand moment for the girder (M_{DE}) in (kNm)	Ratio (M_{DE}/M_{RT})
0	42578.27	0.00	0.00
2	58934.25	19065.35	0.32
4	73400.04	35399.45	0.48
6	85892.89	49700.68	0.58
8	96720.02	61816.30	0.64
10	105881.44	71669.64	0.68
12	113377.15	79449.05	0.70
14	119207.15	85502.31	0.72
16	123371.43	89893.36	0.73
18	125870.00	92476.36	0.73
20	126702.86	92738.97	0.73
22	125870.00	92297.64	0.73
24	123371.43	89966.10	0.73
26	119207.15	85740.54	0.72
28	113377.15	79881.41	0.70
30	105881.44	71710.99	0.68
32	96720.02	61900.19	0.64
34	85892.89	49846.93	0.58
36	73400.04	35543.48	0.48
38	58934.25	19105.97	0.32
40	42578.27	0.00	0.00

Checks for girder at ultimate state shows that, at all girder section moment resistance is greater, therefore no ultimate state additional reinforcement provision is required.

3.3 Sample Calculation Results of Concrete Box Girder C-50

3.3.1 Introduction

This sub chapter describes the calculation results of optimal depth box girder made of C-50 concrete for sample span length of 50m, also analysis result other than this sample span length presented in later chapter and appendix. For detail calculation reference is made to Appendix B. The formulas and values used are taken from [12] and other references, which are then stated in the text.

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3.3.2 External Load effects

3.3.2.1 Load schematization in longitudinal direction

3.3.2.1.1 Vertical loads

Maximum moment Service stress

	SLS moment MNm	Top fiber stress (MPa)	Bottom fiber stress (MPa)
Box girder dead load	36.88	+7.03	-10.67
Utility and attachment dead load	9.68	+1.84	-2.79
Common live load one track loaded	29.76		
Common live load two track loaded	59.69		
Special rail live load one track loaded	9.35		
Special rail live load two track loaded	18.71		
Standard live load Envelop	59.69	+11.58	-17.58
Total	106.25	+20.45	-31.04

Ultimate State Maximum Shear per girder

	SLS shear force kN	Load Factors	ULS shear force kN
Box girder dead load	1627.21	1.3	2115.37
Utility and attachment dead load	419.43	1.3	545.26
Common live load one lane loaded	2805.07		
Common live load two lane loaded	3050.11		
Special railway live load one lane loaded	762.27		
Special railway live load two lane loaded	859.15		
Standard live load Envelop	3050.11	1.15	3507.62
Total	5096.75		6168.26

Ultimate state Maximum Torsional moment

	SLS Torsion moment (kNm)	Load Factors	ULS Torsion moment (kNm)
Standard railway live one lane loaded	5026.98		
Standard railway live two lane loaded	5044.34		
Special railway live one lane loaded	1271.78		

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Special railway live two lane loaded	1291.50		
live load Envelop	5044.34	1.15	5800.99
Total	5044.34		5800.99

3.3.2.2 Load schematization in transversal direction

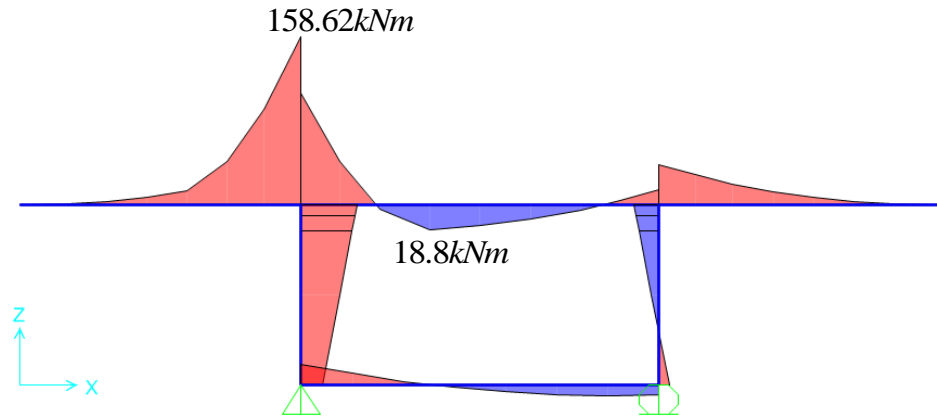


Figure 3-19: Rail load on the left track ultimate moment diagram for 50m span

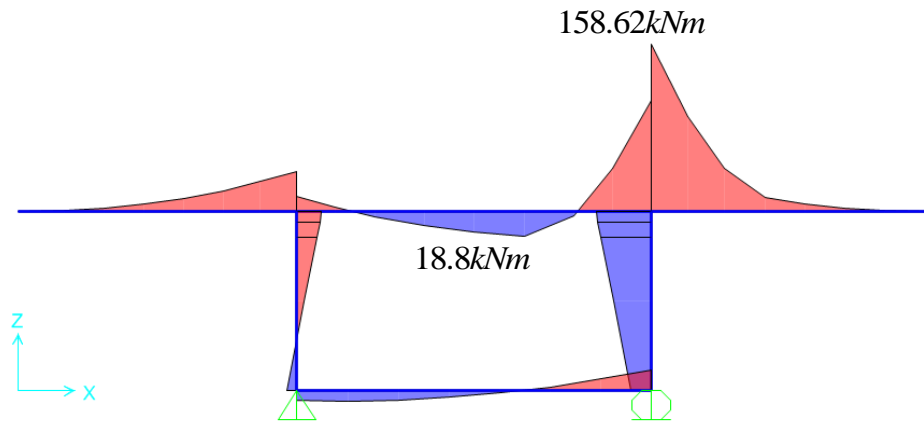


Figure 3-20: Rail load on the right track ultimate moment diagram for 50m span

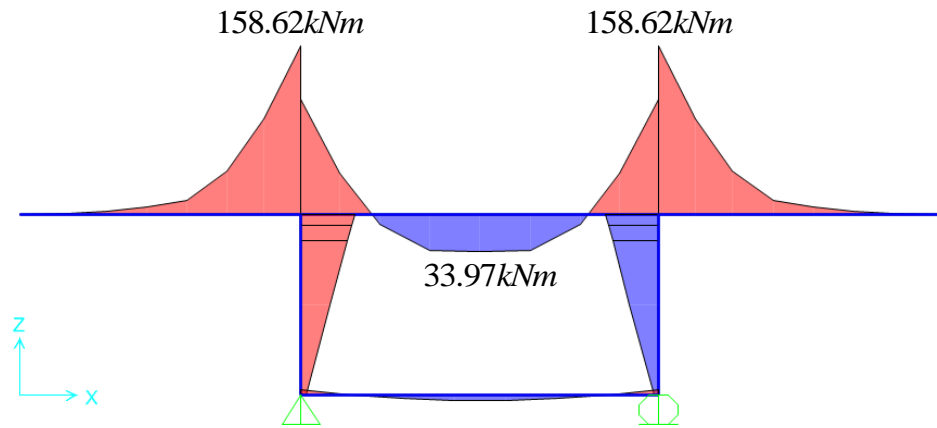
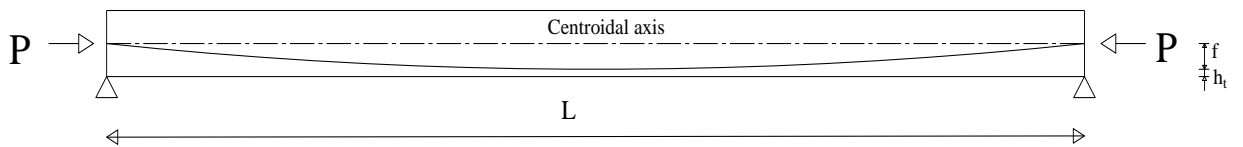


Figure 3-21: Rail load on two tracks ultimate moment diagram for 50m span

3.3.3 Prestressing tendons

3.3.3.1 Layout prestressing tendons



The tendon vertical profile is parabolic with the following ordinate from center:

Tendon distance	0.0	4.17	8.33	12.5	16.67	20.8	25.0	29.17	33.3	37.5	41.67	45.8	50.0
Vertical ordinate	0.0	0.52	0.95	1.27	1.51	1.65	1.70	1.65	1.51	1.27	0.95	0.52	0.0

Distance between the center of the tendons and bottom side at mid-span $h_t = 0.218m$ and

Tendon eccentricity at mid-span $Z_{cp,m} = Z_{cb} - h_t = 1.70m$

3.3.3.2 Prestressing value

$$P = 66504kN$$

Initial Prestress force

$$\text{Total Area of tendon required } A_{ten,total} = P / \sigma_{pm0}$$

$$48900mm^2$$

$$\text{Number of tendon} = A_{ten,total} / A_p$$

18 number of tendon

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3.3.3.3 Prestressing losses

Summary of Total prestressing losses

Tendon distance (m)	prestress force (kN) Prior to setting	prestress force (kN) After long term losses
0	66504.00	60427.28
2	65742.02	59665.30
4	64942.88	58866.16
5	64154.02	58077.30
7	63375.26	57298.54
9	62606.41	56529.69
11	61847.30	55770.59
13	61097.76	55021.04
14	60357.61	54280.89
16	59626.69	53549.98
18	58904.84	52828.13
20	58191.90	52115.19
22	57487.72	51411.00
23	56792.14	50715.43
25	56105.02	50028.30
27	55426.21	49349.50
29	54755.58	48678.86
30	54092.98	48016.26
32	53438.28	47361.56
34	52791.35	46714.63
36	52152.05	46075.34
38	51520.28	45443.56
39	50895.89	44819.17
41	50278.77	44202.05
43	49668.81	43592.09
45	49065.88	42989.16
47	48469.88	42393.16
48	47880.70	41803.98
50	47332.09	41255.37

3.3.4 Service stresses check

Tensile Stress check immediately after prestressing, top side

$$\sigma_{ct\max} = \frac{p_0}{A_c} - \frac{M_{m,p0}}{W_t} + \frac{M_{self}}{W_t} = +2.26\text{Mpa} > -2.74\text{Mpa} \rightarrow \text{Ok}$$

Compressive Stress check under working load, top side

$$\sigma_{ct\max} = \frac{p_\infty}{A_c} - \frac{M_{m,p\infty}}{W_t} + \frac{M_{self}}{W_t} + \frac{M_{utility}}{W_t} + \frac{M_{live}}{W_t} = 16.34\text{Mpa} < 28.33\text{Mpa} \rightarrow \text{Ok}$$

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Where:

$$\alpha_c = (1 + \sigma_{cp} / f_{cd})$$

$$\text{For } 0 \leq \sigma_{cp} \leq 0.25 f_{cd}$$

$$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$$

Girder section distance in (m)	V_{Ed} in (kN)	required A_{sw} in (mm^2 / m)	Type of bar	center to center spacing
0	3203.92	430	Double $\Phi 16$	93cm C/C
1	3121.04	430	Double $\Phi 16$	93cm C/C
2	2890.40	430	Double $\Phi 16$	93cm C/C
3	2661.93	430	Double $\Phi 16$	93cm C/C
4	2566.08	2629	Double $\Phi 16$	15cm C/C
5	2483.21	2544	Double $\Phi 16$	16cm C/C
6	2313.80	2371	Double $\Phi 16$	17cm C/C
7	2095.31	2147	Double $\Phi 16$	19cm C/C
8	2132.80	2185	Double $\Phi 16$	18cm C/C
9	2108.49	2160	Double $\Phi 16$	18cm C/C
10	2025.61	2075	Double $\Phi 16$	19cm C/C
11	1816.96	1862	Double $\Phi 16$	21cm C/C
12	1769.40	1813	Double $\Phi 16$	22cm C/C
13	1847.23	1893	Double $\Phi 16$	21cm C/C
14	1764.35	1808	Double $\Phi 16$	22cm C/C
15	1565.27	1604	Double $\Phi 16$	25cm C/C
16	1442.73	430	Double $\Phi 16$	93cm C/C
17	1608.42	1648	Double $\Phi 16$	24cm C/C
18	1525.54	1563	Double $\Phi 16$	26cm C/C
19	1335.41	430	Double $\Phi 16$	93cm C/C
20	1147.88	430	Double $\Phi 16$	93cm C/C
21	1385.83	430	Double $\Phi 16$	93cm C/C
22	1302.96	430	Double $\Phi 16$	93cm C/C
23	1120.93	430	Double $\Phi 16$	93cm C/C
24	1108.51	430	Double $\Phi 16$	93cm C/C
25	1191.39	430	Double $\Phi 16$	93cm C/C
26	1101.34	430	Double $\Phi 16$	93cm C/C
27	1128.10	430	Double $\Phi 16$	93cm C/C
28	1310.12	430	Double $\Phi 16$	93cm C/C
29	1393.00	430	Double $\Phi 16$	93cm C/C
30	1210.20	430	Double $\Phi 16$	93cm C/C
31	1397.73	430	Double $\Phi 16$	93cm C/C
32	1587.86	1627	Double $\Phi 16$	25cm C/C
33	1670.74	1712	Double $\Phi 16$	23cm C/C

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34	1582.55	1621	Double $\Phi 16$	25cm C/C
35	1745.13	1788	Double $\Phi 16$	22cm C/C
36	1944.20	1992	Double $\Phi 16$	20cm C/C
37	2027.08	2077	Double $\Phi 16$	19cm C/C
38	2041.06	2091	Double $\Phi 16$	19cm C/C
39	2180.82	2234	Double $\Phi 16$	18cm C/C
40	2389.45	2448	Double $\Phi 16$	16cm C/C
41	2494.01	2555	Double $\Phi 16$	16cm C/C
42	2576.89	2640	Double $\Phi 16$	15cm C/C
43	2704.93	2771	Double $\Phi 16$	14cm C/C
44	2923.43	2995	Double $\Phi 16$	13cm C/C
45	3129.92	3207	Double $\Phi 16$	12cm C/C
46	3275.76	3356	Double $\Phi 16$	12cm C/C
47	3501.78	3588	Double $\Phi 16$	11cm C/C
48	3730.25	3822	Double $\Phi 16$	10cm C/C
49	3960.90	4058	Double $\Phi 16$	10cm C/C
50	4043.77	4143	Double $\Phi 16$	10cm C/C

3.3.6.1.1 Additional longitudinal reinforcement due to shear

Girder section distance in (m)	V_{Ed} in (kN)	required A_{sl} in (mm^2)	Number of Bar and type
0	3182.72	0	-
1	3099.84	0	-
2	2869.18	0	-
3	2640.72	0	-
4	2553.18	3208	16 $\Phi 16$
5	2470.30	3104	15 $\Phi 16$
6	2302.30	2892	14 $\Phi 16$
7	2083.88	2619	13 $\Phi 16$
8	2123.08	2666	13 $\Phi 16$
9	2099.12	2636	13 $\Phi 16$
10	2016.24	2532	13 $\Phi 16$
11	1808.26	2271	11 $\Phi 16$
12	1761.87	2212	11 $\Phi 16$
13	1840.42	2309	11 $\Phi 16$
14	1757.55	2205	11 $\Phi 16$
15	1558.19	1957	10 $\Phi 16$
16	1436.80	0	-
17	1603.95	2011	10 $\Phi 16$
18	1521.07	1907	9 $\Phi 16$
19	1331.03	0	-
20	1143.57	0	-
21	1384.12	0	-

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22	1301.24	0	-
23	1119.25	0	-
24	1110.13	0	-
25	1193.01	0	-
26	1103.28	0	-
27	1126.10	0	-
28	1308.08	0	-
29	1390.96	0	-
30	1208.68	0	-
31	1396.13	0	-
32	1586.07	1985	10Φ16
33	1668.94	2088	10Φ16
34	1575.85	1978	10Φ16
35	1735.76	2181	11Φ16
36	1934.79	2430	12Φ16
37	2017.67	2534	13Φ16
38	2033.02	2551	13Φ16
39	2173.34	2726	14Φ16
40	2381.84	2987	15Φ16
41	2486.97	3118	15Φ16
42	2569.84	3221	16Φ16
43	2698.10	3381	17Φ16
44	2916.53	3654	18Φ16
45	3122.13	3912	19Φ16
46	3263.27	4095	20Φ16
47	3489.28	4377	22Φ16
48	3717.74	4663	23Φ16
49	3948.40	4951	24Φ16
50	4031.28	5055	25Φ16

3.3.6.2 Torsion

$$\rho_{w,\min} = A_{t,\min} / (Sb_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk} \quad \text{Minimum torsion reinforcement ratio}$$

$$A_{t,\min} = 430 \text{mm}^2 / \text{m} \quad \text{Minimum area of torsion reinforcement}$$

$$S_{\max} = 35 \text{cm} \leq \begin{cases} u/8 \\ b_w \\ S_{\max} = 0.75d(1 + \cot \alpha) \end{cases} \quad \text{Longitudinal spacing of torsional link}$$

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Girder section distance in (m)	A_{st} required for each inner and outer link leg (mm^2 / m)	Bar and center to center spacing	A_{st} required for each inner and outer layer (mm^2 / m)	Number of Bar and type
0	735	Double $\Phi 16$ C/C 27cm	735	4 $\Phi 16$
1	735	Double $\Phi 16$ C/C 27cm	735	4 $\Phi 16$
2	710	Double $\Phi 16$ C/C 28cm	710	4 $\Phi 16$
3	692	Double $\Phi 16$ C/C 29cm	692	4 $\Phi 16$
4	676	Double $\Phi 16$ C/C 30cm	676	4 $\Phi 16$
5	661	Double $\Phi 16$ C/C 30cm	661	4 $\Phi 16$
6	651	Double $\Phi 16$ C/C 31cm	651	4 $\Phi 16$
7	643	Double $\Phi 16$ C/C 31cm	643	4 $\Phi 16$
8	635	Double $\Phi 16$ C/C 31cm	635	3 $\Phi 16$
9	628	Double $\Phi 16$ C/C 32cm	628	3 $\Phi 16$
10	622	Double $\Phi 16$ C/C 32cm	622	3 $\Phi 16$
11	617	Double $\Phi 16$ C/C 32cm	617	3 $\Phi 16$
12	612	Double $\Phi 16$ C/C 33cm	612	3 $\Phi 16$
13	608	Double $\Phi 16$ C/C 33cm	608	3 $\Phi 16$
14	604	Double $\Phi 16$ C/C 33cm	604	3 $\Phi 16$
15	600	Double $\Phi 16$ C/C 33cm	600	3 $\Phi 16$
16	597	Double $\Phi 16$ C/C 34cm	597	3 $\Phi 16$
17	595	Double $\Phi 16$ C/C 34cm	595	3 $\Phi 16$
18	592	Double $\Phi 16$ C/C 34cm	592	3 $\Phi 16$
19	590	Double $\Phi 16$ C/C 34cm	590	3 $\Phi 16$
20	588	Double $\Phi 16$ C/C 34cm	588	3 $\Phi 16$
21	587	Double $\Phi 16$ C/C 34cm	587	3 $\Phi 16$
22	586	Double $\Phi 16$ C/C 34cm	586	3 $\Phi 16$
23	585	Double $\Phi 16$ C/C 34cm	585	3 $\Phi 16$
24	584	Double $\Phi 16$ C/C 34cm	584	3 $\Phi 16$
25	584	Double $\Phi 16$ C/C 34cm	584	3 $\Phi 16$
26	584	Double $\Phi 16$ C/C 34cm	584	3 $\Phi 16$
27	585	Double $\Phi 16$ C/C 34cm	585	3 $\Phi 16$
28	586	Double $\Phi 16$ C/C 34cm	586	3 $\Phi 16$
29	587	Double $\Phi 16$ C/C 34cm	587	3 $\Phi 16$
30	588	Double $\Phi 16$ C/C 34cm	588	3 $\Phi 16$
31	590	Double $\Phi 16$ C/C 34cm	590	3 $\Phi 16$
32	592	Double $\Phi 16$ C/C 34cm	592	3 $\Phi 16$
33	595	Double $\Phi 16$ C/C 34cm	595	3 $\Phi 16$
34	597	Double $\Phi 16$ C/C 34cm	597	3 $\Phi 16$
35	600	Double $\Phi 16$ C/C 33cm	600	3 $\Phi 16$
36	604	Double $\Phi 16$ C/C 33cm	604	3 $\Phi 16$
37	608	Double $\Phi 16$ C/C 33cm	608	3 $\Phi 16$
38	612	Double $\Phi 16$ C/C 33cm	612	3 $\Phi 16$
39	617	Double $\Phi 16$ C/C 32cm	617	3 $\Phi 16$

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40	622	Double $\Phi 16$ C/C 32cm	622	3 $\Phi 16$
41	628	Double $\Phi 16$ C/C 32cm	628	3 $\Phi 16$
42	635	Double $\Phi 16$ C/C 31cm	635	3 $\Phi 16$
43	643	Double $\Phi 16$ C/C 31cm	643	4 $\Phi 16$
44	651	Double $\Phi 16$ C/C 31cm	651	4 $\Phi 16$
45	661	Double $\Phi 16$ C/C 30cm	661	4 $\Phi 16$
46	676	Double $\Phi 16$ C/C 30cm	676	4 $\Phi 16$
47	692	Double $\Phi 16$ C/C 29cm	692	4 $\Phi 16$
48	710	Double $\Phi 16$ C/C 28cm	710	4 $\Phi 16$
49	735	Double $\Phi 16$ C/C 27cm	735	4 $\Phi 16$
50	735	Double $\Phi 16$ C/C 27cm	735	4 $\Phi 16$

3.3.6.3 Ultimate moment resistance check

Box Girder section distance in (m)	Positive moment resistance of section (M_{RT}) in (kNm)	Positive demand moment for the girder (M_{DE}) in (kNm)	Ratio (M_{DE}/M_{RT})
0	74203.58	0.00	0.00
2	95762.92	23500.94	0.25
4	115462.21	44749.21	0.39
6	132682.09	63798.82	0.48
8	148179.97	80864.19	0.55
10	161955.87	95229.02	0.59
12	174009.78	108089.36	0.62
14	184341.71	118191.36	0.64
16	192951.64	127747.46	0.66
18	199839.59	132450.55	0.66
20	205005.55	137762.91	0.67
22	208449.53	141278.55	0.68
24	210171.52	142941.53	0.68
25	210171.52	142941.53	0.68
26	208449.53	141278.55	0.68
28	205005.55	137762.91	0.67
30	199839.59	132450.55	0.66
32	192951.64	126359.50	0.65
34	184341.71	118191.36	0.64
36	174009.78	107755.31	0.62
38	161955.87	95229.02	0.59
40	148179.97	80605.16	0.54
42	132682.09	63798.82	0.48
44	115462.21	44749.21	0.39
46	95762.92	23500.94	0.25
48	74203.58	0.00	0.00
50	74203.58	0.00	0.00

At all girder section moment resistance is greater than the demand external moment no ultimate state additional reinforcement is required.

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CHAPTER 4 FINDINGS OF THE STUDY

The finding of the study presented in such a way that the objectives set forth in Chapter 1 are met. In this regard, six analysis and design carried out for each span length using, C-30, C-40, C-50 concrete grade and possible superstructure depth.

4.1 Concrete and prestress tendon quantity

Consequently, summary of concrete volume required along with prestress tendon area of the whole design output presented in (table 4-1).

Table 4-1: Design output for each span length and possible depth

30m span length Quantity									
Depth of Girder in (m)	Concrete Grade	initial prestress at post tension end (kN)	mid span prestress immediately after setting (kN)	mid span prestress after long term loss (kN)	Long term loss (kN)	% of long term loss	concrete volume (m ³)	Tendon area (mm ²)	mass of tendon (kg)
1.80	C-30	38760	34734.91	30675.42	4059.49	11.69	122.184	28500	6711.75
1.80	C-40	37536	33637.63	30175.9	3461.73	10.29	122.184	27600	6499.8
1.80	C-50	36312	32539.36	29570.02	2969.34	9.13	122.184	26700	6287.85
1.75	C-40	38760	34755.89	31142.14	3613.75	10.40	121.134	28500	6711.75
1.75	C-50	37536	33657.94	30559.44	3098.5	9.21	121.134	27600	6499.8
1.70	C-50	39168	35145.85	31869.14	3276.71	9.32	120.084	28800	6782.4
40m span length Quantity									
2.50	C-30	51000	44294.11	38655.49	5638.62	12.73	182.512	37500	11775
2.50	C-40	48960	42512.28	37798.43	4713.85	11.09	182.512	36000	11304
2.50	C-50	47736	41450.29	37390.09	4060.2	9.80	182.512	35100	11021.4
2.40	C-40	51408	44683.83	39632.69	5051.14	11.30	179.712	37800	11869.2
2.40	C-50	49776	43260.64	38965.31	4295.33	9.93	179.712	36600	11492.4
2.35	C-50	51408	44708.47	40212.43	4496.04	10.06	178.312	37800	11869.2
50m span length Quantity									
3.20	C-30	67728	57134.1	49185.4	7948.7	13.91	252.64	49800	19546.5
3.20	C-40	64872	54718.42	48110.66	6607.76	12.08	252.64	47700	18722.25
3.20	C-50	62832	52997.95	47390.58	5607.37	10.58	252.64	46200	18133.5
3.10	C-40	67320	56828.08	49848.11	6979.97	12.28	249.14	49500	19428.75
3.10	C-50	64872	54754.79	48933.11	5821.68	10.63	249.14	47700	18722.25
3.05	C-50	66504	56158.86	50082.14	6076.72	10.82	247.39	48900	19193.25
60m span length Quantity									
3.95	C-30	86088	70466.92	59808.5	10658.42	15.13	334.67	63300	29814.3
3.95	C-40	82416	67451.80	58634.67	8817.13	13.07	334.67	60600	28542.6
3.95	C-50	79152	64772.04	57424.1	7347.94	11.34	334.67	58200	27412.2
3.80	C-40	86088	70524.54	61100.88	9423.66	13.36	328.37	63300	29814.3
3.80	C-50	82824	67841.08	59989.56	7851.52	11.57	328.37	60900	28683.9
3.75	C-50	83640	68528.08	60560.56	7967.52	11.63	326.27	61500	28966.5

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4.2 Non-prestressed reinforcements quantity

Non prestress reinforcement consider for comparative study includes shear and torsional reinforcements given there quantity in table 4-2.

Table 4-2: Reinforcement quantity summary (in kg)

30m span reinforcement quantity						
Depth of Girder	Concrete Grade	Total mass Shear reinforcement	Total mass of additional reinforcement due to Shear	Total mass of Torsional reinforcement		Total
				transverse	longitudinal	
1.8	C-30	2178.2	224.1	1446.2	366.1	4214.6
1.8	C-40	2194.8	224.1	1446.2	366.1	4231.2
1.8	C-50	2099.7	208.3	1446.2	366.1	4120.3
1.75	C-40	2195.0	224.1	1453.6	372.4	4245.0
1.75	C-50	2098.4	209.9	1453.6	372.4	4134.3
1.7	C-50	2123.0	216.2	1461.8	378.7	4179.7
40m span reinforcement quantity						
2.50	C-30	3558.0	372.4	2226.4	410.3	6567.0
2.50	C-40	3402.1	345.6	2226.4	410.3	6384.4
2.50	C-50	2999.6	301.4	2226.4	410.3	5937.7
2.40	C-40	3432.4	359.8	2237.6	441.8	6471.7
2.40	C-50	2936.5	326.6	2237.6	441.8	5942.6
2.35	C-50	3067.9	312.4	2243.7	467.1	6091.1
50m span reinforcement quantity						
3.20	C-30	5429.5	620.2	3147.7	463.9	9661.3
3.20	C-40	5121.4	550.7	3147.7	463.9	9283.7
3.20	C-50	4498.3	438.7	3147.7	463.9	8548.6
3.10	C-40	5284.4	534.9	3156.6	479.7	9455.6
3.10	C-50	4523.3	411.9	3156.6	479.7	8571.4
3.05	C-50	4518.8	407.1	3161.0	486.0	8573.0
60m span reinforcement quantity						
3.95	C-30	7377.6	831.6	4250.1	520.7	12980.0
3.95	C-40	6582.9	703.8	4250.1	520.7	12057.5
3.95	C-50	6089.4	588.6	4250.1	520.7	11448.8
3.80	C-40	6991.8	719.6	4260.8	517.6	12489.7
3.80	C-50	6254.5	601.2	4260.8	517.6	11634.1
3.75	C-50	6249.8	602.8	4264.3	517.6	11634.5

4.3 Cost estimation

Summary of the cost estimation based on unit rate cost are given in the table 4-3, Cost assumption and detailed breakdown for each concrete grade attached in appendix C, in congestion with quantity summary tables serves as an input for this estimation.

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Table 4-3: Summary of cost estimations (in Birr)

30m span cost estimation									
Depth of Girder	Concrete Grade	Shear reinforcement cost	cost of additional reinforcement due to Shear	Torsional reinforcement cost		Tendon cost	concrete cost	Total	cost/span length
				transverse	longitudinal				
1.8	C-30	102376.4269	10531.57	67970.90	17206.51	1,421,045.27	585,506.78	2,204,637.46	73,487.92
1.8	C-40	103157.586	10531.57	67970.90	17206.51	1,376,170.16	612,064.71	2,187,101.44	72,903.38
1.8	C-50	98687.86027	9789.91	67970.90	17206.51	1,331,295.04	657,778.96	2,182,729.19	72,757.64
1.75	C-40	103163.1559	10531.57	68319.41	17503.18	1,421,045.27	606,804.87	2,227,367.46	74,245.58
1.75	C-50	98625.8161	9864.08	68319.41	17503.18	1,376,170.16	652,126.28	2,222,608.91	74,086.96
1.7	C-50	99780.93696	10160.74	68705.25	17799.84	1,436,003.64	646,473.59	2,278,924.00	75,964.13
40m span cost estimation									
2.50	C-30	167224.4656	17503.18	104640.41	19283.16	2,493,061.88	874,599.08	3,676,312.16	91,907.80
2.50	C-40	159899.0111	16242.35	104640.41	19283.16	2,393,339.40	914,269.91	3,607,674.24	90,191.86
2.50	C-50	140982.2518	14165.71	104640.41	19283.16	2,333,505.92	982,555.44	3,595,132.89	89,878.32
2.40	C-40	161324.2197	16909.85	105169.08	20766.48	2,513,006.37	900,243.67	3,717,419.67	92,935.49
2.40	C-50	138013.7221	15352.36	105169.08	20766.48	2,433,228.39	967,481.61	3,680,011.64	92,000.29
2.35	C-50	144191.8389	14684.87	105452.96	21953.14	2,513,006.37	959,944.69	3,759,233.87	93,980.85
50m span cost estimation									
3.20	C-30	255188.3852	29147.24	147940.31	21804.80	4138482.713	1,210,653.06	5,803,216.51	116,064.33
3.20	C-40	240705.7555	25883.93	147940.31	21804.80	3963968.381	1,265,566.92	5,665,870.10	113,317.40
3.20	C-50	211421.6803	20618.15	147940.31	21804.80	3839315.288	1,360,090.33	5,601,190.56	112,023.81
3.10	C-40	248366.846	25142.27	148357.87	22546.46	4113552.094	1,248,034.13	5,805,999.68	116,119.99
3.10	C-50	212593.6955	19357.33	148357.87	22546.46	3963968.381	1,341,248.04	5,708,071.78	114,161.44
3.05	C-50	212382.5904	19134.83	148568.42	22843.13	4063690.856	1,331,826.90	5,798,446.72	115,968.93
60m span cost estimation									
3.95	C-30	346747.7684	39085.48	199754.20	24474.78	6312432.668	1,603,731.94	8,526,226.84	142,103.78
3.95	C-40	309395.1842	33078.04	199754.20	24474.78	6043181.985	1,676,475.42	8,286,359.61	138,105.99
3.95	C-50	286201.4345	27663.92	199754.20	24474.78	5803848.045	1,801,689.01	8,143,631.39	135,727.19
3.80	C-40	328612.5369	33819.70	200255.87	24326.45	6312432.668	1,644,916.39	8,544,363.61	142,406.06
3.80	C-50	293961.1613	28257.25	200255.87	24326.45	6073098.728	1,767,772.89	8,387,672.34	139,794.54
3.75	C-50	293741.6828	28331.41	200419.80	24326.45	6132932.213	1,756,467.51	8,436,219.06	140,603.65

4.4 Comparative study

Table 4-4, show increment or reduction change in design parameter and cost for any specific possible alternative change in concrete grade used for each span length considered.

Note: - the increment or reduction value evaluated in the table is for change in concrete grade order from second column of (table 4-4) to fourth column of the same table. Negative values in the table show reduction whereas, positive value shows an increment in quantity of design parameter or cost.

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Table 4-4: 30m span length comparative study

30m span length Quantity										
Depth of Girder in (m)	Concrete Grade from	Depth of Girder in (m)	Concrete Grade to	change in concrete volume (m ³)	change in concrete cost (Birr)	change in tendon area mm ²	change in tendon cost (Birr)	change in reinforcement mass quantity (kg)	change in reinforcement cost (Birr)	change in total cost (Birr)
1.80	C-30	1.80	C-40	0.00	26,557.93	-900.00	-44,875.11	16.62	781.16	-17,536.03
		1.80	C-50	0.00	72,272.18	-1800.00	-89,750.23	-94.26	-4,430.23	-21,908.27
		1.75	C-40	-1.05	21,298.09	0.00	0.00	30.47	1,431.90	22,730.00
		1.75	C-50	-1.05	66,619.50	-900.00	-44,875.11	-80.28	-3,772.93	17,971.45
		1.70	C-50	-2.10	60,966.81	300.00	14,958.37	-34.86	-1,638.64	74,286.54
1.80	C-40	1.80	C-50	0.00	45,714.25	-900.00	-44,875.11	-110.88	-5,211.39	-4,372.25
		1.75	C-40	-1.05	-5,259.84	900.00	44,875.11	13.85	650.74	40,266.02
		1.75	C-50	-1.05	40,061.57	0.00	0.00	-96.90	-4,554.09	35,507.48
		1.70	C-50	-2.10	34,408.88	1200.00	59,833.49	-51.49	-2,419.80	91,822.56
1.80	C-50	1.75	C-40	-1.05	-50,974.09	1800	89,750.23	124.73	5,862.13	44,638.27
		1.75	C-50	-1.05	-5,652.69	900	44,875.11	13.99	657.30	39,879.72
		1.70	C-50	-2.10	-11,305.37	2100	104,708.60	59.40	2,791.58	96,194.81
1.75	C-40	1.75	C-50	0.00	45,321.40	-900	-44,875.11	-110.74	-5,204.83	-4,758.54
		1.70	C-50/60	-1.05	39,668.72	300	14,958.37	-65.33	-3,070.55	51,556.54
1.75	C-50	1.70	C-50	-1.05	-5,652.69	1200	59,833.49	45.41	2,134.29	56,315.09

Table 4-5: 40m span length comparative study

40m span length Quantity											
Depth of Girder in (m)	Concrete Grade from	Depth of Girder in (m)	Concrete Grade to	change in % of long term loss	change in concrete volume (m ³)	change in concrete cost (Birr)	change in tendon area mm ²	change in tendon cost (Birr)	change in reinforcement mass quantity (kg)	change in reinforcement cost (Birr)	change in total cost (Birr)
2.50	C-30/40	2.50	C-40/50	-1.64	0.00	39,670.83	-1500.00	-99,722.48	-182.69	-8,586.28	-68,637.92
		2.50	C-50/60	-2.93	0.00	107,956.37	-2400.00	-159,555.96	-629.35	-29,579.68	-81,179.28
		2.40	C-40/50	-1.43	-2.80	25,644.60	300.00	19,944.49	-95.35	-4,481.59	41,107.51
		2.40	C-50/60	-2.80	-2.80	92,882.53	-900.00	-59,833.49	-624.46	-29,349.57	3,699.48
		2.35	C-50/60	-2.67	-4.20	85,345.62	300.00	19,944.49	-475.92	-22,368.41	82,921.70
2.50	C-40/50	2.50	C-50/60	-1.29	0.00	68,285.53	-900.00	-59,833.48	-446.67	-20,993.41	-12,541.36
		2.40	C-40/50	0.22	-2.80	-14,026.23	1800.00	119,666.97	87.33	4,104.69	109,745.43
		2.40	C-50/60	-1.16	-2.80	53,211.70	600.00	39,888.99	-441.77	-20,763.29	72,337.40
		2.35	C-50/60	-1.03	-4.20	45,674.79	1800.00	119,666.97	-293.24	-13,782.13	151,559.62
2.50	C-50/60	2.40	C-40/50	1.51	-2.80	-82,311.77	2700	179,500.46	534.00	25,098.10	122,286.78
		2.40	C-50/60	0.13	-2.80	-15,073.83	1500	99,722.48	4.90	230.11	84,878.76
		2.35	C-50/60	0.26	-4.20	-22,610.75	2700	179,500.46	153.43	7,211.28	164,100.98
2.40	C-40/50	2.40	C-50/60	-1.38	0.00	67,237.94	-1200	-79,777.98	-529.11	-24,867.98	-37,408.03
		2.35	C-50/60	-1.25	-1.40	59,701.02	0	0.00	-380.57	-17,886.82	41,814.20
2.40	C-50/60	2.35	C-50/60	0.13	-1.40	-7,536.92	1200	79,777.98	148.54	6,981.16	79,222.23

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Table 4-6: 50m span length comparative study

50m span length Quantity											
Depth of Girder in (m)	Concrete Grade from	Depth of Girder in (m)	Concrete Grade to	change in % of long term loss	change in concrete volume (m ³)	change in concrete cost (Birr)	change in tendon area mm ²	change in tendon cost (Birr)	change in reinforcement mass quantity (kg)	change in reinforcement cost (Birr)	change in total cost (Birr)
3.20	C-30/40	3.20	C-40/50	-1.84	0.00	54,913.86	-2100.00	-174,514.33	-377.57	-17,745.93	-137,346.40
		3.20	C-50/60	-3.33	0.00	149,437.28	-3600.00	-299,167.43	-1112.68	-52,295.79	-202,025.94
		3.10	C-40/50	-1.63	-3.50	37,381.07	-300.00	-24,930.62	-205.69	-9,667.28	2,783.17
		3.10	C-50/60	-3.28	-3.50	130,594.99	-2100.00	-174,514.33	-1089.90	-51,225.38	-95,144.72
		3.05	C-50/60	-3.09	-5.25	121,173.84	-900.00	-74,791.86	-1088.34	-51,151.77	-4,769.78
3.20	C-40/50	3.20	C-50/60	-1.50	0.00	94,523.41	-1500.00	-124,653.09	-735.10	-34,549.86	-64,679.54
		3.10	C-40/50	0.21	-3.50	-17,532.79	1800.00	149,583.71	171.89	8,078.65	140,129.57
		3.10	C-50/60	-1.44	-3.50	75,681.12	0.00	0.00	-712.33	-33,479.45	42,201.68
		3.05	C-50/60	-1.26	-5.25	66,259.98	1200.00	99,722.47	-710.76	-33,405.83	132,576.62
3.20	C-50/60	3.10	C-40/50	1.70	-3.50	-112,056.21	3300	274,236.81	906.99	42,628.51	204,809.11
		3.10	C-50/60	0.05	-3.50	-18,842.29	1500	124,653.09	22.77	1,070.42	106,881.22
		3.05	C-50/60	0.24	-5.25	-28,263.44	2700	224,375.57	24.34	1,144.03	197,256.16
3.10	C-40/50	3.10	C-50/60	-1.65	0.00	93,213.92	-1800	-149,583.71	-529.11	-41,558.10	-97,927.90
		3.05	C-50/60	-1.46	-1.75	83,792.77	-600	-49,861.24	-380.57	-41,484.49	-7,552.95
3.10	C-50/60	3.05	C-50/60	0.19	-1.75	-9,421.15	1200	99,722.47	1.57	73.61	90,374.94

Table 4-7: 60m span length comparative study

60m span length Quantity											
Depth of Girder in (m)	Concrete Grade from	Depth of Girder in (m)	Concrete Grade to	change in % of long term loss	change in concrete volume (m ³)	change in concrete cost (Birr)	change in tendon area mm ²	change in tendon cost (Birr)	change in reinforcement mass quantity (kg)	change in reinforcement cost (Birr)	change in total cost (Birr)
3.95	C-30/40	3.95	C-40/50	-2.05	-2.05	72,743.48	-2700.00	-269,250.68	-922.55	-43,360.03	-239,867.23
		3.95	C-50/60	-3.78	-3.78	197,957.07	-5100.00	-508,584.62	-1531.23	-71,967.90	-382,595.45
		3.80	C-40/50	-1.76	-1.76	41,184.46	0.00	0.00	-490.38	-23,047.68	18,136.77
		3.80	C-50/60	-3.55	-3.55	164,040.95	-2400.00	-239,333.94	-1345.99	-63,261.51	-138,554.50
		3.75	C-50/60	-3.50	-3.50	152,735.58	-1800.00	-179,500.46	-1345.59	-63,242.90	-90,007.78
3.95	C-40/50	3.95	C-50/60	-1.73	0.00	125,213.59	-2400.00	-239,333.94	-608.68	-28,607.87	-142,728.21
		3.80	C-40/50	0.29	-6.30	-31,559.02	2700.00	269,250.68	432.18	20,312.35	258,004.01
		3.80	C-50/60	-1.50	-6.30	91,297.47	300.00	29,916.74	-423.44	-19,901.48	101,312.73
		3.75	C-50/60	-1.45	-8.40	79,992.10	900.00	89,750.23	-423.04	-19,882.87	149,859.46
3.95	C-50/60	3.80	C-40/50	2.02	-6.30	-156,772.62	5100	508,584.62	1040.86	48,920.21	400,732.22
		3.80	C-50/60	0.23	-6.30	-33,916.12	2700	269,250.68	185.24	8,706.39	244,040.95
		3.75	C-50/60	0.28	-8.40	-45,221.50	3300	329,084.17	185.64	8,725.00	292,587.67
3.80	C-40/50	3.80	C-50/60	-1.79	0.00	122,856.49	-2400	-239,333.94	-855.61	-40,213.83	-156,691.27
		3.75	C-50/60	-1.74	-2.10	111,551.12	-1800	-179,500.46	-855.22	-40,195.21	-108,144.55
3.80	C-50/60	3.75	C-50/60	0.05	-2.10	-11,305.37	600	59,833.49	0.40	18.61	48,546.72

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Table 4-8: Percentage of cost saving comparisons

Span length in meter		Cylindrical Concrete grade					
		C-30	C-40		C-50		
30	depth in meter	1.8	1.8	1.75	1.8	1.75	1.7
	% of cost saving	3.26%	4.03%	2.26%	4.22%	2.47%	-
40	depth in meter	2.5	2.5	2.4	2.5	2.4	2.35
	% of cost saving	2.21%	4.03%	1.11%	4.37%	2.11%	-
50	depth in meter	3.2	3.2	3.1	3.2	3.1	3.05
	% of cost saving	0.05%	2.41%	-	3.53%	1.69%	0.13%
60	depth in meter	3.95	3.95	3.8	3.95	3.8	3.75
	% of cost saving	0.21%	3.02%	-	4.69%	1.83%	1.27%

4.5 Summary of span findings

Referring to the summary of the results tabulated, the following main observation listed for each span length considered.

4.5.1 30m span length

For span length of 30m, the optimal depth of 1.7m can realized by using C-50 concrete grade with depth decrement of 10cm compared to 1.8m depth of C-30 concrete girder and based on cost assumptions a maximum of 4.22 % cost saving can be enhanced.

And for the same box girder depth considered varying the concrete grade from C-30 to C-40 and from C-40 to C-50, enhance a saving in prestressing tendon area of 900mm^2 , based on the cost assumptions this saving reduce the total cost more than the increment in concrete cost make higher concrete grade box girder more economical.

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Non prestress reinforcement variation behind the concrete grade used is very small and, hence the comparison is more of concrete cost and prestressing cost.



Figure 4-1: Cost comparison chart for 30m span length

4.5.2 40m span length

A total of 15cm box girder depth decrement can be enhanced by varying the concrete grade used from C-30 to C-50. Based on the assumption of prestress concrete material and construction, a maximum of 4.37% cost saving can be enhanced.

Keeping the same depth of girder, while varying the concrete grade from C-30 to C-40, and from C-40 to C-50, enhance a reduction of 1500mm² and 900mm² tendon area respectively. Which make the use of higher strength concrete more significantly economical than, their contribution in 30m span length bridge.

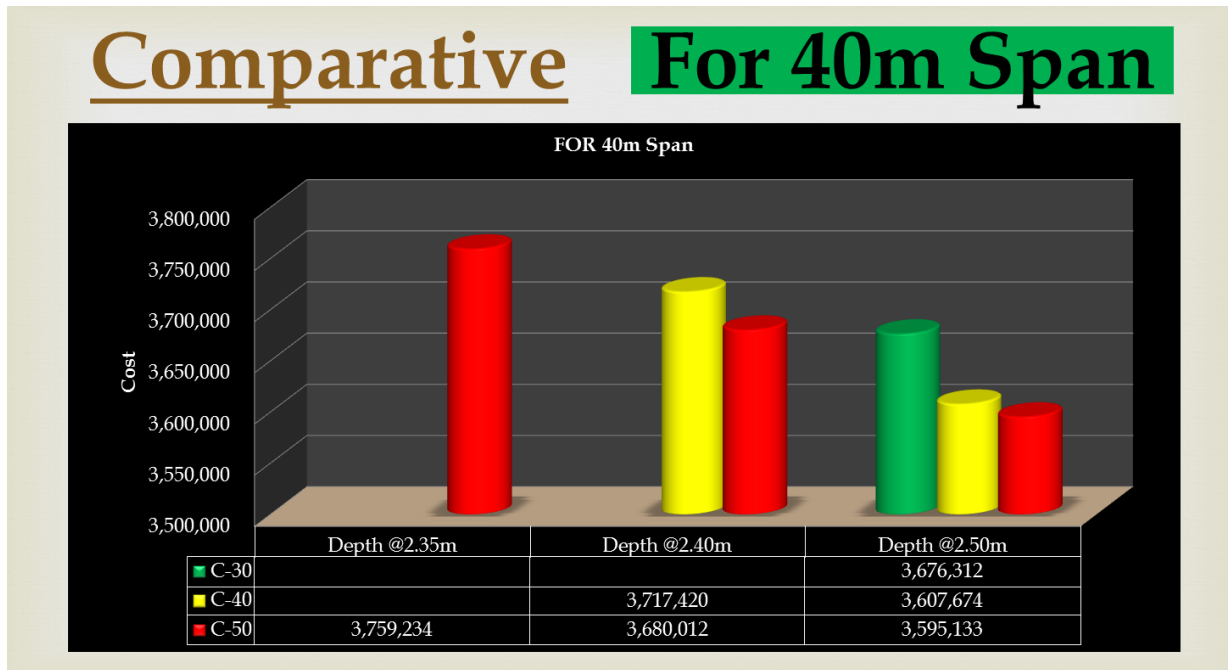


Figure 4-2: Cost comparison chart for 40m span length

4.5.3 50m span length

The reduction of 15cm depth can be made by varying the concrete grade from C-30 to C-50 with a comparative cost and based on cost assumption maximum percentage of cost saving is 3.53%

With the same depth provision 2100mm^2 tendon areas by changing girder concrete grade from C-30 to C-50 and 1500mm^2 from C-40 to C-50 can be reduced.

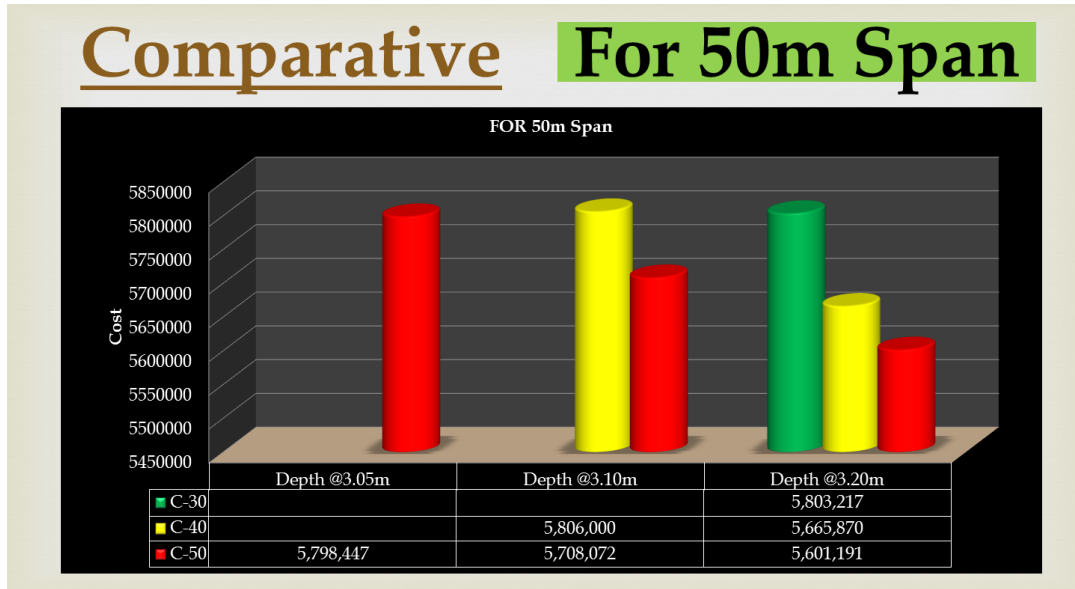


Figure 4-3: Cost comparison chart for 50m span length

4.5.4 60m span length

More than any of the span considered total box girder depth decrement of 20cm can be provided using concrete grade of C-50 with too comparative cost. More tendon area can be saved using higher concrete grade, for instance with the same depth provision 2700mm² and 2400mm² tendon areas can be saved by changing girder concrete grade from C-30 to C-40 and C-40 to C-50 respectively, and 4.69% maximum cost saving benefit can be enhanced.



Figure 4-4: Cost comparison chart for 60m span length

CHAPTER 5 CONCLUSION AND RECOMMENDATION

Concrete grade used in girder governs the prestressing layout, the amount of prestress in bridge girder, and stress limit flexibility.

For variable span lengths and concrete grade considered in this study, with the same possible box girder depth provision and by making the girder concrete strength higher for each span, saving in prestress tendon used amount and a bit more noticeable benefit from non prestress reinforcement reduction can be enhanced, particularly in context design to Euro code.

For variable span lengths and concrete grade considered, as the span get longer, using higher strength concrete for each span:

- More reduced depth can be provided (10cm, 15cm, 15cm and 20cm, total box girder depth decrement can be enhanced for span length of 30m, 40m, 50m and 60m, respectively by varying the concrete grade from 30MPa to 50MPa cylindrical compressive strength)
- More saving in prestress tendon amount and non-prestress reinforcement quantity can be made by keeping the box girder depth the same for each span.
- Based on cost assumptions, more depth decrement can be provided with comparative cost

Also, based on cost assumption, with the same possible box girder depth provision and varying girder concrete from 30MPa to 50MPa for each span length, makes the longer span length box girders, more cost effective or saving in cost/span ratio become significant

And, finally the user can input the unit rate cost of concrete, tendon, reinforcement in any of, table 4-4 to table 4-7, depending on span length, to make any specific cost comparison behind the concrete grade used.

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Appendix A: Statical Calculations Concrete Box Girder C-40

A.1 Introduction

This appendix presents detail calculation of optimal depth box girder in concrete C-40 for sample span length of 40m. In the scope of this study, the loads, load factors and load effects to which the sample box girder is subjected are treated in part A.4.4. In the followed sample calculations, the layout of the prestressing tendons is shown and the stresses in the box girder due to loading and prestressing are calculated. Also prestressing losses, as main concern of concrete grade used evaluated. Furthermore, this appendix includes the calculations on deflection, shear + torsion and the ultimate resistance moment of the sample box girder. The formulas and values used in the calculations are taken from [12] and other references, which are then stated in the text.

Note:-equation terms defined in chapter 3 are also valid for this appendix calculation.

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A.2 Material Characteristics

A.2.3 Concrete C40

Density of Concrete	ρ_c	2500 kg/m ³
Partial factor for concrete	γ_c	1.5
Coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied	α_{cc}	0.85
Compressive cylinder strength of concrete at 28 days	f_{ck}	40 N/mm ²
Modulus of elasticity of Concrete	E_c	3.5*10 ⁴ N/mm ²
Design Tensile strength of concrete	$f_{ctd} = f_{ctk,0.05} / \gamma_c$	6.67 N/mm ²

A.2.4 Reinforcing steel FeB 460

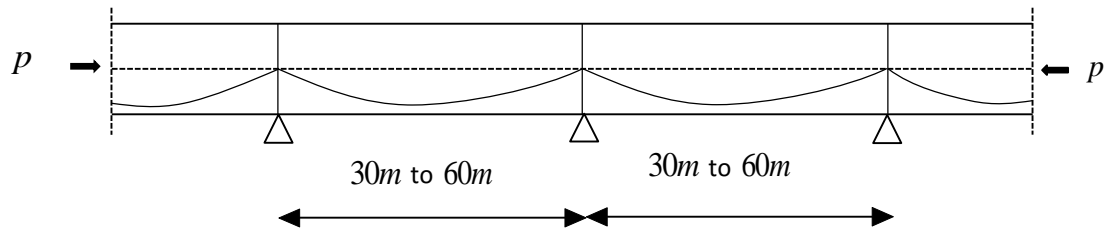
Density of reinforcing steel	ρ_s	7850 kg/m ³
Characteristic yield strength	f_{yk}	460 N/mm ²
Partial factor for reinforcing steel	γ_s	1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_s$	400 N/mm ²
Design value of modulus of elasticity of reinforcing steel	E_s	200,000 N/mm ²

A.2.5 Prestressing steel FeP 1860

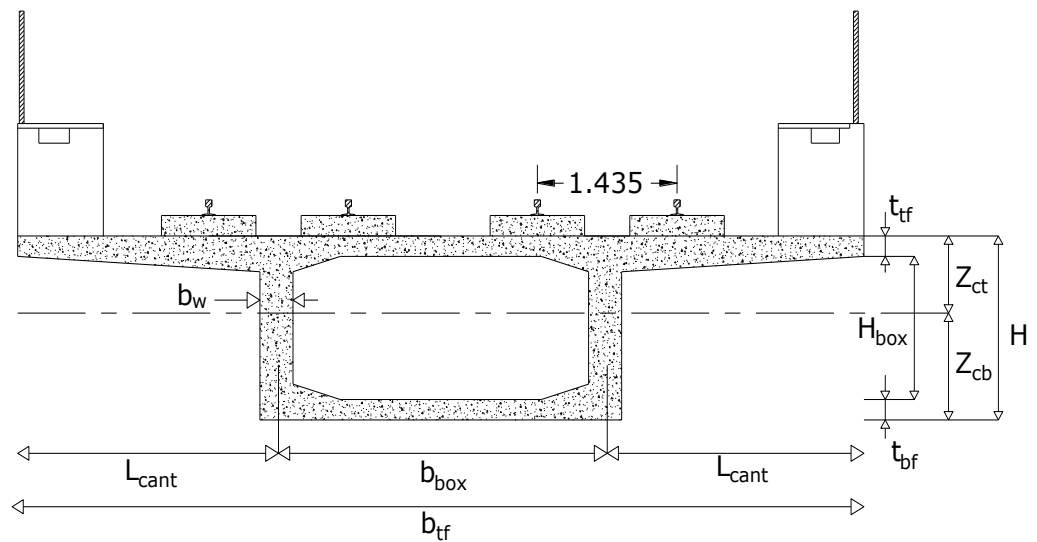
Density of prestressing steel	ρ_s	7850 kg/m ³
Characteristic tensile strength of prestressing steel	f_{pk}	1860 N/mm ²
Partial factor for prestressing steel	γ_s	1.15
Characteristic 0.1% proof-stress of prestressing steel	$f_{p0.1k}$	1600 N/mm ²
Design tensile strength of prestressing steel	$f_{pd} = f_{p0.1k} / \gamma_s$	1391 N/mm ²
Ultimate tensile strength of prestressing steel	f_{pk} / γ_s	1617 N/mm ²
Design value of modulus of elasticity of prestressing steel	E_p	195,000N/mm ²
Maximum tensile stress in the tendon (during tensioning)	min(0.8* f_{pk} or 0.9* $f_{p0.1k}$)	1440 N/mm ²
Maximum tensile stress in the tendon (Immediately after tensioning)	min(0.75* f_{pk} or 0.85* f_{pk})	1360 N/mm ²
Working stress after all losses	$\leq 0.6* f_{pk}$	1116 N/mm ²

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A.3 Geometry box girder



Statically determinate box girders supported by columns



Cross-section of the box girder

A.3.1 General

Length span	L	Variable from 30m to 60m
Depth box girder	H	Design Variable
Center to center girder spacing	b_{box}	3.83m
Thickness top flange	t_{tf}	0.2m
Width web	b_w	Design Variable
Thickness bottom flange	t_{bf}	0.2m
Cantilever length top flange	L_{cant}	2.74m
Width top flange	b_{tf}	8.96m

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A.3.2 Concrete cover

The concrete cover for this box girder made of C40 is:

$$C_{nom} = C_{min} + \Delta C_{dev} = 45mm \text{ Where:}$$

$$\text{Minimum cover } C_{min} = \max \{ C_{min,b}; C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add}; 10mm \} = 35mm$$

$C_{min,b}$ Minimum cover due to bond requirement;

Minimum cover = diameter of bar = 16mm, $C_{min,dur}$ Minimum cover due to environmental conditions; It is assumed that exposure class XF1 and XF3 can be classified as exposure class XD1 for the determination of the concrete cover. With the recommended structural class is S4, exposure class XD1, a design working life of 100 years, class C40, consideration for slab and an ensured special quality control of the concrete production. Therefor for slab class S3 $C_{min,dur} = 30 \text{ mm}$ and for girder S4 $C_{min,dur} = 35mm$.

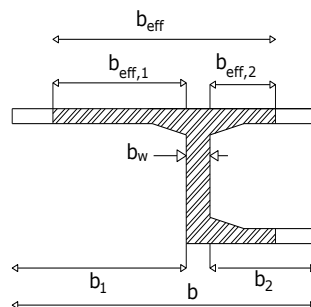
$\Delta C_{dur,\gamma}$ Additive safety element recommended value is 0mm. $\Delta C_{dur,st}$ Reduction for use of stainless steel recommended value is 0 mm. $\Delta C_{dur,add}$ reduction for use of additional protection recommended value is 0 mm. ΔC_{dev} allowance for deviation recommended value is 10mm. for consistency use the same cover for slab and girder

A.4 Sample calculation

A.4.1 Geometric property

Length span	L	40	m
Depth of girder	H	2.4	m
Width web	b_w	0.35	m

A.4.2 Effective width of flanges



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$$b_{eff} \leq \left\{ \begin{array}{l} \sum b_{eff,i} + b_w \\ b \end{array} \right\} \quad \text{Where: } b_{eff,i} \leq \left\{ \begin{array}{l} 0.2b_i + 0.1l_o \\ 0.2l_o \\ b_i \end{array} \right\}$$

Effective width of flanges		Value	
Effective width cantilever length top flange	$b_{eff,1}$	2.56	<i>m</i>
Effective width inner top flange	$b_{eff,2}$	1.56	<i>m</i>
Effective width bottom flange	$b_{eff,3}$	1.56	<i>m</i>
Total effective flange width		Value	
Effective width top flange	$b_{eff,t}$	8.96	<i>m</i>
Effective width bottom flange	$b_{eff,b}$	3.83	<i>m</i>

A.4.3 Cross-sectional properties

Cross-sectional property of concrete

Values cross-sectional properties box girder		Value	
Cross-sectional area of concrete	A_c	4.493	m^2
Distance from bottom to centroid axis	Z_{cb}	1.53	<i>m</i>
Distance from top to centroid axis	Z_{ct}	0.87	<i>m</i>
Second moment of area of the concrete section	I_c	3.556	m^4
Section modulus bottom	W_b	2.32	m^3
Section modulus top	W_t	4.10	m^3
Perimeter concrete box girder	u	22.47	<i>m</i>

A.4.4 Loads

A.4.4.1 General

Acceleration due to gravity	g	9.81	m/s^2
Dynamic factor	$1 + \mu = 1 + \alpha(6/30 + L)$ $\alpha = 4(1 - h) \leq 2$	1.17	
Safety factor for Wight of deck, favorable	$\gamma_{p,fav}$	0.9	
Safety factor for Wight of deck, unfavorable	$\gamma_{p,unfav}$	1.3	
Safety factor for prestress force, favorable	$\gamma_{p,fav}$	1.1	
Safety factor for prestress force, unfavorable	$\gamma_{p,unfav}$	0.9	
Safety factor for variable actions, favorable	$\gamma_{f,fav}$	0.0	
Safety factor for vertical variable actions, unfavorable	$\gamma_{f,unfav}$	1.18	

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A.4.4.2 Load schematization in longitudinal direction

Vertical loads

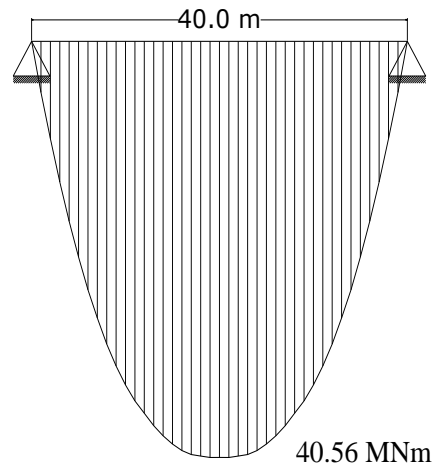
Permanent loads			
Dead load box girder	$g_{dead} = A_c * \rho_c * g$	112.32	kN/m
Concrete plinths		10.0	kN/m per track
Rail		0.97	kN/m per track
Cables		1.2	kN/m per cable duct
Walkway + guard-rail		2	kN/m per walkway
Sound insulation		1.3	kN/m per walkway
Concrete slope (drainage between walkways)		0.5	kN/m^2
Variable loads			
Common live load	Shown in (figure 2-6)		One and two track loaded envelop
Special live load	Shown in (figure 2-6)		One and Two track loaded envelop

Maximum Bending moment due to standard live load

Station (m)	Common live load Moment one track loaded (MNm)	Common live load Moment two track loaded (MNm)	Special rail live load Moment one track loaded (MNm)	Special rail live load Moment two track loaded (MNm)	Enveloped Standard live load Moment (MNm)
0.0	0.00	0	0.00	0.00	0
1.0	2.49	4.98	0.82	1.64	4.98
2.0	4.74	9.49	1.60	3.20	9.49
3.0	6.86	13.71	2.32	4.63	13.71
4.0	8.77	17.54	2.98	5.97	17.54
5.0	10.52	21.03	3.60	7.20	21.03
6.0	12.11	24.22	4.17	8.33	24.22
7.0	13.48	26.96	4.68	9.36	26.96
8.0	14.75	29.51	5.14	10.28	29.51
9.0	15.82	31.64	5.55	11.11	31.64
10.0	16.74	33.49	5.92	11.84	33.49
11.0	17.52	35.04	6.23	12.46	35.04
12.0	18.12	36.24	6.49	12.99	36.24
13.0	18.73	37.46	6.76	13.52	37.46
14.0	19.23	38.45	6.96	13.93	38.45
15.0	19.63	39.25	7.15	14.30	39.25
16.0	19.94	39.88	7.30	14.59	39.88
17.0	20.14	40.27	7.41	14.82	40.27
18.0	20.21	40.42	7.49	14.97	40.42
19.0	20.28	40.56	7.53	15.06	40.56
20.0	20.27	40.55	7.54	15.09	40.55
21.0	20.28	40.56	7.53	15.06	40.56

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22.0	20.21	40.42	7.49	14.97	40.42
23.0	20.14	40.27	7.41	14.82	40.27
24.0	19.94	39.88	7.30	14.59	39.88
25.0	19.63	39.25	7.15	14.30	39.25
26.0	19.23	38.45	6.96	13.93	38.45
27.0	18.73	37.46	6.76	13.52	37.46
28.0	18.12	36.24	6.49	12.99	36.24
29.0	17.52	35.04	6.23	12.46	35.04
30.0	16.74	33.49	5.92	11.84	33.49
31.0	15.82	31.64	5.55	11.11	31.64
32.0	14.75	29.51	5.14	10.28	29.51
33.0	13.48	26.96	4.68	9.36	26.96
34.0	12.11	24.21	4.17	8.33	24.21
35.0	10.52	21.03	3.60	7.20	21.03
36.0	8.77	17.53	2.98	5.97	17.53
37.0	6.86	13.71	2.32	4.63	13.71
38.0	4.74	9.49	1.60	3.20	9.49
39.0	2.49	4.97	0.82	1.64	4.97
40.0	0.00	0	0.00	0.00	0



Moment Envelop line due to Standard live load for 40m span

Service and ultimate state bending moment

Station (m)	Standard live load Enveloped Moment (MNm)	Utility and attachment moment (MNm)	Box girder dead load Moment (MNm)	External load Service limit state moment (SLS)	External load Ultimate limit state moment (ULS)
0.0	0	0.00	0.00	0	0.00
1.0	4.98	0.67	2.31	7.96	9.75
2.0	9.49	1.30	4.50	15.29	18.74
3.0	13.71	1.89	6.53	22.13	27.12
4.0	17.54	2.44	8.43	28.41	34.83
5.0	21.03	2.95	10.18	34.16	41.88

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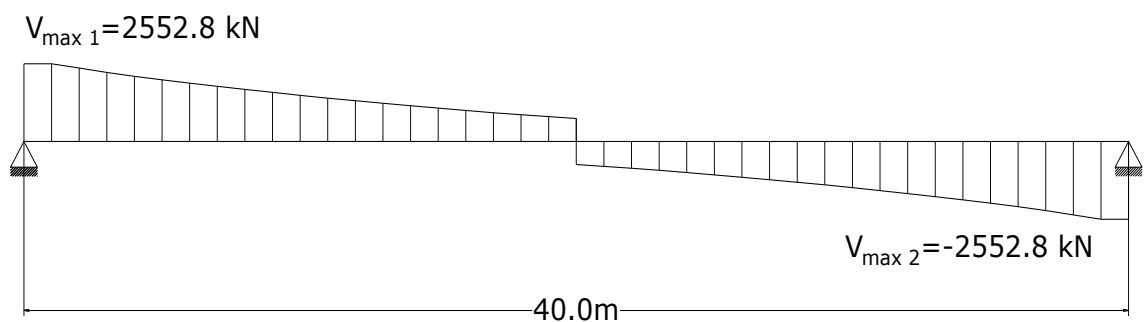
6.0	24.22	3.42	11.80	39.44	48.37
7.0	26.96	3.85	13.27	44.08	54.07
8.0	29.51	4.23	14.60	48.34	59.30
9.0	31.64	4.58	15.80	52.02	63.83
10.0	33.49	4.89	16.87	55.25	67.81
11.0	35.04	5.15	17.79	57.98	71.17
12.0	36.24	5.39	18.58	60.21	73.92
13.0	37.46	5.61	19.35	62.42	76.65
14.0	38.45	5.78	19.96	64.19	78.83
15.0	39.25	5.93	20.45	65.63	80.61
16.0	39.88	6.04	20.85	66.77	82.02
17.0	40.27	6.13	21.15	67.55	82.98
18.0	40.42	6.19	21.37	67.98	83.52
19.0	40.56	6.23	21.49	68.28	83.90
20.0	40.55	6.24	21.52	68.31	83.94
21.0	40.56	6.23	21.49	68.28	83.90
22.0	40.42	6.19	21.37	67.98	83.52
23.0	40.27	6.13	21.15	67.55	82.98
24.0	39.88	6.04	20.85	66.77	82.02
25.0	39.25	5.93	20.45	65.63	80.61
26.0	38.45	5.78	19.96	64.19	78.83
27.0	37.46	5.61	19.35	62.42	76.65
28.0	36.24	5.39	18.58	60.21	73.92
29.0	35.04	5.15	17.79	57.98	71.17
30.0	33.49	4.89	16.87	55.25	67.81
31.0	31.64	4.58	15.80	52.02	63.83
32.0	29.51	4.23	14.60	48.34	59.30
33.0	26.96	3.85	13.27	44.08	54.07
34.0	24.21	3.42	11.80	39.43	48.35
35.0	21.03	2.95	10.18	34.16	41.88
36.0	17.53	2.44	8.43	28.4	34.82
37.0	13.71	1.89	6.53	22.13	27.12
38.0	9.49	1.30	4.50	15.29	18.74
39.0	4.97	0.67	2.31	7.95	9.74
40.0	0	0.00	0.00	0	0.00

Maximum shear per girder due standard live load

Station (m)	common live load Shear one track loaded (kN)	Common live load Shear two track loaded (kN)	Special rail live load Shear one track loaded (kN)	Special rail live load Shear two track loaded (kN)	Enveloped Standard live load Shear (kN)
0.0	2343.21	2552.83	761.84	858.29	2552.83
1.0	2343.21	2552.83	761.84	858.29	2552.83
2.0	2121.62	2411.40	714.95	838.63	2411.40
3.0	1907.15	2267.44	663.96	809.34	2267.44
4.0	1730.24	2140.32	619.71	778.92	2140.32
5.0	1585.86	2024.92	582.84	749.34	2024.92
6.0	1465.94	1915.97	552.07	720.52	1915.97

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

7.0	1365.81	1811.73	526.72	692.67	1811.73
8.0	1281.09	1710.88	505.86	666.01	1710.88
9.0	1208.75	1613.86	488.57	640.84	1613.86
10.0	1143.42	1515.43	474.00	615.98	1515.43
11.0	1087.03	1422.29	462.01	593.11	1422.29
12.0	1037.42	1339.07	452.35	572.58	1339.07
13.0	987.47	1255.44	441.85	551.13	1255.44
14.0	940.72	1174.37	432.22	530.60	1174.37
15.0	897.67	1096.90	423.65	511.42	1096.90
16.0	854.91	1019.76	414.81	492.14	1019.76
17.0	813.44	944.57	406.10	473.21	944.57
18.0	773.07	878.71	397.45	454.60	878.71
19.0	734.19	816.96	388.94	436.44	816.96
20.0	694.35	755.25	379.63	417.04	755.25
20.0	-694.35	-755.25	-379.63	-417.04	-755.25
21.0	-734.19	-816.96	-388.94	-436.44	-816.96
22.0	-773.07	-878.71	-397.45	-454.60	-878.71
23.0	-813.44	-944.57	-406.10	-473.21	-944.57
24.0	-854.91	-1019.76	-414.81	-492.14	-1019.76
25.0	-897.67	-1096.90	-423.65	-511.42	-1096.90
26.0	-940.72	-1174.37	-432.22	-530.60	-1174.37
27.0	-987.47	-1255.44	-441.85	-551.13	-1255.44
28.0	-1037.42	-1339.07	-452.35	-572.58	-1339.07
29.0	-1087.03	-1422.29	-462.01	-593.11	-1422.29
30.0	-1143.42	-1515.43	-474.00	-615.98	-1515.43
31.0	-1208.74	-1613.85	-488.57	-640.84	-1613.85
32.0	-1281.08	-1710.87	-505.86	-666.01	-1710.87
33.0	-1365.81	-1811.73	-526.72	-692.67	-1811.73
34.0	-1465.92	-1915.95	-552.07	-720.52	-1915.95
35.0	-1585.86	-2024.92	-582.84	-749.34	-2024.92
36.0	-1730.24	-2140.32	-619.71	-778.92	-2140.32
37.0	-1907.12	-2267.42	-663.96	-809.34	-2267.42
38.0	-2121.63	-2411.41	-714.95	-838.63	-2411.41
39.0	-2343.19	-2552.81	-761.84	-858.29	-2552.81
40.0	-2343.19	-2552.81	-761.84	-858.29	-2552.81



Shear envelop line due to Standard live load for 40m span

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Ultimate limit state external load shear force per girder

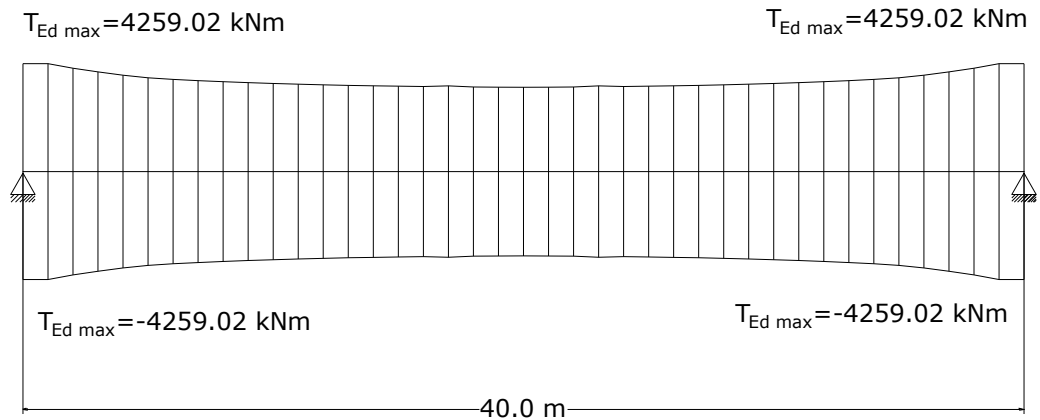
Station (m)	Enveloped rail live load Shear (kN)	Utility and attachment Shear (kN)	Box girder dead load shear (kN)	External load Service limit state shear (SLS)(kN)	External load Ultimate limit state Shear (ULS)(kN)
0.0	2552.83	334.62	1186.17	4073.61	4989.35
1.0	2552.83	334.62	1128.10	4015.54	4913.87
2.0	2411.40	317.04	1067.54	3795.98	4645.40
3.0	2267.44	299.21	1006.15	3572.80	4372.55
4.0	2140.32	280.05	939.52	3359.89	4111.02
5.0	2024.92	261.67	875.91	3162.50	3868.26
6.0	1915.97	243.54	813.47	2972.98	3634.96
7.0	1811.73	225.20	750.02	2786.94	3405.62
8.0	1710.88	206.97	686.63	2604.48	3180.52
9.0	1613.86	189.18	625.36	2428.40	2963.26
10.0	1515.43	171.42	564.20	2251.05	2744.51
11.0	1422.29	154.03	503.44	2079.77	2533.02
12.0	1339.07	136.89	444.40	1920.35	2335.77
13.0	1255.44	119.91	385.93	1761.28	2139.01
14.0	1174.37	103.76	329.78	1607.91	1949.35
15.0	1096.90	87.43	273.24	1457.57	1763.21
16.0	1019.76	71.02	216.75	1307.53	1577.42
17.0	944.57	54.93	161.09	1160.60	1395.43
18.0	878.71	39.55	107.52	1025.78	1228.07
19.0	816.96	23.43	52.06	892.45	1062.15
20.0	755.25	7.28	3.53	759.00	896.07
20.0	-755.25	-7.28	-3.53	-759.00	-896.07
21.0	-816.96	-23.43	-52.06	-892.45	-1062.15
22.0	-878.71	-39.55	-107.52	-1025.78	-1228.07
23.0	-944.57	-54.93	-161.09	-1160.60	-1395.43
24.0	-1019.76	-71.02	-216.75	-1307.53	-1577.42
25.0	-1096.90	-87.43	-273.24	-1457.57	-1763.21
26.0	-1174.37	-103.76	-329.78	-1607.91	-1949.35
27.0	-1255.44	-119.91	-385.93	-1761.28	-2139.01
28.0	-1339.07	-136.89	-444.40	-1920.35	-2335.77
29.0	-1422.29	-154.03	-503.44	-2079.77	-2533.02
30.0	-1515.43	-171.42	-564.20	-2251.05	-2744.51
31.0	-1613.85	-189.18	-625.36	-2428.39	-2963.25
32.0	-1710.87	-206.97	-686.63	-2604.47	-3180.50
33.0	-1811.73	-225.20	-750.02	-2786.94	-3405.62
34.0	-1915.95	-243.54	-813.47	-2972.96	-3634.93
35.0	-2024.92	-261.67	-875.91	-3162.50	-3868.26
36.0	-2140.32	-280.05	-939.52	-3359.89	-4111.02
37.0	-2267.42	-299.21	-1006.15	-3572.78	-4372.55
38.0	-2411.41	-317.04	-1067.54	-3795.99	-4645.41
39.0	-2552.81	-334.62	-1128.10	-4015.52	-4913.87
40.0	-2552.81	-334.62	-1186.17	-4073.59	-4989.35

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Maximum Torsion due to rail live load

Station (m)	Common live load torsion one track loaded (kNm) ±	Common live load torsion two track loaded (kNm) ±	Special rail live load torsion one track loaded (kNm) ±	Special rail live load torsion two track loaded (kNm) ±	Enveloped Standard load torsion (kNm) ±
0.0	4244.63	4259.02	1282.27	1297.61	4259.02
1.0	4244.63	4259.02	1282.27	1297.61	4259.02
2.0	4050.78	4087.65	1210.64	1253.11	4087.65
3.0	3841.30	3929.49	1144.85	1244.47	3929.49
4.0	3653.07	3805.42	1083.36	1256.50	3805.42
5.0	3484.19	3704.16	1034.32	1283.79	3704.16
6.0	3328.83	3639.41	993.26	1316.06	3639.41
7.0	3184.78	3590.72	957.73	1345.02	3590.72
8.0	3049.72	3549.51	928.20	1368.99	3549.51
9.0	2922.37	3513.04	903.71	1391.85	3513.04
10.0	2797.14	3477.23	881.86	1410.39	3477.23
11.0	2677.71	3448.26	862.07	1423.15	3448.26
12.0	2565.29	3427.63	846.32	1436.78	3427.63
13.0	2451.47	3402.51	831.13	1446.96	3402.51
14.0	2343.14	3388.64	817.55	1455.83	3388.64
15.0	2235.10	3372.45	804.64	1461.94	3372.45
16.0	2128.22	3359.09	792.09	1470.25	3359.09
17.0	2022.90	3348.20	779.67	1472.72	3348.20
18.0	1919.01	3339.94	767.89	1474.34	3339.94
19.0	1817.02	3335.26	755.84	1474.34	3335.26
20.0	1713.42	3327.68	743.06	1474.25	3327.68
21.0	1817.02	3335.26	755.84	1474.34	3335.26
22.0	1919.01	3339.94	767.89	1472.72	3339.94
23.0	2022.90	3348.20	779.67	1470.25	3348.20
24.0	2128.22	3359.09	792.09	1467.05	3359.09
25.0	2235.10	3372.45	804.64	1467.05	3372.45
26.0	2343.14	3388.64	817.55	1461.94	3388.64
27.0	2451.44	3402.48	831.13	1455.83	3402.48
28.0	2565.29	3427.63	846.32	1446.96	3427.63
29.0	2677.73	3448.27	862.07	1436.78	3448.27
30.0	2797.14	3477.24	881.86	1423.15	3477.24
31.0	2922.37	3513.04	903.71	1410.39	3513.04
32.0	3049.72	3549.51	928.20	1391.85	3549.51
33.0	3184.78	3590.72	957.73	1368.99	3590.72
34.0	3328.83	3639.41	993.26	1345.02	3639.41
35.0	3484.19	3704.16	1034.32	1316.06	3704.16
36.0	3653.07	3805.42	1083.36	1283.79	3805.42
37.0	3841.30	3929.49	1144.85	1256.50	3929.49
38.0	4050.78	4087.65	1210.64	1253.11	4087.65
39.0	4244.63	4259.02	1282.27	1297.61	4259.02
40.0	4244.63	4259.02	1282.27	1297.61	4259.02

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



Torsion envelop line due to Standard live load for 40m span

Ultimate limit state external load Torsion moment

Station (m)	Enveloped rail live load Torsion (kNm)	External load Service limit state Torsion (SLS)(kNm)	External load Ultimate limit state Torsion (ULS)(kNm)
0.0	4259.02	4259.02	5025.64
1.0	4259.02	4259.02	5025.64
2.0	4087.65	4087.65	4823.43
3.0	3929.49	3929.49	4636.80
4.0	3805.42	3805.42	4490.40
5.0	3704.16	3704.16	4370.91
6.0	3639.41	3639.41	4294.50
7.0	3590.72	3590.72	4237.05
8.0	3549.51	3549.51	4188.42
9.0	3513.04	3513.04	4145.39
10.0	3477.23	3477.23	4103.13
11.0	3448.26	3448.26	4068.95
12.0	3427.63	3427.63	4044.60
13.0	3402.51	3402.51	4014.96
14.0	3388.64	3388.64	3998.60
15.0	3372.45	3372.45	3979.49
16.0	3359.09	3359.09	3963.73
17.0	3348.20	3348.20	3950.88
18.0	3339.94	3339.94	3941.13
19.0	3335.26	3335.26	3935.61
20.0	3327.68	3327.68	3926.66
21.0	3335.26	3335.26	3935.61
22.0	3339.94	3339.94	3941.13
23.0	3348.20	3348.20	3950.88
24.0	3359.09	3359.09	3963.73
25.0	3372.45	3372.45	3979.49
26.0	3388.64	3388.64	3998.60

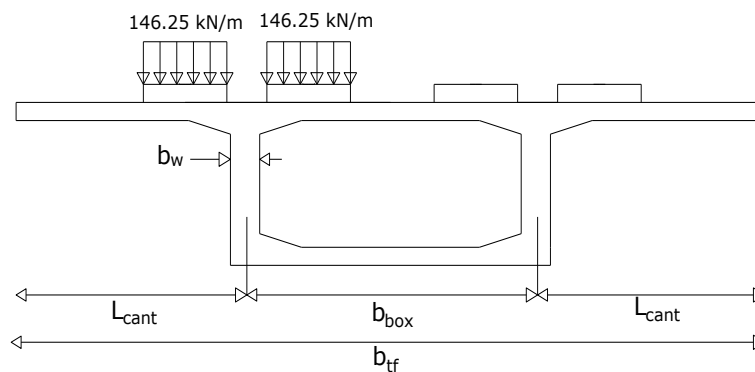
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

27.0	3402.48	3402.48	4014.93
28.0	3427.63	3427.63	4044.60
29.0	3448.27	3448.27	4068.96
30.0	3477.24	3477.24	4103.14
31.0	3513.04	3513.04	4145.39
32.0	3549.51	3549.51	4188.42
33.0	3590.72	3590.72	4237.05
34.0	3639.41	3639.41	4294.50
35.0	3704.16	3704.16	4370.91
36.0	3805.42	3805.42	4490.40
37.0	3929.49	3929.49	4636.80
38.0	4087.65	4087.65	4823.43
39.0	4259.02	4259.02	5025.64
40.0	4259.02	4259.02	5025.64

A.4.4.3 Approximate Load schematization in transversal direction

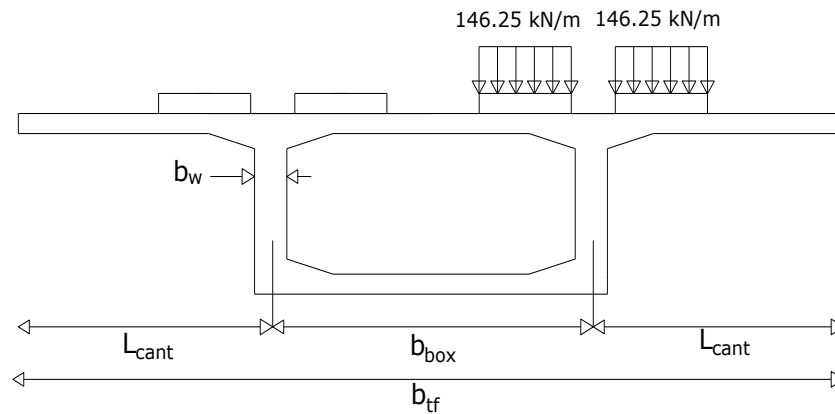
Vertical load

Permanent loads			
Concrete plinths	(/2 plinths per track/1m (width plinth))	5	kN/m
Rail	(/ 2 rails per track)	0.485	kN per rail
Cables		1.2	kN per cable duct
Walkway + guard-rail	(/ 1 m (width walkway))	2	kN/m
Sound insulation		1.3	kN per walkway
Concrete slope (drainage between walkways)		0.5	kN/m
Variable loads			
Concentrated load due to Live (without dynamic factor)	Q_{mob}	250	kN per track
Concentrated load due to Live (with dynamic factor)	$Q_{mob} * (1 + \mu) / 2$	146.25	kN per rail

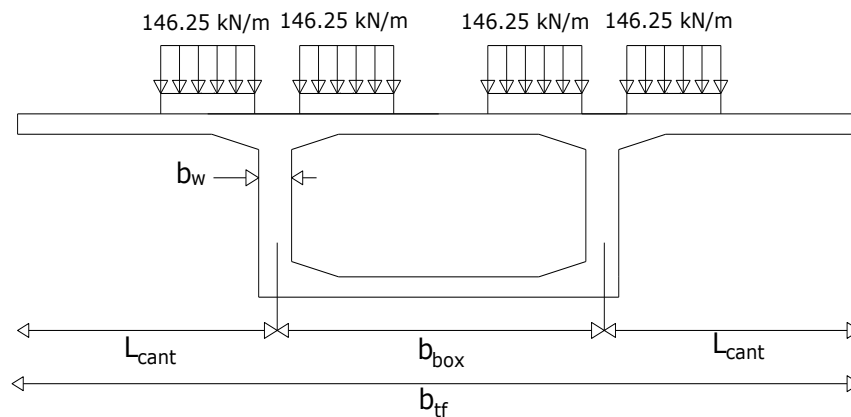


Rail live load on the left track of the deck for 40m span

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



Rail live load on the right track of the deck for 40m span



Rail load on the both track of the deck for 40m span

Top slab Transverse ultimate maximum moment

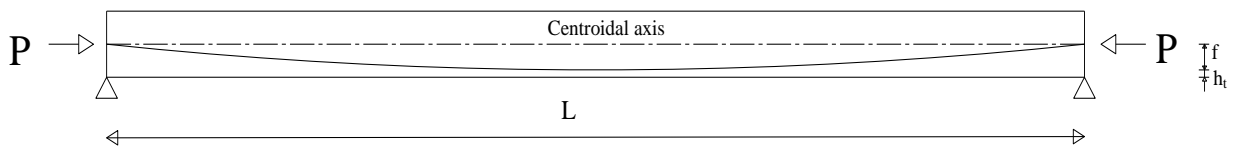
Distance from left edge of deck (m)	Left lane loaded ultimate Moment (kNm)	Right lane loaded ultimate Moment (kNm)	Two lane loaded ultimate Moment (kNm)	Enveloped maximum ultimate moment (kNm)
0.00	0.00	0.00	0.00	0.00
0.46	-0.94	-0.94	-0.94	-0.94
0.85	-3.87	-3.87	-3.87	-3.87
1.24	-8.08	-8.08	-8.08	-8.08
1.63	-14.46	-13.44	-14.46	-14.46
2.02	-42.19	-20.62	-42.19	-42.19
2.38	-93.12	-29.30	-93.12	-93.12
2.74	-163.85	-39.59	-163.85	-163.85
3.12	-42.23	-4.42	-40.89	-42.23
3.50	4.09	4.75	9.48	9.48
4.00	23.81	13.37	34.43	34.43
4.50	19.45	19.45	35.30	35.30

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

5.00	13.37	23.81	34.43	34.43
5.50	4.75	4.09	9.48	9.48
5.84	-4.42	-42.23	-40.89	-42.23
6.22	-39.59	-163.85	-163.85	-163.85
6.60	-29.30	-93.12	-93.12	-93.12
6.94	-20.62	-42.19	-42.19	-42.19
7.33	-13.44	-14.46	-14.46	-14.46
7.72	-8.08	-8.08	-8.08	-8.08
8.11	-3.87	-3.87	-3.87	-3.87
8.51	-0.94	-0.94	-0.94	-0.94
8.96	0.00	0.00	0.00	0.00

A.4.5 Prestressing tendons

A.4.5.1 Layout prestressing tendons



The tendon vertical profile is parabolic with the following ordinate from center:

Tendon distance	0.0	3.33	6.7	10.0	13.33	16.7	20.0	23.33	26.7	30.0	33.33	36.7	40.0
Vertical ordinate	0.0	0.42	0.76	1.02	1.21	1.32	1.36	1.32	1.21	1.02	0.76	0.42	0.0

Distance between the center of the tendons and bottom side at mid-span $h_t = 0.17m$ and

Tendon eccentricity at mid-span $Z_{cp,m} = Z_{cb} - h_t = 1.36m$

A.4.5.2 Prestressing losses

A.4.5.2.1 Losses due to the instantaneous deformation of concrete

Due to successive tensioning of prestressing tendon the box girders shorten and an immediate prestressing loss occur, which can be calculated for each tendon with the following formula:

$$\Delta p_{el} = A_p * E_p * \sum [j * \Delta \sigma_c(t) / E_{cm}]$$

$$P = 51408kN$$

Initial Prestress force

$$E_{cm} = 35000 N/mm^2$$

Secant modulus of elasticity of concrete

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

$\Delta\sigma(t) = \sigma_{pm0} * A_p / A_c = 0.82 N/mm^2$ Is the variation of stress in the concrete at the Center of gravity of the tendons applied at time t.
 $j = (n-1)/2n$ Is a coefficient where n is the number of identical Tendons successively prestressed

This prestressing loss taking into account the order in which the tendons are stressed can be compensated by slightly overstressing the tendons. The maximum over stress is needed in the first prestressed tendon as this tendon has the largest loss due the instantaneous deformation of concrete.

The required over stress $\sigma_{overstr}$ in the first prestressed tendon to compensate the losses due to instantaneous deformation of concrete can be calculated out of the formula below:

$$\Delta p_{el,1} = A_p * E_p * j * \Delta\sigma_{overstr} / E_{cm} = A_p * (\sigma_{overstr} - \Delta\sigma_{pm0})$$

Rearranging the above equation
$$\Delta\sigma_{overstr} = \frac{\sigma_{pm0}}{1 - E_p * \frac{j * A_p}{E_{cm} * A_c}} = 1362.1 N/mm^2$$

Total Area of tendon required $A_{ten,total} = P / \sigma_{pm0} = 37800 mm^2$

Fourteen tendons consist of 27 strands with $100 mm^2$ section area Provide $A_{ten,total}$

Number of tendon = $A_{ten,total} / A_p = 14$ number of tendon

For the first prestressed tendon $n = 14$

$$j = (n-1)/2n = 0.46$$

$$\sigma_{p,max} = 1440 N/mm^2$$

Maximum allowed tensile stress of The tendons during tensioning

A.4.5.2.2 Losses due to friction and wobble

The force at any point due to friction and wobble effect in post-tensioned tendons is:

$$P_x = P_0 e^{-\mu(\sum \alpha + \omega x)}$$

Parameter assumed value as follows:

Where ω = wobble coefficient 0.006 /m

μ = Friction coefficient 0.15

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

A.4.5.2.3 Loss due to Anchor set

As previous chapter, this prestressing loss also assumed to be compensated by overstressing the tendons.

A.4.5.2.4 Time dependent losses of prestress for post-tensioning

$$\Delta P_{c+s+r} = A_p \Delta \sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8 \Delta \sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_o) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} \left(1 + \frac{A_c}{I_c} Z_{cp}^2\right) [1 + 0.8 \varphi(t, t_0)]}$$

Where

Creep

$$\varphi(\infty, t_o) = 1.5$$

Is the final creep coefficient according Figure 3.1b [12]
(Outside conditions; C-40; Class N; $t_0 = 30$ days;
 $h_0 = 2 * A_c / u = 400mm$)

Shrinkage

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 0.00018$$

Is the estimated shrinkage strain in absolute value

Where

$$\varepsilon_{cd}(\infty) = k_h * \varepsilon_{cd,0} = 0.18\%$$

$$\varepsilon_{cd,0} = 0.25\%$$

Is the nominal unrestrained drying shrinkage value
According Table 3.2 [12] (Relative humidity 80%;
C-40)

$$k_h = 0.73$$

A coefficient depending on the notional size h_0
According to Table 3.3 [12]

$$\varepsilon_{ca}(\infty) = 2.5 (f_{ck} - 10) * 10^{-6}$$

$$\varepsilon_{ca}(\infty) = 0.000075\%$$

Is the autogenous shrinkage

Relaxation

Relaxation class 2 (wire or strand):

$$\Delta \sigma_{pr} = (\sigma_{pi}) * 0.66 * \rho_{1000} * e^{9.1\mu} \left(\frac{t}{1000}\right)^{0.75(1-\mu)} * 10^{-5} = 60.93 N/mm^2$$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Where:

$$\sigma_{pi} = \sigma_{pm0} = 1360 \text{ N/mm}^2$$

Relaxation class 2 therefore $\rho_{1000} = 2.5\%$

$$\mu = \sigma_{pi} / f_{pk} = 0.73$$

The long term (final) values of the relaxation losses may be estimated for a time equal to:

$$t = 500,000 \text{ hours.}$$

Concrete stress

$\sigma_{c,QP}$ Is the stress in the concrete adjacent to the tendons, due to self-weight, Initial prestress and other permanent actions depending on the stage Of construction considered. Stress at the Centroid axis at $t=0$ assumed.

This gives:

$$\sigma_{c,QP} = \frac{P_0}{A_c} = 11.44 \text{ N/mm}^2$$

Time dependent loss of prestress for post-tensioning at mid-span

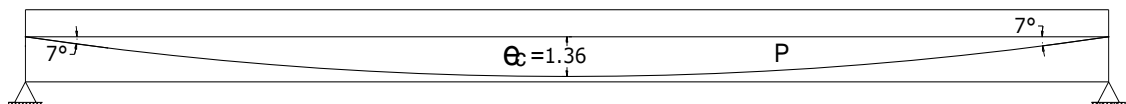
$$\Delta P_{c+s+r,m} = nA_p \Delta \sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8 \Delta \sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} \left(1 + \frac{A_c}{I_{c,m}} Z_{cp,m}^2\right) [1 + 0.8 \varphi(\infty, t_0)]} = 5051.142 \text{ kN}$$

$$Z_{cp,m} = 1362.6 \text{ mm}$$

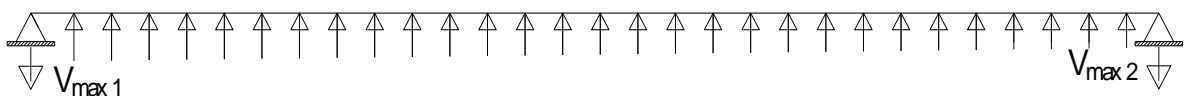
Is the distance between the center of gravity of the Concrete section and the tendons at mid span

A.4.5.2.6 Bending moments and shear force due to prestressing

The moment diagram and structural schematization due to prestressing is shown



Parabolic prestress layout anchored at neutral axis for 40m span

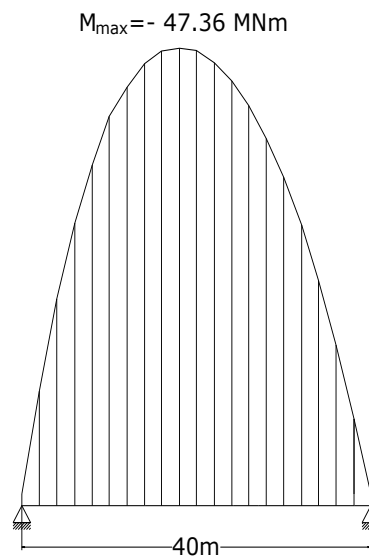


Equivalent load due to prestress action for 40m span

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

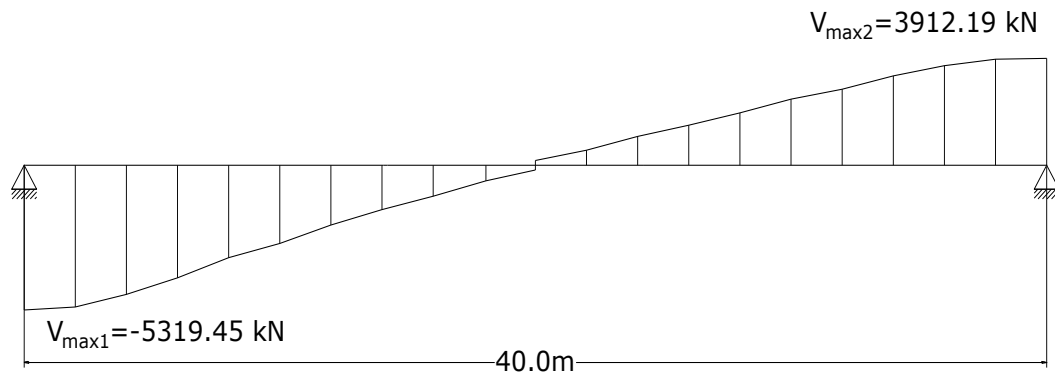
Effective Prestress force shear and moment effect

Tendon distance (m)	Effective prestress Shear force (kN)	Effective prestress moment (MNm)
0.0	-5319.45	0.00
2.0	-5184.82	-10.86
4.0	-4711.40	-20.56
6.0	-4066.94	-28.61
8.0	-3414.92	-34.95
10.0	-2785.62	-39.79
12.0	-2174.52	-43.19
14.0	-1623.69	-45.60
16.0	-1102.41	-46.93
18.0	-607.36	-47.36
20.0	-130.52	-47.00
20.0	129.49	-47.00
22.0	582.92	-45.78
24.0	1025.46	-43.88
26.0	1463.82	-41.27
28.0	1901.45	-37.84
30.0	2357.23	-33.70
32.0	2799.23	-28.70
34.0	3229.52	-22.79
36.0	3625.52	-15.96
38.0	3872.61	-8.24
40.0	3912.19	0.00



Bending moment diagram due to prestress force action for 40m span

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Shear force diagram due to prestress action for 40m span

A.4.6 Table of service stresses

Distance (m)	Bottom fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	0.00	9.78	-0.97	0.00	0.00
	Cumulated		+9.78	+8.82	+8.82	+8.82
2	Partial	-1.94	15.29	-1.52	-0.56	-4.07
	Cumulated		+13.35	+11.83	+11.27	+7.20
4	Partial	-3.59	20.10	-2.04	-1.04	-7.45
	Cumulated		+16.51	+14.47	+13.44	+5.99
6	Partial	-5.01	23.65	-2.43	-1.45	-10.25
	Cumulated		+18.64	+16.21	+14.77	+4.52
8	Partial	-6.17	26.39	-2.75	-1.78	-12.43
	Cumulated		+20.22	+17.47	+15.69	+3.26
10	Partial	-7.09	28.41	-3.01	-2.05	-14.04
	Cumulated		+21.32	+18.31	+16.27	+2.22
12	Partial	-7.79	29.81	-3.20	-2.25	-15.15
	Cumulated		+22.02	+18.82	+16.57	+1.42
14	Partial	-8.33	30.85	-3.36	-2.41	-16.01
	Cumulated		+22.53	+19.17	+16.76	+0.76
16	Partial	-8.68	31.35	-3.46	-2.51	-16.56
	Cumulated		+22.67	+19.21	+16.70	+0.14
18	Partial	-8.88	31.47	-3.52	-2.57	-16.85
	Cumulated		+22.59	+19.07	+16.50	-0.34
20	Partial	-8.94	31.24	-3.55	-2.58	-16.87
	Cumulated		+22.31	+18.76	+16.18	-0.70
22	Partial	-8.88	30.65	-3.53	-2.57	-16.85
	Cumulated		+21.77	+18.25	+15.68	-1.17
24	Partial	-8.68	29.74	-3.47	-2.51	-16.56
	Cumulated		+21.06	+17.59	+15.08	-1.49
26	Partial	-8.33	28.49	-3.37	-2.41	-16.01
	Cumulated		+20.16	+16.79	+14.38	-1.63
28	Partial	-7.79	26.85	-3.22	-2.25	-15.15

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

	Cumulated		+19.07	+15.84	+13.59	-1.55
30	Partial	-7.09	24.87	-3.03	-2.05	-14.04
	Cumulated		+17.78	+14.76	+12.71	-1.33
32	Partial	-6.17	22.43	-2.77	-1.78	-12.43
	Cumulated		+16.27	+13.50	+11.72	-0.71
34	Partial	-5.01	19.51	-2.44	-1.45	-10.25
	Cumulated		+14.51	+12.07	+10.62	+0.37
36	Partial	-3.59	16.04	-2.04	-1.04	-7.45
	Cumulated		+12.45	+10.41	+9.38	+1.93
38	Partial	-1.94	11.80	-1.52	-0.56	-4.07
	Cumulated		+9.87	+8.35	+7.79	+3.72
40	Partial	0.00	7.45	-0.97	0.00	0.00
	Cumulated		+7.45	+6.49	+6.49	+6.49

Distance (m)	Top fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	0.01	9.81	-0.97	0.00	0.00
	Cumulated		+9.83	+8.86	+8.86	+8.86
2	Partial	1.13	7.43	-0.74	0.33	2.37
	Cumulated		+8.56	+7.82	+8.14	+10.51
4	Partial	2.12	4.80	-0.48	0.61	4.41
	Cumulated		+6.93	+6.44	+7.06	+11.47
6	Partial	2.99	2.44	-0.25	0.87	6.12
	Cumulated		+5.43	+5.18	+6.04	+12.16
8	Partial	3.72	0.45	-0.05	1.08	7.51
	Cumulated		+4.17	+4.12	+5.20	+12.71
10	Partial	4.33	-1.16	0.14	1.25	8.58
	Cumulated		+3.17	+3.31	+4.56	+13.14
12	Partial	4.81	-2.38	0.29	1.39	9.36
	Cumulated		+2.43	+2.72	+4.11	+13.47
14	Partial	5.18	-3.28	0.39	1.50	9.95
	Cumulated		+1.90	+2.29	+3.78	+13.74
16	Partial	5.43	-3.88	0.46	1.57	10.36
	Cumulated		+1.56	+2.02	+3.59	+13.95
18	Partial	5.58	-4.20	0.50	1.61	10.59
	Cumulated		+1.38	+1.89	+3.50	+14.09
20	Partial	5.63	-4.26	0.51	1.63	10.63
	Cumulated		+1.36	+1.88	+3.50	+14.13
22	Partial	5.58	-4.07	0.49	1.61	10.58
	Cumulated		+1.51	+2.00	+3.61	+14.19
24	Partial	5.43	-3.65	0.44	1.57	10.36
	Cumulated		+1.78	+2.22	+3.79	+14.15
26	Partial	5.18	-3.00	0.36	1.50	9.95
	Cumulated		+2.18	+2.54	+4.03	+13.99
28	Partial	4.81	-2.12	0.26	1.39	9.36

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	Cumulated		+2.70	+2.95	+4.34	+13.70
30	Partial	4.33	-1.00	0.12	1.25	8.58
	Cumulated		+3.33	+3.45	+4.71	+13.28
32	Partial	3.72	0.37	-0.05	1.08	7.51
	Cumulated		+4.10	+4.05	+5.13	+12.64
34	Partial	2.99	1.99	-0.25	0.87	6.12
	Cumulated		+4.98	+4.73	+5.59	+11.72
36	Partial	2.12	3.84	-0.51	0.61	4.41
	Cumulated		+5.96	+5.45	+6.06	+10.47
38	Partial	1.13	5.81	-0.91	0.33	2.37
	Cumulated		+6.94	+6.03	+6.36	+8.73
40	Partial	0.01	7.47	-0.97	0.00	0.03
	Cumulated		+7.49	+6.52	+6.52	+6.55

Permissible stresses immediately and under working load, Assuming at moment of transfer concrete has an age of 20 days and a normal Portland cement, moist cured is being used.

$$f_{ck}(20) = [t/\alpha + \beta t] * [f_{ck}(28)] = 37.7 \text{ Mpa}$$

Concrete strength at 20 days from Specified 28 day strength value

$$\alpha = 4$$

Constant for normal Portland cement And moist cured concrete

$$\beta = 0.860$$

Constant for normal Portland cement And moist cured concrete

$$f_{ct} = 0.6 * f_{ck}(20) = 22.64 \text{ Mpa}$$

Permissible compressive stress at transfer

$$f_{tt} = 0.21 f_{ck}^{2/3} = -2.45 \text{ Mpa}$$

Permissible tensile stress at transfer

$$f_{cw} = f_{cd} = 22.67 \text{ Mpa}$$

Permissible compressive stress at working Condition

$$f_{tw} = f_{td} = 1.67 \text{ Mpa}$$

Permissible tensile stress at working Condition

A.4.7 Deflection

The deflection due to dead weight is determined with the formula:

$$w = \frac{5}{384} \frac{qL^4}{EI_c}$$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Where:

$$E = E_{C,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)} = 14000 \text{ N/mm}^2$$

Effective modulus of elasticity of concrete
For deflection at $t = \infty$ without variable load

$$E = E_{cm} = 35000 \text{ N/m}^2$$

Secant modulus of elasticity of concrete for
Additional deflection under mobile load

The maximum deflections and unity checks for different Loads are:

Time	Loads	Deflection w	Maximum allowed Deflection w_{max}
At $t = 0$	$g_{boxdead} - q_{pt0}$	$-43mm$	$L/250 = 160mm$
At $t = \infty$ without variable load	$g_{boxdead} + g_{utility} - q_{pt\infty}$	$-64.54mm$	
Additional deflection under mobile load without impact	$q_{var-impact}$	$+49.65mm$	$L/800 = 50mm$
Additional deflection under mobile load with impact	$q_{var+impact}$	$+58mm$	
At $t = \infty$ fully loaded	$g_{boxdead} + g_{utility} + q_{var+impact} - q_{pt\infty}$	$-6.54mm$	

A.4.8 Shear + torsion

A.4.8.1 Shear

$$\rho_{w,min} = A_{sw} / (S b_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk}$$

Minimum shear reinforcement ratio

Where:

$$V_{Rd,c} = \frac{I_c * b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_1 \sigma_{cp} f_{ctd}}$$

The shear resistance of concrete in regions
Uncracked in bending

$$V_{Rd,c} = \left[C_{Rd,c} K (100 \rho_1 f_{ck})^{1/3} + K_1 \sigma_{cp} \right] b_w d$$

The shear resistance of concrete in regions
Cracked in bending

$$V_{Rd,c} = (V_{min} + K_1 \sigma_{cp}) b_w d$$

The shear resistance of concrete without any
Reinforcement

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Values of shear properties per girder		Value	
Live and dead load factor for one girder	$LDFV$	0.5	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	2.28	m
Inner liver arm	Z	1.92	m
Tensile strength of concrete	f_{ctd}	1.67	Mpa
Second moment of area of the girder concrete section about horizontal axis	I_c	1.778	m^4
Yield strength of shear reinforcement	f_{yd}	400	Mpa
Angle of the inclined struts in tress model	θ	45	degree
First moment of area above and about horizontal centroid axis.	S	0.91	m^3
Design strength of concrete	f_{cd}	22.67	Mpa
Prestress Concrete stress at centroid axis	σ_{cp}	4.53	Mpa
Reinforcement ratio of tensile longitudinal reinforcement	ρ_l	0.02	
Maximum shear force imitated by crushing of compression strut.	$V_{Rd,max}$	4606.16	kN

Girder section distance in m	V_{Ed} in kN	$V_{Rd,c}$ in kN	$V_{Ed}/V_{Rd,c}$	$V_{Ed}/V_{Rd,max}$
0	2511.26	2180.18	1.15	0.55
1	2435.77	2180.18	1.12	0.53
2	2270.32	2180.18	1.04	0.49
3	2095.01	991.45	2.11	0.45
4	1975.84	991.45	1.99	0.43
5	1876.80	991.45	1.89	0.41
6	1786.24	991.45	1.80	0.39
7	1699.91	991.45	1.71	0.37
8	1617.54	991.45	1.63	0.35
9	1539.22	991.45	1.55	0.33
10	1471.72	991.45	1.48	0.32
11	1403.69	991.45	1.42	0.30
12	1333.40	991.45	1.34	0.29
13	1269.52	991.45	1.28	0.28
14	1205.65	991.45	1.22	0.26
15	1143.59	991.45	1.15	0.25
16	1083.18	991.45	1.09	0.24
17	1024.24	991.45	1.03	0.22
18	966.90	991.45	0.98	0.21
19	908.54	991.45	0.92	0.20
20	-874.53	991.45	0.88	0.19
21	-909.01	991.45	0.92	0.20
22	-971.23	991.45	0.98	0.21

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

23	-1035.23	991.45	1.04	0.22
24	-1104.30	991.45	1.11	0.24
25	-1181.75	991.45	1.19	0.26
26	-1265.98	991.45	1.28	0.27
27	-1352.09	991.45	1.36	0.29
28	-1450.95	991.45	1.46	0.32
29	-1550.65	991.45	1.56	0.34
30	-1651.97	991.45	1.67	0.36
31	-1772.10	991.45	1.79	0.38
32	-1894.60	991.45	1.91	0.41
33	-2025.01	991.45	2.04	0.44
34	-2162.53	991.45	2.18	0.47
35	-2308.37	991.45	2.33	0.50
36	-2464.19	991.45	2.49	0.53
37	-2638.85	991.45	2.66	0.57
38	-2860.48	991.45	2.89	0.62
39	-3069.04	2180.18	1.41	0.67
40	-3144.52	2180.18	1.44	0.68

A.4.8.1.1 Additional longitudinal reinforcement due to shear

$$\Delta f_{td} = 0.5V_{Ed} * (\cot \theta - \cot \alpha) \quad \text{Additional tensile force in the longitudinal Reinforcement due to shear}$$

A.4.8.2 Torsion

Values of Torsion properties		Value	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	2.28	m
Inner liver arm	Z	1.92	m
Tensile strength of concrete	f_{ctd}	1.67	Mpa
Area enclosed by shear flow path	A_k	7.66	m^2

The capacity of member subjected to shear and torsion limited by the capacity of the concrete struts Verified by the following equation:

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}} \leq 1.0$$

The required cross-sectional area of the longitudinal reinforcement for torsion $\sum A_{Sl}$ calculated from:

$$\frac{\sum A_{Sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta$$

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And, the required cross-sectional area of the transverse reinforcement for torsion A_{St} calculated from:

$$\frac{A_{St} f_{yd}}{S} = \frac{T_{Ed}}{2A_k} \cot \theta$$

Girder section distance in (m)	T_{Ed} in (kNm)	$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}}$	A_{St} required in (mm^2 / m)	A_{St} required in (mm^2 / m)
0	5174.67	0.73	845	845
1	5174.67	0.71	845	845
2	4962.45	0.23	810	810
3	4808.17	0.24	785	785
4	4645.72	0.23	759	759
5	4525.44	0.56	739	739
6	4444.21	0.53	726	726
7	4375.55	0.51	714	714
8	4319.24	0.22	705	705
9	4270.84	0.23	697	697
10	4224.52	0.23	690	690
11	4186.40	0.22	684	684
12	4156.20	0.23	679	679
13	4124.77	0.24	673	673
14	4101.37	0.24	670	670
15	4082.26	0.25	667	667
16	4067.06	0.26	664	664
17	4051.34	0.27	661	661
18	4041.90	0.28	660	660
19	4037.36	0.29	659	659
20	4031.89	0.31	658	658
21	4037.36	0.31	659	659
22	4041.88	0.33	660	660
23	4051.34	0.34	661	661
24	4067.06	0.36	664	664
25	4082.26	0.38	667	667
26	4101.37	0.40	670	670
27	4124.77	0.42	673	673
28	4156.20	0.44	679	679
29	4186.40	0.47	684	684
30	4224.52	0.49	690	690
31	4270.84	0.52	697	697
32	4319.24	0.55	705	705
33	4375.55	0.58	714	714
34	4444.21	0.61	726	726
35	4525.44	0.65	739	739
36	4645.72	0.69	759	759
37	4808.17	0.73	785	785
38	4962.45	0.78	810	810
39	5174.67	0.84	845	845
40	5174.67	0.86	845	845

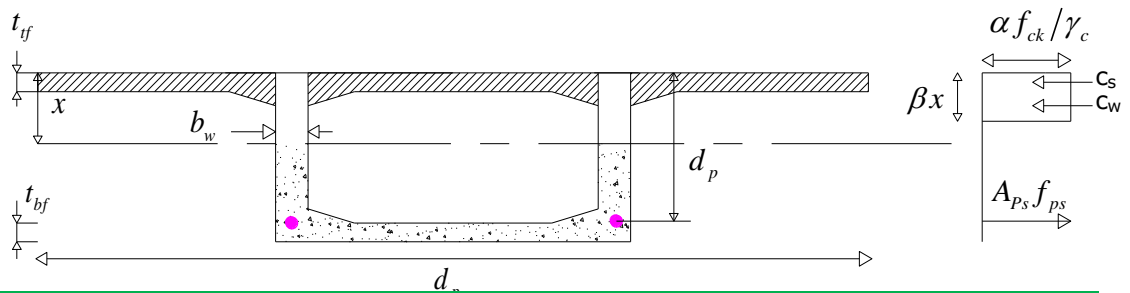
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A.4.9 Ultimate moment resistance

Values of box girder properties		Value	
Area of the top slab	$A_{slab,top}$	2.53	m^2
Equivalent thickness of top slab	$t_{tf, equ}$	0.283	m
Sum of web width	$\sum b_w$	0.7	m
Area of the bottom slab	$A_{slab,bott}$	0.95	m^2
Equivalent thickness of bottom slab	$t_{bf, equ}$	0.248	m
Width of top slab	b_{if}	8.96	

$$A_{PS} = 37800mm^2$$

Total Area of tendon



Box Girder section distance in (m)	Tendon eccentricity from top fiber d_p in (m)	Tendon ultimate force (kN)	Distance between neutral axis and compressive face (x) in (m)	Compressive block depth below top fiber to neutral axis (βx) in (m)
0	0.87	59069.42	0.48	0.38
2	1.126	60607.00	0.60	0.48
4	1.358	61122.60	0.65	0.52
6	1.56	61122.60	0.65	0.52
8	1.74	61122.60	0.65	0.52
10	1.89	61122.60	0.65	0.52
12	2.00	61122.60	0.65	0.52
14	2.107	61122.60	0.65	0.52
16	2.17	61122.60	0.65	0.52
18	2.216	61122.60	0.65	0.52
20	2.23	61122.60	0.65	0.52
22	2.216	61122.60	0.65	0.52
24	2.17	61122.60	0.65	0.52
26	2.107	61122.60	0.65	0.52
28	2.00	61122.60	0.65	0.52
30	1.89	61122.60	0.65	0.52
32	1.74	61122.60	0.65	0.52
34	1.56	61122.60	0.65	0.52
36	1.358	61122.60	0.65	0.52
38	1.126	60607.00	0.60	0.48
40	0.87	59069.42	0.48	0.38

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$$C_s = (\alpha * f_{ck} / \gamma_c) * (b_{tf} - \sum b_w) * h_{tf, equ} \quad \text{Compressive force in flange}$$

$$C_w = (\alpha f_{ck} / \gamma_c) * \beta x * \sum b_w \quad \text{Compressive force in web}$$

$$M_{Rs} = C_s * (d_p - 0.5 * t_{tf}) \quad \text{Resistance moment due to flange compressive force}$$

$$M_{Rw} = C_w * (d_p - 0.5 * \beta x) \quad \text{Resistance moment due to web compressive force}$$

$$M_{RT} = M_{Rs} + M_{Rw} \quad \text{Total positive resistance moment of section}$$

Box Girder section distance in (m)	Positive moment resistance for girder (M_{RT}) in (kNm)	Positive demand moment for the girder (M_{DE}) in (kNm)
0	42578.27	0.00
2	58934.25	19065.35
4	73400.04	35399.45
6	85892.89	49700.68
8	96720.02	61816.30
10	105881.44	71669.64
12	113377.15	79449.05
14	119207.15	85502.31
16	123371.43	89893.36
18	125870.00	92476.36
20	126702.86	92738.97
22	125870.00	92297.64
24	123371.43	89966.10
26	119207.15	85740.54
28	113377.15	79881.41
30	105881.44	71710.99
32	96720.02	61900.19
34	85892.89	49846.93
36	73400.04	35543.48
38	58934.25	19105.97
40	42578.27	0.00

APPENDIX B: Statical Calculations Concrete Box Girder C-50

B.1 Introduction

This appendix presents the calculation of optimal depth box girder in concrete C-50 for sample span length of 50m. First the material characteristics of the concrete and steel along with designation symbol are described. Part B.3 deals with the geometry and the structural schematization of the box girder and its characteristics. The loads, load factors, load combination in the scope of this study application and there effects to which the sample box girder is subjected are treated in part B.4.4. Next the layout of the prestressing tendons is shown and the stresses in the box girder due to loading and prestressing are calculated. It also contains the calculations of the prestressing losses. Furthermore this appendix includes serviceability requirement of deflection and stress calculation of sample span length box girder. And also Shear, torsion and the ultimate moment resistance calculation of the sample box girder addressed for rough estimation of non prestressing reinforcement quantity. The formulas and values used in the calculations are taken from [12] and other references, which are then stated in the text.

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B.2 Material Characteristics

B.2.1 Concrete C-50

Density of Concrete	ρ_c	2500 kg/m ³
Partial factor for concrete	γ_c	1.5
Coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied	α_{cc}	0.85
Compressive cylinder strength of concrete at 28 days	f_{ck}	50 N/mm ²
Mean value of concrete cylinder compressive strength	$f_{cm} = f_{ck} + 8$	58 N/mm ²
Modulus of elasticity of Concrete	E_c	3.7*10 ⁴ N/mm ²
Design Tensile strength of concrete	$f_{ctd} = f_{ctk,0.05} / \gamma_c$	6.93 N/mm ²

B.2.2 Reinforcing steel FeB 460

Density of reinforcing steel	ρ_s	7850 kg/m ³
Characteristic yield strength	f_{yk}	460 N/mm ²
Partial factor for reinforcing steel	γ_s	1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_s$	400 N/mm ²
Design value of modulus of elasticity of reinforcing steel	E_s	200,000 N/mm ²

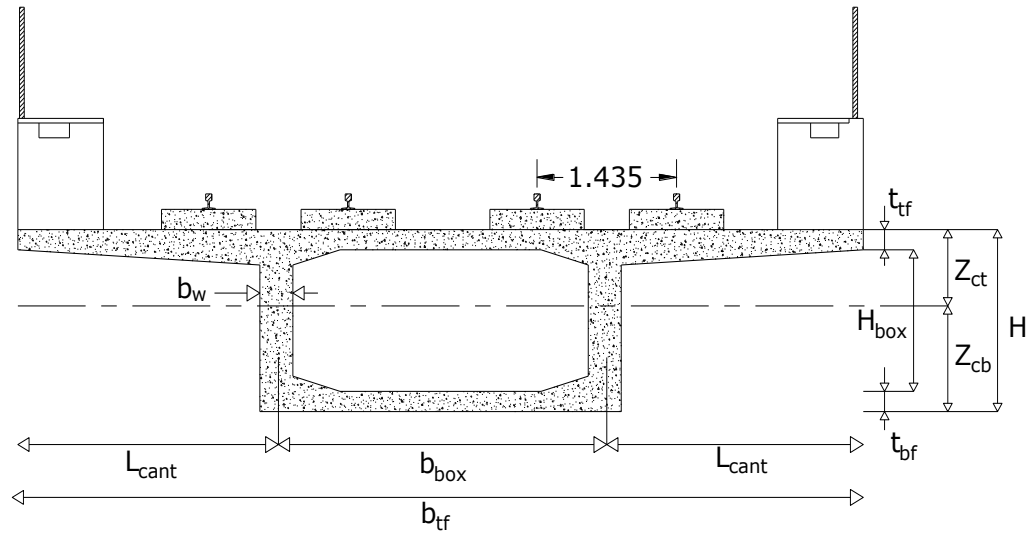
B.2.3 Prestressing steel FeP 1860

Density of prestressing steel	ρ_s	7850 kg/m ³
Characteristic tensile strength of prestressing steel	f_{pk}	1860 N/mm ²
Partial factor for prestressing steel	γ_s	1.15
Characteristic 0.1% proof-stress of prestressing steel	$f_{p0.1k}$	1600 N/mm ²
Design tensile strength of prestressing steel	$f_{pd} = f_{p0.1k} / \gamma_s$	1391 N/mm ²
Ultimate tensile strength of prestressing steel	f_{pk} / γ_s	1617 N/mm ²
Design value of modulus of elasticity of prestressing steel	E_p	195,000N/mm ²
Maximum tensile stress in the tendon (during tensioning)	min(0.8* f_{pk} or 0.9* $f_{p0.1k}$)	1440 N/mm ²
Maximum tensile stress in the tendon (Immediately after tensioning)	min(0.75* f_{pk} or 0.85* f_{pk})	1360 N/mm ²

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Working stress after all losses	$\leq 0.6^*$	1116 N/mm ²
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B.3 Geometry box girder



Cross-section of the box girder

B.3.1 General

Length span	L	Variable from 30m to 60m
Depth box girder	H	Design Variable
Center to center girder spacing	b_{box}	$3.83m$
Thickness top flange	t_{tf}	$0.2m$
Width web	b_w	Design Variable
Thickness bottom flange	t_{bf}	$0.2m$
Cantilever length top flange	L_{cant}	$2.74m$
Width top flange	b_{tf}	$8.96m$

B.3.2 Concrete cover

The concrete cover for this box girder made of C50 is:

$$C_{nom} = C_{min} + \Delta C_{dev} = 45mm$$

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B.4 Sample calculation

B.4.1 Geometric property

Length span	L	50	m
Depth of girder	H	3.05	m
Width web	b_w	0.35	m

B.4.2 Effective width of flanges

Effective width of flanges		Value	
Effective width cantilever length top flange	$b_{eff,1}$	2.56	m
Effective width inner top flange	$b_{eff,2}$	1.56	m
Effective width bottom flange	$b_{eff,3}$	1.56	m
Total effective flange width		Value	
Effective width top flange	$b_{eff,t}$	8.96	m
Effective width bottom flange	$b_{eff,b}$	3.83	m

B.4.3 Cross-sectional properties

Values cross-sectional properties box girder		Value	
Cross-sectional area of concrete	A_c	4.948	m^2
Distance from bottom to centroid axis	Z_{cb}	1.92	m
Distance from top to centroid axis	Z_{ct}	1.13	m
Second moment of area of the concrete section	I_c	6.388	m^4
Section modulus bottom	W_b	3.327	m^3
Section modulus top	W_t	5.65	m^3
Perimeter concrete box girder	u	23.77	m

B.4.4 Loads

B.4.4.1 General

Acceleration due to gravity	g	9.81	m/s^2
Dynamic factor	$1 + \mu = 1 + \alpha(6/30 + L)$ $\alpha = 4(1 - h) \leq 2$	1.15	
Safety factor for Wight of deck, favorable	$\gamma_{p,fav}$	0.9	
Safety factor for Wight of deck, unfavorable	$\gamma_{p,unfav}$	1.3	
Safety factor for prestress force, favorable	$\gamma_{p,fav}$	1.1	

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Safety factor for prestress force, unfavorable	$\gamma_{p,unfav}$	0.9	
Safety factor for variable actions, favorable	$\gamma_{f,fav}$	0.0	
Safety factor for vertical variable actions, unfavorable	$\gamma_{f,unfav}$	1.15	

B.4.4.2 Load schematization in longitudinal direction

Vertical loads

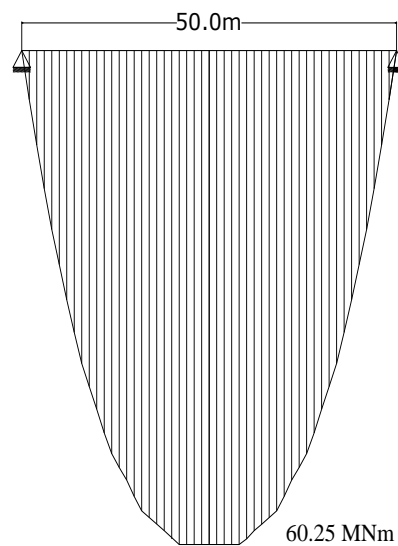
Permanent loads			
Dead load box girder	$g_{dead} = A_c * \rho_c * g$	123.7	kN/m
Concrete plinths		10.0	kN/m per track
Rail		0.97	kN/m per track
Cables		1.2	kN/m per cable duct
Walkway + guard-rail		2	kN/m per walkway
Sound insulation		1.3	kN/m per walkway
Concrete slope (drainage between walkways)		0.5	kN/m^2
Variable loads			
Common live load	Shown in (figure 6)		One and two track loaded envelop
Special live load	Shown in (figure 6)		

Maximum Bending moment due to Standard live load

Station (m)	Common live load Moment one track loaded (MNm)	Common live load Moment two track loaded (MNm)	Special rail live load Moment one track loaded (MNm)	Special rail live load Moment two track loaded (MNm)	Enveloped Standard live load Moment (MNm)
0.0	0.00	0.00	0.00	0.00	0.00
1.0	2.95	5.91	0.82	1.63	5.91
2.0	5.69	11.38	1.60	3.20	11.38
3.0	8.32	16.64	2.34	4.69	16.64
4.0	10.78	21.57	3.06	6.11	21.57
5.0	12.89	25.80	3.68	7.37	25.80
6.0	15.01	30.05	4.30	8.60	30.05
7.0	16.97	33.98	4.89	9.79	33.98
8.0	18.85	37.73	5.46	10.92	37.73
9.0	20.24	40.54	5.90	11.80	40.54
10.0	21.61	43.30	6.33	12.66	43.30
11.0	23.03	46.16	6.78	13.57	46.16
12.0	24.28	48.66	7.21	14.41	48.66
13.0	25.11	50.36	7.50	15.01	50.36
14.0	25.87	51.95	7.74	15.47	51.95
15.0	26.85	53.93	8.05	16.09	53.93
16.0	27.73	55.69	8.33	16.67	55.69
17.0	28.34	56.90	8.54	17.09	56.90
18.0	28.31	57.42	8.68	17.37	57.42
19.0	28.92	58.61	8.88	17.75	58.61
20.0	29.42	59.57	9.05	18.10	59.57

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21.0	29.76	60.25	9.19	18.38	60.25
22.0	29.33	58.92	9.20	18.40	58.92
23.0	29.55	59.34	9.27	18.54	59.34
24.0	29.68	59.62	9.32	18.64	59.62
25.0	29.71	59.69	9.35	18.71	59.69
26.0	29.68	59.62	9.32	18.64	59.62
27.0	29.55	59.35	9.27	18.54	59.35
28.0	29.33	58.92	9.20	18.40	58.92
29.0	29.76	60.02	9.19	18.38	60.02
30.0	29.42	59.57	9.05	18.10	59.57
31.0	28.92	58.60	8.88	17.75	58.60
32.0	28.31	57.42	8.68	17.37	57.42
33.0	28.34	56.90	8.54	17.09	56.90
34.0	27.73	55.69	8.33	16.67	55.69
35.0	26.85	53.93	8.05	16.09	53.93
36.0	25.87	51.95	7.74	15.47	51.95
37.0	25.11	50.37	7.50	15.01	50.37
38.0	24.28	48.66	7.21	14.41	48.66
39.0	23.03	46.16	6.78	13.57	46.16
40.0	21.61	43.30	6.33	12.66	43.30
41.0	20.24	40.55	5.90	11.80	40.55
42.0	18.84	37.72	5.46	10.91	37.72
43.0	16.97	33.97	4.89	9.79	33.97
44.0	15.01	30.04	4.30	8.60	30.04
45.0	12.89	25.79	3.68	7.37	25.79
46.0	10.78	21.57	3.06	6.11	21.57
47.0	8.32	16.64	2.34	4.69	16.64
48.0	5.69	11.38	1.60	3.20	11.38
49.0	2.95	5.91	0.82	1.63	5.91
50.0	0.00	0.00	0.00	0.00	0.00



Moment envelop line due to Standard live load for 50m span

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

Service and ultimate state bending moment

Station (m)	Standard live load Enveloped Moment (MNm)	Utility and attachment moment (MNm)	Box girder dead load Moment (MNm)	External load Service limit state moment (SLS)	External load Ultimate limit state moment (ULS)
0.0	0.00	0.00	0.00	0.00	0.01
1.0	5.91	0.84	3.20	9.95	12.05
2.0	11.38	1.64	6.26	19.28	23.36
3.0	16.64	2.41	9.20	28.25	34.23
4.0	21.57	3.15	12.00	36.72	44.50
5.0	25.80	3.79	14.44	44.03	53.37
6.0	30.05	4.43	16.89	51.37	62.27
7.0	33.98	5.06	19.26	58.30	70.69
8.0	37.73	5.64	21.50	64.87	78.67
9.0	40.54	6.10	23.23	69.87	84.75
10.0	43.30	6.56	24.99	74.85	90.81
11.0	46.16	7.03	26.80	79.99	97.06
12.0	48.66	7.47	28.48	84.61	102.69
13.0	50.36	7.78	29.64	87.78	106.56
14.0	51.95	8.04	30.64	90.63	110.03
15.0	53.93	8.38	31.91	94.22	114.40
16.0	55.69	8.68	33.06	97.43	118.31
17.0	56.90	8.89	33.89	99.68	121.05
18.0	57.42	9.03	34.41	100.86	122.51
19.0	58.61	9.24	35.19	103.04	125.16
20.0	59.57	9.41	35.84	104.82	127.33
21.0	60.25	9.52	36.29	106.06	128.84
22.0	58.92	9.55	36.37	104.84	127.45
23.0	59.34	9.62	36.67	105.63	128.42
24.0	59.62	9.67	36.84	106.13	129.03
25.0	59.69	9.68	36.88	106.25	129.17
26.0	59.62	9.67	36.84	106.13	129.03
27.0	59.35	9.62	36.67	105.64	128.43
28.0	58.92	9.55	36.37	104.84	127.45
29.0	60.02	9.52	36.29	105.83	128.58
30.0	59.57	9.41	35.84	104.82	127.33
31.0	58.60	9.24	35.19	103.03	125.15
32.0	57.42	9.03	34.41	100.86	122.51
33.0	56.90	8.89	33.89	99.68	121.05
34.0	55.69	8.68	33.06	97.43	118.31
35.0	53.93	8.38	31.91	94.22	114.40
36.0	51.95	8.04	30.64	90.63	110.03
37.0	50.37	7.78	29.64	87.79	106.57
38.0	48.66	7.47	28.48	84.61	102.69
39.0	46.16	7.03	26.80	79.99	97.06
40.0	43.30	6.56	24.99	74.85	90.81
41.0	40.55	6.10	23.23	69.88	84.76
42.0	37.72	5.64	21.50	64.86	78.66
43.0	33.97	5.06	19.26	58.29	70.68
44.0	30.04	4.43	16.89	51.36	62.26

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

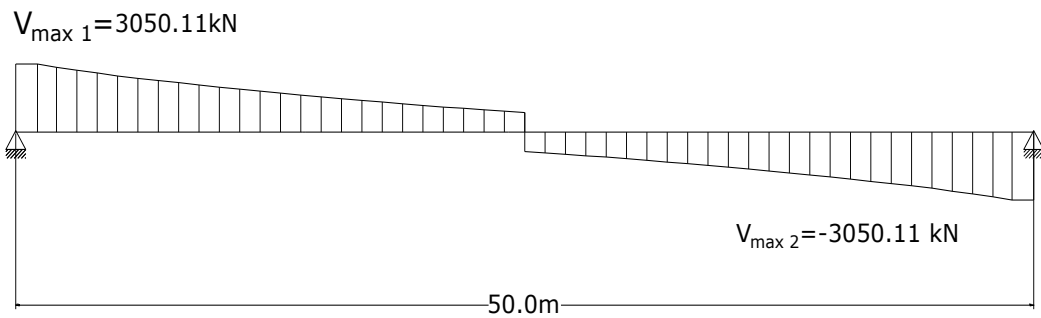
45.0	25.79	3.79	14.44	44.02	53.36
46.0	21.57	3.15	12.00	36.72	44.50
47.0	16.64	2.41	9.20	28.25	34.23
48.0	11.38	1.64	6.26	19.28	23.36
49.0	5.91	0.84	3.20	9.95	12.05
50.0	0.00	0.00	0.01	0.01	0.01

Maximum shear per girder due to Standard live load

Station (m)	Common live load Shear one track loaded (kN)	Common live load Shear two track loaded (kN)	Special rail live load Shear one track loaded (kN)	Special rail live load Shear two track loaded (kN)	Enveloped Standard live load Shear (kN)
0.0	2805.07	3050.11	762.27	859.15	3050.11
1.0	2805.07	3050.11	762.27	859.15	3050.11
2.0	2579.57	2906.45	723.30	843.80	2906.45
3.0	2370.03	2775.22	683.75	825.03	2775.22
4.0	2189.17	2657.19	647.42	804.17	2657.19
5.0	2012.35	2518.00	609.09	773.80	2518.00
6.0	1876.63	2410.94	579.30	749.64	2410.94
7.0	1764.65	2315.00	555.38	728.29	2315.00
8.0	1668.71	2222.63	535.48	708.07	2222.63
9.0	1577.66	2113.88	516.14	683.14	2113.88
10.0	1500.69	2015.97	499.88	660.68	2015.97
11.0	1435.82	1928.69	487.64	642.17	1928.69
12.0	1377.17	1842.79	477.35	624.59	1842.79
13.0	1321.76	1752.21	468.00	606.09	1752.21
14.0	1269.17	1662.24	458.26	586.57	1662.24
15.0	1220.70	1580.78	449.92	569.31	1580.78
16.0	1174.36	1499.93	442.25	552.61	1499.93
17.0	1131.49	1423.62	436.09	538.00	1423.62
18.0	1090.59	1350.67	430.24	524.15	1350.67
19.0	1046.91	1270.68	422.55	507.32	1270.68
20.0	1004.15	1191.48	415.08	490.86	1191.48
21.0	964.34	1119.23	408.63	476.25	1119.23
22.0	932.15	1068.61	404.97	467.05	1068.61
23.0	890.74	1001.00	396.86	450.11	1001.00
24.0	850.19	937.54	388.76	433.30	937.54
25.0	810.98	877.59	380.89	416.75	877.59
25.0	-810.98	-877.59	-380.89	-416.75	-877.59
26.0	-850.19	-937.55	-388.76	-433.30	-937.55
27.0	-890.74	-1001.00	-396.86	-450.11	-1001.00
28.0	-932.15	-1068.61	-404.97	-467.05	-1068.61
29.0	-964.34	-1119.23	-408.63	-476.25	-1119.23
30.0	-1004.15	-1191.48	-415.08	-490.86	-1191.48
31.0	-1046.90	-1270.67	-422.55	-507.32	-1270.67
32.0	-1090.59	-1350.67	-430.24	-524.15	-1350.67
33.0	-1131.49	-1423.62	-436.09	-538.00	-1423.62
34.0	-1174.35	-1499.92	-442.25	-552.61	-1499.92
35.0	-1220.70	-1580.78	-449.92	-569.31	-1580.78

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36.0	-1269.17	-1662.24	-458.26	-586.57	-1662.24
37.0	-1321.76	-1752.21	-468.00	-606.09	-1752.21
38.0	-1377.13	-1842.76	-477.35	-624.59	-1842.76
39.0	-1435.82	-1928.69	-487.64	-642.17	-1928.69
40.0	-1500.69	-2015.97	-499.88	-660.68	-2015.97
41.0	-1577.67	-2113.89	-516.14	-683.14	-2113.89
42.0	-1668.72	-2222.64	-535.48	-708.07	-2222.64
43.0	-1764.65	-2315.00	-555.38	-728.29	-2315.00
44.0	-1876.61	-2410.92	-579.30	-749.64	-2410.92
45.0	-2012.36	-2518.01	-609.09	-773.80	-2518.01
46.0	-2189.13	-2657.15	-647.42	-804.17	-2657.15
47.0	-2370.05	-2775.24	-683.75	-825.03	-2775.24
48.0	-2579.57	-2906.45	-723.30	-843.80	-2906.45
49.0	-2805.08	-3050.11	-762.27	-859.15	-3050.11
50.0	-2805.08	-3050.11	-762.27	-859.15	-3050.11



Shear envelop line due to Standard live load for 50m span

Ultimate limit state external load shear force per girder

Station (m)	Enveloped Standard live load Shear (kN)	Utility and attachment Shear (kN)	Box girder dead load shear (kN)	External load Service limit state shear (SLS)(kN)	External load Ultimate limit state Shear (ULS)(kN)
0.0	3050.11	419.43	1627.21	5096.75	6168.26
1.0	3050.11	419.43	1563.46	5033	6085.38
2.0	2906.45	402.22	1499.71	4808.38	5814.93
3.0	2775.22	385.01	1435.95	4596.18	5558.75
4.0	2657.19	367.80	1372.20	4397.19	5317.77
5.0	2518.00	345.86	1283.98	4147.84	5014.49
6.0	2410.94	327.76	1215.57	3954.27	4778.91
7.0	2315.00	310.55	1151.82	3777.37	4563.33
8.0	2222.63	293.34	1088.07	3604.04	4351.86
9.0	2113.88	273.33	1008.55	3395.76	4097.41
10.0	2015.97	254.76	937.16	3207.89	3867.86
11.0	1928.69	237.55	873.40	3039.64	3662.23
12.0	1842.79	220.34	809.65	2872.78	3458.20
13.0	1752.21	202.56	739.90	2694.67	3240.24
14.0	1662.24	184.77	670.16	2517.17	3022.99

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15.0	1580.78	167.56	606.40	2354.74	2824.05
16.0	1499.93	150.35	544.62	2194.9	2628.38
17.0	1423.62	134.33	484.69	2042.64	2441.89
18.0	1350.67	119.44	420.93	1891.04	2255.75
19.0	1270.68	102.23	357.18	1730.09	2058.52
20.0	1191.48	85.02	296.06	1572.56	1865.61
21.0	1119.23	68.84	244.85	1432.92	1694.91
22.0	1068.61	56.57	181.10	1306.28	1537.87
23.0	1001.00	39.36	117.35	1157.71	1354.87
24.0	937.54	22.15	53.60	1013.29	1176.65
25.0	877.59	4.94	10.15	892.68	1028.85
25.0	-877.59	-4.94	-10.15	-892.68	-1028.85
26.0	-937.55	-22.15	-53.60	-1013.3	-1176.66
27.0	-1001.00	-39.36	-117.35	-1157.71	-1354.87
28.0	-1068.61	-56.57	-181.10	-1306.28	-1537.87
29.0	-1119.23	-68.84	-244.85	-1432.92	-1694.91
30.0	-1191.48	-85.02	-296.06	-1572.56	-1865.61
31.0	-1270.67	-102.23	-357.18	-1730.08	-2058.50
32.0	-1350.67	-119.44	-420.93	-1891.04	-2255.75
33.0	-1423.62	-134.33	-484.69	-2042.64	-2441.89
34.0	-1499.92	-150.35	-544.62	-2194.89	-2628.37
35.0	-1580.78	-167.56	-606.40	-2354.74	-2824.05
36.0	-1662.24	-184.77	-670.16	-2517.17	-3022.99
37.0	-1752.21	-202.56	-739.90	-2694.67	-3240.24
38.0	-1842.76	-220.34	-809.65	-2872.75	-3458.16
39.0	-1928.69	-237.55	-873.40	-3039.64	-3662.23
40.0	-2015.97	-254.76	-937.16	-3207.89	-3867.86
41.0	-2113.89	-273.33	-1008.55	-3395.77	-4097.42
42.0	-2222.64	-293.34	-1088.07	-3604.05	-4351.87
43.0	-2315.00	-310.55	-1151.82	-3777.37	-4563.33
44.0	-2410.92	-327.76	-1215.57	-3954.25	-4778.89
45.0	-2518.01	-345.86	-1283.98	-4147.85	-5014.50
46.0	-2657.15	-367.80	-1372.20	-4397.15	-5317.72
47.0	-2775.24	-385.01	-1435.95	-4596.2	-5558.77
48.0	-2906.45	-402.22	-1499.71	-4808.38	-5814.93
49.0	-3050.11	-419.43	-1563.46	-5033	-6085.38
50.0	-3050.11	-419.43	-1627.21	-5096.75	-6168.26

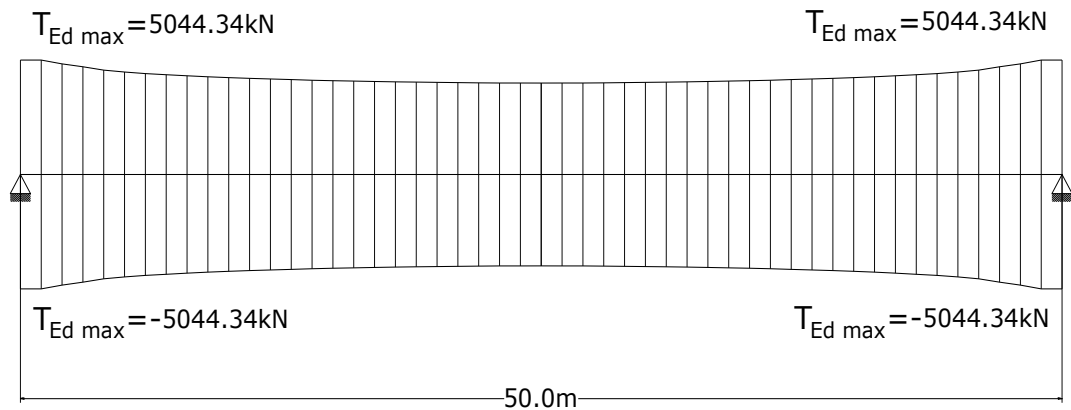
Maximum Torsion due to rail live load

Station (m)	Common live load torsion one track loaded (kNm) ±	Common live load torsion two track loaded (kNm) ±	Special rail live load torsion one track loaded (kNm) ±	Special rail live load torsion two track loaded (kNm) ±	Enveloped Standard live load torsion (kNm) ±
0.0	5026.98	5044.34	1271.78	1291.50	5044.34
1.0	5026.98	5044.34	1271.78	1291.50	5044.34
2.0	4840.12	4878.25	1210.61	1253.93	4878.25
3.0	4662.99	4748.68	1153.82	1266.53	4748.68
4.0	4496.86	4640.11	1103.74	1280.95	4640.11
5.0	4314.61	4516.70	1051.30	1303.29	4516.70
6.0	4162.16	4449.03	1010.08	1328.63	4449.03

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7.0	4022.64	4400.35	976.37	1352.61	4400.35
8.0	3889.74	4352.00	947.59	1367.56	4352.00
9.0	3766.05	4306.88	921.93	1382.71	4306.88
10.0	3643.94	4270.14	898.63	1400.11	4270.14
11.0	3524.18	4239.06	879.64	1415.78	4239.06
12.0	3407.67	4211.68	863.61	1423.93	4211.68
13.0	3298.11	4183.53	850.35	1429.07	4183.53
14.0	3189.29	4155.91	836.75	1438.69	4155.91
15.0	3078.59	4135.21	824.63	1447.26	4135.21
16.0	2970.38	4117.52	814.07	1452.18	4117.52
17.0	2866.07	4100.58	804.99	1452.85	4100.58
18.0	2764.24	4082.73	795.59	1457.46	4082.73
19.0	2660.50	4070.56	786.41	1461.52	4070.56
20.0	2558.33	4060.65	777.92	1464.13	4060.65
21.0	2457.51	4051.67	770.01	1464.13	4051.67
22.0	2357.24	4039.68	761.98	1464.21	4039.68
23.0	2258.00	4034.40	753.35	1465.49	4034.40
24.0	2159.93	4031.20	744.92	1466.38	4031.20
25.0	2062.91	4029.92	736.51	1466.38	4029.92
26.0	2159.93	4031.20	744.92	1466.38	4031.20
27.0	2258.00	4034.40	753.35	1465.49	4034.40
28.0	2357.24	4039.68	761.98	1464.21	4039.68
29.0	2457.52	4051.68	770.03	1464.15	4051.68
30.0	2558.33	4060.65	777.92	1464.15	4060.65
31.0	2660.50	4070.56	786.41	1461.52	4070.56
32.0	2764.24	4082.73	795.59	1457.46	4082.73
33.0	2866.08	4100.59	804.99	1452.85	4100.59
34.0	2970.37	4117.51	814.07	1452.18	4117.51
35.0	3078.58	4135.20	824.63	1447.26	4135.20
36.0	3189.30	4155.91	836.75	1438.69	4155.91
37.0	3298.11	4183.53	850.35	1429.07	4183.53
38.0	3407.66	4211.68	863.61	1423.93	4211.68
39.0	3524.20	4239.08	879.64	1415.78	4239.08
40.0	3643.96	4270.15	898.63	1400.11	4270.15
41.0	3766.05	4306.88	921.93	1382.71	4306.88
42.0	3889.74	4352.00	947.59	1367.56	4352.00
43.0	4022.64	4400.35	976.37	1352.61	4400.35
44.0	4162.16	4449.03	1010.08	1328.63	4449.03
45.0	4314.61	4516.70	1051.30	1303.29	4516.70
46.0	4496.86	4640.11	1103.74	1280.95	4640.11
47.0	4662.99	4748.68	1153.82	1266.53	4748.68
48.0	4840.12	4878.25	1210.61	1253.93	4878.25
49.0	5026.98	5044.34	1271.78	1291.50	5044.34
50.0	5026.98	5044.34	1271.78	1291.50	5044.34

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Torsion envelop line due to Standard live load for 50m span

Ultimate limit state external load Torsion moment

Station (m)	Enveloped Standard load Torsion (kNm)	External load Service limit state Torsion (SLS)(kNm)	External load Ultimate limit state Torsion (ULS)(kNm)
0.0	5044.34	5044.34	5800.99
1.0	5044.34	5044.34	5800.99
2.0	4878.25	4878.25	5609.99
3.0	4748.68	4748.68	5460.98
4.0	4640.11	4640.11	5336.13
5.0	4516.70	4516.70	5194.21
6.0	4449.03	4449.03	5116.38
7.0	4400.35	4400.35	5060.40
8.0	4352.00	4352.00	5004.80
9.0	4306.88	4306.88	4952.91
10.0	4270.14	4270.14	4910.66
11.0	4239.06	4239.06	4874.92
12.0	4211.68	4211.68	4843.43
13.0	4183.53	4183.53	4811.06
14.0	4155.91	4155.91	4779.30
15.0	4135.21	4135.21	4755.49
16.0	4117.52	4117.52	4735.15
17.0	4100.58	4100.58	4715.67
18.0	4082.73	4082.73	4695.14
19.0	4070.56	4070.56	4681.14
20.0	4060.65	4060.65	4669.75
21.0	4051.67	4051.67	4659.42
22.0	4039.68	4039.68	4645.63
23.0	4034.40	4034.40	4639.56
24.0	4031.20	4031.20	4635.88
25.0	4029.92	4029.92	4634.41
26.0	4031.20	4031.20	4635.88
27.0	4034.40	4034.40	4639.56

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

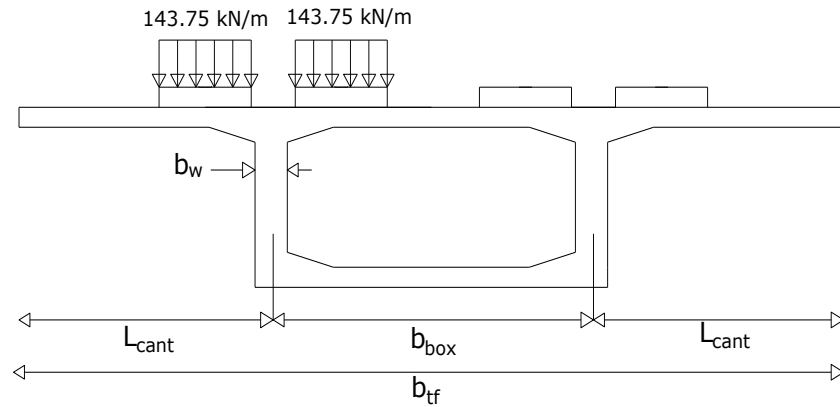
28.0	4039.68	4039.68	4645.63
29.0	4051.68	4051.68	4659.43
30.0	4060.65	4060.65	4669.75
31.0	4070.56	4070.56	4681.14
32.0	4082.73	4082.73	4695.14
33.0	4100.59	4100.59	4715.68
34.0	4117.51	4117.51	4735.14
35.0	4135.20	4135.20	4755.48
36.0	4155.91	4155.91	4779.30
37.0	4183.53	4183.53	4811.06
38.0	4211.68	4211.68	4843.43
39.0	4239.08	4239.08	4874.94
40.0	4270.15	4270.15	4910.67
41.0	4306.88	4306.88	4952.91
42.0	4352.00	4352.00	5004.80
43.0	4400.35	4400.35	5060.40
44.0	4449.03	4449.03	5116.38
45.0	4516.70	4516.70	5194.21
46.0	4640.11	4640.11	5336.13
47.0	4748.68	4748.68	5460.98
48.0	4878.25	4878.25	5609.99
49.0	5044.34	5044.34	5800.99
50.0	5044.34	5044.34	5800.99

B.4.4.3 Load schematization in transversal direction

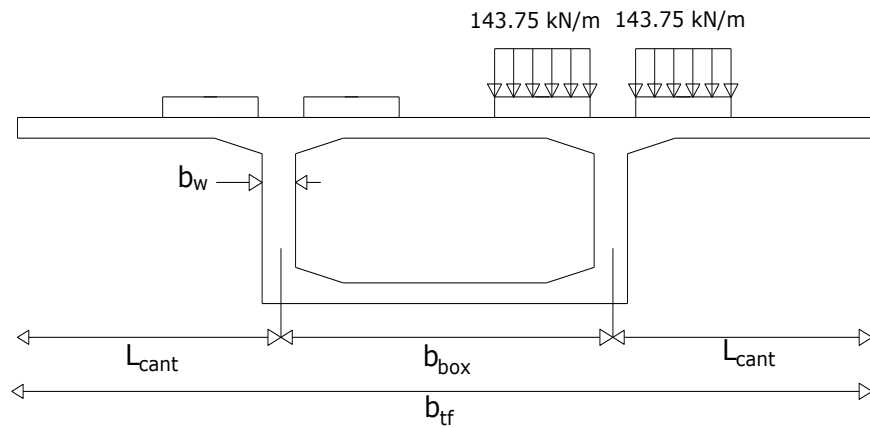
Vertical load

Permanent loads			
Concrete plinths	(/2 plinths per track/1m (width plinth))	5	<i>kN/m</i>
Rail	(/ 2 rails per track)	0.485	<i>kN per rail</i>
Cables		1.2	<i>kN per cable duct</i>
Walkway + guard-rail	(/ 1 m (width walkway))	2	<i>kN/m</i>
Sound insulation		1.3	<i>kN per walkway</i>
Concrete slope (drainage between walkways)		0.5	<i>kN/m</i>
Variable loads			
Concentrated load due to Live (without dynamic factor)	Q_{mob}	250	<i>kN per track</i>
Concentrated load due to Live (with dynamic factor)	$Q_{mob} * (1 + \mu) / 2$	143.75	<i>kN per rail</i>

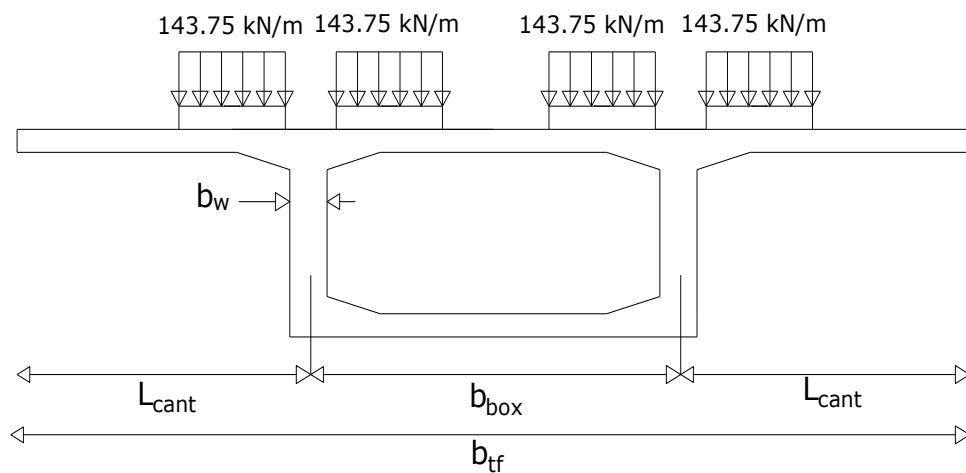
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



Rail live load on the left track of the deck for 50m span



Rail live load on the right track of the deck for 50m span



Rail live load on both track of the deck for 50m span

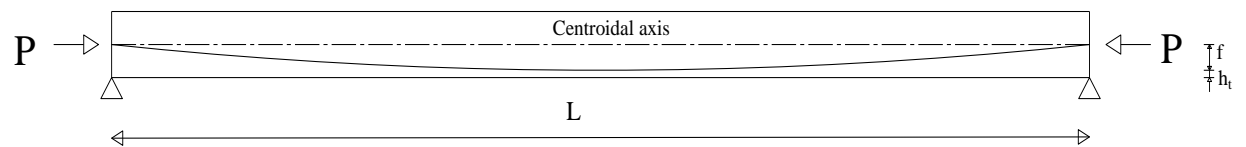
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Top slab Transverse ultimate maximum moment

Distance from left edge of deck (m)	Left lane loaded ultimate Moment (kNm)	Right lane loaded ultimate Moment (kNm)	Two lane loaded ultimate Moment (kNm)	Enveloped maximum ultimate moment (kNm)
0.00	0.00	0.00	0.00	0.00
0.46	-0.94	-0.94	-0.94	-0.94
0.85	-3.87	-3.87	-3.87	-3.87
1.24	-8.08	-8.08	-8.08	-8.08
1.63	-14.41	-13.44	-14.41	-14.41
2.02	-41.28	-20.62	-41.28	-41.28
2.38	-90.43	-29.30	-90.43	-90.43
2.74	-158.62	-39.59	-158.62	-158.62
3.12	-40.70	-4.47	-39.41	-40.70
3.50	3.89	4.54	9.06	9.06
4.00	22.93	12.93	33.10	33.10
4.50	18.79	18.79	33.97	33.97
5.00	12.93	22.93	33.10	33.10
5.50	4.54	3.89	9.06	9.06
5.84	-4.47	-40.70	-39.41	-40.70
6.22	-39.59	-158.62	-158.62	-158.62
6.60	-29.30	-90.43	-90.43	-90.43
6.94	-20.62	-41.28	-41.28	-41.28
7.33	-13.44	-14.41	-14.41	-14.41
7.72	-8.08	-8.08	-8.08	-8.08
8.11	-3.87	-3.87	-3.87	-3.87
8.51	-0.94	-0.94	-0.94	-0.94
8.96	0.00	0.00	0.00	0.00

B.4.5 Prestressing tendons

B.4.5.1 Layout prestressing tendons



The tendon vertical profile is parabolic with the following ordinate from center:

Tendon distance	0.0	4.17	8.33	12.5	16.67	20.8	25.0	29.17	33.3	37.5	41.67	45.8	50.0
Vertical ordinate	0.0	0.52	0.95	1.27	1.51	1.65	1.70	1.65	1.51	1.27	0.95	0.52	0.0

Distance between the center of the tendons and bottom side at mid-span $h_t = 0.218m$ and

Tendon eccentricity at mid-span $Z_{cp,m} = Z_{cb} - h_t = 1.70m$

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B.4.5.2 Prestressing losses

B.4.5.2.1 Losses due to the instantaneous deformation of concrete

$$\Delta p_{el} = A_p * E_p * \sum [j * \Delta \sigma_c(t) / E_{cm}]$$

Where: $A_p = 2700mm^2$ Cross-sectional area per prestressing tendon

$P = 66504kN$ Initial Prestress force

$E_c = 37000 N/mm^2$ Secant modulus of elasticity of concrete

Total Area of tendon required $A_{ten,total} = P / \sigma_{pm0}$ 48900mm²

Sixteen tendons consists of 27 Strands with 100mm² Section area and Two tendon consist of 19 strands with 150mm² section area, Provide

$A_{ten,total}$ Number of tendon = $A_{ten,total} / A_p$ 18 number of tendon

This prestressing loss assumed to be compensated by slightly overstressing the tendons.

B.4.5.2.2 Losses due to friction and wobble

The force at any point due to friction and wobble effect in post-tensioned tendons is:

$$P_x = P_0 e^{-\mu(\sum \alpha + \omega x)}$$

Parameter assumed value as follows:

Where ω = wobble coefficient 0.006 /m

μ = Friction coefficient 0.15

B.4.5.2.3 Loss due to Anchor set

This prestressing loss also assumed to be compensated by overstressing the tendons.

B.4.5.2.4 Time dependent losses of prestress for post-tensioning

Creep

$\varphi(\infty, t_0) = 1.2$ Is the final creep coefficient according Figure 3.1b [12]
(Outside conditions; C-50; Class N; $t_0 = 30$ days;
 $h_0 = 2 * A_c / u = 416.3mm$)

Shrinkage

$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 0.00016231$ Is the estimated shrinkage strain in absolute value

Where

$\varepsilon_{cd}(\infty) = k_h * \varepsilon_{cd,0} = 0.162\%$

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$\varepsilon_{cd,0} = 0.23\%$	Is the nominal unrestrained drying shrinkage value According Table 3.2 [12] (Relative humidity 80%; C-50)
$k_h = 0.72$	A coefficient depending on the notional size h_0 According to Table 3.3 [12]
$\varepsilon_{ca}(\infty) = 2.5(f_{ck} - 10) * 10^{-6}$	
$\varepsilon_{ca}(\infty) = 0.0001\%$	Is the autogenous shrinkage

Relaxation

Relaxation class 2 (wire or strand):

$$\Delta\sigma_{pr} = (\sigma_{pi}) * 0.66 * \rho_{1000} * e^{9.1\mu} \left(\frac{t}{1000} \right)^{0.75(1-\mu)} * 10^{-5} = 60.93 \text{ N/mm}^2$$

Where:

$$\text{Concrete stress } \sigma_{c,QP} = \frac{P_0}{A_c} = 13.44 \text{ N/mm}^2$$

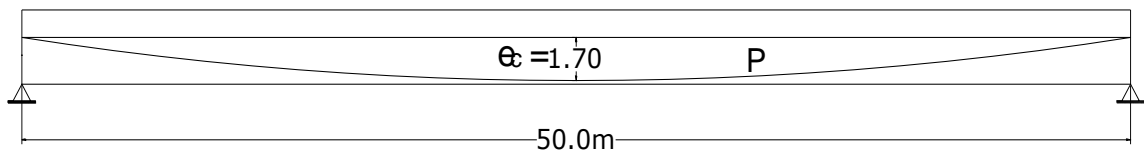
Time dependent loss of prestress for post-tensioning at mid-span

$$\Delta P_{c+s+r,s} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_o) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} \left(1 + \frac{A_c}{I_{c,s}} Z_{cp,s}^2 \right) [1 + 0.8\varphi(t, t_0)]} = 6076.717 \text{ kN}$$

$Z_{cp,m} = 1701.38 \text{ mm}$ Is the distance between the center of gravity of the
Concrete section and the tendons at mid span

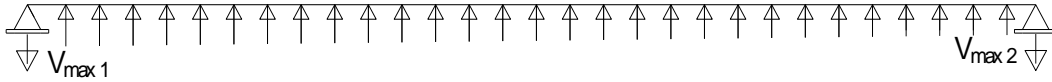
B.4.5.3 Bending moments and shear force due to prestressing

The moment diagram and structural schematization due to prestressing is shown



Parabolic prestress layout anchored at neutral axis for 50m span

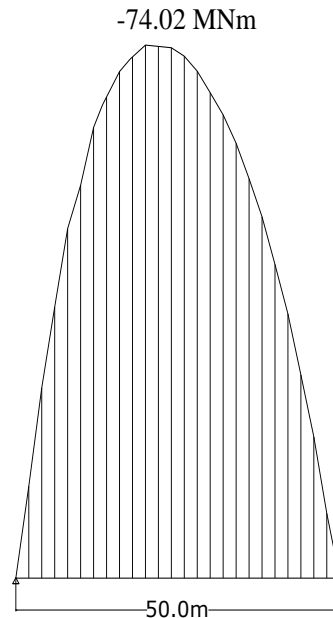
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



Equivalent load due to prestress action for 50m span

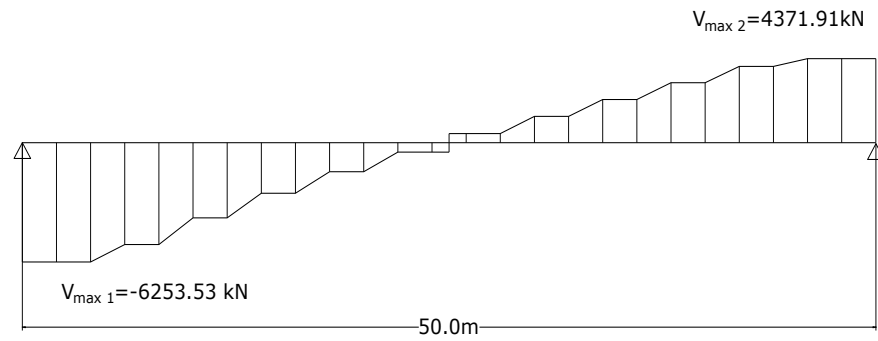
Effective Prestress force effect of shear and moment

Tendon distance (m)	Effective prestress Shear force (kN)	Effective prestress moment (MNm)
0	-6253.53	0.00
5	-5815.37	-32.55
10	-4114.93	-55.06
15	-2699.30	-68.10
20	-1465.65	-73.88
25	-361.91	-74.02
25	352.72	-74.02
30	1338.60	-67.87
35	2302.39	-58.15
40	3272.65	-43.82
45	4281.33	-24.28
50	4371.91	0.00



Bending moment diagram due to prestress force action for 50m span

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Shear force diagram due to prestress action for 50m span

B.4.6 Table of service stresses

Distance (m)	Bottom fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	-0.01	10.30	0.00	0.00	0.00
	Cumulated		+10.29	+10.28	+10.28	+10.28
1	Partial	-0.96	12.40	-0.20	-0.25	-1.77
	Cumulated		+11.44	+11.24	+10.99	+9.22
2	Partial	-1.89	15.92	-1.81	-0.50	-3.42
	Cumulated		+14.03	+12.22	+11.73	+8.31
3	Partial	-2.74	18.82	-2.80	-0.72	-4.95
	Cumulated		+16.08	+13.28	+12.56	+7.61
4	Partial	-3.54	21.06	-3.14	-0.93	-6.35
	Cumulated		+17.52	+14.39	+13.46	+7.11
5	Partial	-4.30	22.89	-2.59	-1.13	-7.66
	Cumulated		+18.59	+16.01	+14.88	+7.22
6	Partial	-5.01	24.54	-2.46	-1.31	-8.89
	Cumulated		+19.53	+17.08	+15.76	+6.87
7	Partial	-5.67	26.04	-2.32	-1.49	-9.98
	Cumulated		+20.37	+18.05	+16.56	+6.58
8	Partial	-6.29	27.39	-2.85	-1.65	-11.01
	Cumulated		+21.10	+18.25	+16.60	+5.59
9	Partial	-6.85	28.59	-3.12	-1.80	-11.92
	Cumulated		+21.74	+18.62	+16.82	+4.90
10	Partial	-7.37	29.65	-2.97	-1.93	-12.73
	Cumulated		+22.28	+19.30	+17.37	+4.64
11	Partial	-7.84	30.58	-2.70	-2.06	-13.46
	Cumulated		+22.74	+20.04	+17.99	+4.53
12	Partial	-8.26	31.40	-2.31	-2.17	-14.08
	Cumulated		+23.14	+20.82	+18.66	+4.58
13	Partial	-8.65	32.11	-2.80	-2.27	-14.64
	Cumulated		+23.46	+20.66	+18.40	+3.76
14	Partial	-8.99	32.72	-3.41	-2.36	-15.16

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	Cumulated		+23.73	+20.32	+17.96	+2.79
15	Partial	-9.30	33.23	-3.11	-2.44	-15.63
	Cumulated		+23.93	+20.83	+18.39	+2.76
16	Partial	-9.57	33.66	-3.25	-2.51	-16.04
	Cumulated		+24.09	+20.84	+18.33	+2.30
17	Partial	-9.81	33.93	-3.87	-2.57	-16.39
	Cumulated		+24.13	+20.25	+17.68	+1.29
18	Partial	-10.00	34.24	-3.72	-2.62	-16.67
	Cumulated		+24.25	+20.53	+17.91	+1.24
19	Partial	-10.18	34.46	-3.47	-2.67	-16.94
	Cumulated		+24.28	+20.81	+18.14	+1.20
20	Partial	-10.34	34.58	-3.34	-2.71	-17.16
	Cumulated		+24.25	+20.91	+18.20	+1.04
21	Partial	-10.45	34.62	-3.99	-2.74	-17.32
	Cumulated		+24.17	+20.18	+17.44	+0.13
22	Partial	-10.54	34.62	-3.83	-2.76	-17.43
	Cumulated		+24.08	+20.25	+17.49	+0.05
23	Partial	-10.60	34.51	-3.57	-2.78	-17.51
	Cumulated		+23.91	+20.34	+17.56	+0.05
24	Partial	-10.27	34.43	-3.34	-2.79	-17.53
	Cumulated		+24.16	+20.82	+18.03	+0.51
25	Partial	-10.64	34.21	-3.97	-2.79	-17.53
	Cumulated		+23.57	+19.60	+16.81	-0.72
26	Partial	-10.63	33.93	-3.83	-2.79	-17.49
	Cumulated		+23.30	+19.47	+16.68	-0.81
27	Partial	-10.60	33.65	-3.70	-2.78	-17.50
	Cumulated		+23.05	+19.35	+16.57	-0.93
28	Partial	-10.54	33.27	-3.47	-2.76	-17.44
	Cumulated		+22.73	+19.26	+16.50	-0.93
29	Partial	-10.45	32.86	-3.72	-2.74	-17.32
	Cumulated		+22.41	+18.68	+15.94	-1.38
30	Partial	-10.34	32.38	-3.73	-2.71	-17.16
	Cumulated		+22.05	+18.31	+15.60	-1.56
31	Partial	-10.18	31.85	-3.62	-2.67	-16.94
	Cumulated		+21.67	+18.04	+15.37	-1.57
32	Partial	-10.00	31.25	-3.45	-2.62	-16.67
	Cumulated		+21.25	+17.81	+15.18	-1.48
33	Partial	-9.81	30.62	-3.50	-2.57	-16.39
	Cumulated		+20.81	+17.31	+14.74	-1.65
34	Partial	-9.65	29.91	-3.53	-2.53	-16.17
	Cumulated		+20.26	+16.73	+14.20	-1.97
35	Partial	-9.30	29.14	-3.45	-2.44	-15.63
	Cumulated		+19.84	+16.39	+13.95	-1.68
36	Partial	-8.99	28.31	-3.32	-2.36	-15.16
	Cumulated		+19.32	+16.00	+13.64	-1.52
37	Partial	-8.65	27.41	-3.23	-2.27	-14.64
	Cumulated		+18.76	+15.54	+13.27	-1.37

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38	Partial	-8.26	26.44	-3.20	-2.17	-14.08
	Cumulated		+18.18	+14.98	+12.81	-1.27
39	Partial	-7.84	25.40	-3.12	-2.06	-13.46
	Cumulated		+17.57	+14.44	+12.39	-1.07
40	Partial	-7.37	24.29	-2.98	-1.93	-12.73
	Cumulated		+16.92	+13.94	+12.01	-0.71
41	Partial	-6.85	23.09	-2.83	-1.80	-11.92
	Cumulated		+16.24	+13.41	+11.61	-0.31
42	Partial	-6.29	21.82	-2.79	-1.65	-11.01
	Cumulated		+15.53	+12.74	+11.10	+0.08
43	Partial	-5.67	20.45	-2.65	-1.49	-9.98
	Cumulated		+14.78	+12.13	+10.65	+0.67
44	Partial	-5.01	19.00	-2.42	-1.31	-8.89
	Cumulated		+13.99	+11.57	+10.26	+1.37
45	Partial	-4.30	17.44	-2.35	-1.13	-7.66
	Cumulated		+13.15	+10.80	+9.68	+2.02
46	Partial	-3.54	15.76	-3.10	-0.93	-6.35
	Cumulated		+12.22	+9.12	+8.19	+1.84
47	Partial	-2.74	13.85	-2.54	-0.72	-4.95
	Cumulated		+11.11	+8.57	+7.85	+2.90
48	Partial	-1.89	11.56	-1.60	-0.50	-3.42
	Cumulated		+9.68	+8.07	+7.58	+4.16
49	Partial	-0.96	8.91	-0.30	-0.25	-1.77
	Cumulated		+7.95	+7.65	+7.40	+5.63
50	Partial	-0.01	7.40	-0.13	0.00	0.00
	Cumulated		+7.39	+7.26	+7.26	+7.26

Distance (m)	Top fiber stress in (MPa)					
		Self-Wight	Initial Prestress	Prestress loss	Utility and attachment	Live load
0	Partial	0.02	11.33	-1.05	0.00	0.05
	Cumulated		+11.34	+10.29	+10.29	+10.34
1	Partial	0.58	10.09	-0.93	0.15	1.08
	Cumulated		+10.67	+9.74	+9.89	+10.97
2	Partial	1.13	8.85	-0.82	0.30	2.06
	Cumulated		+9.99	+9.17	+9.46	+11.52
3	Partial	1.67	8.03	-1.12	0.44	3.02
	Cumulated		+9.70	+8.58	+9.02	+12.04
4	Partial	2.18	6.79	-0.78	0.57	3.91
	Cumulated		+8.97	+8.19	+8.76	+12.67
5	Partial	2.66	5.59	-0.54	0.70	4.73
	Cumulated		+8.25	+7.72	+8.41	+13.15
6	Partial	3.11	5.04	-0.35	0.82	5.52
	Cumulated		+8.15	+7.81	+8.62	+14.14
7	Partial	3.54	3.35	-0.22	0.93	6.22
	Cumulated		+6.89	+6.67	+7.60	+13.82

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8	Partial	3.94	2.34	-0.17	1.03	6.89
	Cumulated		+6.28	+6.11	+7.14	+14.03
9	Partial	4.31	1.40	-0.09	1.13	7.50
	Cumulated		+5.71	+5.62	+6.75	+14.25
10	Partial	4.66	0.54	0.04	1.22	8.04
	Cumulated		+5.19	+5.23	+6.45	+14.49
11	Partial	4.98	-0.25	0.11	1.31	8.55
	Cumulated		+4.73	+4.84	+6.15	+14.69
12	Partial	5.27	-0.95	0.13	1.38	8.99
	Cumulated		+4.32	+4.45	+5.84	+14.82
13	Partial	5.55	-1.59	0.21	1.45	9.39
	Cumulated		+3.96	+4.17	+5.62	+15.01
14	Partial	5.79	-2.16	0.30	1.52	9.76
	Cumulated		+3.63	+3.94	+5.45	+15.22
15	Partial	6.01	-2.66	0.33	1.58	10.10
	Cumulated		+3.35	+3.68	+5.25	+15.36
16	Partial	6.21	-3.11	0.34	1.63	10.41
	Cumulated		+3.10	+3.44	+5.07	+15.48
17	Partial	6.39	-3.49	0.42	1.67	10.67
	Cumulated		+2.90	+3.32	+4.99	+15.66
18	Partial	6.54	-3.82	0.48	1.71	10.90
	Cumulated		+2.71	+3.19	+4.91	+15.81
19	Partial	6.67	-4.10	0.48	1.75	11.10
	Cumulated		+2.57	+3.05	+4.80	+15.90
20	Partial	6.78	-4.33	0.46	1.78	11.26
	Cumulated		+2.45	+2.92	+4.69	+15.95
21	Partial	6.87	-4.50	0.53	1.80	11.39
	Cumulated		+2.37	+2.90	+4.70	+16.08
22	Partial	6.94	-4.63	0.57	1.82	11.48
	Cumulated		+2.31	+2.87	+4.69	+16.17
23	Partial	6.99	-4.71	0.56	1.83	11.54
	Cumulated		+2.28	+2.83	+4.67	+16.21
24	Partial	7.02	-4.76	0.51	1.84	11.58
	Cumulated		+2.26	+2.77	+4.61	+16.19
25	Partial	7.03	-4.74	0.55	1.84	11.58
	Cumulated		+2.28	+2.83	+4.67	+16.25
26	Partial	7.02	-4.69	0.58	1.84	11.57
	Cumulated		+2.33	+2.91	+4.75	+16.32
27	Partial	6.99	-4.59	0.57	1.83	11.55
	Cumulated		+2.40	+2.97	+4.80	+16.34
28	Partial	6.94	-4.45	0.52	1.82	11.48
	Cumulated		+2.49	+3.00	+4.82	+16.30
29	Partial	6.87	-4.27	0.49	1.80	11.39
	Cumulated		+2.60	+3.09	+4.89	+16.28
30	Partial	6.78	-4.05	0.51	1.78	11.26
	Cumulated		+2.73	+3.24	+5.01	+16.27
31	Partial	6.67	-3.79	0.50	1.75	11.10
	Cumulated		+2.88	+3.37	+5.12	+16.22

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32	Partial	6.54	-3.49	0.44	1.71	10.90
	Cumulated		+3.05	+3.49	+5.21	+16.11
33	Partial	6.39	-3.15	0.38	1.67	10.67
	Cumulated		+3.24	+3.62	+5.29	+15.96
34	Partial	6.21	-2.76	0.36	1.63	10.41
	Cumulated		+3.45	+3.81	+5.44	+15.84
35	Partial	6.01	-2.34	0.34	1.58	10.10
	Cumulated		+3.68	+4.02	+5.60	+15.70
36	Partial	5.79	-1.87	0.28	1.52	9.76
	Cumulated		+3.92	+4.21	+5.73	+15.49
37	Partial	5.55	-1.36	0.19	1.45	9.39
	Cumulated		+4.19	+4.38	+5.83	+15.22
38	Partial	5.27	-0.80	0.13	1.38	8.99
	Cumulated		+4.47	+4.60	+5.98	+14.97
39	Partial	4.98	-0.21	0.10	1.31	8.55
	Cumulated		+4.77	+4.87	+6.18	+14.72
40	Partial	4.66	0.49	-0.03	1.22	8.04
	Cumulated		+5.15	+5.12	+6.34	+14.38
41	Partial	4.31	1.80	-0.50	1.13	7.50
	Cumulated		+6.11	+5.61	+6.74	+14.24
42	Partial	3.94	1.84	-0.20	1.03	6.89
	Cumulated		+5.78	+5.58	+6.61	+13.51
43	Partial	3.54	2.60	-0.25	0.93	6.22
	Cumulated		+6.14	+5.89	+6.81	+13.04
44	Partial	3.11	3.40	-0.33	0.82	5.52
	Cumulated		+6.51	+6.18	+7.00	+12.52
45	Partial	2.66	4.23	-0.43	0.70	4.72
	Cumulated		+6.89	+6.45	+7.15	+11.87
46	Partial	2.18	5.07	-0.60	0.57	3.89
	Cumulated		+7.24	+6.65	+7.22	+11.11
47	Partial	1.67	5.85	-0.97	0.44	2.99
	Cumulated		+7.52	+6.55	+6.99	+9.98
48	Partial	1.13	6.42	-0.75	0.30	2.03
	Cumulated		+7.55	+6.80	+7.10	+9.13
49	Partial	0.58	7.40	-0.93	0.15	1.08
	Cumulated		+7.98	+7.05	+7.20	+8.28
50	Partial	0.02	8.31	-1.05	0.00	0.05
	Cumulated		+8.32	+7.27	+7.27	+7.32

$$f_{ck}(20) = [t/\alpha + \beta t] * [f_{ck}(28)] = 47.17 \text{ Mpa}$$

Concrete strength at 20days from
Specified 28 day strength value

$$f_{ct} = 0.6 * f_{ck}(20) = 28.30 \text{ Mpa}$$

Permissible compressive stress at transfer

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$$f_{tt} = 0.21f_{ck}^{2/3} = -2.74\text{Mpa}$$

Permissible tensile stress at transfer

$$f_{cw} = f_{cd} = 28.33\text{Mpa}$$

Permissible compressive stress at working Condition

$$f_{tw} = f_{td} = -1.93\text{Mpa}$$

Permissible tensile stress at working Condition

B.4.7 Deflection

The deflection due to dead weight is determined with the formula:

$$w = \frac{5}{384} \frac{qL^4}{EI_c} \quad \text{Where:}$$

$$L = 50\text{m}$$

Length span

$$I_c = 6.388\text{m}^4$$

Moment of inertia of concrete section

$$E = E_{c,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)} = 16818.2\text{ N/mm}^2$$

Effective modulus of elasticity of concrete

For deflection at $t = \infty$ without variable load

$$E = E_{cm} = 37000\text{ N/m}^2$$

Secant modulus of elasticity of concrete for

Additional deflection under mobile load

The maximum deflections and unity checks for different Loads are:

Time	Loads	Deflection w	Maximum allowed Deflection w_{max}
At $t = 0$	$g_{boxdead} - q_{pt0}$	-45mm	$L/250 = 200\text{mm}$
At $t = \infty$ without variable load	$g_{boxdead} + g_{utility} - q_{pt\infty}$	-62mm	
Additional deflection under mobile load without impact	$q_{var-impact}$	$+61\text{mm}$	$L/800 = 62.5\text{mm}$
Additional deflection under mobile load with impact	$q_{var+impact}$	$+71\text{mm}$	
At $t = \infty$ fully loaded	$g_{boxdead} + g_{utility} + q_{var+impact} - q_{pt\infty}$	$+9\text{mm}$	

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B.4.8 Shear + torsion

B.4.8.1 Shear

$\rho_{w,\min} = A_{sw} / (Sb_w \sin \alpha) = 0.08 f_{ck}^{0.5} / f_{yk}$	Minimum shear reinforcement ratio
	Longitudinal axis
$S_{\max} = 0.75d(1 + \cot \alpha)$	Maximum longitudinal spacing of links
$V_{Rd,c} = \frac{I_c * b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_1 \sigma_{cp} f_{ctd}}$	The shear resistance of concrete in regions
	Uncracked in bending
$V_{Rd,c} = \left[C_{Rd,c} K (100 \rho_l f_{ck})^{1/3} + K_1 \sigma_{cp} \right] b_w d$	The shear resistance of concrete in regions
	Cracked in bending
$V_{Rd,c} = (V_{\min} + K_1 \sigma_{cp}) b_w d$	The shear resistance of concrete without any
	Reinforcement

Values of shear properties per girder		Value	
Live and dead load factor for one girder	$LDFV$	0.5	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	2.89	m
Inner liver arm	Z	2.44	m
Tensile strength of concrete	f_{ctd}	1.93	Mpa
Second moment of area about horizontal centroid axis.	I_c	3.194	m^4
Yield strength of shear reinforcement	f_{yd}	400	Mpa
Angle of the inclined struts in tress model	θ	45	degree
First moment of area above and about horizontal centroid axis.	S	1.277	m^3
Design strength of concrete	f_{cd}	28.33	Mpa
Prestress Concrete stress at centroid axis	σ_{cp}	5.66	Mpa
Reinforcement ratio of tensile longitudinal reinforcement	ρ_l	0.02	
Maximum shear force imitated by crushing of compression strut.	$V_{Rd,\max}$	6968.64	kN

Girder section distance in m	V_{Ed} in kN	$V_{Rd,c}$ in kN	$V_{Ed} / V_{Rd,c}$	$V_{Ed} / V_{Rd,\max}$
0	3203.92	3319.15	0.97	0.46
1	3121.04	3319.15	0.94	0.45
2	2890.40	3319.15	0.87	0.41
3	2661.93	3319.15	0.80	0.38
4	2566.08	1465.85	1.75	0.37
5	2483.21	1465.85	1.69	0.36

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6	2313.80	1465.85	1.58	0.33
7	2095.31	1465.85	1.43	0.30
8	2132.80	1465.85	1.45	0.31
9	2108.49	1465.85	1.44	0.30
10	2025.61	1465.85	1.38	0.29
11	1816.96	1465.85	1.24	0.26
12	1769.40	1465.85	1.21	0.25
13	1847.23	1465.85	1.26	0.27
14	1764.35	1465.85	1.20	0.25
15	1565.27	1465.85	1.07	0.22
16	1442.73	1465.85	0.98	0.21
17	1608.42	1465.85	1.10	0.23
18	1525.54	1465.85	1.04	0.22
19	1335.41	1465.85	0.91	0.19
20	1147.88	1465.85	0.78	0.16
21	1385.83	1465.85	0.95	0.20
22	1302.96	1465.85	0.89	0.19
23	1120.93	1465.85	0.76	0.16
24	1108.51	1465.85	0.76	0.16
25	1191.39	1465.85	0.81	0.17
26	1101.34	1465.85	0.75	0.16
27	1128.10	1465.85	0.77	0.16
28	1310.12	1465.85	0.89	0.19
29	1393.00	1465.85	0.95	0.20
30	1210.20	1465.85	0.83	0.17
31	1397.73	1465.85	0.95	0.20
32	1587.86	1465.85	1.08	0.23
33	1670.74	1465.85	1.14	0.24
34	1582.55	1465.85	1.08	0.23
35	1745.13	1465.85	1.19	0.25
36	1944.20	1465.85	1.33	0.28
37	2027.08	1465.85	1.38	0.29
38	2041.06	1465.85	1.39	0.29
39	2180.82	1465.85	1.49	0.31
40	2389.45	1465.85	1.63	0.34
41	2494.01	1465.85	1.70	0.36
42	2576.89	1465.85	1.76	0.37
43	2704.93	1465.85	1.85	0.39
44	2923.43	1465.85	1.99	0.42
45	3129.92	1465.85	2.14	0.45
46	3275.76	1465.85	2.23	0.47
47	3501.78	1465.85	2.39	0.50
48	3730.25	3319.15	1.12	0.54
49	3960.90	3319.15	1.19	0.57
50	4043.77	3319.15	1.22	0.58

B.4.8.1.1 Additional longitudinal reinforcement due to shear

$$\Delta f_{td} = 0.5V_{Ed} * (\cot \theta - \cot \alpha)$$

Additional tensile force in the longitudinal Reinforcement due to shear

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B.4.8.2 Torsion

Values of Torsion properties		Value	
Web Width	b_w	0.35	m
Effective depth	d_{eff}	2.89	m
Inner liver arm	Z	2.44	m
Tensile strength of concrete	f_{ctd}	1.93	Mpa
Area enclosed by shear flow path	A_k	9.918	m^2

Where:

$$T_{Rd,max} = 2v\alpha_c f_{cd} A_k t_{ef,i} \sin \theta \cos \theta$$

The design torsional resistance moment

$$\alpha_c = (1 + \sigma_{cp} / f_{cd})$$

$$0 \leq \sigma_{cp} \leq 0.25 f_{cd}$$

$$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$$

$$t_{ef,i} = \frac{A}{u}$$

Effective thickness

A
 u

Total area of the cross-section
Is the outer circumference of the
Cross-section

V_{Ed}

Is the design transverse shear force

T_{Ed}

The design torsional moment

$$V_{Rd,max} = \alpha_c b_w z v f_{cd} / (\cot \theta + \tan \theta)$$

Maximum design shear resistance

The required cross-sectional area of the longitudinal reinforcement for torsion $\sum A_{Sl}$ calculated from:

$$\frac{\sum A_{Sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta$$

Where

u_k Is the perimeter of the area A_k

T_{Ed} Torsional design moment

A_k Area enclosed by the center line through girder

f_{yd} Is the design yield stress of the longitudinal reinforcement A_{Sl}

θ Is the angle of compression struts

And, the required cross-sectional area of the transverse reinforcement for torsion A_{St} calculated from:

$$\frac{A_{St} f_{yd}}{S} = \frac{T_{Ed}}{2A_k} \cot \theta \quad \text{Where:}$$

S Is the spacing of transverse reinforcement for torsion

T_{Ed} Torsional design moment

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Girder section distance in (m)	T_{Ed} in (kNm)	$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max}$	A_{st} required in (mm^2 / m)	A_{st} required in (mm^2 / m)
0	5831.37	0.56	735	735
1	5831.37	0.55	735	735
2	5636.22	0.51	710	710
3	5487.48	0.48	692	692
4	5361.78	0.46	676	676
5	5248.21	0.45	661	661
6	5165.26	0.42	651	651
7	5102.99	0.39	643	643
8	5039.68	0.39	635	635
9	4984.99	0.39	628	628
10	4937.54	0.38	622	622
11	4895.52	0.35	617	617
12	4857.56	0.34	612	612
13	4825.66	0.35	608	608
14	4791.40	0.34	604	604
15	4763.58	0.31	600	600
16	4738.96	0.29	597	597
17	4723.20	0.31	595	595
18	4698.34	0.30	592	592
19	4681.97	0.27	590	590
20	4668.06	0.25	588	588
21	4660.43	0.28	587	587
22	4647.45	0.27	586	586
23	4640.63	0.24	585	585
24	4636.12	0.24	584	584
25	4633.88	0.25	584	584
26	4636.12	0.24	584	584
27	4640.63	0.24	585	585
28	4647.45	0.27	586	586
29	4660.41	0.28	587	587
30	4668.06	0.26	588	588
31	4681.97	0.28	590	590
32	4698.34	0.31	592	592
33	4723.19	0.32	595	595
34	4738.96	0.31	597	597
35	4763.58	0.33	600	600
36	4791.40	0.36	604	604
37	4825.63	0.38	608	608
38	4857.56	0.38	612	612
39	4895.52	0.40	617	617
40	4937.54	0.43	622	622
41	4984.99	0.45	628	628
42	5039.68	0.46	635	635
43	5102.99	0.48	643	643
44	5165.26	0.51	651	651

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45	5248.21	0.54	661	661
46	5361.78	0.56	676	676
47	5487.48	0.60	692	692
48	5636.22	0.63	710	710
49	5831.37	0.67	735	735
50	5831.37	0.68	735	735

B.4.9 Ultimate moment resistance

Values of box girder properties		Value	
Area of the top slab	$A_{slab,top}$	2.53	m^2
Equivalent thickness of top slab	$t_{if, equ}$	0.283	m
Sum of web width	$\sum b_w$	0.7	m
Area of the bottom slab	$A_{slab,bott}$	0.95	m^2
Equivalent thickness of bottom slab	$t_{bf, equ}$	0.248	m
Width of top slab	b_{if}	8.96	m

$$A_{ps} = 48900mm^2$$

Total Area of tendon

Box Girder section distance in (m)	Tendon ultimate force	Distance between neutral axis and compressive face (x) in (m)	Compressive block depth below top fiber to neutral axis (βx) in (m)
0	76122.24	0.63	0.50
2	78081.80	0.75	0.60
4	79071.30	0.81	0.65
6	79071.30	0.81	0.65
8	79071.30	0.81	0.65
10	79071.30	0.81	0.65
12	79071.30	0.81	0.65
14	79071.30	0.81	0.65
16	79071.30	0.81	0.65
18	79071.30	0.81	0.65
20	79071.30	0.81	0.65
22	79071.30	0.81	0.65
24	79071.30	0.81	0.65
26	79071.30	0.81	0.65
28	79071.30	0.81	0.65
30	79071.30	0.81	0.65
32	79071.30	0.81	0.65
34	79071.30	0.81	0.65
36	79071.30	0.81	0.65
38	79071.30	0.81	0.65
40	79071.30	0.81	0.65
42	79071.30	0.81	0.65
44	79071.30	0.81	0.65

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46	79071.30	0.81	0.65
48	78081.80	0.75	0.60
50	76122.24	0.63	0.50

Box Girder section distance in (m)	Positive moment resistance of section (M_{RT}) in (kNm)	Positive demand moment for the girder (M_{DE}) in (kNm)
0	74203.58	0.00
2	95762.92	23500.94
4	115462.21	44749.21
6	132682.09	63798.82
8	148179.97	80864.19
10	161955.87	95229.02
12	174009.78	108089.36
14	184341.71	118191.36
16	192951.64	127747.46
18	199839.59	132450.55
20	205005.55	137762.91
22	208449.53	141278.55
24	210171.52	142941.53
25	210171.52	142941.53
26	208449.53	141278.55
28	205005.55	137762.91
30	199839.59	132450.55
32	192951.64	126359.50
34	184341.71	118191.36
36	174009.78	107755.31
38	161955.87	95229.02
40	148179.97	80605.16
42	132682.09	63798.82
44	115462.21	44749.21
46	95762.92	23500.94
48	74203.58	0.00
50	74203.58	0.00

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APPENDIX C: Cost Assumptions

Assume the same production rate for all concrete grades considered.




DETAILED BREAKDOWN OF WORK ITEM														
UNIT PRICES														
Project:														
Work Item: Concrete Grade C-30												Performance Rate	18.00	m ³ /day
Total Quantity of Work Item:													2.25	m ³ /hr
MATERIAL COST					LABOR COST					EQUIPMENT COST				
Material Type	Unit	Qty	Rate	Cost/Unit	Title	Qty	UF	Indexed hrly cost	Total hrly cost	Equipment type	Qty	UF	Rental Rate hrly	Total
Cement	Qtl	3.78	350.0	1,323.00	Construction Forman	1.00	0.50	54.99	27.50	Conceret Mixer (5-6m3)	1.00	1.00	77.82	77.82
Sand	m ³	0.20	470.0	94.00	Labour Forman	1.00	1.00	31.94	31.94	Conceret Vib. (4HP)	2.00	1.00	12.61	25.21
Quary Rock Prod.	m ³	0.50	160.0	80.00	Masson	2.00	1.00	33.87	67.75	Water Truck	1.00	0.50	376.00	188.00
Quary Rock Hauling	m ³	0.50	65.0	32.50	Carpenter	1.00	1.00	33.87	33.87	Carrier Truck	1.00	0.20	843.18	168.64
Crushing Con.Agg.	m ³	0.50	250.0	125.00	Conceret Mixer Oper.	1.00	1.00	39.50	39.50	Hand Tools	10.00	1.00	10.22	102.18
Hauling Con.Agg.	m ³	0.50	350.0	175.00	Water Truck Driver	1.00	0.50	57.25	28.62					
Form Work	m ³	0.05	11,700.0	585.00	Carrier Truck Driver	1.00	0.20	57.25	11.45					
Eucalyptus Pole	no	10.00	50.0	500.00	Labour	20.00	1.00	17.27	345.33					
Nails/Assorted	kg	2.50	50.0	125.00										
Total				3,039.50	Total				585.96	Total				561.85
A = Material Unit Cost :		3,039.50	Birr/m ³	B = ManPower Unit Cost:		260.43	Birr/m ³	C = Equipment Unit Cost:		249.71	Birr/m ³			
Direct Cost of Work Item =A+B+C=										3,549.64		Birr/m³		
Total Cost =										4,792.01		Birr/m³		

DETAILED BREAKDOWN OF WORK ITEM														
UNIT PRICES														
Project:														
Work Item: Concrete Grade C-40												Performance Rate	18.00	m ³ /day
Total Quantity of Work Item:													2.25	m ³ /hr
MATERIAL COST					LABOR COST					EQUIPMENT COST				
Material Type	Unit	Qty	Rate	Cost/Unit	Title	Qty	UF	Indexed hrly cost	Total hrly cost	Equipment type	Qty	UF	Rental Rate hrly	Total
Cement	Qtl	4.25	350.00	1,488.38	Construction Forman	1.00	0.50	54.99	27.50	Conceret Mixer (5-6m3)	1.00	1.00	77.82	77.82
Sand	m ³	0.18	470.00	84.60	Labour Forman	1.00	1.00	31.94	31.94	Conceret Vib. (4HP)	2.00	1.00	12.61	25.21
Quary Rock Prod.	m ³	0.51	160.00	80.98	Masson	2.00	1.00	33.87	67.75	Water Truck	1.00	0.50	376.00	188.00
Quary Rock Hauling	m ³	0.51	65.00	32.90	Carpenter	1.00	1.00	33.87	33.87	Carrier Truck	1.00	0.20	843.18	168.64
Crushing Con.Agg.	m ³	0.51	250.00	126.53	Conceret Mixer Oper.	1.00	1.00	39.50	39.50	Hand Tools	10.00	1.00	10.22	102.18
Hauling Con.Agg.	m ³	0.51	350.00	177.14	Water Truck Driver	1.00	0.50	57.25	28.62					
Form Work	m ³	0.05	11,700.00	585.00	Carrier Truck Driver	1.00	0.20	57.25	11.45					
Eucalyptus Pole	no	10.00	50.00	500.00	Labour	20.00	1.00	17.27	345.33					
Nails/Assorted	kg	2.50	50.00	125.00										
Total				3,200.51	Total				585.96	Total				561.85
A = Material Unit Cost :		3,200.51	Birr/m ³	B = ManPower Unit Cost:		260.43	Birr/m ³	C = Equipment Unit Cost:		249.71	Birr/m ³			
Direct Cost of Work Item =A+B+C=										3,710.64		Birr/m³		
Total Cost =										5,009.37		Birr/m³		

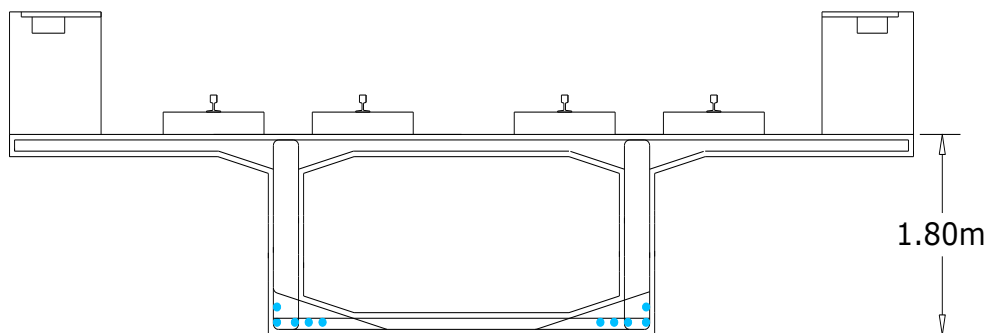
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

APPENDIX D: Cross Sectional Drawings

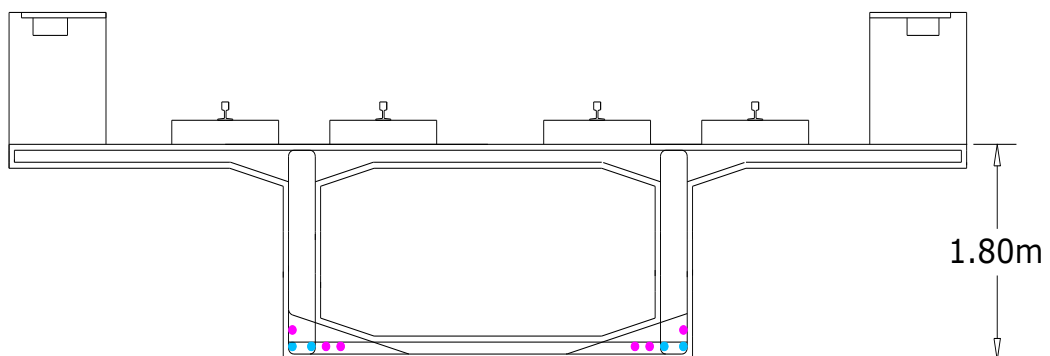
This appendix contains sectional drawing of box girder made of different concrete grade. Also, for each concrete grades used sectional layout of prestress tendons at mid span shown. The prestressing tendons section area designated by colors defined by the following table.

Color designation			
Area per stand in mm ²	100	100	150
Number of strand/tendon	12	27	19
Area per tendon in (mm ²)	1200	2700	2850

D.1 30m span length

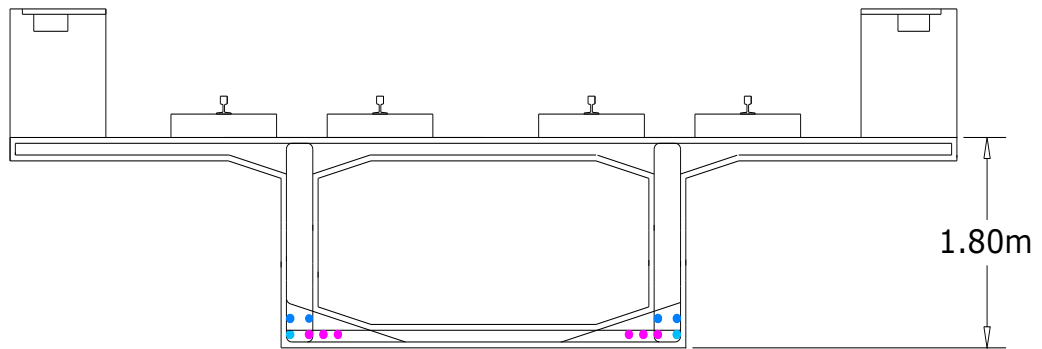


C-30 concrete grade box girder section, $e = 0.138\text{m}$

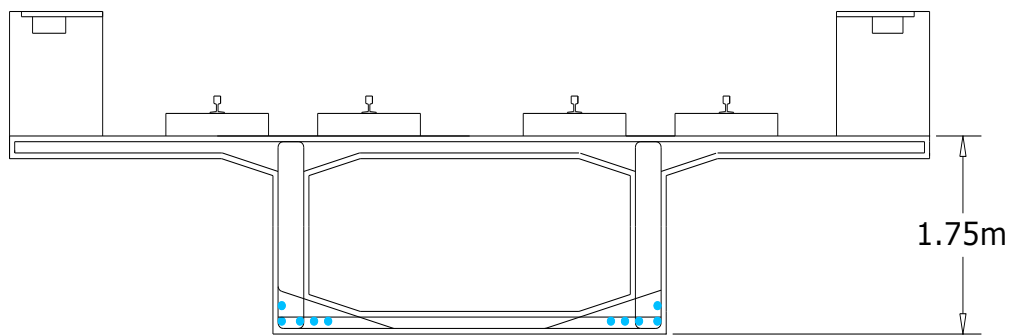


C-40 concrete grade box girder section, $e = 0.137\text{m}$

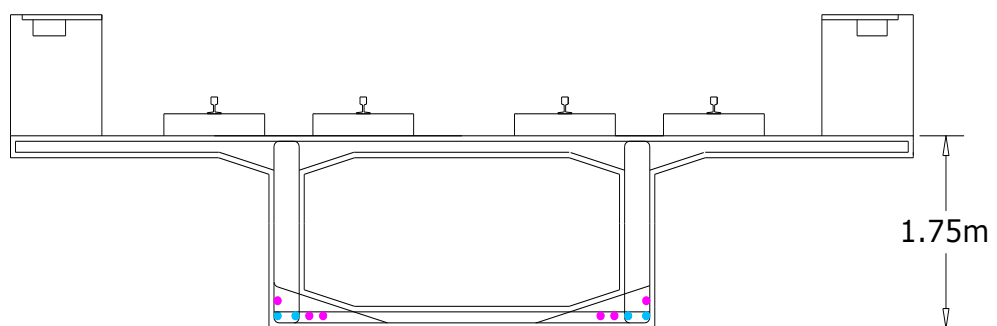
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C-50 concrete grade box girder section, $e = 0.135\text{m}$

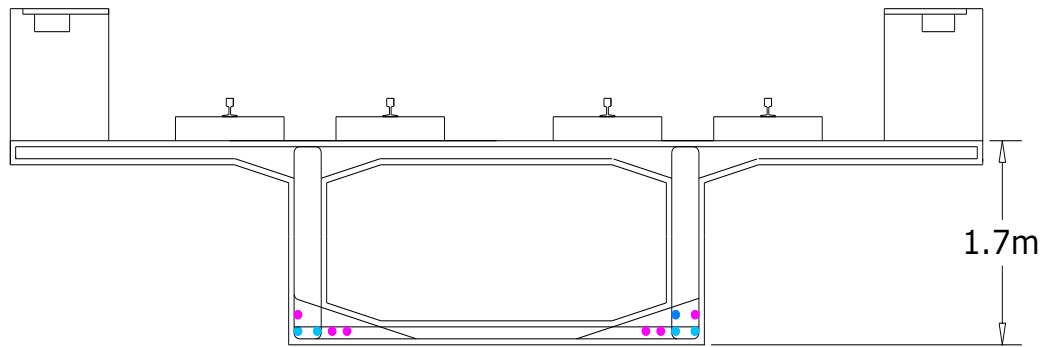


C-40 concrete grade box girder section, $e = 0.138\text{m}$



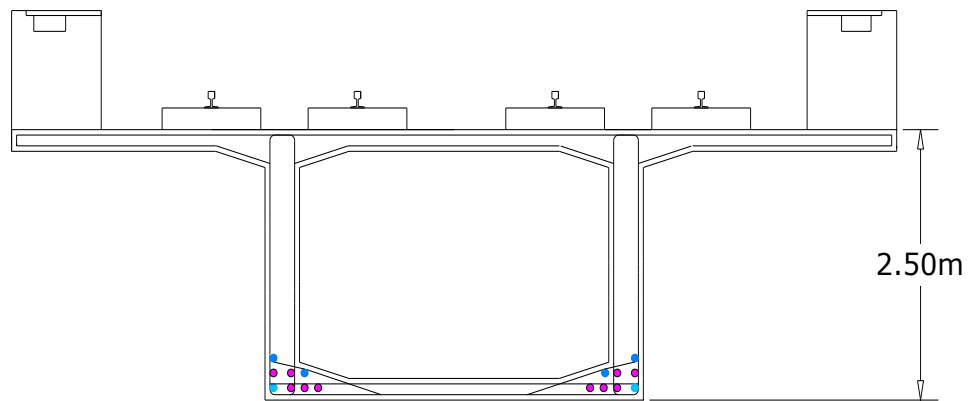
C-50 concrete grade box girder section, $e = 0.137\text{m}$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

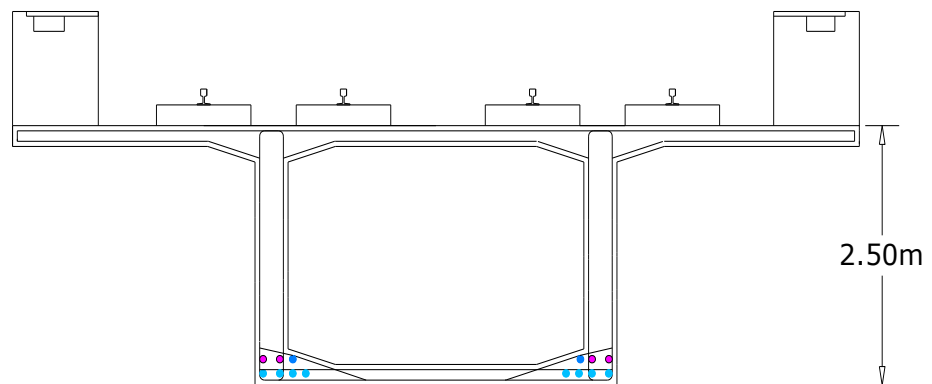


C-50 concrete grade box girder section, $e = 0.142\text{m}$

D.2 40m span length

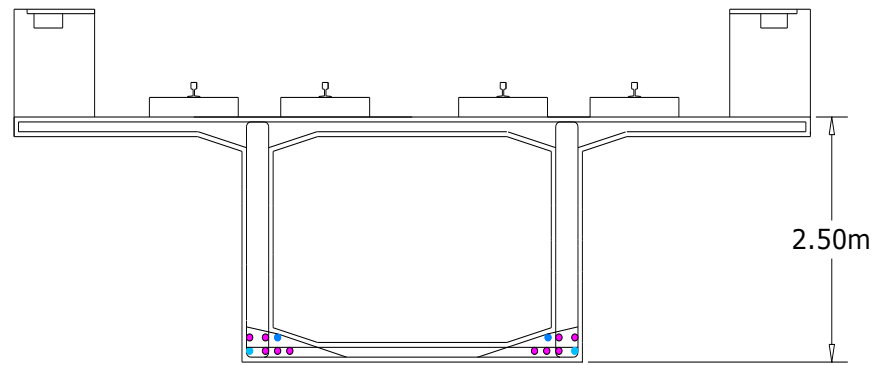


C-30 concrete grade box girder section, $e = 0.177\text{m}$

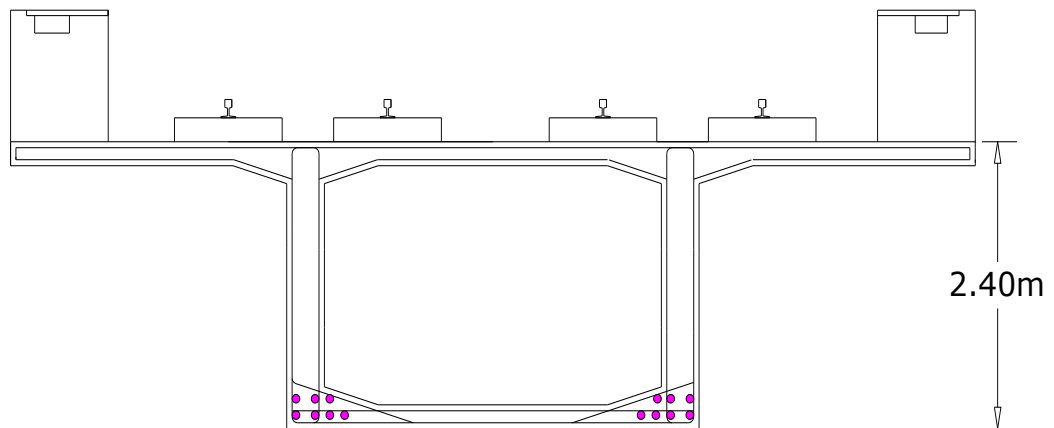


C-40 concrete grade box girder section, $e = 0.16\text{m}$

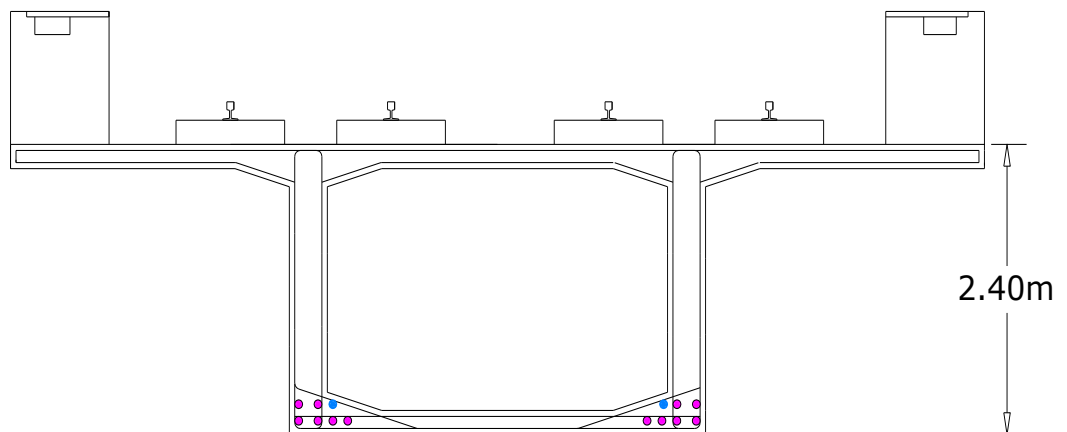
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



C-50 concrete grade box girder section, $e = 0.163\text{m}$

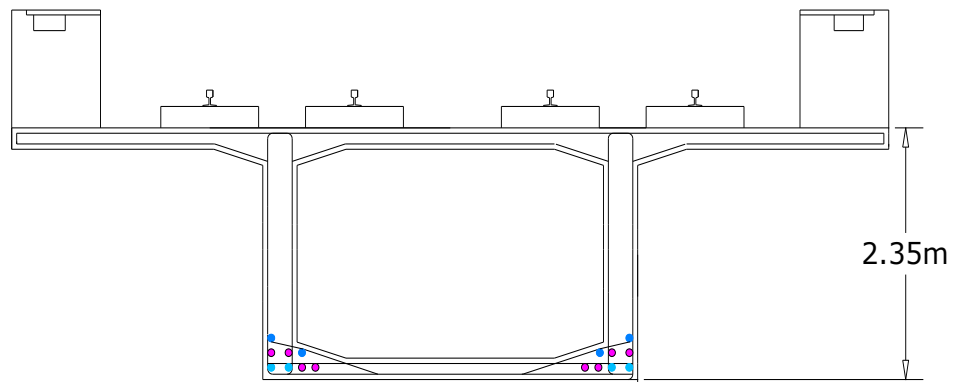


C-40 concrete grade box girder section, $e = 0.17\text{m}$



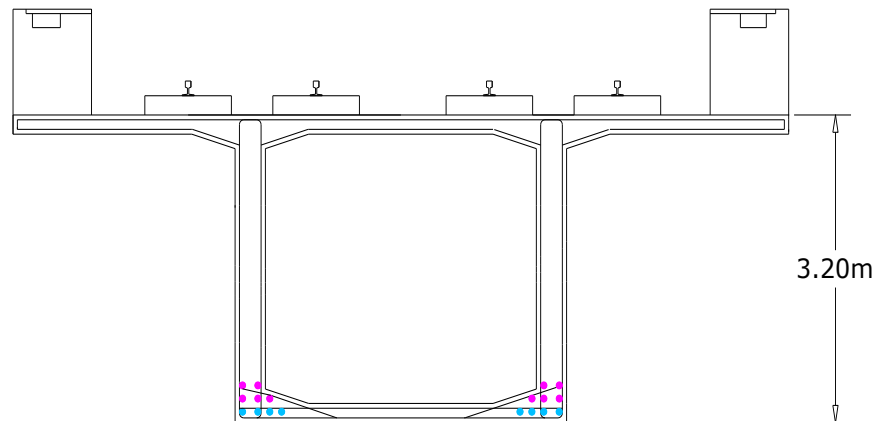
C-50 concrete grade box girder section, $e = 0.163\text{m}$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge

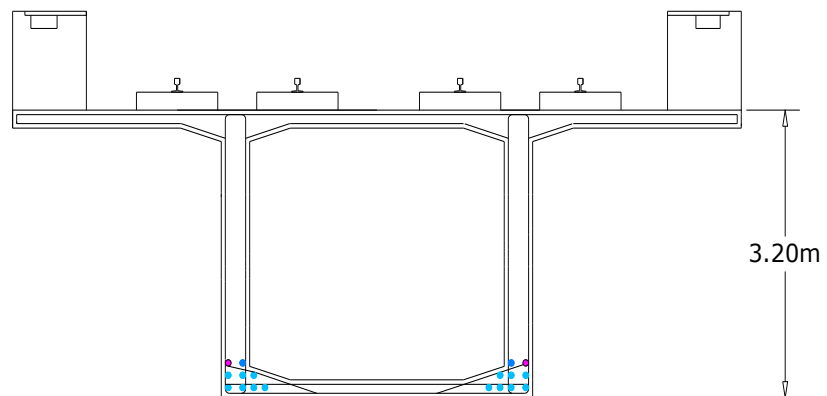


C-50 concrete grade box girder section, $e = 0.177\text{m}$

D.3 50m span length

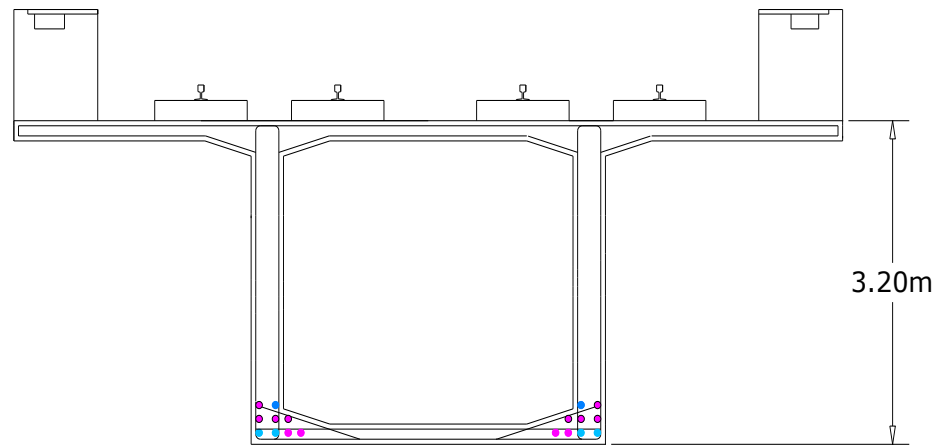


C-30 concrete grade box girder section, $e = 0.216\text{m}$

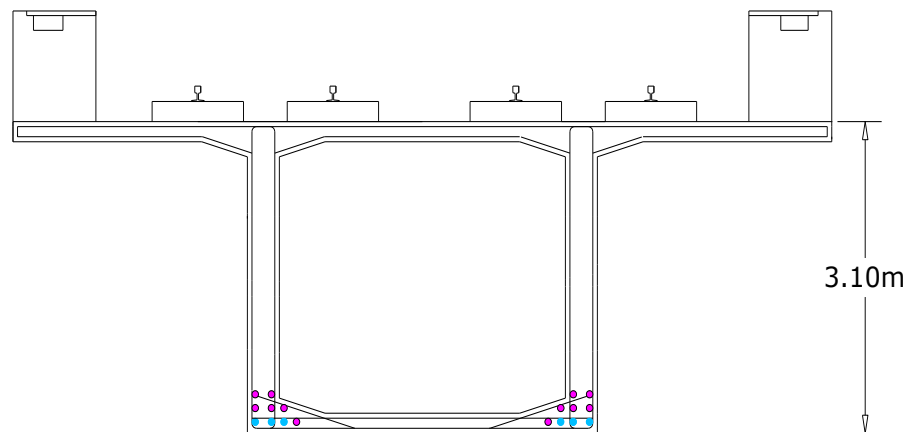


C-40 concrete grade box girder section, $e = 0.206\text{m}$

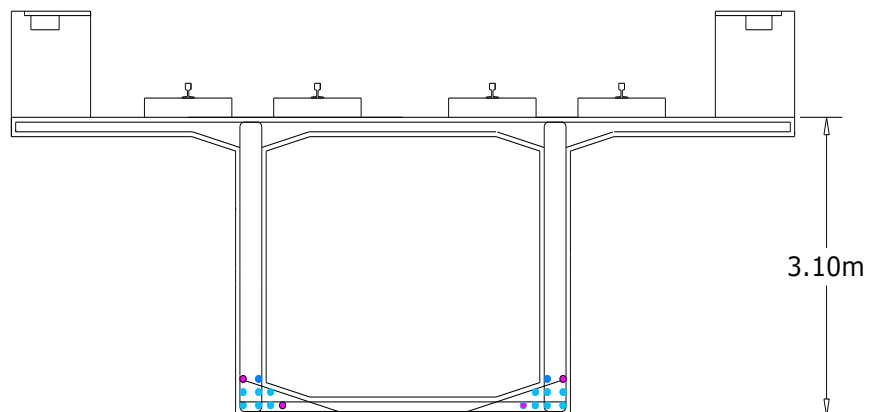
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



C-50 concrete grade box girder section, $e = 0.206\text{m}$

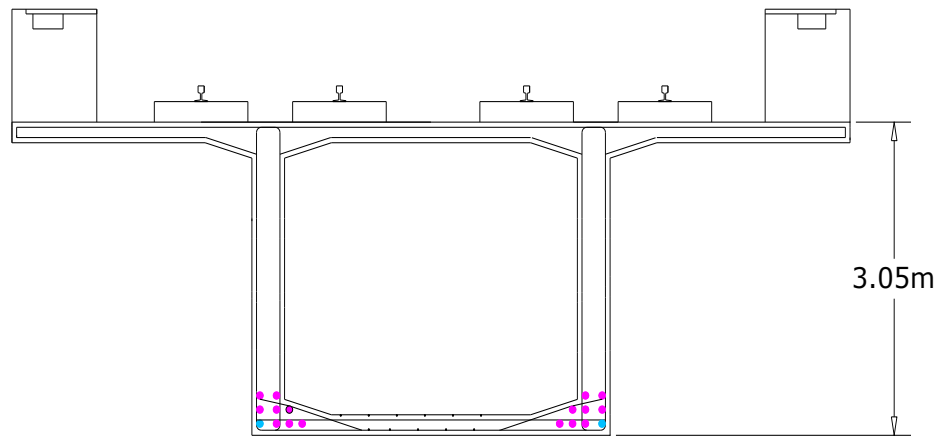


C-40 concrete grade box girder section, $e = 0.217\text{m}$



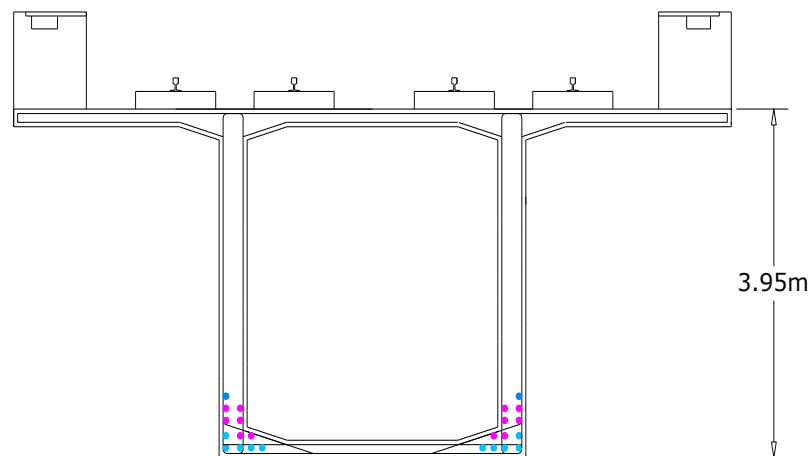
C-50 concrete grade box girder section, $e = 0.206\text{m}$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



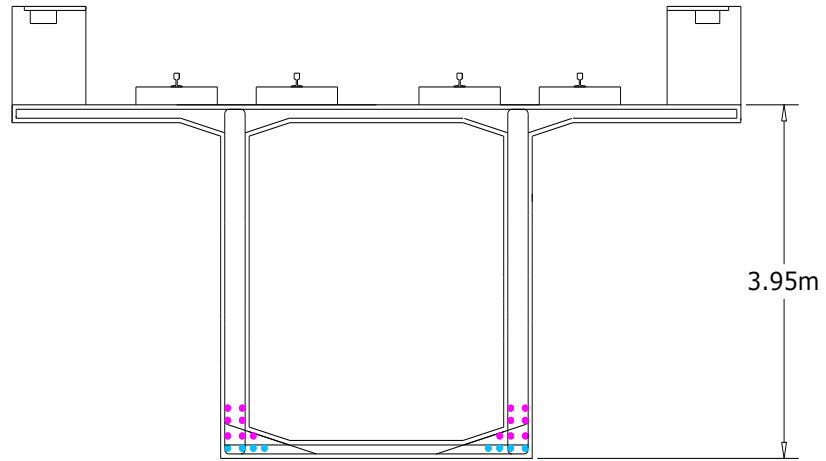
C-50 concrete grade box girder section, $e = 0.218\text{m}$

D.4 60m span length

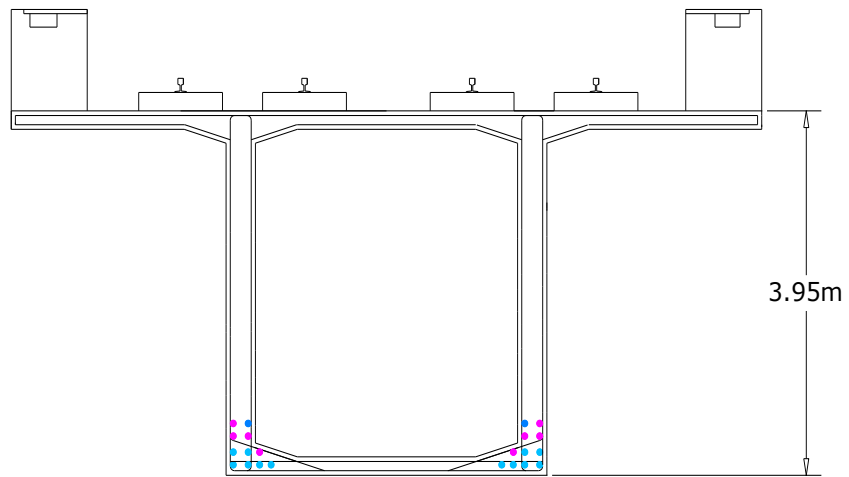


C-30 concrete grade box girder section, $e = 0.287\text{m}$

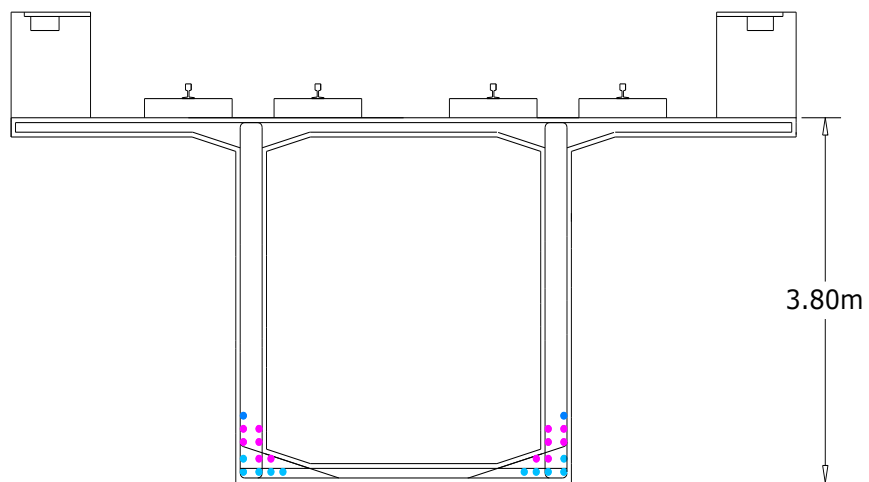
Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



C-40 concrete grade box girder section, $e = 0.27\text{m}$

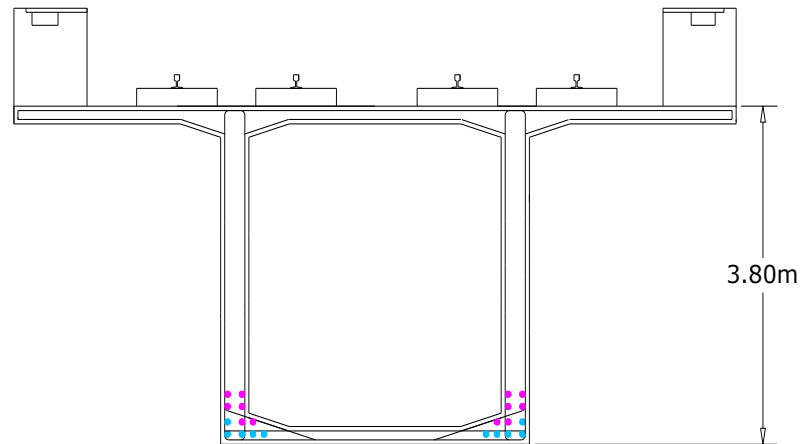


C-50 concrete grade box girder section, $e = 0.258\text{m}$

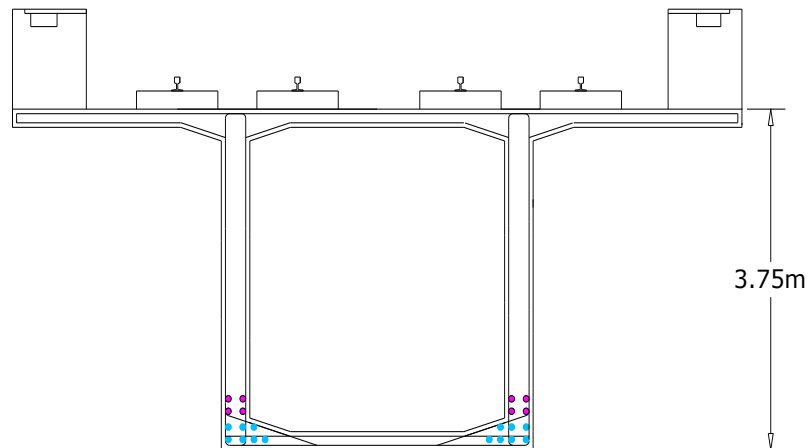


C-40 concrete grade box girder section, $e = 0.287\text{m}$

Economical Concrete Grade for design of variable span cast in-situ prestressed box girder railway bridge



C-50 concrete grade box girder section, $e = 0.272\text{m}$



C-50 concrete grade box girder section, $e = 0.272\text{m}$