



ADDIS ABABA INSTITUTE OF TECHNOLOGY

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DEPARTMENT OF CIVIL AND ENVIRONMENTAL

ENGINEERING

**Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit
of Double Cell Pre-stressed Box girder Railway Bridges**

By

Misiker Jemberu

**A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial
Fulfillment of the Degree of Masters of Science in Civil Engineering**

Advisor

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Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

DECLARATION

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

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Name

Signature

Date

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

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List of Symbols

a_f = total long time deflection

a_{il} = initial deflection due to transverse loads

a_{ip} = initial deflection due to pre stress force

A_{ps} = area of pre-stressing steel (m^2)

A_s = area of non-prestressed tension reinforcements (mm^2)

A_v = Area of shear reinforcement

b = width of flange of flanged member or width of rectangular member (mm)

b' = width of a web of a flanged member (mm)

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

d_b = nominal diameter of prestressing strand (mm)

d_p = distance from extreme compression fiber to centroid of prestressing steel (mm)

e = eccentricity

E_c = modulus of elasticity of concrete (MPa)

E_{ci} = modulus of elasticity of concrete at transfer (MPa)

e_m = average prestressing steel eccentricity at critical sections (m)

e_{max} = maximum eccentricity

E_p = modulus of elasticity for pre-stressing tendons (MPa)

E_s = modulus of elasticity of steel reinforcement (MPa) f'_c = compressive strength of concrete at 28 days (MPa)

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

f'_{cir} = average concrete stress at the c.g. of the prestressing steel at time of release (MPa)

f'_s = tensile strength of the pre-stressing steel (1860 MPa)

f'_y = yield strength of non-prestressed conventional reinforcement in compression (MPa)

f_b = concrete stress at the bottom fiber of the girder beam (KN/m^2)

f_{cd} = design compressive stress in concrete

f_{cd} = design compressive stress in concrete (MPa)

f_{cds} = stress in concrete at centroid of prestressing reinforcement, due to all dead load not included in calculation of f_{cir} (MPa)

f_{cir} = stress in concrete at the centroid of pre-stressing reinforcement immediately after transfer due to total pre-stress force and dead load acting at transfer (MPa)

f_{cit} = extreme fiber stress in tension of concrete immediately after prestress transfer (MPa)

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f_{ck} = 28 days cylinder compressive strength of concrete (MPa)

f_{ct} = extreme fiber stress in tension of concrete at service loads (MPa)

f_{ct} = permissible compressive stress in concrete at the moment of prestressing

f_{ctd} = design tensile stress in concrete

f_{ctd} = design tensile stress in concrete

f_{pbt} = stress in prestressing steel immediately prior to transfer (MPa)

f_{pe} = loads stress in tendons after all prestressing losses

f_{pi} = stress in tendons immediately after pre stress

f_{pj} = jacking stress, (MPa)

f_{pk} = transfer Ultimate tensile strength of pre-stressing tendons

f_{pu} = ultimate tensile strength of post-tensioned tendon (MPa)

f_{su} = average stress in prestressing steel at ultimate load (MPa)

f_{ut} = permissible tensile stress in concrete at the moment of pre stressing

f_{tw} = allowable tensile stress permissible in concrete under working loads

f_y = yield stress of steel (MPa)

$\gamma_{sand,gravel,ballast}$ = Unit weight of sand, gravel, ballast

h = overall depth of member in (mm)

I = moment of inertia about the centroid of the cross section in (m⁴)

I_g = moment of inertia of the gross concrete section (m⁴)

M_g = Unfactored bending moment at critical sections due to self weight (KN-m)

M_{LL+I} = Unfactored BM due to live load plus impact (KN-m)

M_{max} = maximum factored moment at section due to externally applied loads in (KN-m)

M_q = live load bending moment,

M_{SDL} = Unfactored bending moment due to Superimposed dead load

N = number of tendons

ϕ = Strength Reduction Factor

ρ = A_s/bd , ratio of non-prestressed tension reinforcements

P_i = pre stressing force immediately after prestressing

P_{se} = effective prestress force after allowing for all losses (KN)

S_b = bottom fiber section modulus (m³)

t = compression flange thickness (m)

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V_c =nominal shear strength provided by concrete

V_{cr} =shear force at critical section due to unfactored dead load

V_{cw} =nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

V_d =shear force at section due to unfactored dead load

V_{LL+I} =design shear for working loads plus impact

w_c = weight of concrete (Kg/m^3)

WL=Working loads

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

β_1 = factor for concrete strength

γ = factor for type of prestressing steel

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Abstract

Currently, railway transport system is being the most efficient, environmentally friendly, space-saving means of land transport system. Due to the new technologies and innovations made to such way of transporting goods and passengers, a lot of studies and documentations are being politicized.

Among the new technologies added to this transport system is segmental bridge construction where simple span railway bridges are placed over two successive piers. As such long distances of railway bridges are being constructed by assembling simple span pre-fabricated segmental girders placed on the piers. One of the ways to prefabricate girders before on site erection on the piers is to pre-stress the structural member and modify its material properties (i.e. a reduced weight of concrete, reaching the full strength of the concrete and steel).

In this research, an attempt is made to economically compare analysis and design of two units of single cell pre-stressed box girders and one unit of double cell pre-stressed box girder railway bridges and to select an economical one on the basis of construction material cost. For both cases, cost versus span is made so as to meet the economic comparison objectives of the study.

Bases on the study, it is obtained that applying two units of single cell pre-stressed box girder railway bridges is uneconomical way of arrangement than one unit of double cell pre-stressed box girder railway bridge for simple spans greater than 34m.

Key words: segmental, girders, piers, pre-stressed, single cell, double cell

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1. Introduction

1.1. Background

The Economic growth and prosperity of a given country depends on many factors such as political stability, employment rate, health, transportation...etc. Currently, it is widely accepted that railway transportation is suitable for movement of bulk goods and products within a short period of time. This means of transport enhance the economic development of a nation by providing cargos and containers from productive regions within the nation and from neighboring countries. In addition, railway transport systems greatly ease the passenger movement within cities as it consumes a smaller transport corridor with a more efficiency in relation to road transports. Such advantages make the rail-road users more productive to the city's income as one can reach his/her destination within fast duration of time from his/her work place and business areas.

One of the distinctive features of railway transport system is that it cannot tolerate grades vertical grades exceeding 5% under normal situations. This forces the railway alignment designers to provide railway tunnels and bridges wherever they encounter unfavorable terrain and topography along the route the railway passes. In such scenario, railway bridges and tunnels are designed. Railway Tunnels are commonly employed in subways and metro lines in urban cities and in circumstances when it is obliged to cut through a mountain. In the same manner railway bridges are better ways of solving problems in circumstances when an urban city is crowded and congested and at-grade construction is very difficult to apply.

Nowadays, there are different types of railway bridges being under construction. This includes girder railway, arch railway bridges, truss railway bridges, suspension railway bridges and cable stayed railway bridges. Girder railway bridges are further classified into slab, I-section, T-girder and box girder railway bridges. Based on structural arrangement, box girder railway bridges are classified into three types: single cell, double cell and multi cell box girder railway bridges. Railway designers employ commonly used parameters and considerations while selecting among the above types of bridges. These considerations include structural considerations, aesthetics, economy, harmony, contrast.

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In this research, an attempt is made to make economical comparison between two single cell pre-stressed concrete railway bridges and a double cell pre-stressed concrete railway bridge. Pre-stressing (preferably Post-tensioning) which is an engineering technique to increase the load carrying capacity with a smaller construction materials usage in box-girder bridge sections is employed during the structural analysis and design.

1.2. Statement of the problem

Economy is a prime consideration in most of the railway bridges being under construction, especially in developing countries. Therefore pre-stressed box girder railway bridges require economic evaluation studies so as to choose between two single cell and double cell pre-stressed concrete railway bridges. If the right decision of economically selecting between the above two ways of construction is not made, it could result in a higher material, equipment, and labor costs.

1.3. Objectives of the research

The general objective of working on such area of study is to justify whether double cell box girder pre-stressed concrete railway bridges are more economical than two single cell box girder pre-stressed concrete railway bridges.

Specific objectives of the research include:

1. To investigate variations in span length by using two single cell box girders pre-stressed concrete railway bridges and a double cell box girder pre-stressed concrete Railway Bridge.
2. Redesigning two single cell box girder pre-stressed concrete railway bridges to obtain an economical double cell box girder pre-stressed concrete railway bridge.
3. To select a demarcation span between double cell and two single cell box girders pre-stressed concrete railway bridges types on the basis of cost analysis. This done by plotting on cost-span graph for both types of bridges and at the point where these two graphs intersect, the demarcation span length is calculated.

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1.4. Research Methodology

The research methodology for this study involved the following major tasks: literature review; design program preparation using computer program and AREMA and AASHTO LRFD Design manuals; designing two single cell box girder railway pre-stressed concrete railway bridges for various span length using the design program; designing a double cell box girder railway pre-stressed concrete railway bridges for various span length using the design program and cost versus span length values are plotted on the same graph to find the demarcation span length and for economic selection between the two different designs.

The design codes used in the design process are obtained from the AREMA and AASHTO LRFD design manuals. The analysis type applied is gravity load analysis. The ultimate moment capacity and the shear design are done by Load Factor Design using ultimate strength theory (Strength Limit State) for factored design loads.

1.5. Applications and Limitations

Applications

- Consulting firms and parties in the bridge construction can benefit from the output of this research work.
- It will increase awareness of practicing architects and structural engineers on the cost advantages that prevails because of railway box girder structural arrangement only.
- It will thus be used as a guide to optimize between material costs in railway box girder constructions.

Limitations

- The Economic Analysis and Design for the Selection of two units of Single cell and one unit of Double cell Pre-stressed Box girder Railway Bridges is limited to post-tensioning pre-stressing systems only. This is also supported from the fact that pre-tensioned pre-stressing system is mostly applicable for shorter span length.
- In addition, the analysis during the design is limited to superstructure gravity load analysis for a typical section only.

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2. Literature review

2.1 Introduction

This particular paper is a result of feeds from different publications directly and indirectly related to the work. A lot of design manuals, design examples, presentations, books and research papers done on simply supported box girder bridges covers principles, methodologies and approaches related with the content of this work. Literatures done related to simply supported post-tensioned box girder bridges are common and mostly available for highway bridges than railway bridges. Yet, employing the few publications on pre-stressed post-tensioned box girder railway bridges provides this paper work a better and intensified content.

2.2. Historical background of pre-stressed concrete railway bridges

Prior to the application of pre-stressing to bridges, there were remarkable bridge designs using the conventional reinforced concrete. But a lot of factors contributed for the application of pre-stressing systems to concrete railway and highway bridges.

The earliest investigations of pre-stressed concrete beams were conducted in the ninetieth century. In 1888, the German Engineer W. Döring patented a system for the construction of slabs, by which cracking was reduced through the use of pre-stressed wires, pre-stressing saw no further practical application in construction until its significance and potential were recognized by Freyasinet. From 1928 to 1936, he worked on the development of stressing systems and patented pre-stressing jacks and anchors. He was then ready to begin applying his ideas to the construction of bridges and buildings. From the very beginning, Freyalsnet insisted on the use of high-strength wires bonded to concrete. His success with pre-stressing is largely due to the practical implementation of the concept. [Christian Menn Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel. Boston. Berlin].

Although pre-stressing was well received in Germany, it was not initially implemented according to Freyasinets system. Dischinger, the country's most experienced reinforced concrete designer, chose to use large-diameter bars of low strength steel, unbounded to the concrete. His method had several major shortcomings. Due to its low tensile strength, the pre-stressing steel could only be restrained to barely twice the expected plastic strain in the concrete due to creep and shrinkage. Large pre-stressing losses, and hence cracking and large deformations, were therefore unavoidable. [Christian Menn .Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel. Boston. Berlin].

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Dischinger used his system to construct the world's first pre-stressed concrete bridge in Aue, Germany completed in 1937. The pre-stressed portion of the bridge consisted of three spans of 25.2m, 69m, and 23.4m. The central part of the main span was suspended at internal hinges, making the system statically determinate. Cantilever tendons were draped over the two intermediate piers; the suspended segment was pre-stressed as a simply supported beam. Dischinger used 70mm diameter bars for the tendons, pre-stressing and redistribution of sectional forces in cantilever construction can be solved with sufficient accuracy using simple methods based on a few well-defined material parameters. [Christian Menn, Birkhauser, Verlag (1986), Prestressed Concrete Bridges: Basel, Boston, Berlin].

Post-war reconstruction, followed by rapid economic growth, produced a boom in bridge construction in Europe beginning in the late 1940s. The early years after the war were an important development period for pre-stressing concrete, in which new design and construction techniques were tested and improved. The analysis and design of pre-stressed concrete structures was one of the most important research goals at schools of engineering throughout the world. Unfortunately, it took a relatively long time for a clear picture of the behavior of pre-stressed concrete structures to emerge from the countless scientific papers published. During this period, one of the first important contributions in this regard was the book *Spannbeton für die Praxis*, first published in 1954 and later translated into English as *Pre-stressed Concrete - Design and Construction*, by Fritz Leonhardt. [Christian Menn, Birkhauser, Verlag (1986), Prestressed Concrete Bridges: Basel, Boston, Berlin].

In relation to segmented precast bridge construction, period reduction and ease of constructability had become more realized for pre-stressed bridge constructions.

Segmental box girder bridges externally post-tensioned are one of the major new developments in bridge engineering in the last years. In contrast to 'classical' monolithic constructions a segmental bridge consists of small precast elements stressed together by external tendons. The many advantages of this type of structure like fast and versatile construction, no disruption at ground level, high controlled quality and cost savings have made them the preferred solution for many long elevated highways, especially in South East Asia, and bridges. [Prof. Dr.-Ing. G. Rombach (2002), *Precast segmental box girder bridges with external pre-stressing - design and construction*, Technical University: Hamburg-Harburg, Germany].

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The first cantilever constructed bridges using precast segments were built in the early 1960s in the Soviet Union and France. The detailing of the joints between the segments proved to be a particularly difficult problem. The 148 m span of the bridge at the Licha6er Factory in Moscow was designed with 0.2 m thick cast-in place concrete joints between the segments, at which longitudinal reinforcing steel was lap spliced. Due to considerable delays in construction, however, continuous mild reinforcing steel was abandoned in subsequent bridges. Another Russian structure, the Irtysh Bridge (110 m main span), used 20 mm thick mortar joints; dry joints were used for the Ojat Bridge (64 m main span), erected in winter. [Christian Menn .Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel.Bosten.Berlin].

In France, the erection of precast segments was known since the construction of Freyssinet's Marne bridges. French engineers developed new solutions to the problems of mortar joints, which slowed construction, and dry joints which complicated the grouting of tendons. The Choisy-le-Roi Bridge over the Seine, completed in 1965 (span lengths 37.5 m, 55.0 m, and 37.5 m) used 1 mm thick epoxy-coated segment joint faces. Similar joint details were also used in 1964 for the 3 km bridge linking the Mainland of western France to Oleron Island. Precast segments were transported over the already completed portion of the bridge and erected with a launching truss. This system, which enabled the erection of eight 3 m long segments per day, was successfully used on several other bridges, including the Chillon Viaducts in Switzerland and the Rio-Niteroi Bridge in Brazil. The former, completed in 1967, was 2.1 km long, sharply curved in plan, and had spans varying from 96 m to 106 m. The latter, completed in 1973, was a 6 km long prismatic girder with 80 spans. [Christian Menn .Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel.Bosten.Berlin].

By the beginning of the 1960s, the Dutch were leaders in the erection of heavy elements with ship-mounted cranes, based on their experience with large sea-works. The 5 km long Osterschelde Bridge was built in record time between 1962 and 1965. The entire bridge, including foundations and piers, consisted of precast components weighing up to 600 tonnes. The segments used in the cantilever construction of the 95 m long spans - were 12.5 m long and were connected with 0.4 m thick concrete joints. Construction of the superstructure reached an average rate of 3500 m³ per month. The Dutch again demonstrated - their leadership in the erection of large precast segments in the construction of the - Saudi Arabia-Bahrain Causeway, for which complete spans weighing up to 1400 tonnes were placed from a floating crane. The causeway, completed in 1985, included five bridges with lengths 934 m, 2034 m, 5194 m, 3334 m, and 934 m. [Christian Menn .Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel.Bosten.Berlin].

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In England and in Australia, segmental bridges were constructed by erecting precast segments onto false work and pre-stressing them together. The 100 mm wide joints between segments were filled before stressing with all-fines concrete. This method was used, for example, for the 600 m long Hammersmith Flyover in London, completed in 1961.[Christian Menn .Birkhauser. Verlag (1986) .Pre-stressed Concrete Bridges: Basel.Bosten.Berlin].The design, details, and construction methods used for the Brotonne Bridge and the Pasco-Kennewick Bridge guided the development of concrete cable-stayed bridges into the 1980s. Although most economical for long spans, cable-stayed bridges have been recently been built economically for spans of less than 150 m.[Christian Menn .Birkhauser. Verlag (1986) .Prestressed Concrete Bridges: Basel.Bosten.Berlin].

The Gateway Bridge in Brisbane, Australia is the longest spanning cantilever-constructed girder bridge, with a 260 m span. It was opened to traffic in 1986. The total height of the structure was limited to 24.7 m, due to its proximity to an airport. The 22 m wide single cell box girder increases in depth from 5.2 m at midspans to 15.0 m at the piers. The choice of 750 mm thick webs,-reinforced with vertical stirrups, is unfortunate; their thickness could have been reduced by almost 50 percent if inclined web reinforcement, which can be easily accommodated in cantilever construction, had been used. If not for the severe height restriction, a cantilever-constructed girder bridge would probably not: have been built at this site; cable-stayed bridges are more economical and more elegant for spans of this range. The record of the Gateway Bridge is also, therefore, unlikely to be exceeded.[Christian Menn .Birkhauser. Verlag (1986) .Prestressed Concrete Bridges: Basel.Bosten.Berlin].

In general, the world had reached to the stage when both concrete and the steel materials which are employed in the construction of huge railway and highway bridges are allowed to yield their maximum natural strength and material behaviors. This phase is different from the early ages of reinforced concrete system when both the steel and concrete are limited to small grades like up to c-30 and s-300(i.e. most common grades in reinforced concrete structures); however, thanks to the pre-stressed concrete technology, the concrete grades used for construction are raised up to C-50 and S-600(i.e. most commonly used concrete and steel grades for pre-stressed concrete structures).

2.3. Historical development of pre-stressed box girder bridges

The first box girder cross section possessed deck slabs that cantilevered out only slightly from the box portion shown in figs a to e. With the pre-stressed concrete the length of cantilever could be increased. The high form work costs caused a reduction in the number of cells fig (f, g, h) in order to reduce the construction loads to

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minimum possible extent or to require only one longitudinal girder in working states even with multiple traffic lanes.[Box Girder Bridges Prayash Gomdenn Binesh Puliyelil Joy Aalto University School of Engineering Department of Structural Engineering and Building Technology Rak-11.3001 Design of Bridges].

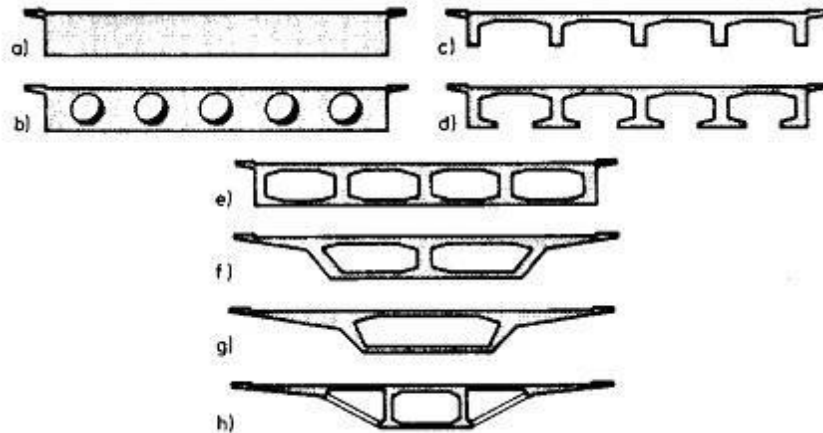


Figure 2.1. evolution of box girder beams.[Aalto University School of Engineering Department of Structural Engineering and Building Technology Rak-11.3001 Design of Bridges].

The first pre-stressed concrete bridges, most of I-cross sections were built towards the end of the 1920's. The great breakthrough was achieved only after 1945. "THE SCLAYN" bridge over the river Maas, which was built by Magnel in 1948, was the first continuous pre-stressed concrete box-girder bridge with 2 spans of 62.70m.

The box girder bridge was a popular choice during the road building expansion of the 1960s and many new bridge projects were in progress simultaneously. A serious blow to this use was a sequence of three serious disasters, when new bridges collapsed in 1970 (West Gate Bridge and Cleddau Bridge) and 1971 (Koblenz Bridge). Fifty-one people were killed in these failures, leading in the UK to the formation of the Merrison Committee and considerable investment in new research into steel box girder behavior. Most of the bridges still under construction at this time were delayed for investigation of the basic design principle. Some were abandoned and rebuilt as a different form of bridge altogether. Most of those that remained as box girder bridges, such as Erskine Bridge, were either redesigned, or had additional stiffening added later. Some bridges were strengthened a few years after opening and then further strengthened years later, although this was often due to increased traffic load as much as better design standards. The Irwell Valley bridge of 1970 was strengthened in 1970 and again in

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2000.[Box Girder Bridges Prayash Gomdenn Binesh Puliyelil Joy Aalto University School of Engineering Department of Structural Engineering and Building Technology Rak-11.3001 Design of Bridges].

Generally, prestressed Box girder bridges had evolved from Slab bridges and I-girder bridges since they gave freedom for designers to employ more cantilever sections, to use only one or two girders to carry heavy traffic loads during service period, and to benefit from the advantage prestressing concepts by reducing concrete volume.

2.4. Cost comparison between single cell and double cell box girder bridges

Single cell box girders are generally used in relatively narrow bridges. As the width increases, the bending moments in the deck slab increase and hence the thickness must increase. Beyond some critical width it becomes more economical to use a multi-cell box or multiple single cell boxes. In the case of multiple single cell box girders, the basic single cell units are cast separately and are connected after erection with a concrete joint and transverse post-tensioning. Thus, in general, it will be possible to have smaller basic units in this case than with a multi-cell box. On the other hand, a multi-cell box, of relatively small base width as in the Western Avenue Viaduct, may be advantageous when narrow piers are desirable.[G. C. Lacey and J. E. Breen. long span concrete bridges of segmental construction state of the art].

A variety of box girder shapes and pre-stressing systems have been used. The most suitable cross-sectional shapes for bridges of 30 ft.-50 ft. width appear to be a one or two-cell box or a pair of single cells connected by the deck slab.

The material economy of multi-cell box girder decks is a figment of the designer's imagination. The absolute minimum thickness for the top slab of a highway bridge spanning across 2 m wide voids is 170 mm, subject to checking the requirements of punching shear to the relevant loading and reinforced concrete codes of practice. However with the addition of 30 mm of concrete, a far more robust 200 mm thick slab may span up to 6 m between webs. There are more webs than are necessary to carry the shear force in the deck. In attempting to make the webs as thin as possible to save weight, the deck becomes much more difficult to build. It is economical to minimize the number of webs and to make them thicker for easier casting.[Robert Benaim (2008).The Design of Prestressed Concrete Bridges, Concepts and Principles].

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The absolute minimum thickness of the bottom slab of a bridge deck is 150 mm. With the addition of a further 20–30 mm, this bottom slab may span 6 m or more.

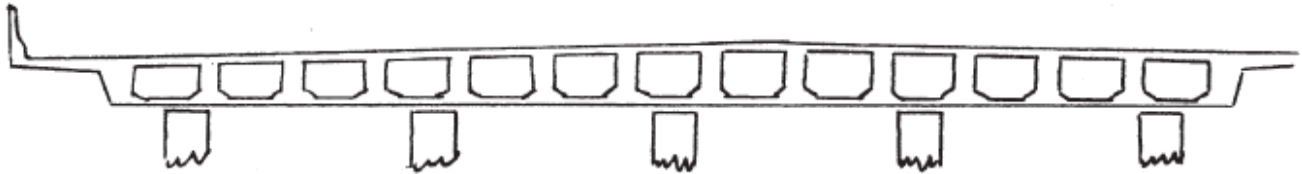


Figure 3.1 Multicell box varying in width, depth and cross fall

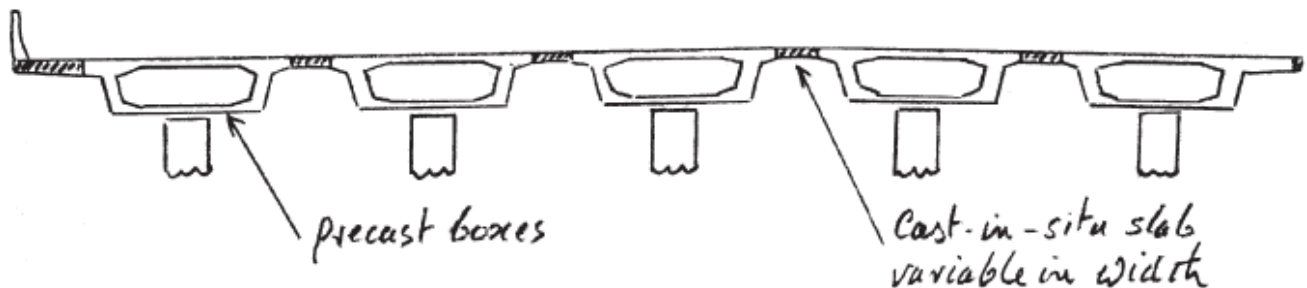


Figure 3.2 Alternative with standard precast glued segmental boxes.

The need to clad both faces of each web and each face of the top and bottom slabs with a mesh of reinforcement inevitably leads to the use of expensive small bars, slow fixing rates and a high rate of reinforcement, typically over 230 kg/m³. As a consequence, multi-cell box decks are heavier than well-designed ribbed slab or single-cell box section decks, have more reinforcement content, and are far more labor intensive and costly to build. [Robert Benaim (2008). *The Design of Prestressed Concrete Bridges, Concepts and Principles*].

The designer faced with one of these decks, perhaps specified by a client, should investigate alternatives. These may be voided slabs, ribbed slabs for cast-in-situ construction, or precast segmental single cell boxes attached by their top slabs. As the deck width and the cross fall varied along the deck, the height and width of each cell was variable and the proposed design was virtually unbuildable. The alternative proposed was a series of identical precast segmental boxes, linked by cast-in-situ top slabs whose width could be extended as necessary. The variable geometry of the conforming design could thus be matched exactly with a standard precast product. Alternatively, if it had been essential to maintain a flat soffit, the cross section could have been made far easier to

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build by adopting circular voids. [Robert Benaim (2008).The Design of Prestressed Concrete Bridges, Concepts and Principles].

2.5. Concrete Box Girder Beam and pre-stressing

Box girder bridges contain top deck, vertical web, and bottom slab and are often used for spans of 15 to 36 m with girders spaced at 1.5 times the structure depth. Beyond this range, it is probably more economical to consider a different type of bridge, such as post-tensioned box girder or steel girder superstructure. This is because of the massive increase in volume and materials. They can be viewed as T-beam structures for both positive and negative moments. The high tensional strength of the box girder makes it particularly suitable for sharp curve alignment, skewed piers and abutments, super elevation, and transitions such as interchange ramp structures. [Wai Fah Chen and Lian Duan (2000).Bridge Engineering Handbook: CRC Press].

Michael P Collins defines pre-stressing as a concrete material subjected to pre-compression: Pre-stressed concrete is a type of reinforced concrete in which the steel reinforcements has been tensioned against the concrete. This tensioning operation results in a self-equilibrating system of internal stresses (tensile stresses in the steel and compressive stresses in the concrete) which improves the response of the concrete to external loads. While concrete is strong and ductile in compression it is weak and brittle in tension and hence its response to external loads is improved to by applying a pre-compression. [Michael, P., & Denis (1997) Pre-stressed Concrete Structures: Canada: Response Publications].

Pre-stressing is more a philosophy than a specific technique. It means preparing a structure to receive a load by applying a pre-emptive countervailing load.[Robert Benaim(2008).The Design of Prestressed Concrete Bridges, Concepts and Principles].

Typical installed costs in the UK for reinforcement and pre-stressing steel are £700 per ton and £2,000 per ton respectively, a ratio of approximately three, whereas the ratio of the ultimate strengths is approximately four. When the weight of steel is governed by the ULS, as in this example, the pre-stressed version would show a saving of some 30 per cent. However, whereas the cost of reinforcing steel varies generally by not more than about ± 10 per cent between applications of a similar size, more than 50 per cent of the cost of pre-stressing consists of site labor, of which a large part is due to the operations of stressing and then grouting the tendons. Consequently the cost of pre-stressing varies considerably, depending principally on the length of the tendons, with the cost rising steeply as the length falls below about 25 m. A 10 m long tendon costs approximately twice as much per ton of steel as one 40 m long. One of the aims of a designer should be to make tendons as long as is practicably possible, although the

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lesser cost of the tendons needs to be balanced against their lower force due to friction losses. In countries with labor costs that are lower than those of the UK, pre-stressing becomes relatively cheaper in comparison with reinforcing. [Robert Benaim (2008).The Design of Prestressed Concrete Bridges, Concepts and Principles].

In addition to reduced cost, pre-stressing offers other advantages which will be described in later chapters, including considerations of build ability and speed and security of construction, as well as enhanced quality due to cancelled dead load deflections, elastic behavior and the absence of cracking.[Robert Benaim (2008).The Design of Prestressed Concrete Bridges, Concepts and Principles].

Wai-Fah Chan and Lian Duan classified pre-stressing systems into two: There are two types of pre-stressing systems: pre-tensioning and post-tensioning systems. Pre-tensioning systems are methods in which the strands are tensioned before the concrete is placed. This method is generally used for mass production of pretension members. Post-tensioning systems are methods in which the tendons are tensioned after concrete has reached a specified strength. [Wai Fah Chen and Lian Duan (2000).Bridge Engineering Handbook: CRC Press].

There are currently two types of pre-stressing: post tensioning and pre-tensioning [Bridge Design Practice. (1993). Section 3-Pres-tressed Concrete].

Pre-tensioning: - involves stretching of tendons between external anchorages before the concrete is placed, and the jacking force is released after the fresh concrete hardens and reached the desired strength.

- Tensioning is applied using hydraulic or mechanical device
- High early strength concrete is used
- Steam curing to accelerate hardening of concrete
- Expensive and anchorage hard ware avoided since the force is transferred by bond from steel to concrete

This technique is well suited to the mass production of beams using long line method and the simultaneous pre stressing of members at once results in great saving of labor

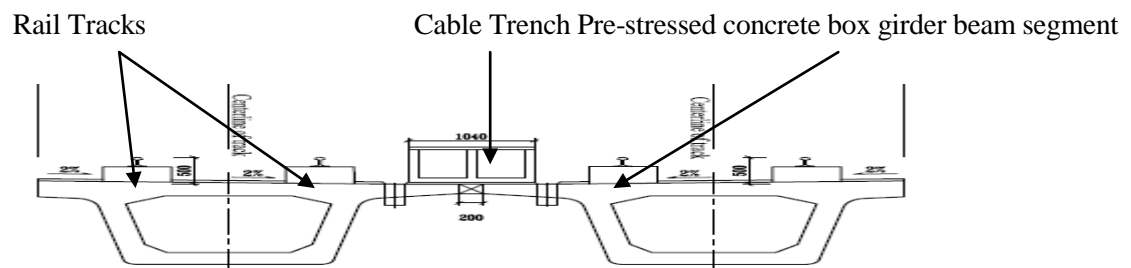
Post-tensioning: - tendons tensioned after hardening of concrete

Hollow conduits containing the unstressed tendons are placed in the forms to the desired profile before pouring

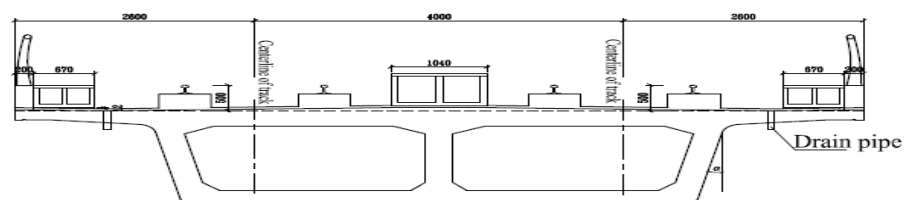
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of the concrete. When the concrete hardens and gains sufficient strength the tendons are tensioned where they are anchored by special fittings at the far end of the member and then anchored at the jacking and by similar fitting after which the jack is removed.[Bridge Design Practice. (1993). Section 3-Pre-stressed Concrete].To improve the performance of the member, the tendons are grouted with cement paste in their conduit after being stressed resulting bonds between tendons and inner wall of conduit

Advantage: - the ease with which the tendons eccentricity can be varied along the span to provide the desired counter moment.



(a)



(b)

Figure 2.4. Types of Railway box Girder Bridges (a) Two units of Single cell Railway Box Girder Bridges connected with cast in-situ wet joint (b) One unit of Double cell Box Girder Railway Bridges, Addis Ababa Light Railway Transit (AA-LRT) system elevated structures.

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2.6. The stressing tendon

Compressive forces are induced in a concrete structure by tensioning steel tendons of strands or bars placed in ducts embedded in the concrete. The tendons are installed after the concrete has been placed and sufficiently cured to a prescribed initial compressive strength. A hydraulic jack is attached to one or both ends of the tendon and pressurized to a predetermined value while bearing against the end of the concrete beam. This induces a predetermined force in the tendon and the tendon elongates elastically under this force. After jacking to the full, required force, the force in the tendon is transferred from the jack to the end anchorage. [Post-Tensioning Tendon Installation and Grouting Manual. (2004), U.S. Department of Federal Highway Administration].

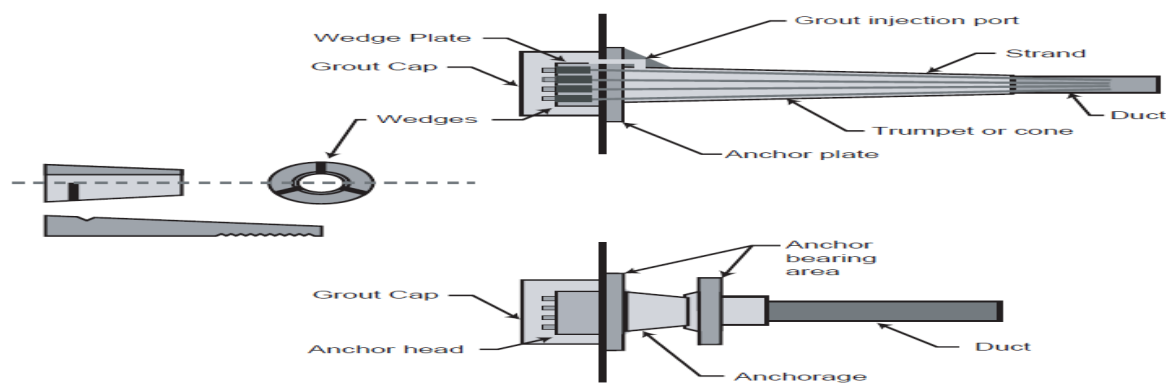


Figure 2.5. Prestressing tendons

Pre-stress losses are summarized as below:

Elastic Shortening - The concrete beam shortens at transfer when the prestressed strands are released and the force in them is transferred to the concrete. This elastic shortening is immediate and results in a reduction in the strain of the pre-stressing steel and therefore a pre-stress loss. The loss from elastic shortening should be included in both initial and total loss computations.

Concrete Creep - The time dependent increase of strain in hardened concrete subjected to sustained stress is defined as concrete creep.

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Steel Relaxation - Steel relaxation is very similar to concrete creep. With steel relaxation the length of the strand is held constant under stress and there is a time dependent loss in stress.

Anchorage Set - Some loss of pre-stress occurs to post-tensioned tendons as the anchorage hardware deforms and sets at the transfer of tension. The amount of set is a function of the type of anchorage system used. The amount of pre-stress loss is a function of this anchorage set and the length between anchorages. Power seating of the chucks tends to reduce this loss.

Friction - Tendons also lose some pre-stress due to friction inside the ducts during stressing operations.

Total Losses - Some of the losses mentioned above are interdependent. Shrinkage and concrete creep reduce the strain in the pre-stressing steel, which reduces the force in the pre-stressing steel. The reduction in force in the pre-stressing steel affects elastic shortening, future concrete creep and steel relaxation.[Dagim Tadesse(2015). An assessment of Pre-stressing in post-tensioned Concrete Box Girder Railway Elevated Structures: a case study on Comparative Analysis of Continuous Vs Simply Supported Elevated Structure of Addis Ababa's Light Rail Transit: Addis Ababa University School of Graduate Studies Institute of Technology Department of Civil Engineering].

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2.8. Advantages of pre-stressed concrete over ordinary reinforced concrete

Pre-stressed concrete has advantages over ordinary reinforced concrete summarized as below. [Bridge Design Practice. (1993). Section 3-Pre-stressed Concrete].

- Reduction of concrete and steel quantities
- Considerable reduction in depth of section, not only relative to reinforced concrete, but also relative to structural steel
- An assessment of Pre-stressing in Post-tensioned Concrete Box Girder Railway Elevated Structures
- Crack less concrete within a known range of load. This results in greater durability under severe conditions of exposure
- Possesses maximum rigidity under working loads and maximum flexibility under excessive overloads
- Provides capacity to support a load in excess of the design load in which cracks appear but disappear completely on removal of the excess load
- Provides resistance to repeating and alternating loads even when exceeding the design load
- Produce definite reduction in diagonal tension which leads to fewer stirrups needed for shear
- Makes it possible to control and/or reduce long term deflections
- Added flexibility for construction of continuous multi-span frame

2.9. Disadvantages of pre-stressed concrete over ordinary reinforced concrete

Pre-stressed concrete has disadvantages over ordinary reinforced concrete summarized as below.[Bridge Design Practice. (1993). Section 3-Prestressed Concrete].

- The pre-stressed structure is more sensitive to quality of workmanship and materials
- Creep and shrinkage of the concrete and relaxation of the pre-stressing steel are important considerations which need to be considered by the designer.

In general, pre-stressed concrete railway bridges have the following advantages:

- No cracking: for durable structure, weather resistant saucers, water tight structures are in tanks, reservoirs' ...etc.
- Deflection is controlled (small)
- Smaller cross-section can be used compared with ordinary concrete --- lower own weight, longer spans

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- Efficient use of high strength materials, (the saving amounts 15 to 30% of concrete and 60 to 80% steel)
- Improved resistance to shearing force
- Improved resistance to repeating loads (vibration)
- Ability to resist over load – cracks close up after removing over load
- Possibility to connect prefabricated members by moment and shear resisting joints

2.10. Advantages of box girder bridges over other types of bridges

Advantages of box girder bridges over other types of bridges are summarized as below.[Box Girder Bridges Prayash Gomdenn Binesh Puliyelil Joy Aalto University School of Engineering Department of Structural Engineering and Building Technology Rak-11.3001 Design of Bridges].

- High stiffness against torsion compared to normal plate girders
- Good option when building bridges that curve in plane
- Smaller economical construction depth when compared to plate girders
- It has high structural efficiency and tensional strength which minimizes the pre-stressing force required to resist a given bending moment. Pre-stressing is not required for spans up to 60m.
- Their closed shape makes them more efficient in corrosion resistance than plate girders, as the shape drastically reduces the exposed surface area
- The upper flange can act also as a part of the deck structure
- Bracings can be hidden in the space available inside the girder and the structure becomes more aesthetic from outside

2.11. Disadvantages of box girder bridges over other types of bridges

Disadvantages of box girder bridges over other types of bridges are summarized as below.[Box Girder Bridges Prayash Gomdenn Binesh Puliyelil Joy Aalto University School of Engineering Department of Structural Engineering and Building Technology Rak-11.3001 Design of Bridges].

- .One of the main disadvantages of box decks is that they are difficult to cast in-situ due to the inaccessibility of the bottom slab and the need to extract the internal shutter.
- Either the box has to be designed so that the entire cross section may be cast in one continuous pour, or the cross section has to be cast in stages.

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- Box girders are more expensive to fabricate, and they are more difficult to maintain, because of the need for access to a confined space inside the box.
- Girders are not efficient as trusses in resisting loads over long spans.
- Typically higher construction costs compared to I-section steel girder

3. Analysis Overview

This chapter describes the analysis method, material properties, applied loads, strength and serviceability limit states design requirements. The analysis is performed using the excel spreadsheet design template and SAP2000.

The purpose of the analysis is to determine the demarcation span between two post-tensioned single cell box girder railway bridges and a double cell post-tensioned box girder railway bridge both of them are analyzed and designed for the same carriage way width.

The general step by step procedures followed in the excel spreadsheet design program for the analysis of the above two types of scenario are:

- One unit of single cell box girder railway bridge with initial assumed girder depth is loaded with dead loads, secondary dead loads and live loads .Then the initial deflection (i.e. immediately after pre-stressing) and the long time deflection are checked for the allowable deflection. If deflection is not satisfied the section is revised but if satisfied the section is checked and designed for shear and bending moment.
- In the same manner one unit of double cell box girder Railway Bridge with initial assumed girder depth is checked for deflection requirements and designed for shear stress and flexural stress.

3.1 Analysis Models

The analysis models are girders which could be classified as one unit of single cell and one unit of double cells. The single cell: from 20m-span simply supported up to 50m-span simply supported and a double cell: 20m-span simply supported up to 50m-span simply supported.

3.2 Materials

The following table summarizes the material properties to be used in the analysis. A concrete compressive strength of 50MPa is applied.

Material Property		Values
Concrete	compressive strength	$f'_c = 50 \text{ MPa}$, $f_{ci} = 0.6 \times 50 = 30 \text{ MPa}$ (for
	Tensile strength	$f_{ct} = 0 \text{ MPa}$, $f_{cit} = 0.25 f_{ci}^{1/2} = 1.369 \text{ MPa}$
	Elastic modulus	$E_c = 0.043 w_c^{1.5} f_c^{0.5} = 42,689,387.17 \text{ MPa}$,
		$E_{ci} = 0.043 w_c^{1.5} f_{ci}^{0.5} = 33,067,057.12 \text{ MPa}$
Pre-stressing tendons	Ultimate strength	$f_{pu} = 1860 \text{ MPa}$
	Yield stress	$f_{py} = 0.9 f_{pu} = 1674 \text{ MPa}$
	Effective prestress after all	$f_{pj} = 0.7 f_{pu} = 1,302 \text{ MPa}$
	losses	
	Elastic modulus	$E_p = 195,000 \text{ MPa}$
Reinforcing steel bars	Yield stress	$f_y = 335 \text{ MPa}$
	Elastic modulus	$E_s = 210 \text{ GPa}$

Table 3. 1. Material properties

The ducts for multiple wire, strand, or bar tendons shall have an inside cross sectional area not less than two times the net area of tendons. And the minimum clear distance between pre-stressing tendons at each end of a member shall not be less than 1-1/3rd times the maximum size of the coarse aggregate. The minimum spacing center-to-center of tendon size of 15.24mm shall be 50mm. Again, the minimum clear distance between adjacent post-tensioning ducts is 40mm. The horizontal spacing among the duct diameter is used to provide sufficient space for concrete placement and vibration AREMA 2010 (CL.8.17.5.5).

3.3. Concrete Covers

For the Cast-in-Place Concrete the following minimum concrete covers provided for prestressing tendons and non-prestressed reinforcements.

Reinforcement or steel ducts	Concrete covers (mm)	
	Cast-in-place	Pre-cast
Post-tensioning ducts	75 mm	40 mm
Non-pre-stressed reinforcement	50mm	40 mm
Stirrups, ties and spirals	50mm	25mm

Table 3.2: Concrete cover requirements, AREMA 2010 [CL.8.17.5.2]

3.4 Loads

The basic loadings that used in the analysis are Dead Load, Additional dead load, Live load and prestress load. Others loadings like wind load, earth quake load, etc could be considered. Yet, they do not have major influence on the relative percentage values of pre-stressing.

3.4.1 Dead Load (DL)

The dead load carried by the box girder consists of its own self weight which is based on the volume weight of concrete, varies with the span lengths from 20m up to 50m.

3.4.2 Secondary Dead Load (SDL)

They are loads of auxiliary facilities such as the track side equipment, cable ducts, railings, pipelines and the supporting devices, cast-in-place wet joints of bridge deck boards, cross slabs, waterproof layers, and paving layers of bridge decks, etc. It is taken to be 25KN/m (for the single cell) & 77KN/m (for the double cells).

3.4.3 Live Load

The Live load scheme as per AREMA 2010 [2] is Copper E-80 live load configuration. The loads of tram cars are shown in the figure below:

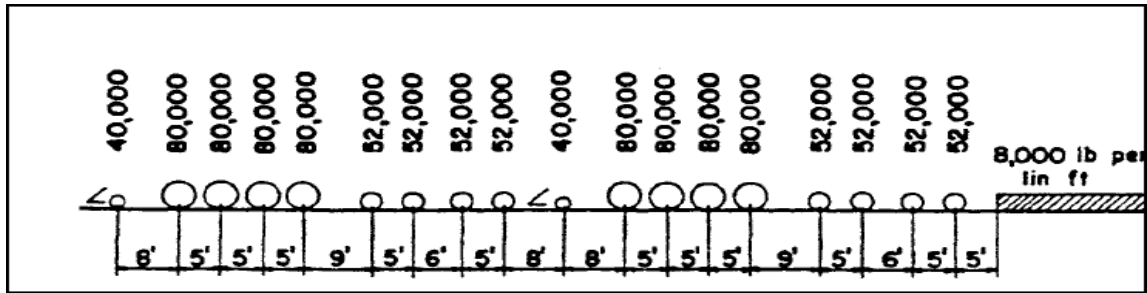


Figure 3.1. Copper E-80 1 Live load configuration [American Railway Engineering and Maintenance-Of-Way Association (AREMA), (2010). Manual for Railway Engineering. ISSN 1542-8036 - Print Version. Volume II: 8-17-9].

3.5. Load Combinations and Load Factors

There are several limit states that need to be considered in the design of the box-girders. The following groups from AREMA [CL.8.2.2.4] represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure shall be proportioned for the group of loads that produce the most critical design condition.

Strength limits state: basic load combination by which flexural, shear strength are investigated. Service limit state: a Load combination relating to normal operational use of the bridge. Concrete stresses, deflection and camber are investigated at service limit states.

Service I: Crack control and limiting compression in pre-stressed concrete

Service II: Crack control and tension in pre-stressed concrete

The load combinations and load factors used in the analysis under strength and service limit states are summarized in the table below. Extreme event limit state is not considered in the analysis for the reason that earthquake and appropriate collision forces are out of the scope of this study. Again, fatigue limit state is not considered because fatigue of the reinforcement need not be checked for fully pre-stressed concrete members satisfying the requirements of service limit state.

Loads	Permanent Loads		Transitory Load
	DL	ADL	LL
Strength limit state (Strength I)	1.4	1.4	2.33
Service limit state (Service I)	1	1	1
Service limit state (Service II)	1.25	0	0

Table 3.2 Table 3.3: Load Combinations simple spans pre-stressed box girder beams (N.B. DL = dead load, ADL = Additional Dead Load or secondary dead load, LL = live load = secondary pre-stress effects)

As shown in the table above, the load combinations Service II only consider permanent loads without live loads; the tendons push the bridge upward to make camber, longitudinally, while there might not be enough dead load to weigh the bridge down. Therefore, these load combinations must be checked whether they fulfill Strength limit state and Service limit state requirements. This, specifically, is critical for slender bridges with large pre-stressing forces and small dead loads. Considering only live load can be used for superstructure vibration check.

Strength Capacity Reduction Factors

The following strength capacity reduction factors used according to AREMA [CL.8.17.15.1]

For flexure: $\phi = 0.95$

For shear: $\phi = 0.90$

3.6. Design Requirements

The analyses are done to satisfy AREMA 2010's Load factor (Strength limit state) and Service load design requirements. The strength limit state deals with flexure and shear strength requirements where as the Service limit state deals with stress, vibration and deflection limitations. The design check considers only longitudinal behavior.

Flexural strength

According to AREMA [CL.8.17.18] pre-stressed concrete members may be assumed to act as uncracked members subjected to combined axial and bending stresses within specified service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included in pre-tensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted.

Rectangular Sections

For rectangular or flanged sections having pre-stressing steel only, in which the depth of the equivalent rectangular stress block, defined as $(A_s f_{su}) / (0.85 f'_c b)$, is not greater than the compression flange thickness t , and which satisfy the reinforcement index for pre-stressing steel only, the design flexural strength shall be assumed as:

$$\phi M_n = \phi [A_s f_{su} d \{1 - 0.6(\rho f_{su} / f'_c)\}]$$

For rectangular or flanged sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as $(A_s f_{su} + A_s f_s y) / (0.85 f_c b)$, is not greater than the compression flange thickness “t” and which satisfy the reinforcement index with non-prestressed tension reinforcement included, the design flexural strength shall be assumed as:

$$\phi M_n = \phi \{ A_{ps} f_{su} d [1 - 0.6((\rho f_{su} / f_c) + (d_t / d)(\rho f_s y / f_c))] + A_s f_s y d_t [1 - 0.6((d / d_t)(\rho f_{su} / f_c) + (\rho f_s y / f_c))] \}$$

The reinforcement indices of sections with non-prestressed reinforcement tension reinforcement included (f_{su} is for bonded Members with non-prestressed tension reinforcement included):

Non-Prestressed Reinforcement

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

$$\text{For rectangular sections: } (\rho f_s y / f_c) d_t / d + (\rho f_{su} / f_c) - (\rho f_s y / f_c) < 0.36 \beta_1$$

$$\text{For flanged sections: } (A_s f_s y) / (b' d f_c) + (A_{sr} f_{su}) / (b' d f_c) - (A_s f_s y) / (b' d f_c) < 0.36 \beta_1$$

For members with reinforcement indices greater than $0.36 \beta_1$, the design flexural strength shall be assumed not greater than:

$$\text{For rectangular sections: } \phi M_n = \phi [0.36 \beta_1 - 0.08 \beta_1^2] f_c b d^2$$

$$\text{For flanged sections: } \phi M_n = \phi [0.36 \beta_1 - 0.08 \beta_1^2] f_c b d^2 + 0.85 f_c (b - b') t (d - 0.5t)$$

3.7. Shear Strength

Pre-stressed concrete flexural members shall be reinforced for shear and diagonal tension stresses as per AREMA [CL.8. 17.21]. Members subject to shear shall be designed so that

$$V_n = \phi (V_c + V_s)$$

Shear strength provided by Concrete

For members with effective prestress force not less than 40 percent of the total tensile strength of flexural reinforcement:

$$V_c = \left(\frac{\sqrt{f'_c}}{20} + 5 \frac{V_u d_p}{M_u} \right) b_w d$$

But, V_c need not be taken less than $1/6(f'_c)^{1/2} b_w d$ and nor shall V_c be taken greater than $0.4\sqrt{f'_c} b_w d$.

Shear strength provided by web reinforcement

Shear reinforcement shall consist of stirrups perpendicular to axis of member.

The shear strength provided by web reinforcement shall be taken as $V_s = (A_v f_{sy} d)/s$.

Where,

A_v is the area of web reinforcement with in a distance s .

V_s shall not be taken greater than $0.66 b' d$ and d need not be taken less than $0.8h$.

The spacing of web reinforcing shall not exceed $0.75h$ or 600mm when V_s exceeds $0.332b' d$

Stress in Concrete at immediately at transfer of pre-stress force

For bottom fiber: $f_b = P_{se}/A + P_{se}e_c/s_b + M_g/s_b$

For top fiber: $f_t = P_{se}/A + P_{se}e_c/s_t + M_g/s_t$

Stress in Concrete at immediately at service loads

For bottom fiber: $f_b = P_{se}/A + P_{se}e_c/s_b + (M_g + M_{SDL} + M_{LL+I})/s_b$

For top fiber: $f_t = P_{se}/A + P_{se}e_c/s_t + (M_g + M_{SDL} + M_{LL+I})/s_t$

Where,

P_{se} = effective prestress force after allowing for all losses (KN)

A=Cross sectional area

e_c = eccentricity of strands

M_g =bending moment due to self weight

M_{SDL} = Unfactored bending moment due to Superimposed dead load

M_{LL+I} = bending moment due live load and impact load

s_b = bottom fiber section modulus (m^3)

s_b = bottom fiber section modulus (m^3)

Initial Concrete Strength

Service I limit state is used to determine the concrete compressive stress. According to AREMA [CL.8.17.16.2] the concrete stresses in concrete immediately after pre-stress transfer (before time-dependent prestress losses- Creep and Shrinkage) is limited to $0.55 f'_{ci}$.

For extreme fiber stress in tension again not to exceed $0.25 \sqrt{f'_{ci}}$.

Final Concrete Strength

Service I and Service II limit state are used to determine the concrete compressive stress. Live load only also used to check the vibration.

Stresses in the concrete at service loads shall not to exceed $0.4f'_{ci}$ in Compression and 0 in tension.

3.8. Analysis assumptions

- The centre of gravity of tendons is assumed to coincide with the centroid of the duct.
- Designs of substructure are not considered in this analysis.
- The beams are designed as a standard beam but the real shape of the girders deviate in the reality to a little.

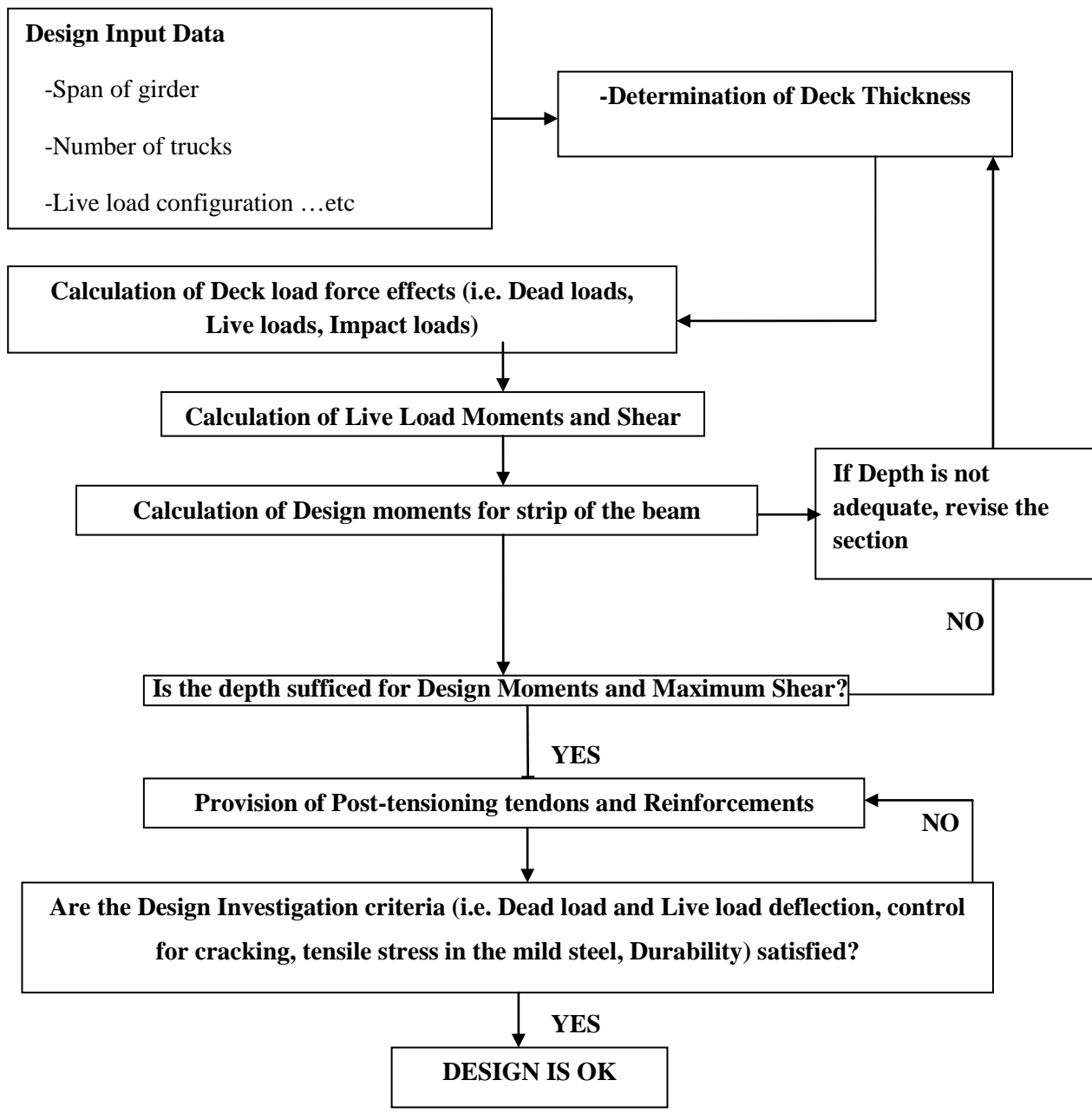
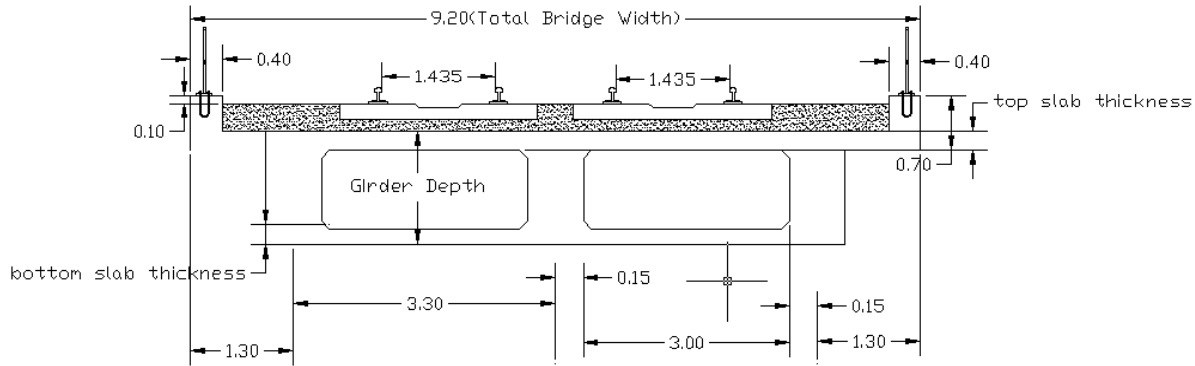


Figure 3.2. Design flow chart for the analysis and design of single cell and double cell box girder beams

4. Gravity load analysis and design of one unit of double cell box girder Railway Bridge with span of 20m using AREMA and LRFD bridge design manual

Preliminary Dimensions



Typical Section

Figure 4. 1 preliminary dimensions for one unit of double cell box girder Railway Bridge (typical section)

Grade of Concrete=C- 50

Grade for mild steel= S- 335

Grade for tendons, $f_{pu}=1860$ Mpa

Unit weight for normal concrete= 24KN/m^3

Unit weight for reinforced concrete, $\gamma_{rc}= 25\text{KN/m}^3$

Unit weight for sand, gravel ballast= 19KN/m^3

Unit Weight of Ballast including Ties = 19KN/m^3

Thickness of the ballast= 0.35m

Length of Tie =2.74m

Width of Tie=0.33m

Depth of Tie =0.25m

Span length (L) = 20m

Total width of the box girder beam=9.2m

Girder depth from AREMA MRE 2010 Volume-2, Chapter 26.17.4

$$1/15 > d_o/L > 1/3 ,$$

Where,

d_o = girder depth, feet

L = span length between supports, feet

Girder web thickness= 0.3m

Top Slab Thickness (t) = $a/30$ to 150mm = 0.11m to 150m(Section 17.11 of AREMA manual)

Where,

a = C/C of Girder Spacing=3.3m

Therefore, take $t=0.15$ m

Width and height of Fillets =0.1m

Bottom slab thickness = 0.15m (i.e. minimum is 140mm from Section 17.11 of AREMA manual)

Overhang Distance (c) =1 .3m

Total width of the deck slab, $b_s=9.2$ m

Clear spacing between end box cells and exterior vertical edge of the beam =30mm

Rail gauge dist, $B_g=1.5$ m

Live load configuration -Cooper E-80

Thickness of web, $b_w= 700$ mm

Girder Depth=1 .20m

4.1. Design for tendons area and pre-stressing force (Jacking Force)

4.1.1. Material properties for the pre-cast concrete box girder

$$f_{ck}=0.8f_{cu}=0.8*50=40\text{Mpa}$$

$$f_{ctd}=0.21 f_{ctk}^{2/3}/1.5=1.64\text{Mpa}$$

$$f_{cd}=0.85 f_{ck}/1.5=22.67 \text{ MPa}$$

$$f_{ct}=0.6*f_{ck}=0.6*40=24\text{Mpa}$$

$$f_{tr}=f_{cw}-hf_{it}, h=0.82 \text{ for post tensioned member}$$

$$f_{cw}=0.5f_{ck}=0.5*40=20\text{Mpa}$$

$$f_{it} \leq 0.21 (f_{ctk})^{2/3}$$

$$f_{it} \leq 0.21 (40)^{2/3}=2.46\text{Mpa}$$

$$f_{it}=2.46\text{mpa}$$

$$f_{tr}=20\text{mpa}-0.82*2.46\text{mpa}=17.99\text{Mpa}$$

$$f_{br}=hf_{ct}-f_{tw}$$

$$f_{tw}=0 \text{ since full prestressing}$$

$$f_{tw}=0.75f_{ck} \text{ for partial prestressing}$$

$$f_{br}=hf_{ct}=0.82*24=19.68\text{Mpa}$$

$$f_{br}=19.68\text{mpa}$$

Permissible stresses in concrete

$$f'_c=50\text{MPa}$$

$$f_{ci}=0.25f'_c=12.5\text{MPa}$$

$$f_{ct}=0.00\text{MPa}$$

At transfer (before time dependent prestress losses)

$$\text{Compression}=0.25*f'_c=12.5\text{MPa}$$

$$\text{Tension}=0.55f_{ci}=13.75\text{MPa}$$

At service loads (after allowances for all prestress losses)

$$\text{Compression}=0.4f'_c=20.00\text{MPa}$$

$$\text{Tension in pre-compressed tensile zone}=0.00\text{MPa}$$

4.1.2. Loadings for the pre-cast concrete box girder

Superimposed dead load

a) Dead load due Edge beams

$$h_1=0.55\text{m}$$

$$b_1=0.40\text{m}$$

$$A_1=b_1*h_1=0.22\text{m}^2$$

$$DL_1=2*A_1*\gamma_{rc}=10.56\text{KN/m}$$

$$DL_1=10.56\text{KN/m}$$

b) Superimposed loads from ballast including track ties

$$DL_2=\text{Unit Weight of Ballast including Ties}*(\text{Thickness of the ballast} + \text{Depth of Tie})=19\text{KN/m}^3 *$$

$$(0.35+0.25)=10.45\text{KN/m}^2$$

c) Dead loads from Railing

DL3=1.0KN/m (i.e. from AREMA Manual)

d) Self weight of track rail and fastening

From AREMA Manual, for a single track=3KN/m

Since we have double tracks on a single beam, a load of 6KN/m is taken. DL4=6KN/m

$$DL_{tot}=DL1 +DL2+DL3+DL4=28.96KN/m$$

Superimposed live load due to the axle load

From the AREMA PCI bridge design manual, chapter 17, which is obtained from AREMA Table 1.17, load can be obtained in respective to the span length.

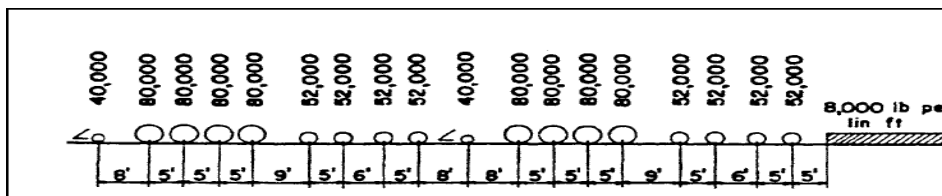


Figure 4. 2. Copper E-80 1 Live load configuration [American Railway Engineering and Maintenance-Of-Way Association (AREMA), (2010). Manual for Railway Engineering. ISSN 1542-8036 - Print Version. Volume II: 8-17-9].

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

Span, ft	Maximum bending moment ,ft-kips	Maximum bending moment at quarter point, ft-kips
5	50	37.5
6	60	45
7	70	55
8	80	70
9	93.89	85
10	112.5	100
11	131.36	115
12	160	130
13	190	145
14	220	165
16	280	210
18	340	255
20	412	300
24	570.42	420
28	730.98	555
32	910.85	692
36	1097.3	851
40	1311.3	1010.5
45	1601.2	1233.6
50	1901.8	1473
55	2233.1	1732.3
60	2597.8	2010
70	3415	2608.2
80	4318.9	3298
90	5339.1	4158
100	6446.3	5060.5
120	9225.4	7098
140	12406	9400
160	15908	11932
180	19672	14820

Span, ft	Maximum shear forces ,kips			Maximum pier reaction, kips
	end	quarter point	mid span	
5	40	30	20	40
6	46.67	30	20	53.33
7	51.43	31.43	20	62.86
8	55	35	20	70
9	57.58	37.78	20	75.76
10	60	40	20	80
11	65.45	41.82	21.82	87.28
12	70	43.33	23.33	93.33
13	73.24	44.61	24.61	98.46
14	77.14	47.14	25.71	104.29
16	85	52.5	27.5	113.74
18	93.33	56.67	28.89	121.33
20	100	60	28.7	131.1
24	110.83	70	31.75	147.92
28	130.86	77.14	34.29	164.58
32	131.44	83.12	37.5	181.94
36	141.12	88.9	41.1	199.06
40	150.8	93.55	44	215.9
45	163.38	100.27	45.9	237.25
50	174.4	106.94	49.73	257.52
55	185.31	113.58	52.74	280.67
60	196	120.21	55.69	306.42
70	221.04	131.89	61.45	354.08
80	248.4	143.41	67.41	397.7
90	274.46	157.47	73.48	437.15
100	300	173.12	78.72	474.24
120	347.35	202.19	88.92	544.14
140	392.59	230.23	101.64	614.91
160	436.51	265.51	115.2	687.5
180	479.57	281.96	128.12	762.22

Table 4.1. Maximum Bending Moments, Shear Forces and Pier Reactions for Cooper E-80
 Maximum moments, shears and pier (floor beam) reactions for Cooper E-80 Live [AREMA Manual chapter 17]

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

For the span Length of L =20m=65.62ft

Interpolating between L1= 60ft and L2=70ft, the following result is obtained

Span,ft	Maximum bending moment ,ft-kips	Maximum bending moment at quarter point, ft-kips
60.00	2597.80	2010.00
65.62	3056.80	2346.00
70.00	3415.00	2608.20

Span,ft	Maximum shear forces ,kips			Maximum pier reaction, kips
	end	quarter point	mid span	
60.00	196.00	120.21	55.69	306.42
65.62	210.06	126.77	58.93	333.19
70.00	221.04	131.89	61.45	354.08

Figure 4.2. Interpolation between span (ft) and values in TABLE 17.3.2.1.1 of AREMA manual, chapter 17 for one unit of double cell box girder Railway Bridge

$M_{LL} = 16580.11 \text{ KN-m}$

For pre-stressed concrete, the impact load is a percentage of the live load based on span length in ft.

$L \leq 60\text{ft}, I = 35 - L^2 / 500 = 26.39\%$

$60 < L \leq 135 \text{ ft}, I = 14 + 800 / (L - 2) = 26.58\%$

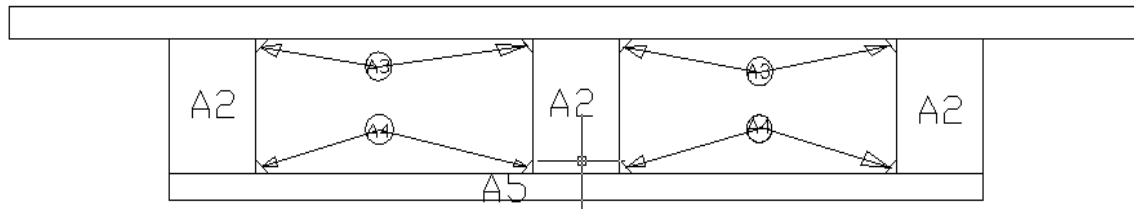
$L > 135 \text{ ft}, I = 20\% \dots\dots\dots \text{AREMA Eq.17-1,}$

Where L=Span length of member in ft

Therefore for a span length= 65.62ft

Impact load factor=26.58%

26.58% of impact load will be added $M_{LL+I} = 16580.11 \text{ KNm} + 0.2658 * 16580.11 \text{ KNm} = 20986.33 \text{ KNm}$



Typical Section

Figure 4. 3Regular regions used for calculation of moment of inertia of 20m double cell box girder

No.	b	h	y	yi	Ayi	I	di	Adi2	I+Adi2
A1	9.20	0.20	0.10	1.40	2.58	0.006	-0.691	0.879	0.885
3*A2	0.30	1.15	0.58	0.43	0.44	0.114	0.284	0.083	0.197
4*A3	0.10	0.10	0.07	0.97	0.02	0.000	-0.258	0.001	0.001
4*A4	0.10	0.10	0.03	-0.12	0.00	0.000	0.826	0.014	0.014
A5	6.90	0.15	0.08	-0.23	-0.23	0.002	0.934	0.903	0.905
Σ					2.80	0.122		1.880	2.002

Figure 4. 3. Regular regions used for calculation of moment of inertia of 20m one unit of double cell box girder

Yb=	0.71	Sb=	2.82
Yt=	0.49	St=	4.08

Figure 4. 4. Regular regions used for calculation of moment of inertia of 20m one unit of double cell box girder beam.

$$M_{SDL} = SDL * L^2 / 8 = 1448.00 \text{ KNm}$$

$$Q_{g1} = A * w_c = 84.84 \text{ KN/m}$$

$$M_{g1} = Q_{g1} * L^2 / 8 = 4740.00 \text{ KNm}$$

4.1.3. The pre-stressing force and calculation of number of strands

Concrete cover=40.00mm

Try eccentricity of strands at mid span= $e_c=y_b$ -concrete cover=0.68m

$$f_{tt} \leq 0.21(f_{ckt})^{2/3}$$

$$f_{tt} \leq 0.21(0.8*f_c)^{2/3}=2.46\text{MPa}$$

$$F_{sup} = f_{tt} - M_{g1}/z_t = 1.29\text{MPa}$$

$$f_{inf} \geq [f_{tw}/h + (M_q + M_{SDL} + M_{LL+I})]/h * z_b = 11.73\text{MPa}$$

$$f_{tw} = 0 \text{ and } h = 0.82$$

$$e_{max} = [z_t * z_b [f_{inf} - f_{sup}] / [A(f_{sup} * z_t + f_{inf} * z_b)]]$$

$$e_{max} = 0.73\text{m}$$

Therefore eccentricity of strands=0.68m

$$\text{Bottom tensile stress due to applied loads} = f_b = ((M_g + M_{SDL} + M_{LL+I})/S_b) = 9.62\text{MPa}$$

Since the allowable tensile stress in bottom fiber is zero, the required pre-compression is =9.62MPa

$$\text{Bottom fiber stress due to prestress after all losses} = f_b = P_{se}/A + P_{se} * e_c/S_b$$

Here all the values are known except P_{se} , P_{se} can be calculated in the equation

$$P_{se} = ((f_b * A * S_b) / (S_b + A * e_c))$$

$$P_{se} = 19636.32\text{KN}$$

An average of 22% loss in case of pretension and 18% in post-tension may be assumed with condition of over tensioning has been applied to overcome friction and anchorage losses.

Allowable tensile stress in prestressing tendons immediately after prestress transfer is the larger of

$$0.82f_{py} \text{ (} 0.82 * 0.9f_{pu} = 0.738f_{pu} \text{) or } 0.75f_{pu}$$

$$0.75f_{pu} = 1395.00\text{MPa}$$

$$\text{Area of strands} = 33337.00\text{MPa}$$

$$\text{Area for a 15.2mm diameter single strand} = 181.37\text{mm}^2$$

$$\text{No of 15.2mm strands} = 194$$

4.1.4. Check for concrete stresses

Stress at transfer at mid span

Concrete stress at the top fiber of the beam, f_t

$$f_t = P_{si}/A - P_{si}e_c/s_t + M_g/S_t$$

M_g is based on the overall length of span=20m

$$M_g = w_c A * L^2 / 8 = 4740 \text{KNm}$$

$$f_t = 2.91 \text{MPa} < 6.25 \text{MPa} > -27.50 \text{MPa}$$

Stress at transfer at mid span

Concrete stress at the bottom fiber of the beam, f_b

$$f_b = P_{si}/A + P_{si}e_c/s_b - M_g/S_b$$

M_g is based on the overall length of span=20m

$$M_g = w_c A * L^2 / 8 = 0.63 \text{KNm}$$

$$f_b = 7.94 \text{MPa} < 20 \text{MPa} > 0 \text{MPa}$$

Stress at transfer at the end

Since the strands are straight and all the strands are bonded to for the full length of the beam, the concrete stress at the end are simply the stresses at the midspans without the stress due to dead load

Stress at service loads midspan

Concrete stress at the top fiber of the beam, f_t

$$f_t = P_{se}/A - P_{se}e_c/s_t + M_g + M_{SDL} + M_{LL+I}/S_t$$

M_g is based on the overall length of span=20m

$$M_g = w_c A * L^2 / 8 = 4740 \text{KNm}$$

$$f_t = 8.42 \text{MPa} < 20 \text{MPa} > 0.00 \text{MPa}$$

The pre-stress force is at its maximum value at the release and service loads don't affect stresses at the end of the beam.

Therefore stresses at the release will govern at the end of the beam; there is no need to check the stresses at the ends of the beam at service loads.

Flexural strength

Stress in strands at flexural strength

$$f_{ps} = f_{pu} (1 - 0.5 p_p f_{pu} / f'_c)$$

Provided that f_{se} is greater than $0.5 f_{pu}$

$$f_{se} = 559.14 < 0.5 f_{pu} = 930 \text{ MPa}$$

4.1.5. Check for the prevailing deflection at various stages of the loadings

Based on the AREMA design manual, the deflection of simple or continuous spans shall be designed so that the deflection due to service load plus impact load should not exceed $L/640$ of the span length.

Deflection immediately after pre-stressing

Total immediate Deflection = $a_p + a_q$

$$\text{Where } a_p = -P_i e L^2 / 8 E I_c$$

$$a_q = 5 * \sum DL * L^2 / E I_c$$

$$P_i = 23200.05 \text{ KN}$$

$$E = 42689.39 \text{ MPa}$$

$$e = 0.68 \text{ m}$$

$$I_c = 1.81 \times 10^{11} \text{ m}^4$$

$$L = 20 \text{ m}$$

$$\sum DL = 94.80 \text{ KN/m}$$

Therefore, $a_p = -8.72 \text{ mm}$ (upward deflection)

Therefore, $a_q = 2.41 \text{ mm}$ (downward deflection)

Hence total immediate deflection = $-6 \text{ mm} \leq$ Maximum deflection allowed = -31.25 mm

Hence OK!

Deflection under working loads after pre-stress losses and total long time deflection

$$a_f = a_{il}(1 + \Phi) - a_{ip} [(1 - \Delta p/P_i) + ((1 - \Delta p/P_i) * \Phi)]$$

Where,

$$\Delta p = P_i - P \text{ and } P \text{ is permissible stress under working condition}$$

$$\Phi = 2$$

$$P = f_{pe} * A_s$$

$$f_{pe} = 0.82 f_{pi} < 0.6 f_{pk},$$

Assuming 22% pre-stress loss for such a post-tensioned box girder beam

$$f_{pi} = 0.75 * f_{pk} = 1395 \text{ MPa}$$

$$\text{But, } f_{pe} = 0.6 f_{pk} = 1116 \text{ MPa}$$

$$\text{Therefore, } f_{pe} = 1116 \text{ MPa}$$

$$\text{Take } f_{pe} = 1116 \text{ MPa}$$

$$P = f_{pe} * A_s = 37204.09 \text{ KN}$$

$$\text{Implies } \Delta p = P_i - P = 8184.90 \text{ KN}$$

$$\text{Where } P_w = \sum DL + w(LL+I)$$

$$w(LL+I) = 8 * M(LL+I) / L^2 = 419.73 \text{ KN/m}$$

$$\sum DL = 84.84 \text{ KN/m}$$

$$P_w = \sum DL + w(LL+I) = 504.57 \text{ KN/m}$$

$$a_{il} = 5 * P_w * L^4 / 384 * E_{ci} = 0.01 \text{ m}$$

$$a_f = a_{il}(1 + \Phi) - a_{ip} [(1 - \Delta p/P_i) + ((1 - \Delta p/P_i) * \Phi)] = 8.39 \text{ mm (upward)}$$

Hence long time deflection = 8.39 mm (upward) ≤ Maximum deflection allowed = -31.25 mm

Hence OK!

4.1.6. Shear Design

Non prestressed reinforcement properties

$$f_y = 335 \text{ Mpa}$$

$$E_s = 210000 \text{ MPa}$$

$$A_p = \text{Area of a two legged 12mm diameter stirrup} = 0.009 \text{ m}^2$$

$$b = \text{width of compressive component} = 9.2 \text{ m}$$

$$d_p = \text{distance from extreme compression fiber to centroid of prestressing steel} = 0.59 \text{ m}$$

$$h = \text{height of the section} = 1.2 \text{ m}$$

$$d_v = 0.5 + 0.8h = 1.46 \text{ m}$$

$$b_v = \text{available web width} = 3 * 0.3 \text{ m} = 0.9 \text{ m}$$

Shear force in the section due to live load + Impact

At $h/2$ distance from the face of end support (i.e. $1.20/2 = 0.6 \text{ m}$), critical shear is calculated.

From table 4.1. , for span length of $= 65.62 \text{ ft}$ (20 m)

The support shear as calculated $= 3738.93 \text{ KN}$ and for impact factor $= 26.58\%$

$$V_{LL+I} = 4732.56 \text{ KN}$$

Therefore by similarity of triangle, $V_{\text{supp}}/0.5*L = V_{\text{cr}}/(0.5*L - 0.5*h)$

$$V_{\text{cr}} = 4448.61 \text{ KN}$$

Shear force in the section due to dead loads

Total Dead Load $= 28.96 \text{ KN/m}$

$$V_{\text{supp}} = wL/2$$

$$V_{\text{supp}} = 289.60 \text{ KN}$$

By similarity of triangle,

$$V_{\text{supp}}/0.5L = V_{\text{cr}}/(0.5L - 0.5h), h \text{ is depth of girder}$$

$$V_{\text{cr}} = 272.22 \text{KN}$$

$$\text{Therefore, } V_D = 1.4(DL + 5/3(LL + I))$$

$$V_D = 10761.20 \text{KN}$$

$$V_{\text{supp}} = (1.3V_D + 1.6(LL + I))$$

$$V_{\text{supp}} = 6357.77 \text{KN}$$

$$M_u = 1.4(M_g + M_{\text{sdl}}) + 2.33M_{LL+I}$$

$$M_u = 57561.34 \text{KNm}$$

$$V_c = (f_c^{1/2}/20 + 5V_u d_p / M_u) b_w d = 723.00 \text{ KN/m} < 0.4 f_c^{1/2} b_w d = 3716.55 \text{KN/m}$$

$$V_s = V_u / \phi - V_c$$

Substituting the above values of V_u , V_c and $\phi = 0.90$

$$V_s = 6342.30 \text{KN/m}$$

$$\text{Spacing of stirrups, } S = A_v f_y d_v / V_s = 69.74 \text{mm} < S_{\text{max}} = 600 \text{mm}$$

Use a Stirrup spacing of 70mm

4.1.7. Check for flexural strength requirements

Concrete properties

$$f_c = 50 \text{MPa}$$

Cross sectional properties

$$D, \text{total depth} = 1.20 \text{m}$$

$$b, \text{width of compression component} = 9.2 \text{m}$$

$$y_b, \text{distance from base to centroid of the section} = 0.63 \text{m}$$

$$I_g, \text{Moment of inertia about horizontal axis of the section} = 1.81 \text{m}^4$$

$$M_u = M_{\text{SLS}} = 57561.34 \text{KNm}$$

$$d_p = \text{distance from extreme compression fiber to centroid of pre-stressing steel} = 0.59 \text{m}$$

$$f_{py} = 1860 \text{MPa}$$

Number of strands=194

$$A_{ps}=0,034m^2$$

For $(A_{ps}f_{su})/(0.85 f'cb) < t$, $\phi M_n = \phi [A_{ps}f_{su}d\{1-0.6(\rho f_{su}/ f'c)\}] = \phi [A_{ps}f_{su}(d - a/2)]$
(AREMA,CL.8.17.18.2).

$$C = (A_{ps} f_{su})/(0.85 f'cb)$$

$$C=0.16<0.2$$

Therefore the section is a rectangular section.

$$f_{su} = f'_s [1-(\gamma/\beta_1)(\rho f'_s/ f'c)]$$

$$f_{su}=1796.76MPa$$

$$a=c\beta_1=0.13$$

$$T = A_{ps}f_{su}=63219.15KN$$

$$Z=d-a/2=0.63m$$

$$\phi M_u=39600.54KNm$$

$$M_u=M_{SLS}=57561.34KNm$$

$$M_u=M_{SLS}=57561.34KNm > \phi M_u=39600.54KNm$$

The section is not safe!

Therefore, we need to provide additional strands for the moment= $M_{SLS}-\phi MU=17960.80KNm$

$$N_p=28673.01KN$$

$$A_p=15958.18KNm$$

Additional no of strands provided to be =88

Total no of strands for the section=194+88=282

4.2. Deck slab design

General

No of girder=1

C/c support=20m

No. of tracks= 2

Loading used is AREMA-Copper E-80 Live load configuration

Thickness of slab=0.20m

Thickness of ballast below bottom of tie = 0.25m

Effect of multiple track =2

Materials used

Concrete:C-50

$f_c' = 40\text{KN/m}$

$E_c = 0.043 \cdot \gamma_c^{1.5} \cdot (f_c')^{1/2} = 33.99\text{Mpa}$

$E_s = 210000\text{Mpa}$

Track rail, inside guard rail and fastening: Unit weight, $\gamma_t = 3\text{KN/m}$

From the LRFD design Methodology

Strength reduction factor

For flexure= 0.9

For shear = 0.8

$b = 0.85$

Load combination as per AREMA manual

Strength load design

Group 1 Loading= $1.4 \cdot (DL + 5/3(L+I))$

Service load design

Group 1 Loading= $DL + LL + IL$

Impact load factor=26.58%

Loadings for the Deck Slab

Dead Loads

Uniform = 17.21kN/m^2

Concentrated for edge beam & railing = 5.78kN/m

$W = 7.82\text{KN/m}$ and $P = 5.78\text{KN}$

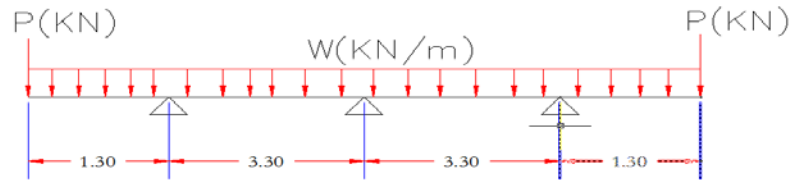


Figure 4. 4 Dead loads applied for 1m strip of 20m span one unit of double cell box girder railway bridge deck

From SAP 2000 Version 2015 analysis, the Maximum Positive and Negative Moment due to dead loads for 1m strip of 20m span double cell box girder railway bridge deck are:

Maximum Positive moment = 8.10 kNm/m
Maximum Negative moment = 22.12 kNm/m

Live loads from concrete sleepers (Ties) uniformly distributed over a strip width of $2 \times 3.3\text{m} = 6.6\text{m}$

$W = 72.73 \text{ kN/m}$

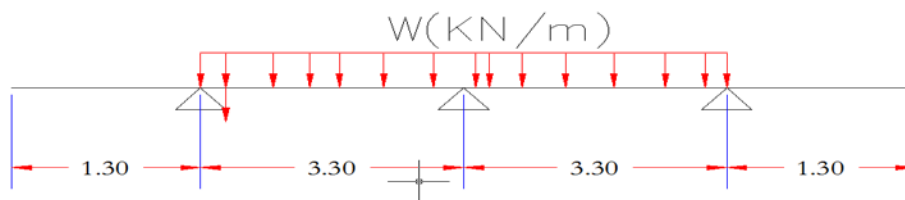


Figure 4. 5 Live load applied for 1m strip of 20m span one of double cell box girder railway bridge deck

From SAP 2000 Version 2015 analysis, the Maximum Positive and Negative Moment due to live loads for 1m strip of 20m span double cell box girder railway bridge deck are:

Maximum Positive moment = 60.88 kNm/m
Maximum Negative moment = 109.84 kNm/m

Summary of Maximum Moments

Due to Dead loads

$$MDL-VE=22.12\text{KNm/m}$$

$$MDL+VE=8.10\text{KNm/m}$$

Due to Live Load plus Impact

$$MLL-VE=53.70\text{KNm/m}$$

$$MLL+VE=24.60\text{KNm/m}$$

Factored Design Moments

$$M_{Dneg}=1.4*(DL+5/3*(L+I))=156.29\text{KNm/m}$$

$$M_{Dpost}=1.4*(DL+5/3*(L+I))=68.74\text{KNm/m}$$

Determination of top reinforcement

$$M_u=156.29\text{KNm/m}$$

Maximum reinforcement is limited by ductility requirement, which is given by

$$a/d < 0.42, a < 0.42\beta d$$

$$\beta = 0.85 \text{ for C-50 concrete}$$

Then, $a=0.357d$, $\Phi=0.9$ for flexure.....LRFD Art.5.7.2.2

$$M_u=0.85 \Phi f_c * a * b * (d-a/2) = 0.2244 f_c b d^2$$

$$b=1000.0 \text{ mm (i.e. taking 1m strip of the deck slab)}$$

$$d = M_u / (0.2244 f_c b) = 118.04 \text{ mm}$$

$$\text{Concrete cover} = 50 \text{ mm}$$

$A_s=314.16\text{mm}^2$ (i.e. cross sectional area of one 20mm deformed steel bar)

$d_{\text{provided}}=140\text{mm}$

Bar diameter=20mm

Depth is Adequate

Reinforcement

$f_y=335\text{Mpa}$

$a=A_s f_y / 0.85 f' c b$

$a=A_s / 76.99$

$a=13.14$

$\rho=M_u / (\Phi f_y b d (d-a/2))=0.028$ is greater than $\rho_{\min}=0.03 f' c / f_y=0.0045$

$A_s = \rho b d = 3885.10\text{mm}^2$

Spacing=80.86mm

Number of bars per meter width=13pcs

Use 13 bars

Provide Φ 20mm bars at c/c spacing of 77 mm as top reinforcement at interior parts of deck slab.

$A_{s\text{provided}}=4084.07\text{mm}^2 > 3885.10\text{mm}^2$

Determination of bottom reinforcement

$M_u=68.74\text{KNm/m}$

Maximum reinforcement is limited by ductility requirement, which is given by

$a/d \leq 0.42$, $a \leq 0.42 \beta d$ $\beta = 0.85$ for C-50 concrete

Then $a = 0.357d$ $\Phi=0.9$ for flexure.....LRFD Art.5.7.2.2

$$M_u = 0.85 \Phi f_c a b (d - a/2) = 0.2244 f_c b d^2$$

$$b = 1000.0 \text{ mm}$$

$$d = M_u / (0.2244 f_c b) = 78.28 \text{ mm}$$

$$\text{Concrete cover} = 25 \text{ mm}$$

$$a_s = 314.16 \text{ mm}^2$$

$$d_{\text{provided}} = 165 \text{ mm}$$

$$\text{Bar diameter} = 20 \text{ mm}$$

Depth is Adequate

$$f_y = 335 \text{ Mpa}$$

Reinforcement

$$a = A_s f_y / 0.85 f_c b \quad a = A_s / 76.99$$

$$a = 15.49$$

$$\rho = M_u / (\rho f_y b d (d - a/2)) = 0.009 \text{ greater than } \rho_{\text{min}} = 0.03 f_c / f_y = 0.002$$

$$A_s = \rho b d = 1449.70 \text{ mm}^2$$

$$\text{Spacing} = 216.70 \text{ mm}$$

$$\text{Number of bars per meter width} = 4.61 \text{ pcs}$$

Use 5 bars

Provide Φ 20mm bars c/c spacing of 200 mm as bottom reinforcement at interior parts of deck slab.

$$A_{s\text{provided}} = 1570.80 \text{ mm}^2 > 1449.70 \text{ mm}^2$$

Distribution Reinforcement

For Primary reinforcement Parallel to traffic,

$$S = 3.3 \text{ m} - 0.3 \text{ m (web width)} = 3.00 \text{ m}$$

Percentage of Distribution reinforcement= $3840 / (S)^{1/2} \leq 67\% = 70.11\%$

Take percentage of Reinforcement =67%

$A_s = 1052.43 \text{ mm}^2$

Diameter (mm)=12mm

$A_s = 113.1 \text{ mm}^2$

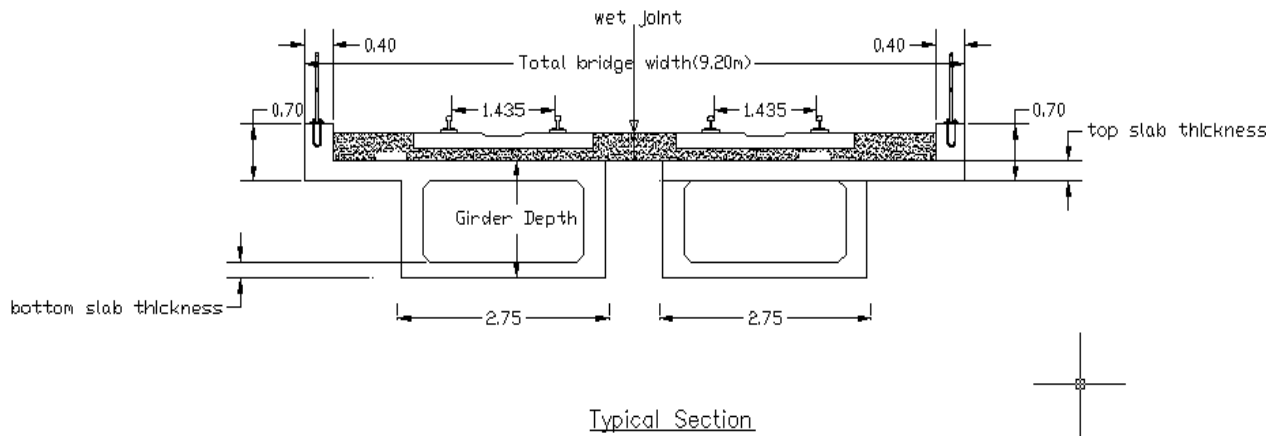
Spacing=107.50mm

Use spacing=76mm

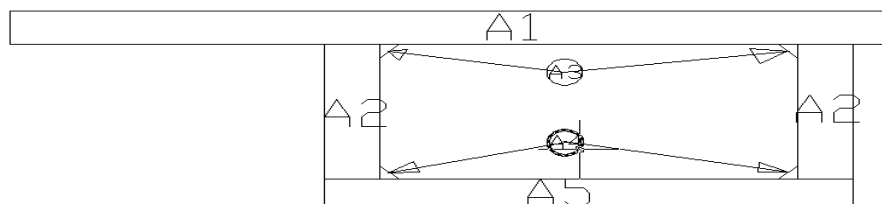
Provide Ø 20mm bars at c/c spacing of 107 mm at top and bottom slab in the longitudinal direction.

5. Gravity load analysis and design of one unit of single cell box girder Railway Bridge with span of 20m using AREMA and LRFD bridge design manual

Preliminary Dimensions



(a)



(b)

Figure 5. 1 (a) preliminary dimensions and regions for two units of single cell box girder Railway Bridges (typical section) ,(b)regions used for the calculations of moment of inertia for one unit of single cell box girder

Grade of Concrete=C- 50

Grade for mild steel= S- 335

Grade for tendons, $f_{pu}=1860$ Mpa

Unit weight for normal concrete= 24KN/m^3

Unit weight for reinforced concrete, $\gamma_{rc}= 25\text{KN/m}^3$

Unit weight for sand, gravel ballast= 19KN/m^3

Unit Weight of Ballast including Ties = 19KN/m^3

Thickness of the ballast= 0.35m

Length of Tie = 2.74m

Width of Tie= 0.33m

Depth of Tie = 0.25m

Span length (L) = 20m

Total width of the box girder beam= 4.6m

Girder depth from AREMA MRE 2010 Volume-2, Chapter 26.17.4

$$1/15 > d_o/L > 1/3 ,$$

Where,

d_o = girder depth, feet

L = span length between supports, feet

Girder web thickness= 0.3m

Top Slab Thickness (t)= $a/30$ to 150mm = 0.11m to 150m (Section 17.11 of AREMA manual)

Where,

$$a= \text{C/C of Girder Spacing}=2.95\text{m}$$

Therefore, take $t=0.15\text{m}$

Width and height of Fillets = 0.1m

Bottom slab thickness = 0.15m (i.e. minimum is 140mm from Section 17.11 of AREMA manual)

Overhang Distance (c) = 1.3m

Total width of the deck slab, $b_s=4.6\text{m}$

Clear spacing between end box cells and exterior vertical edge of the beam = 30mm

Rail gauge dist, $B_g=1.5\text{m}$

Live load configuration -Cooper E-80

Thickness of web, $b_w= 300\text{mm}$

Girder Depth= 1.25m

5.1. Design for tendons area and pre-stressing force (Jacking Force)

5.1.1. Material properties for the precast concrete box girder

$$f_{ck} = 0.8 f_{cu} = 0.8 * 50 = 40 \text{Mpa}$$

$$f_{ctd} = 0.21 f_{ctk}^{2/3} / 1.5 = 1.64 \text{Mpa}$$

$$f_{cd} = 0.85 f_{ck} / 1.5 = 22.67 \text{MPa}$$

$$f_{ct} = 0.6 * f_{ck} = 0.6 * 40 = 24 \text{Mpa}$$

$$f_{tr} = f_{cw} - h f_{tt}, h = 0.82 \text{ for post-tensioned member}$$

$$f_{cw} = 0.5 f_{ck} = 0.5 * 40 = 20 \text{Mpa}$$

$$f_{tt} \leq 0.21 (f_{ckt})^{2/3}$$

$$f_{tt} \leq 0.21 (40)^{2/3} = 2.46 \text{Mpa}$$

$$f_{tt} = 2.46 \text{mpa}$$

$$f_{tr} = 20 \text{mpa} - 0.82 * 2.46 \text{mpa} = 17.99 \text{Mpa}$$

$$f_{br} = h f_{ct} - f_{tw}$$

$$f_{tw} = 0 \text{ since full prestressing}$$

$$f_{tw} = 0.75 f_{ck} \text{ for partial prestressing}$$

$$f_{br} = h f_{ct} = 0.82 * 24 = 19.68 \text{Mpa}$$

$$f_{br} = 19.68 \text{mpa}$$

Permissible stresses in concrete

$$f'_c = 50 \text{MPa}$$

$$f_{ci} = 0.25 f'_c = 12.5 \text{MPa}$$

$$f_{ct} = 0.00 \text{MPa}$$

At transfer (before time dependent prestress losses)

$$\text{Compression} = 0.25 * f_{ci} = 6.25 \text{MPa}$$

$$\text{Tension} = 0.55 f_{ci} = 30.625 \text{MPa}$$

At service loads (after allowances for all prestress losses)

$$\text{Compression} = 0.4 f_{ci} = 20.00 \text{MPa}$$

$$\text{Tension in pre-compressed tensile zone} = 0.00 \text{MPa}$$

5.1.2. Loadings for the pre-cast concrete box girder

Superimposed dead load

a) Dead load due Edge beams

$$h1=0.55\text{m}$$

$$b1=0.40\text{m}$$

$$A1=b1*h1=0.22\text{m}^2$$

$$DL1=A1*\gamma_{rc}=5.28\text{KN/m}$$

$$DL1=5.28\text{KN/m}$$

b) Superimposed loads from ballast including track ties

$$DL2=\text{Unit Weight of Ballast including Ties}*(\text{Thickness of the ballast} + \text{Depth of Tie})=19\text{KN/m}^3 * (0.35+0.25)=10.45\text{KN/m}^2$$

c) Dead loads from Railing

$$DL3=0.5\text{KN/m (i.e. from AREMA Manual)}$$

d) Self weight of track rail and fastening

$$\text{From AREMA Manual, for a single track}=3\text{KN/m}$$

Since we have a single track on a single beam, a load of 3KN/m is taken. $DL4=3\text{KN/m}$

$$DL_{tot}=DL1 +DL2+DL3+DL4=20.18\text{KN/m}$$

Superimposed live load due to the axle load

From the AREMA PCI bridge design manual, chapter 17, which is obtained from AREMA Table 1.17, load can be obtained in respective to the span length.

From table 4.1, for the above live load configuration and span Length of $L =20\text{m}=65.62\text{ft}$

Interpolating between $L1= 60\text{ft}$ and $L2=70\text{ft}$, the following result is obtained

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

Span,ft	Maximum bending moment ,ft-kips	Maximum bending moment at quarter point, ft-kips
60.00	2597.80	2010.00
65.62	3056.80	2346.00
70.00	3415.00	2608.20

Span,ft	Maximum shear forces ,kips			Maximum pier reaction, kips
	end	quarter point	mid span	
60.00	196.00	120.21	55.69	306.42
65.62	210.06	126.77	58.93	333.19
70.00	221.04	131.89	61.45	354.08

Table 5.1.interpolation between span (ft) and values in TABLE 17.3.2.1.1 of AREMA manual, chapter 17 for one unit of double cell box girder railway bridge

$M_{LL} = 8290.05 \text{ KN-m}$

For pre-stressed concrete, the impact load is a percentage of the live load based on span length in ft.

$L \leq 60 \text{ft}, I = 35 - L^2 / 500 = 26.39\%$

$60 < L \leq 135 \text{ ft}, I = 14 + 800 / (L - 2) = 26.58\%$

$L > 135 \text{ ft}, I = 20\% \dots\dots\dots \text{AREMA Eq.17-1,}$

Where L=Span length of member in ft

Therefore for a span length= 65.62ft

Impact load factor=26.58%

26.58% of impact load will be added $M_{LL+I} = 8290.05 \text{ KNm} + 0.2658 * 8290.5 \text{ KNm} = 10493.16 \text{ KNm}$

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

No.	b	h	ref	A	y	yi	Ayi	I	di	Adi ²	I+Adi ²
A2	4.60	0.15	1.10	0.69	0.08	1.18	0.81	0.001	0.639	0.282	0.283
2*A3	0.30	0.95	0.15	0.57	0.48	0.63	0.36	0.043	0.239	0.033	0.075
2*A4	0.15	0.15	0.95	0.02	0.10	1.05	0.02	0.000	0.614	0.008	0.009
2*A5	0.15	0.15	0.15	0.02	0.05	0.20	0.00	0.000	0.664	0.010	0.010
A6	2.75	0.15	0.00	0.41	0.08	0.08	0.03	0.001	0.639	0.168	0.169
Σ				1.72			1.23	0.045		0.501	0.546

Table 5.2 Moment of Inertia calculations for a 20m single cell box girder beam (dimensions are in metric units)

Yb=	0.71	Sb=	0.76
Yt=	0.54	St=	1.02

Table 5.3. Section modulus calculation for a 20m one unit of single cell box girder beam (dimensions are in metric units)

$$M_{SDL} = SDL * L^2 / 8 = 1009 \text{KNm}$$

$$Q_{g1} = A * w_c = 41.22 \text{KN/m}$$

$$M_{g1} = Q_{g1} * L^2 / 8 = 2061.00 \text{KNm}$$

5.1.3. The pre-stressing force and calculation of number of strands

Concrete cover=40.00mm

Try eccentricity of strands at mid span=ec=yb-concrete cover=0.67m

$$f_{it} \leq 0.21(f_{ckt})^{2/3}$$

$$f_{it} \leq 0.21(0.8 * f_c)^{2/3} = 2.46 \text{MPa}$$

$$F_{sup} = f_{it} * M_{g1} / z_t = 0.43 \text{MPa}$$

$$f_{inf} \geq [f_{tw} / h + (M_q + M_{SDL} + M_{LL+I})] / h * z_b = 21.63 \text{MPa}$$

$$f_{tw} = 0 \text{ and } h = 0.82$$

$$e_{max} = [z_t * z_b (f_{inf} - f_{sup})] / [A (f_{sup} * z_t + f_{inf} * z_b)]$$

$$e_{max} = 0.54 \text{m}$$

Therefore eccentricity of strands=0.54m

Bottom tensile stress due to applied loads= $f_b = ((M_g + M_{SDL} + M_{LL+I}) / S_b) = 17.74 \text{MPa}$

Since the allowable tensile stress in bottom fiber is zero, the required pre-compression is =17.74MPa

Bottom fiber stress due to pre-stress after all losses= $f_b = P_{se} / A + P_{se} * e_c / S_b$

Here all the values are known except P_{se} , P_{se} can be calculated in the equation

$$P_{se} = ((f_b A S_b) / (S_b + A e_c))$$

$$P_{se} = 12119.65 \text{KN}$$

An average of 22% loss in case of pretension and 18% in post-tension may be assumed with condition of over tensioning has been applied to overcome friction and anchorage losses.

Allowable tensile stress in pre-stressing tendons immediately after pre-stress transfer is the larger of $0.82f_{py}$ ($0.82 * 0.9f_{pu} = 0.738f_{pu}$) or $0.75f_{pu}$

$$0.75f_{pu} = 1395.00 \text{MPa}$$

Area of strands=21675.54MPa

Area for a 15.2mm diameter single strand=181.37mm²

No of 15.2mm strands=120

5.1.4. Check for concrete stresses

Stress at transfer at mid span

Concrete stress at the top fiber of the beam, f_t

$$f_t = P_{si} / A - P_{si} e_c / S_t + M_g / S_t$$

M_g is based on the overall length of span=20m

$$M_g = w_c A * L^2 / 8 = 2061.00 \text{KNm}$$

$$f_t = 2.56 \text{MPa} < 6.25 \text{MPa} > -27.50 \text{MPa}$$

Stress at transfer at mid span

Concrete stress at the bottom fiber of the beam, f_b

$$f_b = P_{si} / A + P_{si} e_c / S_b - M_g / S_b$$

M_g is based on the overall length of span=20m

$$M_g = w_c A L^2 / 8 = 0.99 \text{ KNm}$$

$$f_t = 11.32 \text{ MPa} < 20 \text{ MPa} > 0 \text{ MPa}$$

Stress at transfer at the end

Since the strands are straight and all the strands are bonded to for the full length of the beam, the concrete stress at the end are simply the stresses at the midspans without the stress due to dead load

Stress at service loads midspan

Concrete stress at the bottom fiber of the beam

$$f_t = P_{se} / A - P_{se} e_c / S_t + M_g + M_{SDL} + M_{LL+I} / S_t$$

M_g is based on the overall length of span = 20m

$$M_g = w_c A L^2 / 8 = 2061.00 \text{ KNm}$$

$$f_t = 10.90 \text{ MPa} < 20.00 \text{ MPa} > 0.00 \text{ MPa}$$

The pre-stress force is at its maximum value at the release and service loads don't affect stresses at the end of the beam.

Therefore stresses at the release will govern at the end of the beam; there is no need to check the stresses at the ends of the beam at service loads.

Flexural strength

Stress in strands at flexural strength

$$f_{ps} = f_{pu} (1 - 0.5 p_p f_{pu} / f'_c)$$

provided that f_{se} is greater than $0.5 f_{pu}$

$$f_{se} = 559.14 < 0.5 f_{pu} = 930 \text{ MPa}$$

5.1.5. Check for the prevailing deflection at various stages of the loadings

Based on the AREMA design manual, the deflection of simple or continuous spans shall be designed so that the deflection due to service load plus impact load should not exceed $L/640$ of the span length.

Deflection immediately after prestressing

$$\text{Total immediate Deflection} = a_p + a_q$$

Where $a_p = -P_i e / 8EI_c$

$$a_q = 5 * \sum DL * L^2 / EI_c$$

$$P_i = 14785.98 \text{KN}$$

$$E = 42689.39 \text{MPa}$$

$$e = 0.54 \text{m}$$

$$I_c = 1.81 \times 10^{11} \text{m}^4$$

$$L = 20 \text{m}$$

$$\sum DL = 41.22 \text{KN/m}$$

Therefore, $a_p = -18.42 \text{mm}$ (upward deflection)

Therefore, $a_q = 4.25 \text{mm}$ (downward deflection)

Hence total immediate deflection = $-14 \text{mm} \leq$ Maximum deflection allowed = -31.25mm

Hence OK!

Deflection under working loads after prestress losses and total long time deflection

$$a_f = a_{il}(1 + \Phi) - a_{ip} [(1 - \Delta p / P_i) + ((1 - \Delta p / P_i) * \Phi)]$$

where,

$$\Delta p = P_i - P \text{ and } P \text{ is permissible stress under working condition}$$

$$\Phi = 2$$

$$P = f_{pe} * A_s$$

$$f_{pe} = 0.82 f_{pi} < 0.6 f_{pi},$$

Assuming 22% prestress loss for such a post-tensioned box girder beam

$$f_{pi} = 0.75 * f_{pk} = 1395 \text{MPa}$$

$$\text{But, } f_{pe} = 0.6 f_{pk} = 1116 \text{MPa}$$

$$\text{Therefore, } f_{pe} = 1116 \text{MPa}$$

$$\text{take } f_{pe} = 1116 \text{MPa}$$

$$P = f_{pe} * A_s = 24189.90 \text{KN}$$

$$\text{Implies } \Delta p = P_i - P = 5321.78 \text{KN}$$

$$\text{Where } P_w = \sum DL + w(LL+I)$$

$$w(LL+I)=8*M(LL+I)/L^2 =209.86\text{KN/m}$$

$$\sum DL=41.22\text{KN/m}$$

$$P_w=\sum DL + w(LL+I)=251.08\text{KN/m}$$

$$a_{il}=5*P_w*L^4/384*E_cI=0.02\text{m}$$

$$a_f=a_{il}(1+\phi)-a_{ip}[(1-\Delta p/P_i)+((1-\Delta p/P_i)*\phi)]=16.02\text{mm(upward)}$$

Hence long time deflection= 16.02mm (upward) \leq Maximum deflection allowed =-31.25mm

Hence OK!

5.1.6. Shear Design

Non prestressed reinforcement properties

$$f_y=335\text{Mpa}$$

$$E_s=210000\text{MPa}$$

$$A_p=\text{Area of a two legged 12mm diameter stirrup}=0.009\text{m}^2$$

$$b=\text{width of compressive component}=4.60\text{m}$$

$$d_p=\text{distance from extreme compression fiber to centroid of prestressing steel}=0.59\text{m}$$

$$h=\text{height of the section}=1.25\text{m}$$

$$d_v=0.5+0.8h=1.50\text{m}$$

$$b_v=\text{available web width}=2*0.3\text{m}=0.6\text{m}$$

Shear force in the section due to live load +Impact

At $h/2$ distance from the face of end support (i.e. $1.20/2=0.6\text{m}$), critical shear is calculated. From table 4.1. , for span length of=65.62ft (20m)

The support shear as calculated= 3738.93KN and for impact factor=26.58%

$$V_{LL+I}=2366.28\text{KN}$$

Therefore by similarity of triangle, $V_{\text{supp}}/0.5*L=V_{\text{cr}}/(0.5*L-0.5*h)$

$$V_{cr}=2218.39\text{KN}$$

Shear force in the section due to dead loads

$$\text{Total Dead Load}=20.18\text{KN/m}$$

$$V_{\text{supp}}=wL/2$$

$$V_{\text{supp}}=201.80\text{KN}$$

By similarity of triangle,

$$V_{\text{supp}}/0.5L=V_{cr}/(0.5L-0.5h), h \text{ is depth of girder}$$

$$V_{cr}=189.19\text{KN}$$

$$\text{Therefore, } V_D=1.4*(DL+5/3*(LL+I))$$

$$V_D=5441.10\text{KN}$$

$$V_{\text{supp}}=(1.3*V_D+1.6*(LL+I))$$

$$V_{\text{supp}}=3253.48\text{KN}$$

$$M_u=1.4(M_g+M_{sd})+2.33M_{LL+I}$$

$$M_u=28747.07\text{KNm}$$

$$V_c=(f_c^{1/2}/20+5V_u d_p/M_u)b_w d=724.88\text{KN/m} < 0.4*f_c^{1/2}*b_w*d=3716.55\text{KN/m}$$

$$V_s=V_u/\phi - V_c$$

Substituting the above values of V_u , V_c and $\phi=0.90$

$$V_s=2880.10\text{KN/m}$$

$$\text{Spacing of stirrups, } S=A_v f_y d_v / V_s=157.78\text{mm} < S_{\text{max}}=600\text{mm}$$

Use a Stirrup spacing of 157mm

5.1.7. Check for flexural strength requirements

Concrete properties

$$f_c=50\text{MPa}$$

Cross sectional properties

$$D, \text{total depth}=1.25\text{m}$$

$$b, \text{width of compression component}=4.60\text{m}$$

$$y_b, \text{distance from base to centroid of the section}=0.71\text{m}$$

$$I_g, \text{Moment of inertia about horizontal axis of the section}=0.55\text{m}^4$$

$$M_u=M_{SLS}=28747.07\text{KNm}$$

d_p =distance from extreme compression fiber to centroid of pre-stressing steel=0.59m

f_{py} =1860MPa

Number of strands=120

A_{ps} =0,022m²

For $(A_{ps} f_{su}) / (0.85 f'c b) < t$, $\phi M_n = \phi [A_{ps} f_{su} d \{1 - 0.6(p f_{su} / f'c)\}] = \phi [A_{ps} f_{su} (d - a/2)]$ (AREMA, CL.8.17.18.2).

$C = (A_{ps} f_{su}) / (0.85 f'c b)$

$C = 0.2 > 0.15$

Therefore the section is a flanged section.

$\phi M_n = \phi \{ A_{sr} f_{su} d [1 - 0.6(A_{sr} f_{su} / b' d f'c)] + 0.85 f'c (b - b')(t)(d - 0.5t) \}$

Where $A_{sr} = A_{ps} - 0.85 (b - b')t / f_{su}$

$A_{sr} = 0.006m^2$

$\phi M_n = 38688.02KNm$

$\phi M_u = 39600.54KNm$

$M_u = M_{SLS} = 28747.07KNm$

$M_u = M_{SLS} = 28747.07KNm < \phi M_u = 39600.54KNm$

The section is safe!

5.2. Deck slab design

General

No of girder=1

C/c support=20m

No. of tracks= 2

Loading used is AREMA-Copper E-80 Live load configuration

Thickness of slab=0.15m

Thickness of ballast below bottom of tie = 0.25m

Effect of multiple track =2

Materials used

Concrete:C-50

$f'c = 40KN/m$

$E_c = 0.043 * \gamma_c^{1.5} * (f'c)^{1/2} = 33.99Mpa$

$E_s = 210000 \text{ Mpa}$

Track rail, inside guard rail and fastening: Unit weight, $\gamma_t = 3 \text{ KN/m}$

From the LRFD design Methodology

Strength reduction factor

For flexure = 0.9

For shear = 0.8

$b = 0.85$

Load combination as per AREMA manual

Strength load design

Group 1 Loading = $1.4 * (DL + 5/3(L+I))$

Service load design

Group 1 Loading = $DL + LL + IL$

Impact load factor = 26.58%

Loadings for the Deck Slab

Dead Loads

Uniform = 16.10 kN/m^2

Concentrated for edge beam & railing = 5.78 kN/m

$W = 16.10 \text{ kN/m}$ and $P = 5.78 \text{ kN}$

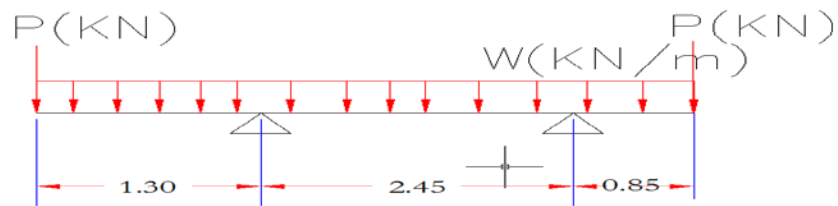


Figure 5. 2 Dead loads applied for 1m strip of 20m span single cell box girder railway bridge deck

From SAP 2000 Version 2015 analysis, the Maximum Positive and Negative Moment due to dead loads for 1m strip of 20m span one unit of double cell box girder railway bridge deck are:

Maximum Positive moment = 0.00 kNm/m

Maximum Negative moment = 21.12 kNm/m

Live loads from concrete sleepers (Ties) uniformly distributed over a strip width of $1*3.3\text{m}=3.3\text{m}$

$$W=72.73\text{KN/m}$$

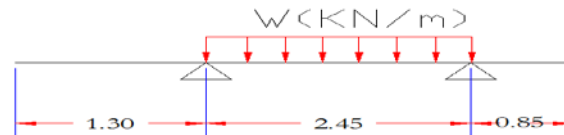


Figure 5. 3Live load applied for 1m strip of 20m span one unit of single cell box girder railway bridge deck

From SAP 2000 Version 2015 analysis, the Maximum Positive and Negative Moment due to live loads for 1m strip of 20m span double cell box girder railway bridge deck are:

$$\text{Maximum Positive moment} = 57.28\text{KNm/m}$$

$$\text{Maximum Negative moment} = 0.00\text{KNm/m}$$

Summary of Maximum Moments

Due to Dead loads

$$\text{MDL-VE} = 22.12\text{KNm/m}$$

$$\text{MDL+VE} = 0.00\text{KNm/m}$$

Due to Live Load plus Impact

$$\text{MLL-VE} = 0.00\text{KNm/m}$$

$$\text{MLL+VE} = 25.10\text{KNm/m}$$

Factored Design Moments

$$M_{\text{Dneg}} = 1.4 * (\text{DL} + 5/3 * (\text{L} + \text{I})) = 29.57\text{KNm/m}$$

$$M_{D_{post}} = 1.4 * (DL + 5/3 * (L+I)) = 58.56 \text{KNm/m}$$

Determination of top reinforcement

$$M_u = 29.57 \text{KNm/m}$$

Maximum reinforcement is limited by ductility requirement, which is given by

$$a/d < 0.42, a < 0.42\beta d$$

$$\beta = 0.85 \text{ for C-50 concrete}$$

Then, $a = 0.357d$, $\Phi = 0.9$ for flexure.....LRFD Art.5.7.2.2

$$M_u = 0.85 \Phi f'_c a b (d - a/2) = 0.2244 f'_c b d^2$$

$b = 1000.0 \text{ mm}$ (i.e. taking 1m strip of the deck slab)

$$d = M_u / (0.2244 f'_c b) = 51.34 \text{mm}$$

Concrete cover = 50mm

$A_s = 314.16 \text{mm}^2$ (i.e. cross sectional area of one 20mm deformed steel bar)

$$d_{provided} = 90 \text{mm}$$

Bar diameter = 20mm

Depth is Adequate

Reinforcement

$$f_y = 335 \text{Mpa}$$

$$a = A_s f_y / 0.85 f'_c b$$

$$a = A_s / 76.99$$

$$a = 13.14$$

$$\rho = M_u / (\Phi f_y b d (d - a/2)) = 0.013 \text{ is greater than } \rho_{min} = 0.03 f'_c / f_y = 0.0045$$

$$A_s = \rho b d = 1175.50 \text{mm}^2$$

$$\text{Spacing} = 237.26 \text{mm}$$

Number of bars per meter width = 3.74 pcs

Use 4 bars

Provide Φ 20mm bars at c/c spacing of 250 mm as top reinforcement at interior parts of deck slab.

$$A_{sprovided} = 1256.64 \text{mm}^2 > 1175.47 \text{mm}^2$$

Determination of bottom reinforcement

$$M_u = 58.56 \text{ kNm/m}$$

Maximum reinforcement is limited by ductility requirement, which is given by

$$a/d \leq 0.42, a \leq 0.42 \beta d \quad \beta = 0.85 \text{ for C-50 concrete}$$

Then $a = 0.357d$ $\Phi = 0.9$ for flexure.....LRFD Art.5.7.2.2

$$M_u = 0.85 \Phi f_c' a b (d - a/2) = 0.2244 f_c' b d^2$$

$$b = 1000.0 \text{ mm}$$

$$d = M_u / (0.2244 f_c' b) = 72.25 \text{ mm}$$

$$\text{Concrete cover} = 25 \text{ mm}$$

$$A_s = 314.16 \text{ mm}^2$$

$$d_{\text{provided}} = 115 \text{ mm}$$

$$\text{Bar diameter} = 20 \text{ mm}$$

Depth is Adequate

$$f_y = 335 \text{ Mpa}$$

Reinforcement

$$a = A_s f_y / 0.85 f_c' b \quad a = A_s / 76.99$$

$$a = 15.49$$

$$\rho = M_u / (\phi f_y b d (d - a/2)) = 0.016 \text{ lesser than } \rho_{\text{min}} = 0.03 f_c' / f_y = 0.002$$

$$A_s = \rho b d = 1810.80 \text{ mm}^2$$

$$\text{Spacing} = 173.50 \text{ mm}$$

$$\text{Number of bars per meter width} = 5.76 \text{ pcs}$$

Use 6 bars

Provide Φ 20mm bars c/c spacing of 160 mm as bottom reinforcement at interior parts of deck slab.

$$A_{s\text{provided}} = 1884.96 \text{ mm}^2 > 1810.80 \text{ mm}^2$$

Distribution Reinforcement

For Primary reinforcement Parallel to traffic,

$$S = 2.95\text{m} - 0.3\text{m (web width)} = 2.65\text{m}$$

$$\text{Percentage of Distribution reinforcement} = 3840 / (S)^{1/2} \leq 67\% = 74.59\%$$

Take percentage of Reinforcement = 67%

$$A_s = 1262.92\text{mm}^2$$

$$\text{Diameter (mm)} = 12\text{mm}$$

$$A_s = 113.1\text{mm}^2$$

$$\text{Spacing} = 89.60\text{mm}$$

Use spacing = 89mm

Provide Ø 20mm bars at c/c spacing of 89 mm at top and bottom slab in the longitudinal direction.

6. Design Outputs

The summary of necessary outputs obtained from the design program for a 20m, 25m.30m, 35m, 40m, 45m and 50m span two units of single cell box Girder Bridge and one unit of double cell box girder bridges are provided in Table 6.2.

6.1. Summary on Outputs

For cost analysis, recent values of construction material costs to the nearest hundred including overhead cost are used here

The unit price for concrete and pre-stressing tendons are \$250.00/m³ and \$8.5/kg respectively.[Dagim Tadesse (2015). An assessment of Pre-stressing in post-tensioned Concrete Box Girder Railway Elevated Structures: a case study on Comparative Analysis of Continuous Vs Simply Supported Elevated Structure of Addis Ababa's Light Rail Transit:Addis Ababa University School of Graduate Studies Institute of Technology Department of Civil Engineering].

Unit cost for hoisting pre-cast segments is taken from the Addis Ababa Light Railway Transit System project.

Material	Unit	Unit prices(in ETB)
Concrete (f ['] c=50MPa)	m ³	6200.00
Longitudinal pre-stressing tendons	Kg	420.00
Machine cost for hoisting precast segments	m ³	100400.00
12mm diameter non-pre-stressed reinforcement bar	kg/m	24.00
20mm diameter non-pre-stressed reinforcement bar	kg/m	65.00
Formwork	m ²	300

Table 6.1.material unit prices

6.2. Cost Analysis

The materials volume obtained from the output of the program and current material prices are used for the cost analysis in order to identify the economical span of the two types of bridges (i.e. Two units of Single cells and one unit of Double cell box girder bridges).

All unit prices include installation costs (i.e. concrete placement and vibration, grouting of tendons). For cost analysis, recent values of construction costs including overhead cost are used here.

No	Span(m)	Total Cost(ETB)	
		Total cost (ETB) for two units of single cell box girder beam	Total cost (ETB) for one unit of double cell box girder beam
1	20	542,650.38	576,106.58
2	25	563,682.61	579,072.42
3	30	621,613.30	674,412.13
4	35	687,963.15	670,216.70
5	40	760,801.32	676,343.32
6	45	832,748.10	697,461.06
7	50	959,328.20	719,913.79

Table 6.2. Summary of construction materials cost of per 1m span length for two units of single cells and one unit of double cell pre-stressed concrete box girder railway bridges.

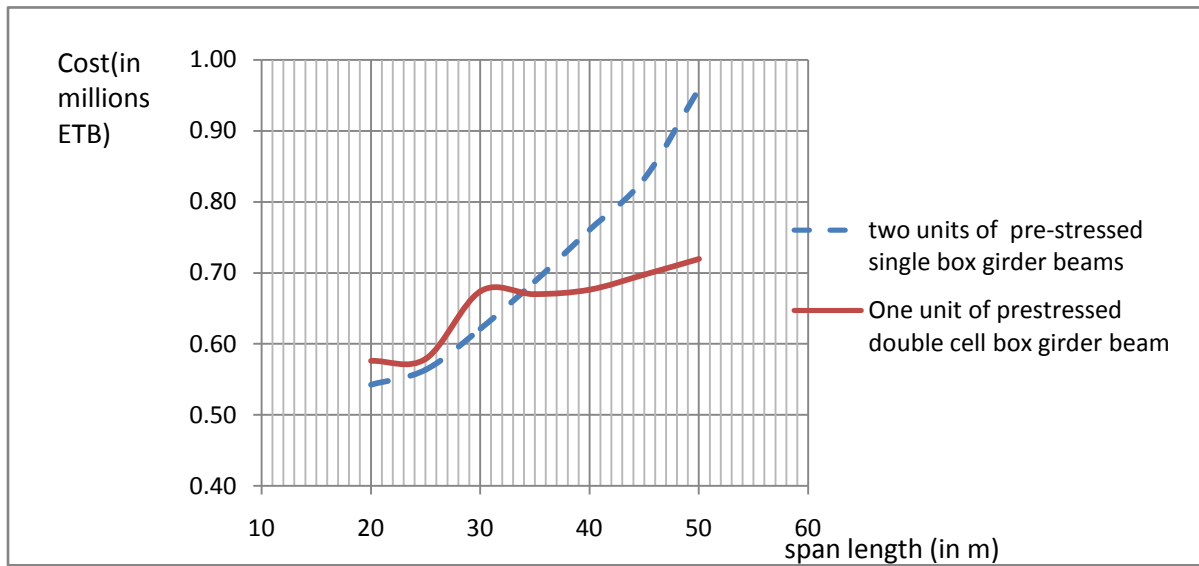


Figure 6.1 Material cost versus span length relationship for two units of single cell box girder railway bridges and a double cell box girder railway bridge (vertical axis: cost (ETB)/m and horizontal axis: span length in m unit).

7. Conclusion and Recommendations

7.1. Conclusion

Basically this research is attempted to find out the demarcation span and to solve the problem of selecting between constructing and designing between two units of pre-stressed concrete single cell box girder railway and one unit of double cell one. Finally the results from the study are discussed as:

1. The demarcation span from the graph on fig. 6.2.above is for a span of 34m.This gives an implication that for a span of 34m, one can choose either one unit of double cell pre-stressed concrete box girder Railway Bridge or an economical pre-stressed concrete two units of single cell box girder railway bridges.

2.From the same graph ,for the span less than 34m ,the graph for the pre-stressed concrete two units of single cell box girder railway bridges lies below the graph plotted for the one of pre-stressed concrete double cell box girder railway bridge. This implies the former one is more economical from the material cost point of view than the later design. Therefore, it is recommended that the designer should choose two units of single cells box girders than one unit of double cell box girder for spans less than 34m and vice versa for spans greater than 34m.

7.2. Recommendations for future works

This paper work has attempted to find the demarcation span and to solve the problem of selecting between constructing and designing between two units of pre-stressed concrete single cell box girder railway and one unit of double cell one. However, due to financial constraints and time limitations the present research work did not cover detail information on pre-stressed concrete box girder railway bridges.

In view of this work, it would be desirable to consider the following recommendations for the future work for the development of complete information on the topic.

1. Though pre-tensioning pre-stressing system is mostly applied for small structures and short spans, further study need to be done to apply such system of pre-stressing on long span and huge structures too.
2. Only gravity loads were analyzed as service loads making other loads (i.e. wind loads, earthquake loads, stream flow pressure, longitudinal force, buoyancy, earth pressure) as controlled variables. So it is strongly recommended to analyze the super structure taking into account these loads under various design data.
3. Although the costs resulting from the construction methods (i.e. machinery and equipment costs) is not the objectives of this research scope, further studies including such costs are recommended.
4. At last but not the least, since only super structure design is made in this study, the sub structure design (i.e. pier design, pile cap design and subsurface investigation design) is strongly recommended to provide more confidence for the designer to choose between the before mentioned types of pre-stressed concrete box girder railway bridge design methods.
5. Only typical bridge section is designed due to time limitations.

8. Annex

8.1.Total cost of concrete, post-tensioning tendons and reinforcement bars per 1m length of two units of single cell box girders with span lengths of 20m,25m,30m,35m,40m,45m and 50m

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	1.92	6200.00	11,888.50	23,777.00
Machine cost for hoisting precast segments	m ³	1.92	100400.00	192,517.00	385,034.00
prestressing strands	kg/m	137	420.00	57,746.77	115,493.54
Web Reinforcements 12mm(4 leged stirrups)	kg/m	63.21	24	1,516.95	3,033.90
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m ²	9.02	300	2,705.25	5,410.50
			TOTAL(ETB)	272,258.61	544,517.21

Table 8.1. Cost of materials for two units of single cell box girder railway bridges (span=20m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	1.89	6200.00	11,733.50	23,467.00
Machine cost for hoisting precast segments	m3	1.89	100400.00	190,007.00	380,014.00
prestressing strands	kg/m	168	420.00	70,723.63	141,447.26
Web Reinforcements 12mm(4 legged stirrups)	kg/m	79.31	24	1,903.50	3,807.00
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m2	8.99	300	2,697.75	5,395.50
			TOTAL(ETB)	282,949.52	565,899.04

Table 8.2. Cost of materials for two units of single cell box girder railway bridges (span=25m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	2.04	6200.00	12,663.50	25,327.00
Machine cost for hoisting precast segments	m3	2.04	100400.00	205,067.00	410,134.00
prestressing strands	kg/m	199	420.00	83,379.79	166,759.57
Web Reinforcements 12mm(4 leged stirrups)	kg/m	92.15	24	2,211.55	4,423.09
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m2	9.64	300	2,892.75	5,785.50
			TOTAL(ETB)	312,098.72	624,197.44

Table 8.3. Cost of materials for two units of single cell box girder railway bridges (span=30m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	2.22	6200.00	13,779.50	27,559.00
Machine cost for hoisting precast segments	m3	2.22	100400.00	223,139.00	446,278.00
prestressing strands	kg/m	230	420.00	96,607.55	193,215.10
Web Reinforcements 12mm(4 leged stirrups)	kg/m	122.11	24	2,930.64	5,861.28
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m2	10.42	300	3,126.75	6,253.50
			TOTAL(ETB)	345,467.58	690,935.15

Table 8.4. Cost of materials for two units of single cell box girder railway bridges (span=35m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	2.46	6200.00	15,267.50	30,535.00
Machine cost for hoisting precast segments	m3	2.46	100400.00	247,235.00	494,470.00
prestressing strands	kg/m	254	420.00	106,649.85	213,299.70
Web Reinforcements 12mm(4 legged stirrups)	kg/m	149.48	24	3,587.58	7,175.17
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m2	11.39	300	3,417.75	6,835.50
			TOTAL(ETB)	382,041.82	764,083.64

Table 8.5. Cost of materials for two units of single cell box girder railway bridges (span=40m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	2.67	6200.00	16,569.50	33,139.00
Machine cost for hoisting precast segments	m3	2.67	100400.00	268,319.00	536,638.00
prestressing strands	kg/m	285	420.00	119,713.12	239,426.23
Web Reinforcements 12mm(4 leged stirrups)	kg/m	167.15	24	4,011.70	8,023.40
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m2	12.37	300	3,711.75	7,423.50
			TOTAL(ETB)	418,209.20	836,418.40

Table 8.5. Cost of materials for two units of single cell box girder railway bridges (span=45m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB) for a single cell beam	TOTAL (ETB) for two single cell beam
concrete volume	m ³	2.97	6200.00	18,429.50	36,859.00
Machine cost for hoisting precast segments	m ³	2.97	100400.00	298,439.00	596,878.00
prestressing strands	kg/m	358	420.00	150,519.90	301,039.81
Web Reinforcements 12mm(4 leged stirrups)	kg/m	190.31	24	4,567.48	9,134.97
Top slab Reinforcement bar 20mm	kg/m	34	65	2,188.68	4,377.36
Bottom slab Reinforcement bar 20mm	kg/m	45	65	2,918.24	5,836.48
Distribution Reinforcement bar 12mm	kg/m	32.384	24	777.22	1,554.43
Formwork	m ²	13.67	300	4,101.75	8,203.50
			TOTAL(ETB)	481,941.77	963,883.55

Table 8.7 Cost of materials for two units of single cell box girder railway bridges (span=50m)

8.2.Total cost of concrete, post-tensioning tendons and reinforcement bars per 1m length of one unit of double cell box girder beams with span lengths of 20m,25m,30m,35m,40m,45m and 50m

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	282	420.00	118,286.51
Web Reinforcements 12mm(4 leged stirrups)	kg/m	241.32	24	5,791.56
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	15.55	300	4,665.00
			TOTAL(ETB)	578,020.27

Table 8.8.Cost of materials for one unit of double cell box girder railway bridges (span=20m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	285.76	420.00	120,018.24
Web Reinforcements 12mm(4 leged stirrups)	kg/m	294	24	7,049.25
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	15.55	300	4,665.00
			TOTAL(ETB)	581,009.69

Table 8. 9 Cost of materials for one unit of double cell box girder railway bridges (span=25m)

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	483.50	420.00	203,068.65
Web Reinforcements 12mm(4 leged stirrups)	kg/m	241.32	24	5,791.56
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	17.15	300	5,145.00
			TOTAL(ETB)	663,282.41

Table 8.10. Cost of materials for one unit of double cell box girder railway bridges (span=30m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	508	420.00	213,175.66
Web Reinforcements 12mm(4 leged stirrups)	kg/m	228.95	24	5,494.70
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	18.25	300	5,475.00
			TOTAL(ETB)	673,422.57

Table 8.11. Cost of materials for one unit of double cell box girder railway bridges (span=35m)

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	522	420.00	219,291.59
Web Reinforcements 12mm(4 leged stirrups)	kg/m	217.86	24	5,228.68
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	19.45	300	5,835.00
			TOTAL(ETB)	679,632.47

Table 8.12. Cost of materials for one unit of double cell box girder railway bridge (span=40m)

Economic Analysis and Design for the Selection of Two Units of Single Cell and One Unit of Double Cell Pre-stressed Box girder Railway Bridges

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	572	420.00	240,423.21
Web Reinforcements 12mm(4 leged stirrups)	kg/m	220.52	24	5,292.56
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	20.15	300	6,045.00
			TOTAL(ETB)	701,037.97

Table 8.13.Cost of materials for one unit of double cell box girder railway bridge (span=45m)

ITEM	UNIT	QUANTITY	UNIT PRICE (ETB)	TOTAL (ETB)
concrete volume	m ³	3.95	6200.00	24,490.00
Machine cost for hoisting precast segments	m ³	3.95	100400.00	396,580.00
prestressing strands	kg/m	626	420.00	263,040.73
Web Reinforcements 12mm(4 leged stirrups)	kg/m	212.74	24	5,105.77
Top slab Reinforcement bar 20mm	kg/m	292	65	18,968.56
Bottom slab Reinforcement bar 20mm	kg/m	112	65	7,295.60
Distribution Reinforcement bar 12mm	kg/m	80.96	24	1,943.04
Formwork	m ²	21.25	300	6,375.00
			TOTAL(ETB)	723,798.70

Table 8.14.Cost of materials for one unit of double cell box girder railway bridge (span=50m)

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