ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

COMPUTER PROGRAM FOR COMPARATIVE STUDY OF THE ANALYSIS AND DESIGN OF REINFORCED CONCRETE T-GIRDER AND BOX-GIRDER BRIDGES

A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

By

Abiy Alemu

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Declaration

I certify that this research work entitled “Computer Program for Comparative Study of the Analysis and Design of Reinforced Concrete T-Girder and Box-Girder Bridges” is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been duly acknowledged.

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A_{di}	=	area of longitudinal reinforcing bars provided
A_{dn}	=	depth of concrete under compression, negative moment
A_{dp}	=	depth of concrete under compression, positive moment
A_{ig} (nb)	=	compression depth of concrete in the slab at the n^{th} bar iteration
A_{pn}	=	area of negative moment reinforcing bars provided
A_{pp}	=	area of positive moment reinforcing bars provided
A_{p3}	=	area of overhang moment reinforcing bars provided
A_{sbl}	=	area of bottom slab reinforcement in the longitudinal direction
A_{sbt}	=	area of bottom slab reinforcement in the transversal direction
A_{sc} (nb)	=	area of steel required to balance the longitudinal compression in the web
A_{sfi}	=	area of steel required to balance the longitudinal compression in the overhang
A_{si} (nb)	=	area of reinforcement at the n^{th} bar iteration
A_{ski}	=	area of skin reinforcing bars
A_{sp3}	=	area of provided reinforcement in the longitudinal direction, bottom slab
A_{sp4}	=	area of provided reinforcement in the transversal direction, bottom slab
A_{st}	=	area of temperature reinforcing bars
A_{s1}	=	area of positive moment reinforcing bars
A_{s2}	=	area of negative moment reinforcing bars
A_{s3}	=	area of overhang moment reinforcing bars
A_{xg} (nb)	=	compression depth of concrete in the web at the n^{th} bar iteration
B_{cs}	=	clear distance between parallel bars in a layer
b_{eii}	=	effective width of exterior girder
b_{ii}	=	effective width of interior girder
b_{min}	=	the minimum web width
b_t	=	bitumen thickness
C_{cbs}	=	center to center distance between parallel bars in a layer
C_d	=	additional curb depth
C_s	=	clear span of the bridge
C_w	=	additional curb width
db	=	diameter of girder main reinforcing bars
dd1	=	diameter of distribution reinforcing bars
d_{efl}	=	deflection due to lane load

d_{eftl} = maximum deflection from 25% of truck load plus lane load

D_{lc} = dead load from curb

D_{ld} = maximum dead load deflection

D_{ld1} = dead load deflection due to girder and deck slab

D_{ld2} = deflection due to diaphragm dead load

D_{lge} = the total dead load on exterior girder

D_{lgi} = the total dead load on interior girder

d_{lmax} = maximum live load deflection of the bridge

D_{loh} = dead load from overhang slab

D_{ls} = dead load from deck slab

D_{lw} = dead load from web

D_{lwe} = dead load from future wearing surface, exterior girder

D_{lwi} = dead load from future wearing surface, interior girder

d_{max} = maximum allowable live load deflection of the bridge

d_n = effective depth for negative reinforcement

d_{od} = effective depth for overhang main reinforcement

d_p = effective depth for positive reinforcement

d_{ppr} (nb) = effective depth of interior girder at the n^{th} bar iteration

D_{rp} = dead load from posts and railing

dsk = diameter of skin reinforcing bars

d_{v1} = effective shear depth

d_{xi} = deflection due to each truck load

d1t = diameter of temperature reinforcing bars

E_c = modulus of elasticity of concrete

E_i = equivalent interior transverse strip width for negative live load moment

E_{oh} = equivalent interior transverse strip width for overhang negative live load moment

E_s = modulus of elasticity of steel

E_w = equivalent interior transverse strip width for positive live load moment

F_c = cylindrical compressive strength of concrete

F_{fr} = limit for fatigue stress range

F_r = modulus of rupture

F_s = tensile stress in the bottom reinforcement at the service limit state

F_{se} = tensile stress in the top reinforcement at the service limit state

F_{s1} = tensile stress in main reinforcing bars at the service limit state

F_{smax} = maximum fatigue stress due to total load

F_{smin} = minimum fatigue stress due to dead load

F_{sran} = fatigue stress range

F_y = yield strength of reinforcing bars

G_s = girder spacing

I_{cr} = moment of inertia of transformed cracked section, deck slab

I_{cr1} = moment of inertia of transformed cracked section, girder

I_{eff} = effective moment of inertia

I_{gt} = gross moment of inertia of about the centroid

ILOa, ILOb = influence line ordinates for support reaction

ILOc, ILOd = influence line ordinates for span moment

ILOe, ILOf = influence line ordinates for support moment

ILOg = influence line ordinate for girder reaction

L_1 = center-to-center span length of a bridge

L_{1a} = position of the wheel load

m = multiple presence factor

M_{ad} = maximum moment in the component at stage for which deformation is computed

M_{cr} = cracking moment

M_{dl} (L_{1a}) = moment due to dead load at L_{1a} distance from support for interior girder.

M_{dle} (L_{1a}) = moment due to dead load at L_{1a} distance from support for exterior girder.

Mge = distribution factor for moment of exterior beams

Mgi = distribution factor for moment of interior beams

Mgve = distribution factor for shear of exterior beams

Mgvi = distribution factor for shear of interior beams

M_{ni} (nb) = section capacity of the girder for flexure

M_{nn} = section capacity for negative moment

M_{np} = section capacity for positive moment

M_{tml} (L_{1a}) = bending moment at a distance L_{1a} due to tandem and lane load

M_{trf1} (L_{1a}) = bending moment for fatigue stress computation at a distance L_{1a} due to truck load moving to the right

M_{trf2} (L_{1a}) = bending moment for fatigue stress computation at a distance L_{1a} due to truck load moving to the left

M_{trl1} (L_{1a}) = bending moment at a distance L_{1a} due to truck moving to the right and lane load

M_{trl2} (L_{1a}) = bending moment at a distance L_{1a} due to truck moving to the left and lane load

M_{un} = factored design negative moment

M_{up} = factored design positive moment

m_1, m_2, m_3 = influence line coefficeints for shear force due to truck loads

m_1', m_2', m_3' = influence line coefficeints for moment due to truck loads

M_{200} = the maximum overhang moment at the face of support

M_{200b} = support moment at B due to own weight of barrier

M_{200o} = support moment at B due to own weight of overhang

M_{200ol} = overhang negative live load moment at support B

M_{200w} = support moment at B due to own weight of wearing surface

M_{204} = the maximum span/ positive moment

M_{204b} = span moment on span BC at a distance 0.4*Gs from support B due to own weight of barrier

M_{204d} = span moment on span BC at a distance 0.4*Gs from support B due to own weight of deck slab

M_{204o} = span moment on span BC at a distance 0.4*Gs from support B due to own weight of overhang

M_{204pl} = maximum positive live load moment

M_{204s} = maximum positive moment for service limit state investigation

M_{204w} = span moment on span BC at a distance 0.4*Gs from support B due to own weight of wearing surface

M_{300} = the maximum support/ negative moment at the face of support

M_{300b} = support moment at C due to own weight of barrier

M_{300d} = support moment at C due to own weight of deck slab

M_{300nl} = maximum interior negative live load moment

M_{300o} = support moment at C due to own weight of overhang

M_{300s} = maximum negative moment for service limit state investigation

M_{300w} = support moment at C due to own weight of wearing surface

n = modular steel ratio

N_g = total number of girders

N_L = number of design lanes

n_1, n_2 = influence line coefficeints for shear force due to tandem loads

n_1', n_2' = influence line coefficeints for moment due to tandem loads

P_d = post depth

P_e = percentage of distribution reinforcement

P_h	=	post height
P_{ll1}	=	design truck load
P_n	=	total number of posts
P_s	=	post spacing
P_w	=	post width of concrete
P_1, P_2	=	truck loads
P_3	=	tandem load
R_d	=	railing depth of concrete
R_w	=	roadway width of the bridge
R_{ww}	=	railing width of concrete
R_3	=	steel reinforcement ratio
R_8 (nb)	=	steel reinforcement ratio at the n^{th} bar iteration
R_{200}	=	the maximum reaction on the exterior girder
R_{200b}	=	support reaction at B due to own weight barrier
R_{200d}	=	support reaction at B due to own weight of deck slab
R_{200el}	=	maximum live load reaction on the exterior girder
R_{200o}	=	support reaction at B due to own weight of overhang
R_{200pl}	=	exterior girder reaction
R_{200w}	=	support reaction at B due to own weight of wearing surface
S_{a3}	=	spacing of reinforcement in the longitudinal direction, bottom slab
S_{b3}	=	spacing of top and bottom reinforcement in the transversal direction, bottom slab
S_{di}	=	spacing of distribution reinforcing bars
S_e	=	spacing of negative main reinforcing bars
S_{e3}	=	spacing of overhang main reinforcing bars
S_i	=	spacing of posetive main reinforcing bars
S_{kio}	=	spacing of skin reinforcing bars
S_{maxi}	=	maximum spacing of main reinforcing bars for crack control
S_{maxn}	=	maximum spacing of negative moment reinforcement for crack control
S_{maxp}	=	maximum spacing of positive moment reinforcement for crack control
S_{rip}	=	spacing of shear reinforcing bars provided
S_{st}	=	spacing of temperature reinforcing bars
S_x	=	clear spacing between girders
t_s	=	thickness of deck slab
t_{sb}	=	thickness of bottom slab

V_{cc1} = the nominal shear strength provided by the concrete

V_{dl} (L_{1a}) = shear due to dead load at L_{1a} distance from support for interior girder

V_{dle} (L_{1a}) = shear due to dead load at L_{1a} distance from support for exterior girder

V_{dv1} = design maximum factored shear force at L_{1a} distance from center of support

V_{si1} = the nominal shear strength provided by the shear reinforcement

V_{tml} (L_{1a}) = shear force at a distance L_{1a} due to tandem and lane load

V_{trl} (L_{1a}) = shear force at a distance L_{1a} due to truck and lane loads

W_{dw} = weight of future wearing surface

W_{ls} = beam seat width, left

w_{l1} = design lane load

W_{oh} = weight of overhang slab

W_{rs} = beam seat width, right

W_s = weight of deck slab

X = position of the wheel load from exterior girder

X_2 = neutral axis depth, positive moment

X_{22} = neutral axis depth from top extreme fiber

X_3 = neutral axis depth, negative moment

Y_b = bituminous density

Y_c = concrete density

Y_{sg} = centroid of the cross section measured from bottom of girder

Y_{st} = centroid of transformed cross section measured from bottom of girder

Abstract

Bridges affect people. People use them, and engineers design them and later build and maintain them. Bridges do not just happen. They must be planned and engineered before they can be constructed. Planning and designing of bridges is part art and part compromise, the most significant feature of structural engineering.

Design standards recommend span lengths to be the basic criteria for selecting bridge types, such as T-or Box-Girder bridge. Ethiopian Roads Authority (ERA) bridge design manual recommends reinforced concrete T-Girder bridges can be economical for spans from 12 to 20m and Box-Girder bridges for span lengths of between 30-90m. According to Addis Ababa City Roads Authority (AACRA) bridge design manual of 2004, the recommended span length range for a reinforced concrete T-Girder bridge is 10 - 20m and that of the Box-Girder bridge is 15-35m.

In some design standards, such as AACRA bridge design manual of 2004 and AASHTO bridge design specifications of 2005, overlap in there recommendation of economical span range for T-Girder and Box-Girder bridges have been perceived. Furthermore, inconsistency in selection of the bridge type based on provided span length is also verified while conducting the site visit for this study. Hence, identifying a clear demarcation of span length for selecting the bridge type (T- / Box-Girder), on the basis of economy, happens to be found indispensable.

This study basically concentrates on developing a computer program using “FORTRAN 95” programming language software for analysis, design, and cost estimate of superstructures of reinforced concrete T-Girder and Box-Girder bridges. As a result, it come up with a solution for the design problems associated with the economical span range of the two types of bridges.

A comparative study on the basis of cost of materials (construction cost) is conducted to select and identify the economical span range of T-Girder and Box-Girder bridges. The developed program is demonstrated using particular examples of both T-Girder and Box-Girder bridges.

The study has procured a solution to benefit clients, consulting firms and contractors working in road infrastructure sector in the design and design review (checking) process.

1 Introduction

1.1 Background

The structural design scheme of the bridge presents a complex problem for the structural designers despite the presence of modern technology and advanced computer facilities. The scope of such a problem encompasses the rational for the determination of general dimensions of the structure, the span system (i.e. number and length of spans), the choice for type of substructure. Also, within this scope, there is a demand to find the most advantageous solution to the design problem in order to determine the maximum safety with minimum cost and compatible with structural engineering principles [8].

The analysis and design of reinforced concrete T-girder and Box-girder bridges using excel spread sheet template takes longer time, demands more effort and requires high degree of accuracy. Moreover, the economical span range for the two types of bridges is not clearly demarcated. As referred to many literatures [6, 7, 8], there is overlap of span range for the two types of bridges. Accordingly, most of bridge designers here in our country experience obscurity in selecting the more appropriate bridge type, on the basis of economy, for the span overlap range.

A thesis submitted to the school of graduate studies of Addis Ababa University by Abrham Gebre [1], a Computer Program for Comparative Study of the Analysis and Design of Slab and T-Girder Bridges, simplifies the drawbacks associated with excel spread sheet templates for analyzing and designing of slab and T- Girder bridges and also demarcates the economical span range of the two types bridges. Accounting the prevailing component cost for construction of Bridges.

Economy is the governing factor among other requirements of a bridge like aesthetics, maintainability, for developing countries like Ethiopia. Hence, in view of saving investments also facilitating the design and design checking process, a comparative study, using computer program, to select and identify the economical span range of T- and Box- girder bridges on the basis of cost of materials (construction cost) is the focus of this study.

1.2 Objective of the Thesis

- To prepare a computer program for Analysis and Design of the superstructure of T- and Box-girder bridges following AASHTO LRFD Bridge Design Specifications with reference to Ethiopian Roads Authority (ERA) Bridge Design Manual, and any other internationally & nationally accepted design standards; and to demarcate the economical span with a better degree of accuracy using cost as basis of comparison;

- To selected with the aid of the developed program, the more appropriate type of bridge used or to be used

1.3 Content of the Thesis

Seven chapters are contained within this study, where, Chapter 1 deals with introducing the general background and objectives of the thesis work.

Chapter 2 is devoted to literature survey where, historical development of bridge construction in Ethiopia, design standards and the criteria for selection of design standards, type and classification of bridges, determination of opening and span length of a bridge briefed, in addition to materials selection during design for construction.

Chapter 3 addresses the background and objective of the condition survey, inventory and inspection of selected bridges which encompass the site visit in aggregate and observations during the visit. Moreover, site findings and discussions on the findings are also addressed in this chapter.

Chapter 4 focused on the considerations that need to be taken in to account while designing T - and Box - Girder Bridges, with specific attention to loadings, materials properties and design assumption. Moreover, it also addresses the design specifications considered during analysis and design of bridges as per the design standards. The core and specific input of the researcher is presented under this chapter, indicating all the necessary steps and calculations used in the developed source program.

Chapter 5 describes the basis of selection of bridges and the general overview of the computer program. Moreover, the whole analysis, design and cost estimate process for the two types of bridges addressed using algorithms and flowcharts.

Chapter 6 presents illustrative design examples, applications and limitations of the developed program, and summary of outputs for different span lengths of the two types of bridges. Discussion on the summary of outputs is also incorporated.

The last chapter of this thesis is made to address the conclusion drawn from the outputs based on the developed program and recommendations on the basis of the findings.

2 Review of State of Art of Bridges

2.1 Historical Development of Bridges in Ethiopia

Bridge construction in Ethiopia was early started on Blue Nile River near Alata, Almeida Bridge, where thick log is placed across the narrow rocky banks. During the period following Fassiledes (after 1667) it is said that many bridges were constructed in Gonder and Lake Tana area [1].

In Addis Ababa, the first standard bridge was constructed on Kebena River in 1902 by a Russian Engineer after their compatriot was drowned. The second was Ras Mekonnen Bridge, arch type bridge, in 1908 [1]. Quite considerable numbers of bridges were constructed in the years 1935-1945, the years of occupation by Italians. Notable bridges have been constructed since 1970's (i.e., during the last 30 years). A bridge on Baro River with a total span of 305m is one of the longest span bridges in the country, constructed during this period [1]. Slab, T-Girder, and Box-Girder bridges made using reinforced concrete, mostly with simple supports but either single span or spans in series are the most commonly used bridges in Ethiopia

2.2 Design Standards and Selection of Design Standards

The design of Highway Bridge, like many other civil engineering projects, is dependent on certain standards and criteria. Design of highway bridges in a modern transportation system would require a set of rigorous design specifications to ensure the safety and overall quality of the constructed project.

2.2.1 Design Standards

AASHTO LRFD Bridge Design Specifications

On December 12, 1914, the American Association of State Highway Officials (AASHO) was formed, and in 1921 its Committee on Bridges and Allied Structures was organized. The charge to this committee was the development of standard specifications for the design, materials, and construction of highway bridges. The first edition of the Standard Specifications for Highway Bridges and Incidental Structures was published in 1931 by AASHO, the predecessor to AASHTO [7].

In the beginning, the design philosophy utilized in the standard specification was working stress design (also known as allowable stress design). In the 1970s, variations in the uncertainties of loads, such as vehicular loads and wind forces, were considered and load factor design was introduced as an alternative method. In 1986, the Subcommittee on Bridge and Structures initiated a study on incorporating the load and resistance factor design (LRFD) philosophy in to the standard specification. This study recommended that LRFD be utilized in the design of highway bridges. The LRFD method is introduced by AASHTO since 1994 [7].

ERA Bridge Design Manual

ERA Bridge Design Manual deal with small and medium sized structures and used for all structures within the Ethiopian Roads Authority (ERA) throughout the country [5]. This standard is based mainly on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 2nd edition, 1998, with modifications to Ethiopian conditions, requirements, and applicable laws of the Federal Democratic Republic of Ethiopia.

The Design Standard provides all the necessary procedural guidance, dimensions, and materials advice to enable a civil engineer with some field experiences to prepare appropriate preliminary and detailed designs for small and medium sized bridges [5].

The manual covers the entire design process in terms of both the:

- Time-related process, proceeding from feasibility study through preliminary design and final design to bridge inspection and strength evaluation of old bridges; and
- Total design from foundations to superstructure to bearing and railing.

2.2.2 Selection of Design Standards

For design and construction of Highway Bridge in Ethiopia, the manual that need to be referenced first is ERA Bridge Design Manual, 2002, since it is nationally accepted bridge design manual throughout the country. ERA Bridge Design Manual, 2002 is prepared closely following the universally recognized bridge design specification, AASHTO LRFD Bridge Design Specification 2nd edition 1998. Since, ERAs' bridge design manual prepared based on older version of AASHTOs' specification whereas, the AASHTO LRFD Bridge Design Specification is frequently updated since 1998, bridge engineers are strongly recommended to have a good reference of the most recent version of these specification while designing and constructing bridges in the country. Besides the two above-mentioned design standards, reference also made to Addis Ababa City Roads Authority (AACRA) Bridge Design Manual, 2004, while designing Bridges for urban streets.

2.3 Bridge Types and Classifications

2.3.1 Types of Bridges

In the selection of a bridge type, there is no unique method of obtaining the “correct” answer. For each span length range, more than one bridge type will satisfy the design criteria. Due to regional differences and preferences of available materials, skilled workers and knowledgeable contractors, the type of bridge selected may vary in different areas for the same set of geometric and subsurface circumstances [7].

The types of bridges for general use fall under the following category [4]:

- Simply supported bridges, suitable for short spans.
- Balanced cantilever bridges, good for increased span and are mostly of reinforced concrete and / or pre-stressed concrete construction.
- Multiple but simple spans in series, used for medium or long span bridges.
- Continuous beams: advantages over simple spans for reduced weight, greater stiffness (smaller deflection), and less number of bearings. Continuous spans also provide redundancy and hence greater overload capacity than simple spans.
- Arch bridges: suitable for large spans.
- Rigid frame bridges: where the horizontal deck slabs are made monolithic with the vertical abutment walls and are economically suitable for moderate-medium span lengths.

2.3.2 Classifications of Bridges

Different methods may be used to classify bridges among which [7]:

- Materials as concrete, steel, wood, or combination of any two or more
- Usage as pedestrian, highway, or railroad
- Span as short, medium, or long
- Structural form as slab, girder, truss, arch, suspension, or cable-stayed

Reinforced Concrete is used extensively in highway bridges in short and medium spans because of its economy, durability, low maintenance costs, adaptability to any geometric alignment, and ease of construction. The major advantage in the use of concrete bridge is the wide variation that can be achieved in form [4].

Concrete can be used in many different ways and often many different configurations are feasible. However, market forces, projects and site conditions affect the relative economy of each option and this lead to selecting the different forms of bridge as slab, T- or Box-Girder, Arch etc.

T-Girder Bridges

The T-beam construction consists of a transversely reinforced slab deck which spans across to the longitudinal support girders. These require a more-complicated formwork, particularly for skewed bridges, compared to the other superstructure forms [8]. Based on span lengths T-beam bridges are the next class of structures beyond the range of longitudinally reinforced slabs and may be considered for spans 45 to 90 ft. (13.5-27 m) long [6]. They shall be used for span lengths between 12-20m [5]. They are generally economical for spans 10-20 m [2] & [7] and for spans 12 to 18 m [8].

T-Girder bridges usually are constructed on ground-supported falsework and require a good finish on all surfaces [7]. The girder stem thickness usually varies from 35 to 55 cm and is controlled by the required horizontal spacing of the positive moment reinforcement. Optimum lateral spacing of longitudinal girders is typically between 1.8 and 3.0 m for a minimum cost of formwork and structural materials. However, where vertical supports for the formwork are difficult and expensive, girder spacing can be increased accordingly [8].

Box-Girder Bridges

Multicell reinforced concrete box girders become practical at about the maximum optimum span length of a T-girder bridge. They 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 [8]. These bridges are used for spans of 15-35 m and are often more economical than steel girders and precast concrete girders [2] & [7]. They are usually considered for spans of 95 to 140 ft (28.5-42 m). 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, due to massive increment in volume and materials [6].

Cast-in-place box-girder bridges can be constructed to follow any desired alignment in plan, so that straight, skew, and curved bridges of various shapes are common in the highway system. Because of the high torsional resistance, a box girder structure is particularly suited to bridges with significant curvature [6]. Appearance is good from all directions and they are excellent choices in metropolitan areas [7].

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2.4 Openings and Span Selections

2.4.1 Openings

Bridges over rivers will normally require a certain opening of hydraulic reasons. The opening must be large enough not to cause any damage due to backwater [5]. Sometimes it shall be necessary to compensate for the backwater by means of training or relining the stream. For bridges above roadways, the opening width shall not be less than that of the approach roadway section, including shoulders or curbs, gutters, and sidewalks [3]. Sometimes aesthetical reasons require a larger opening than necessary [5].

Bridges over water shall normally have a minimum clearance (free board) height according to Table 2.1 unless a refined hydraulic analysis has been made [5]. The standard minimum headroom or vertical clearance for under bridges or tunnels of all classes of roads shall be 5.1 m [5]. This clearance should be maintained over the roadway(s) and shoulders. Where future maintenance of the roadway is likely to lead to a raising of the road level, then an additional clearance up to 0.1 m may be provided.

Table 2.1 Vertical Clearance from Superstructure to Design Flood Level

Design Flow at Bridge (m^3/s)	Vertical Clearance (m)
5 to 30	0.6
30 to 300	0.9
> 300	1.2

2.4.2 Span Selections

The most economic overall bridge length and span length depends to a large extent upon the foundation costs [5]. If the piers have to be founded deep under the water or if piling is needed, then longer spans up to a certain limit shall be more economical. At larger bridges and difficult soil conditions the most favorable locations of the pier should be sought. If no such locations can be located it may in some cases be more economical to realign the road to a more suitable bridge site, although all aspects should be re-investigated.

A bridge aligned at right angles to the river results in the shortest superstructure length. A skewed bridge requires more material and is more complicated to design and construct. If a skew is unavoidable the angle should preferably not to exceed 20° [5].

Span length should be [5]:

- For simple spans: the distance center to center of supports but need not exceed clear span plus thickness of slab.
- For members that are not built integrally with their supports: the clear span plus the depth of the member but need not exceed the distance between centers of supports.

Span length should give the placing of the piers regardless of type or dimensions selected at a later stage. It is normally measured at the alignment and given as stations.

2.5 Materials

Reinforced concrete has become universally accepted material for bridge work in a variety of dominant structural forms. The range of applications covers all-concrete structures and nobel combinations of concrete decks supported on steel beams or girders. The broad acceptability of reinforced concrete is related to the availability of structural materials such as reinforcing bars and the constituents of concrete: sand, aggregate, and cement. The choice is enhanced by the relative simple skills required at the site [6].

2.5.1 Concrete

Concrete is a conglomerate artificial stone. It is a mixture of large and small particles held together by a cement paste that hardens and will take the shape of the formwork in which it is placed. The proportions of the coarse and fine aggregate, Portland cement, and water in the mixture influence the properties of the hardened concrete [7].

In most cases a bridge engineer will select a particular class of concrete from a series of predesigned mixes, usually on the basis of the desired 28-day compressive strength, f_c'. These classes intended for use as follows [7]:

- Class A concrete is generally used for all elements of structures and especially for concrete exposed to salt water.
- Class B concrete is used in footings, pedestals, massive pier shafts, and gravity walls.
- Class C concrete is used in thin sections under 100 mm in thickness, such as reinforced railings and for filler in steel grid floors.
- Class P concrete is used when strengths exceeding 28 MPa are required.
- Class S concrete is used for concrete deposited under water in cofferdams to seal out water.

The water-cement ratio (W/C) by weight is the single most important strength parameter in concrete. The lower the W/C ratio, the greater is the strength of the mixture. Obviously, increasing the cement content increases the strength for a given amount of water in the mixture. For each class of concrete a minimum amount of cement in (kg/m^3) is specified.

To obtain quality concrete that is durable and strong, it is necessary to limit the water content, which may produce problems in workability and placement of the mixture in the forms [7].

2.5.2 Reinforcing Steel

Reinforced concrete is simply concrete with embedded reinforcement, usually steel bars or tendons. Reinforcement is placed in structural members at locations where it will be of the most benefit. It is usually thought of as resisting tension, but it is also used to resist compression [7].

Concrete can be reinforced with welded wire fabric, deformed steel bars, and prestressing tendons [9]. Among three of them, deformed steel bars are commonly employed as reinforcement in most reinforced concrete bridge construction. The surface of a steel bar is rolled with lugs or protrusions called deformations in order to restrict longitudinal movement between the bars and the surrounding concrete.

Reinforcing bar steel can be made of billet steel grades 40 and 60 having minimum specific yield stress of 40,000 and 60,000 psi, respectively (276 and 414 MPa) [9].

3 Condition Survey of Selected Bridges

3.1 Background

A bridge must be inspected on regular basis to ensure public safety and to protect public investment. Condition survey of existing bridges helps to identify the needs of bridges for repair, maintenance, preservation, reconstruction, and replacement. Many bridges have been collapsed worldwide prior to the bridge service life span due to lack of proper periodic inspection and maintenance.

The key premises of this study is the inconsistency observed in the selection of the bridge type (T- / Box-Girder Bridge) for a particular span length, during the design and/or design review process. Accordingly, the condition survey of existing bridges happens to be found compulsory to justify the discrepancy in the selection of the bridge type with regard to the provided span length.

3.2 Objective of Condition Survey

Basically, the condition survey of existing bridges is conducted to

- Investigate or assess the selection of the bridge type for that particular crossing from the provided span length point of view.

- Verify consensus of the provided span length with the recommendations of span ranges for the two types of bridges (T- & Box-Girder bridges) specified on different design standards.

- Check whether the design requirements are satisfied or not from economy as well as safety point of view. Moreover, to further investigate the safety of the bridge taking the defects observed at time of conducting the survey and the associated potential risks they may cause on the users of the road in to consideration.

3.3 Inventory and Inspection of Selected Bridges

Bridge inspections are conducted to determine the physical and functional conditions of a bridge and assist to determine the basis for the evaluation and take preventive and maintenance actions. Bridge inventory on the other hand deals with record of quantifiable items of a bridge which may be carried once in a bridges' life span. However, inspection need to be carried frequently as it deals with condition assessment [10].

In order to appreciate the design problem, condition survey of existing bridges has been performed by means of site visit accompanied by desk study. During the visit, attention was given only to the two types of bridges (T- & Box-Girder bridges), which is related to the study work. Brief survey summary and conditions of selected bridges are provided underneath.

3.3.1 Abiya River Bridge

Bridge Name:	Abiya River Bridge
Crossing:	Abiya River
Bridge Location:	About 623 km North of Addis Ababa
Bridge Type:	Reinforced Concrete T-Girder Bridge
Number of Spans:	1
Clear Opening Length:	25.80m
Roadway Width:	6.00m
Length of Overhang:	0.55m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.50m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	Exposure of reinforcements and honeycomb on deck slab and Longitudinal girders

Figure 3.1 Abiya River Bridge

3.3.2 Koga River Bridge

Bridge Name:	Koga River Bridge
Crossing:	Koga River
Bridge Location:	About 581 km North of Addis Ababa
Bridge Type:	Reinforced Concrete Box-Girder Bridge
Number of Spans:	1
Clear Opening Length:	27.20m
Roadway Width:	6.00m
Length of Overhang:	0.60m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.80m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	Water leakage on the deck slab and exposure of reinforcements on deck and longitudinal girders

Figure 3.2 Koga River Bridge

3.3.3 Dilbazia River Bridge

Bridge Name:	Dilbazia River Bridge
Crossing:	Dilbazia River
Bridge Location:	About 669 km North of Addis Ababa
Bridge Type:	Reinforced Concrete T-Girder Bridge
Number of Spans:	1
Clear Opening Length:	20.20m
Roadway Width:	6.00m
Length of Overhang:	0.55m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.45m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	Exposure of reinforcements and honeycomb on longitudinal girders

Figure 3.3 Dilbazia River Bridge

3.3.4 Teda River Bridge

Bridge Name:	Teda River Bridge
Crossing:	Teda River
Bridge Location:	About 658 km North of Addis Ababa
Bridge Type:	Reinforced Concrete T-Girder Bridge
Number of Spans:	1
Clear Opening Length:	25.50m
Roadway Width:	6.00m
Length of Overhang:	0.55m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.60m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Fair Condition
Problems Observed:	Exposure of reinforcements and honeycomb on deck slab and voids on Longitudinal girders.

Figure 3.4 Teda River Bridge

3.3.5 Burure River Bridge

Bridge Name:	Burure River Bridge
Crossing:	Burure River
Bridge Location:	About 588 km North of Addis Ababa
Bridge Type:	Reinforced Concrete Box-Girder Bridge
Number of Spans:	1
Clear Opening Length:	40.00m
Roadway Width:	7.30m
Length of Overhang:	0.75m (both sides)
Deck Slab Thickness:	0.25m
Total Depth of Girder:	1.80m
Bearing Types:	Elastomeric Bearing
Abutment Walls:	Reinforced Concrete
Wingwalls:	Reinforced Concrete
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	None

Figure 3.5 Burure River Bridge

3.3.6 Warkena River Bridge

Bridge Name:	Warkena River Bridge
Crossing:	Warkena River
Bridge Location:	About 723.5 km North of Addis Ababa
Bridge Type:	Reinforced Concrete T-Girder Bridge
Number of Spans:	1
Clear Opening Length:	25.10m
Roadway Width:	6.60m
Length of Overhang:	0.50m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.80m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	Honeycomb on deck slab and longitudinal girders due to excess vibration

Figure 3.6 Warkena River Bridge

3.3.7 Mille River Bridge

Bridge Name: Mille River Bridge (No. 3)

Crossing: Mille River

Bridge Location: About 521 km North East of Addis Ababa

Bridge Type: Reinforced Concrete Box-Girder main & Reinforced Concrete T-Girder approach

Number of Spans: 4

Span Length: 10.00m (both exterior spans) & 36.00m (both interior spans)

Roadway Width: 7.32m

Length of Overhang: 0.80m (both sides)

Deck Slab Thickness: 0.20m

Total Depth of Girder: 2.20m

Bearing Types: Elastomeric Bearing

Abutment Walls: Stone Masonry

Wingwalls: Stone Masonry

Piers: Reinforced Concrete

Present Status: Fair Condition

Problems Observed: Hair line cracks on bottom face of box girder due to shortage of water during curing of concrete; exposure of rebar and peeling off concrete cover on surface of deck slab

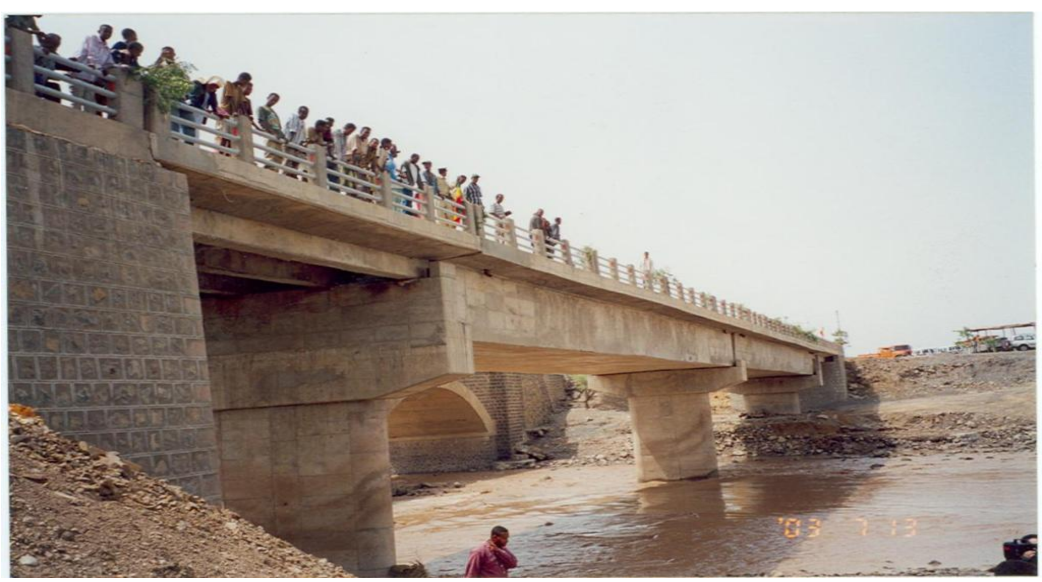

Figure 3.7 Mille River Bridge

3.3.8 Semeno River Bridge

Bridge Name:	Semeno River Bridge
Crossing:	Semeno River
Bridge Location:	About 621 km North of Addis Ababa
Bridge Type:	Reinforced Concrete T-Girder Bridge
Number of Spans:	1
Clear Opening Length:	19.00m
Roadway Width:	6.00m
Length of Overhang:	0.55m (both sides)
Deck Slab Thickness:	0.20m
Total Depth of Girder:	1.50m
Bearing Types:	Steel Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	None
Present Status:	Generally in a Good Condition
Problems Observed:	Exposure of reinforcements on deck slab

Figure 3.8 Semeno River Bridge

3.3.9 Meterie River Bridge

Bridge Name:	Meterie River Bridge
Crossing:	Meterie River
Bridge Location:	About 567 km North East of Addis Ababa
Bridge Type:	Reinforced Concrete Box-Girder main & Reinforced Concrete T-Girder approach
Number of Spans:	3
Span Length:	20.00m (both exterior spans) & 40.00m (interior span)
Roadway Width:	7.30m
Length of Overhang:	0.75m (both sides)
Deck Slab Thickness:	0.25m
Total Depth of Girder:	1.80m
Bearing Types:	Elastomeric Bearing
Abutment Walls:	Stone Masonry
Wingwalls:	Stone Masonry
Piers:	Reinforced Concrete
Present Status:	Generally in a Good Condition
Problems Observed:	honey comb on deck slab

Figure 3.9 Meterie River Bridge

3.4 Site Findings

While conducting the site visit, the following observations were made:

- Most of the inspected T-Girder bridges are constructed not following design standards' span length recommendation for a reinforced concrete T-Girder bridge. Although the upper most bound span length recommendation set by various design standards for a T-Girder bridge is 20m, Abiya, Teda, and Warkena river bridges, having span lengths 25.8m, 25.5m, and 25.1m respectively, are way longer than 20m.

- In Ethiopia, the total number of reinforced concrete box-girder bridges can be considered as small as compared to reinforced concrete T-Girder bridges since their construction have began recently in the country, approximately not more than last 15 years. For these reason, it has been found difficult to get a box-girder bridge having span length that deviate from design standards' span range recommendation in the vicinity where the condition survey conducted.

- Honeycomb due to excessive vibration during construction stage, exposure and corrosion of reinforcements on deck and girder in addition to water leakage are the predominant structural defects observed on superstructures of the inspected bridges.

- The installation of deck drains in most of the bridges is not per the recommendation of AASHTOs' Bridge Design Specification. Furthermore, Most of them got clogged with debris and trashes due to lack of proper periodic maintenance, which have significant contribution for further damages and exposure and corrosion of reinforcing bars on the deck system.

- Some of expansion joints elements such as structural steel angles for instance, on Mille river bridge are missing while on others got deformed due to frequent exposure for heavily loaded vehicles.

- Approach slab, except on Burure and Meterie river bridges, and Protection barriers are not provided on the approach roadway of the existing bridges.

- Though some minor defects are observed on substructures of the inspected bridges, such as minor longitudinal cracks on mortar joints of masonry walls and clogging of weep hole with washed backfill material, the substructures can generally be said in a good condition. Moreover, while conducting the condition survey, no major scour problem encountered, since the bridges are constructed on a good foundation bed material.

3.5 Discussions

Even though major structural problems associated with the span lengths are not examined on Abiya, Teda, and Warkena river bridges, they could have been constructed with lesser cost if they were using box-girder bridges.

Steel/Concrete post Guardrails, as per ERA bridge design manual recommendation; have to be provided on the approach roadways of the existing bridges with the aim of providing the proper safety for the users of the road.

Approach slabs, having the appropriate dimension, need to be provided on both sides of the bridges in order to minimize the adverse effects associated with settlement.

Although minor defects are observed on the bridges under consideration, they have to be regularly inspected and appropriately maintained by the concerned bodies before they changed to major defects that lead the structure to a completely collapsed state.

4 Analysis and Design of T-Girder and Box-Girder Bridges

4.1 Loadings, Material Properties and Design Assumptions

4.1.1 Loadings

Designers must consider all the loads that are expected to be applied to the bridge during its service life. Such loads may be divided in to two broad categories: permanent loads and transient loads [7].

Permanent loads are those that remain on the bridge for an extended period of time, perhaps for the entire service life. Such loads include:

- Dead load of structural components and non-structural attachments
- Dead load of wearing surfaces and utilities
- Dead load of earth fill
- Earth pressure, earth surcharge and downdrag
- Locked-in erection stresses

Transient loads, as the name implies, change with time, highly variable and may be applied from several directions and/ or locations. Typically, such loads include:

- Gravity loads due to vehicular and pedestrian traffic
- Horizontal vehicular loads, such as those due to braking, centrifugal forces
- Lateral loads due to wind, water and earthquake
- Collision loads, caused either by a service vehicle striking the structure, or by some moving object in the channel beneath the bridge
- Load due to thermal effects

A bridge is usually constructed to fulfill a well-defined function. Bridge design loadings are directly governed by the functionality requirement [1].

The design vehicular live load was replaced in 1993 because of heavier truck configurations on the road today, and because a statically representative, notional load was needed to achieve a "consistent level of safety." The notional load that was found to best represent "exclusion vehicles," i.e. trucks with loading configurations greater than allowed but routinely granted permits by agency bridge rating personnel, was adapted by AASHTO and named "Highway Load '93" or HL 93. It is notional in that it does not represent any specific vehicle [8].

The AASHTO "design vehicular live load," HL 93, is a combination of a "design truck" or "design tandem" and a "design lane".

Design Truck: is a model load that resembles the typical semitrailer truck. The front axle is 35 kN, the drive axle of 145 kN is located 4.3 m behind, and the rear trailer axle is also 145 kN and is positioned at a variable distance ranging between 4.3 m and 9 m. The variable range means that the spacing used should cause critical load effect [7].

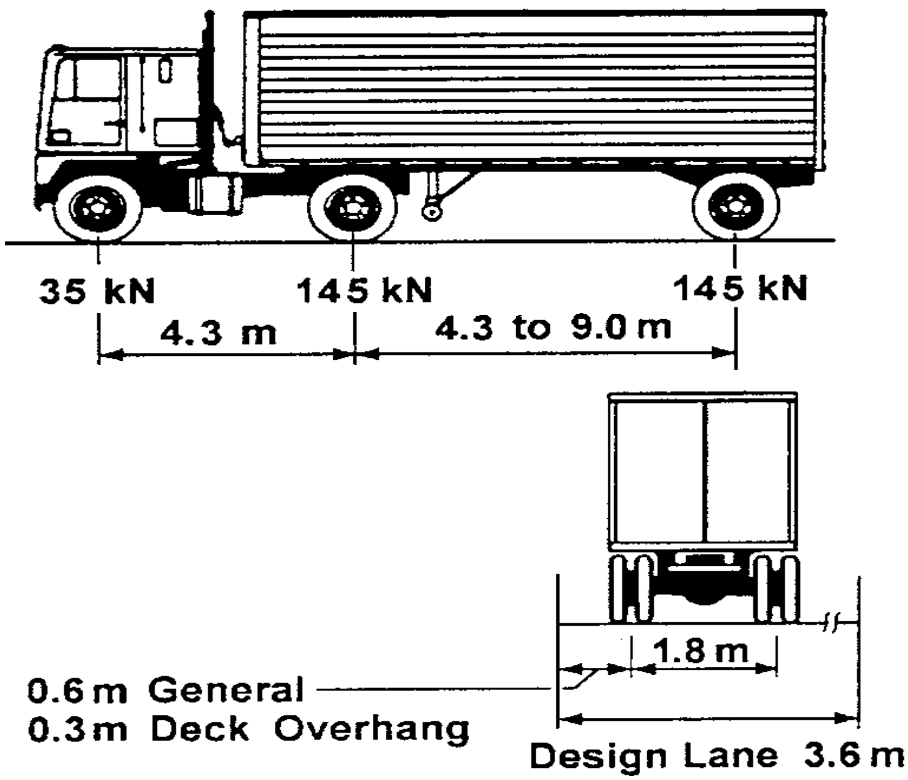

Figure 4.1 AASHTO-LRFD Design Truck

Design Tandem: The design tandem shall consist of a pair of 110 kN axles spaced 1.2 m apart. The transverse spacing of wheels shall be taken as 1.8 m. A dynamic load allowance shall be considered [3].

Design Lane Load: The design lane load shall consist of a load of 9.3 kN/m uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over 3.0 m width. The force effects from the design lane load shall not be subjected to a dynamic load allowance [AASHTO LRFD Bridge Design Specifications, 2005, Article 3.6.1.2.4].

4.1.2 Material Properties

According to AASHTO LRFD Bridge Construction Specifications, 2005, all materials and tests must conform to the appropriate standards included in the AASHTO Standard Specifications for Transportation Materials [3]. Accordingly, materials used in designing bridges of this program are as follows:

i. Steel strength of 420MPa (Grade 60) for reinforcing bars of diameter greater than 20mm and 300MPa (Grade 40) for less diameters
ii. The strength of the concrete is the 28th day cylindrical compressive strength of concrete. (f_c'=0.8*f_c, where f_c is the 150mm cubic strength of the same age)
iii. The density of concrete is taken as 2400kg/m^3 for computation of the modulus of elasticity of concrete, E_c, and 25kN/m^3 for dead load computations
iv. The modulus of elasticity, E_s, is constant and which is equal to 200,000MPa
v. The density of wearing surface is taken as 2250 kg/m^3 for dead load computations.

4.1.3 Design Assumptions

The following assumptions are made in developing the program.

- Abutments are assumed to be the same for both T-Girder and Box Girder bridges of the same span length. Hence, design of abutments and its associated cost are not taken in to account for cost comparisons,

- Although the dead loads of railings, curbs and posts are considered during the analysis, design and in quantity computations, it is assumed that the loads are the same for the same span of both T-Girder and Box Girder bridges,

- The cost of elastomeric bearing for both T-Girder and Box Girder bridges is the same

4.2 Design Specification

Bridge Site Arrangement

The choice of location of bridges shall be supported by analyses of alternatives with consideration given to economic, engineering, social, and environmental concerns as well as cost of maintenance and inspection associated with the structures and with the relative of the above-noted concerns [5].

Attention, commensurate with the risk involved, shall be directed towards providing for favorable bridge locations that:

- Fit the conditions created by the obstacle being crossed
- Facilitate practical cost effective design, construction, operation, inspection and maintenance
- Provide for the desire level of traffic service and safety
- Minimize adverse highway impacts

Width of Bridge Deck

The width of the bridge should correspond with the roadway or carriage way width and it is to be measured between the inside of the railings or the curbs. The bridge width shall not be less than that of the approach roadway section, including shoulders or curbs, gutters, and sidewalks [3] & [5].

The inside face of a barrier should not be closer than 600 mm to either the face of the object or the edge of a designated traffic lane. The specified minimum distance between the edge of the traffic lane and fixed object are intended to prevent collision with slightly errant vehicles and those carrying wide loads.

If not otherwise stated in the ERA Geometric Design Manual, 2002, a one-lane bridge shall not be less than 4.2 m wide and a two-lane bridge not less than 7.0 m wide.

Table 4.1 Bridge Widths

Application	Width (m)
Two-lane in “urban” area	10.30
Two-lane in “rural” area	7.30
Single Lane	4.20
Pedestrian Overpass	3.0

Number of Design Lanes

The number of design lanes should be determined by taking the integer part of the ratio w/3600, where w is the clear roadway width in mm between curbs and/or barriers [3].

$$N_L = INT((R_w * 10^3)/3600)$$

Where:

N_L is number of design lanes

R_w is roadway width of the bridge (m)

Multiple Presence Factor

Trucks will be present in adjacent lanes on roadway with multiple design lanes, but it is unlikely that three adjacent lanes will be loaded simultaneously with the heavy loads. Therefore, some adjustments in the design loads are necessary. To account for this effect, AASHTO LRFD Bridge Design Specifications, 2005, provides an adjustment factor for the multiple presence [7].

Table 4.2 Multiple Presence Factors, m

Number of Loaded Lanes	1	2	3	>3
Multiple Presence Factors “m”	1.20	1.00	0.85	0.65

Dynamic Load Allowance

This factor accounts for hammering when riding surface discontinuities exist, and long undulations when settlement or resonant excitation occurs [8].

Unless otherwise permitted in AASHTO LRFD Bridge Design Specifications, 2005, the static effects of the design truck or tandem, other than centrifugal and braking forces, shall be increased by the percentage specified in Table 4.3 for dynamic load allowance [7].

The factor to be applied to the static load shall be taken as (1+IM/100). The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.

Table 4.3 Dynamic Load Allowance, IM

Component	IM
Deck Joints-All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit States	15%
• All other Limit States	33%

Load Factors and Load Combinations

In Load and Resistance Factor Design (LRFD) method, load factors per the provision of AASHTO LRFD Bridge Design Specifications, 2005, are applied to the loads and resistance factors to the internal resistances or capacities of sections [3] & [5]. Moreover, the load factors and load combination are considered accordingly.

4.3 Analysis of Internal Forces and Design of T-Girder Bridges

4.3.1 Design Loads

Loads taken in to account in the design of T- Girder bridges are:

- Self-weight of the deck slab including curbs, railings, and posts
- Self-weight of the beams (girders)
- Self- weight of the future wearing surface if exists and
- Vehicular live loads (A standard design truck, a standard design tandem, and the design lane load are used to compute the extreme force effects).

4.3.2 Load Distribution

The permanent load is distributed to the girders by assigning to each all loads from superstructure elements within half the distance to the adjacent girder. The dead loads due to concrete barrier, sidewalks and curbs, however, may be equally distributed to all girders [8].

Load distribution tables and the “lever rule” are approximate methods for distributing live load and intended for most designs. The lever rule considers the slab between two girders to be simply supported. The reaction is determined by summing the reaction from the slabs on either side of the beam under consideration.

4.3.3 Analysis Procedures

Analysis of Deck Slab

An approximate method of analysis in which the deck is subdivided in to strips perpendicular to the supporting components shall be considered acceptable for decks [3].

The strips shall be treated as continuous beams with span lengths equal to the center-to-center distance between girders. The girders shall be assumed to be rigid [7].

Depth Determination:

The minimum thickness for concrete deck slab is 175mm excluding any provision for grinding, grooving, and sacrificial surfaces [3].

Traditional minimum depths of slabs are based on the deck span length to control deflection.

For durability, adequate cover shall be used as per the specification of AASHTO LRFD Bridge Design manual, 2005.

$$t_s = \mathrm{Max}[((G_s * 10^3) + 3000)/30, 175] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.1)$$

where:

t_s = depth of deck slab (mm)

G_s = girder spacing (m)

Weights of Components:

$$W_s = 25 * t_s / 10^3$$

$$W_{oh} = 25 * t_{oh} / 10^3$$

$$W_{cb} = 25 * C_d$$

$$W_{ra} = 25 * R_{ww} * R_d$$

$$W_p = 25 * P_w * P_h * P_d * P_n / L_1$$

$$W_{dw} = Y_b * 9.81 * b_t / 10^6$$

where:

W_s = weight of deck slab (kN/m^2)

W_{oh} = weight of overhang slab (kN/m^2)

W_{dw} = weight of future wearing surface (kN/m^2)

Bending Moment Force Effects

An approximate analysis of strips perpendicular to girders is considered.

The extreme positive moment in any deck panel between girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any girder shall be taken to apply to all negative moment regions [7].

For ease in applying load factors, the bending moments are determined separately for the deck slab, future wearing surface, overhang, barrier, and vehicular live load.

1. Deck Slab

A deck analysis design aid based on influence lines, one dimensional influence functions, is presented in Annex I. For a uniform load, the tabulated areas are multiplied by G_s for shears and by G_s^2 for moments. A one-meter wide strip is taken for the analysis.

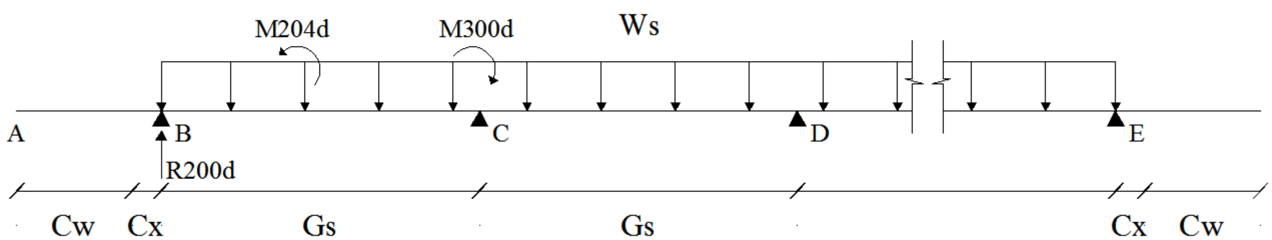

Figure 4.2 Dead Load of Deck Slab

Support reaction and moments due to dead load of deck slab are given as follows:

$$R_{200d} = W_s * [\text{net area w/o cantilever}] * G_s$$
$$= 0.3928 * W_s * G_s$$
$$M_{204d} = W_s * [\text{net area w/o cantilever}] * G_s^2$$
$$= 0.0772 * W_s * G_s^2$$
$$M_{300d} = W_s * [\text{net area w/o cantilever}] * G_s^2$$
$$= -0.1071 * W_s * G_s^2$$

Where:

R_{200d} = support reaction at B due to dead load of deck slab

M_{204d} = span moment on span BC at a distance 0.4*Gs from support B

M_{300d} = support moment at C due to dead load of deck slab

2. Future Wearing Surface

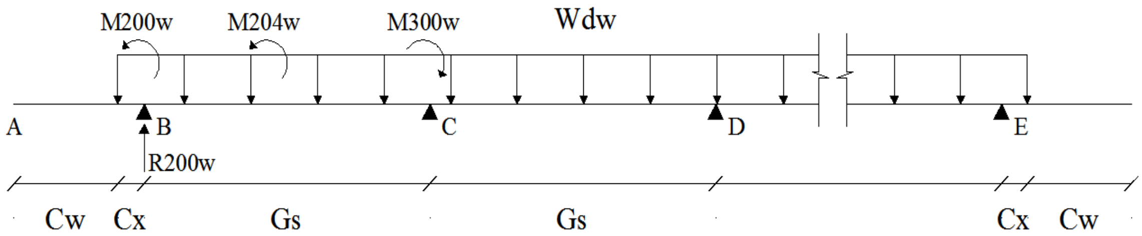

Figure 4.3 Future Wearing Surface Dead-Load Placement

Support reaction and moments due to dead load of wearing surface are given as follows:

$$R_{200w} = W_{dw} * [\text{net area cantilever}] * C_x + W_{dw} * [\text{net area w/o cantilever}] * G_s$$
$$= W_{dw} * [(1.0 + 0.635 * (C_x / G_s)) * C_x + (0.3928 * G_s)]$$
$$M_{200w} = W_{dw} * [\text{net area cantilever}] * C_x^{\ 2}$$
$$= -0.50 * W_{dw} * C_x^{\ 2}$$
$$M_{204w} = W_{dw} * [\text{net area cantilever}] * C_x^{\ 2} + W_{dw} * [\text{net area w/o cantilever}] * G_s^{\ 2}$$
$$= W_{dw} * [(-0.2460 * C_x^{\ 2}) + (0.0772 * G_s^{\ 2})]$$
$$M_{300w} = W_{dw} * [\text{net area cantilever}] * C_x^{\ 2} + W_{dw} * [\text{net area w/o cantilever}] * G_s^{\ 2}$$
$$= W_{dw} * [(0.1350 * C_x^{\ 2}) + (-0.1071 * G_s^{\ 2})]$$

Where:

R_{200w} = support reaction at B due to wearing surface

M_{200w} = support moment at B due to wearing surface

M_{204w} = span moment on span BC at a distance 0.4*Gs from support B

M_{300w} = support moment at C due to wearing surface

3. Deck overhang

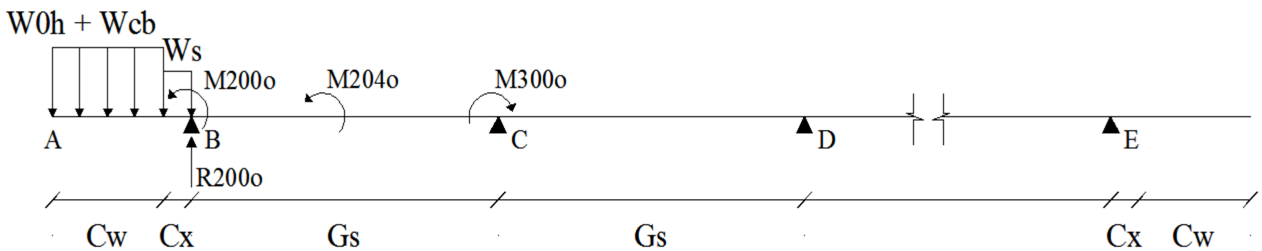

Figure 4.4 Deck Overhang Dead-Load Placement

Support reaction and moments due to dead load of deck overhang are given as follows:

$$R_{200o} = W_s * [\text{net area cantilever}] * C_x + (W_{ho} + W_{cb}) * [\text{net area cantilever}] * C_w$$
$$= W_s * ((1.0 + 0.635 * (C_x / G_s)) * C_x) + (W_{ho} + W_{cb}) * ((1.0 + 0.635 * (C_w / G_s)) * C_w)$$
$$M_{200o} = W_s * [\text{net area cantilever}] * C_x^{\ 2} + (W_{ho} + W_{cb}) * [\text{net area cantilever}] * C_w^{\ 2}$$
$$= -0.50 * [(W_s * C_x^{\ 2}) + ((W_{ho} + W_{cb}) * C_w^{\ 2})]$$
$$M_{204o} = W_s * [\text{net area cantilever}] * C_x^{\ 2} + (W_{ho} + W_{cb}) * [\text{net area cantilever}] * C_w^{\ 2}$$
$$= -0.2460 * [(W_s * C_x^{\ 2}) + ((W_{ho} + W_{cb}) * C_w^{\ 2})]$$
$$M_{300o} = W_s * [\text{net area cantilever}] * C_x^{\ 2} + (W_{ho} + W_{cb}) * [\text{net area cantilever}] * C_w^{\ 2}$$
$$= 0.1350 * [(W_s * C_x^{\ 2}) + ((W_{ho} + W_{cb}) * C_w^{\ 2})]$$

Where:

R_{200o} = support reaction at B due to deck overhang

M_{200o} = support moment at B due to deck overhang

M_{204o} = span moment on span BC at a distance 0.4*Gs from support B

M_{300o} = support moment at C due to deck overhang

4. Barrier

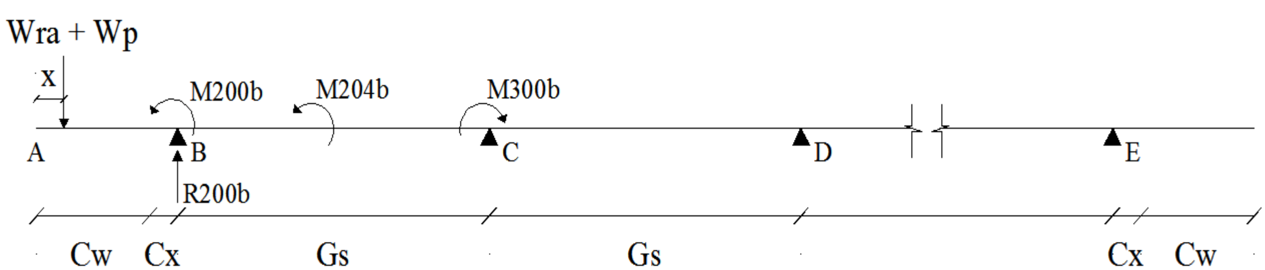

Figure 4.5 Barrier Dead-Load Placement

The intensity of the load is multiplied by the influence line ordinate for shears and reaction. For bending moments, the influence line ordinate is multiplied by the cantilever length.

Support reaction and moments due to dead load of barrier

$$R_{200b} = (W_{ra} + W_p) * [\text{influence line ordinate}]$$
$$= (W_{ra} + W_p) * (1.0 + 1.270 * ((C_x + C_w - x) / G_s))$$
$$M_{200b} = (W_{ra} + W_p) * [\text{influence line ordinate}] * L$$
$$= -1.0000 * (W_{ra} + W_p) * (C_x + C_w - x)$$

$$M_{204b} = (W_{ra} + W_p) * [\text{influence line ordinate}] * L$$
$$= -0.4920 * (W_{ra} + W_p) * (C_x + C_w - x)$$
$$M_{300b} = (W_{ra} + W_p) * [\text{influence line ordinate}] * L$$
$$= 0.2700 * (W_{ra} + W_p) * (C_x + C_w - x)$$

Where:

R_{200b} = support reaction at B due to barrier

M_{200b} = support moment at B due to barrier

M_{204b} = span moment on span BC at a distance 0.4*Gs from support B

M_{300b} = support moment at C due to barrier

Vehicular Live Load

Where decks are designed using the approximate strip method, the strips are transverse and shall be designed for the 145-kN axle of the design truck [7].

The design truck shall be positioned transversely to produce maximum force effects such that the center of any wheel load is not closer than 300 mm from the face of the curb for the design of the deck overhang and 600mm from the edge of the 3600mm –wide design lane for the design of all other components [3].

1. Overhang Negative Live-Load Moment

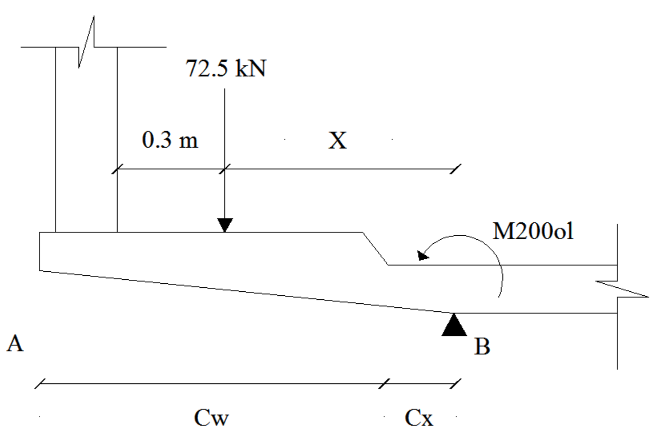

Figure 4.6 Wheel Load Placement on Overhang

The critical placement of a single wheel load is shown in the figure above.

The equivalent interior transverse strip width over which the live load is applied is:

$$E_{oh} = 1.140 + (0.833 * X)$$

Where:

E_{oh} = equivalent interior transverse strip width for overhang negative live load moment (m)

X = position of the wheel load from exterior girder (m)

$$M_{200ol} = (-1 * m * 72.5 * X) / E_{oh}$$

Where:

M_{200ol} = overhang negative live load moment at support B (kN-m /m)

m = multiple presence factor

2. Maximum Positive Live-Load Moment

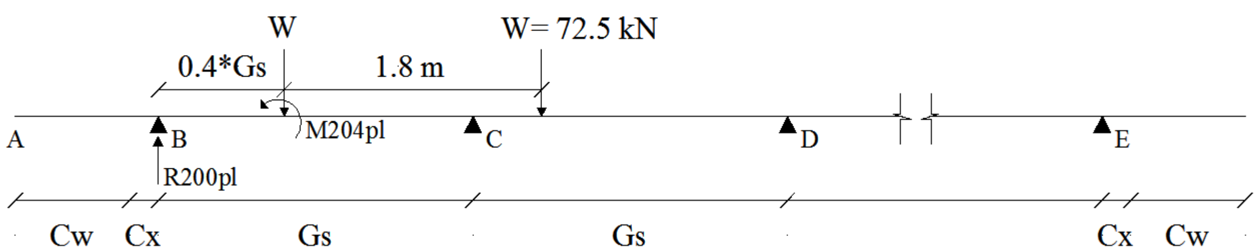

Figure 4.7 Live-Load Placement for Maximum Positive Moment

For repeating equal spans, the maximum positive bending moment occurs near the 0.4 point of the first interior span, that is, at location 204 [7].

The equivalent interior transverse strip width over which the live load is applied is:

$$E_w = 0.66 + (0.55 * G_s)$$

Where:

E_w = equivalent interior transverse strip width for positive live load moment (m)

$$R_{200pl} = (m * (ILOa + ILOb) * 72.5) / E_w$$

$$M_{204pl} = (m * (ILOc + ILOd) * G_s * 72.5) / E_w$$

Where:

R_{200pl} = exterior girder reaction (kN)

M_{204pl} = maximum positive live load moment (kN-m /m)

ILOa & ILOb are influence line ordinates for support reaction

ILOc & ILOd are influence line ordinates for span moment

3. Maximum Interior Negative Live-Load Moment

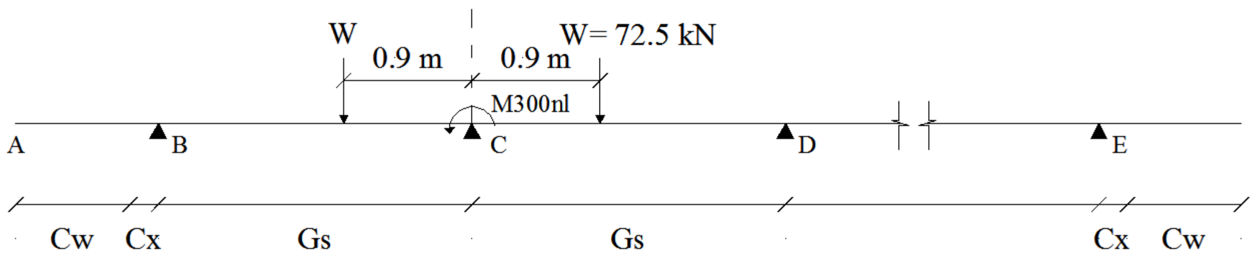

Figure 4.8 Live-Load Placement for Maximum Negative Moment

The critical placement of live load for maximum negative moment is at the first interior deck support with one loaded lane (m = 1.2) [7].

The equivalent interior transverse strip width over which the live load is applied is:

$$E_i = 1.220 + (0.25 * G_s)$$

Where:

E_i = equivalent interior transverse strip width for negative live load moment (m)

$$M_{300nl} = (1.2 * (ILOe + ILOf) * G_s * 72.5) / E_i$$

Where:

M_{300nl} = maximum interior negative live load moment (kN-m /m)

ILOe & ILOf are influence line ordinates for support moment

4. Maximum Live-Load Reaction on Exterior Girder

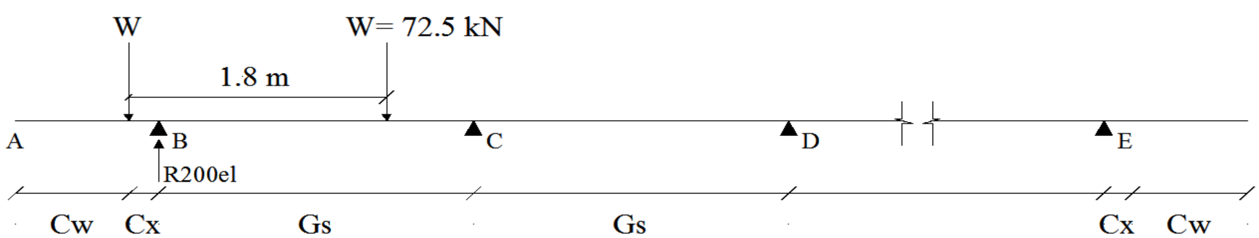

Figure 4.9 Live-Load Placement for Maximum Reaction at Exterior Girder

The maximum live load reaction on the exterior girder is obtained when the exterior wheel is placed 300mm from the face of the curb [7].

$$R_{200el} = (1.2 * (1 + 0.381 * ((C_x + C_w) / G_s) + ILOg) * 72.5) / E_{oh}$$

Where:

R_{200el} = maximum live load reaction on the exterior girder (kN)

ILOg is influence line ordinate for girder reaction

Analysis of Longitudinal Girders

Structural Depth

The minimum structural depth for simple span T-beams is given as follows:

$$D_w = 0.07 * L_1 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.2)$$

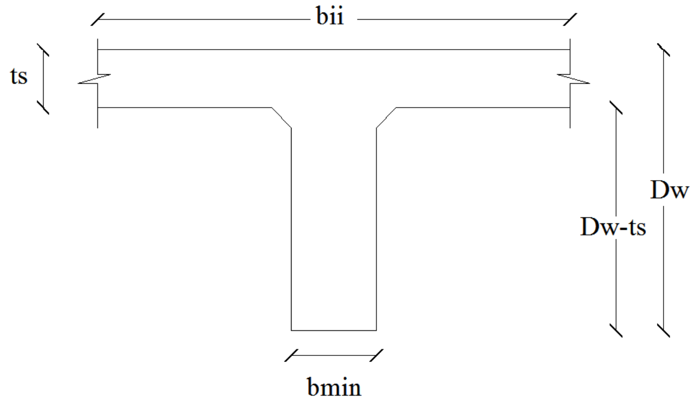

Figure 4.10 Cross Section of an Interior Girder

Effective Flange Width

It depends on effective span length, which is defined as the distance between points of permanent load inflection for continuous beams [7].

a) Interior girder

$$b_{ii} = Min((0.25*(10^3 * L_1)), ((12*t_s) + b_{min}), (10^3 * G_s)) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.3)$$

b) Exterior girder

$$b_{eii} = (0.5*b_{ii}) + Min((0.125*(10^3 * L_1)), ((6*t_s) + (0.5*b_{min})), (10^3 * (C_x + C_w) - (0.5*b_{min}))) \ldots(4.4)$$

Dead Load Force Effects

a) Interior girder

$$D_{lsi} = b_{ii} * t_s * \gamma_c * 9.81*10^{-9}$$
$$D_{lw} = b_{min} * (D_w - t_s) * \gamma_c * 9.81*10^{-9}$$
$$D_{lwi} = b_{ii} * b_t * \gamma_b * 9.81*10^{-9}$$
$$D_{rp} = (2*(W_{ra} + W_p))/N_g$$
$$D_{lgi} = D_{lsi} + D_{lw} + D_{lwi} + D_{rp} + 0.25\,kN/m$$
$$D_{Le} = (25*0.5*((G_s * 10^3) - b_{min}) * 10^{-3}) * ((D_t * (D_w - t_s - 200) * 10^{-6}) + (2*0.5*0.1*0.1))$$
$$D_{Li} = 2*D_{Le}$$

Where:

D_{lsi} is dead load from deck slab (kN/m)

D_{lw} is dead load from web (kN/m)

D_{lwi} is dead load from future wearing surface (kN/m)

D_{rp} is dead load from posts and railing (kN/m)

D_{lgi} is the total dead load on interior girder (kN/m)

D_{Le} is diaphragm dead load on exterior girder (kN)

D_{Li} is diaphragm dead load on interior girder (kN)

b) Exterior girder

$$D_{lse} = (0.5*(b_{eii} + b_{min}))*t_s*\gamma_c*9.81*10^{-9}$$
$$D_{lwe} = ((0.5*(b_{eii} + b_{min})) + (10^3*C_x))*b_t*\gamma_b*9.81*10^{-9}$$
$$D_{lc} = (10^3*C_w)*(10^3*C_d)*\gamma_c*9.81*10^{-9}$$
$$D_{loh} = (((10^3*C_w)*t_{oh}) + (((10^3*C_x) - (0.5*b_{min}))*t_s))*\gamma_c*9.81*10^{-9}$$
$$D_{lge} = D_{lse} + D_{lw} + D_{lwe} + D_{lc} + D_{loh} + D_{rp} + 0.25\,\text{kN/m}$$

Where:

D_{lse} is dead load from deck slab (kN/m)

D_{lwe} is dead load from future wearing surface (kN/m)

D_{lc} is dead load from curb (kN/m)

D_{loh} is dead load from overhang slab (kN/m)

D_{lge} is the total dead load on exterior girder (kN/m)

The bending moment and shear force due to dead loads at every distance L_{1a} from the support is computed for both interior and exterior girders.

$$L_{1a} = (0.01 + ((a_{11} - 1)*0.1))*L_1$$

For center-to-center support length (L_1) lesser or equal to 12m

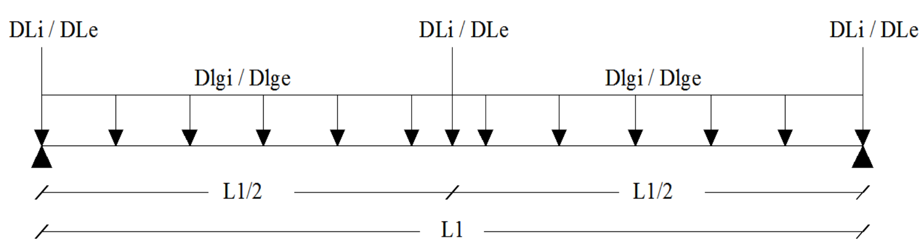

Figure 4.11 Dead-Load Placement for Interior/Exterior Girder ($L_1 \leq$ 12m)

a) Interior girder

For $L_{1a} \leq L_1/2$

$$M_{dl}(L_{1a}) = 0.5 * L_{1a} * ((D_{lgi} * (L_1 - L_{1a})) + D_{Li})$$

$$V_{dl}(L_{1a}) = D_{lgi} * ((0.5 * L_1) - L_{1a}) + (0.5 * D_{Li})$$

For $L_{1a} > L_1/2$

$$M_{dl}(L_{1a}) = 0.5 * (D_{lgi} * (L_{1a}^{\,2} - (L_1 * L_{1a})) + D_{Li} * (L_{1a} - L_1))$$

$$V_{dl}(L_{1a}) = (0.5 * D_{Li}) + D_{lgi} * (L_{1a} - (0.5 * L_1))$$

Where:

$M_{dl}(L_{1a})$ = Moment due to dead load at L_{1a} distance from support for interior girder.

$V_{dl}(L_{1a})$ = Shear due to dead load at L_{1a} distance from support for interior girder.

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain dead load moment and shear

For center-to-center support length (L_1) greater than 12m

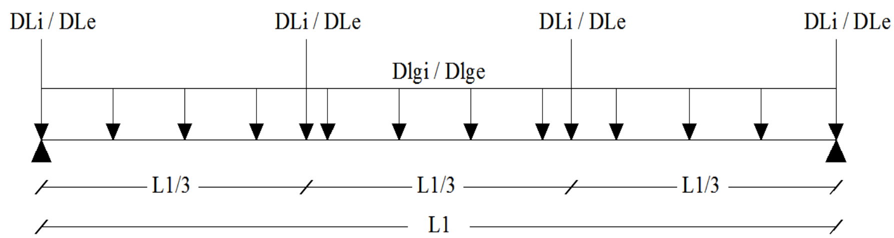

Figure 4.12 Dead-Load Placement for Interior/Exterior Girder (L_1> 12m)

a) Interior Girder

For $L_{1a} \leq L_1/3$

$$M_{dl}(L_{1a}) = (L_{1a} * (D_{Li} + (0.5 * D_{lgi} * L_1))) - (0.5 * D_{lgi} * L_{1a}^{\,2})$$

$$V_{dl}(L_{1a}) = D_{Li} + (D_{lgi} * ((0.5 * L_1) - L_{1a}))$$

For $L_{1a} > L_1/3$

$$M_{dl}(L_{1a}) = (L_{1a} * (0.5 * D_{lgi} * L_1)) + (D_{Li} * L_1) - (0.5 * D_{lgi} * L_{1a}^{\,2})$$

$$V_{dl}(L_{1a}) = D_{lgi} * ((0.5 * L_1) - L_{1a})$$

Where:

$M_{dl}(L_{1a})$ = Moment due to dead load at L_{1a} distance from support for interior girder.

$V_{dl}(L_{1a})$ = Shear due to dead load at L_{1a} distance from support for interior girder.

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain dead load moment and shear.

Distribution Factors for Live-Load Moment and Shear

The distribution factors for moments and shears for both internal and external girders are obtained from the empirical formula provided in AASHTO LRFD Manual, 2005.

Distribution Factors for Moment

1. Interior beams with concrete decks

$$A_{db} = (t_s * (R_{wt} * 10^3)) + (N_g * (b_{min} * (D_w - t_s)))$$
$$e_{gi} = (0.5 * t_s) + (0.5 * (D_w - t_s))$$
$$I_{db} = N_g * (1/12) * b_{min} * (D_w - t_s)^3$$
$$K_{gi} = I_{db} + (A_{db} * (e_{gi})^2)$$

One design lane loaded

$$Mgi1 = 0.06 + ((G_s * 10^3)/4300)^{0.4} * (G_s/L_1)^{0.3} * (K_{gi}/((L_1 * 10^3) * (t_s)^3))^{0.1}$$

Two or more design lane loaded

$$Mgi2 = 0.075 + ((G_s * 10^3)/2900)^{0.6} * (G_s/L_1)^{0.2} * (K_{gi}/((L_1 * 10^3) * (t_s)^3))^{0.1}$$
$$Mgi = 2 * (Max(Mgi1, Mgi2))$$

Where:

Mgi is distribution factor for moment of interior beams

2. Exterior beams with concrete decks

One design lane loaded-lever rule

$$d_1 = 600 - ((C_x * 10^3) + (0.5 * b_{min}))$$
$$d_2 = (G_s * 10^3) - d_1 - 1800$$
$$Mge1 = 1.2 * (d_2 + ((G_s * 10^3) - d_1))/(G_s * 10^3)$$

Two or more design lane loaded

$$ex = Max((0.77 + ((C_w + C_x - (0.5 * (b_{min}/10^3)) - P_d)/2.8)), 1)$$
$$Mge2 = ex * Mgi2$$
$$Mge = Max(Mge1, (2 * Mge2))$$

Where:

Mge is distribution factor for moment of exterior beams

Distribution Factors for Shear

1. Interior beams

One design lane loaded

$$Mgvi1 = 0.36 + ((G_s * 10^3)/7600)$$

Two or more design lane loaded

$$Mgvi2 = 0.2 + ((G_s * 10^3)/3600) - ((G_s * 10^3)/10700)^2$$
$$Mgvi = 2 * (Max(Mgvi1, Mgvi2))$$

Where:

Mgvi is distribution factor for shear of interior beams

2. Exterior beams

One design lane loaded-lever rule

$$Mgve1 = 1.2 * (d_2 + ((G_s * 10^3) - d_1))/(G_s * 10^3)$$

Two or more design lane loaded

$$ev = 0.6 + ((C_w + C_x - (0.5 * (b_{min}/10^3)) - P_d)/3)$$
$$Mgve2 = ev * Mgvi2$$
$$Mgve = Max(Mgve1, (2 * Mgve2))$$

Where:

Mgve is distribution factor for shear of exterior beams

Influence Lines for Shear Forces and Bending Moments Due to Live Loads

The concentrated and distributed live loads per effective flange width of girder is given by:

Design Truck Load: P_1 = 1.33*18.125; P_2 = 1.33*72.5

(One line of wheel truck load of 18.125 and 72.5 kN, respectively)

Design Tandem Load: P_3 = 1.33*55

(One line of wheel tandem load of 55 kN, 1.33 is an impact factor)

Design Lane Load: W = 9.3 kN/m

$$K = P_1/P_2$$

1. Shear Force and Bending Moment for Truck and Lane Load

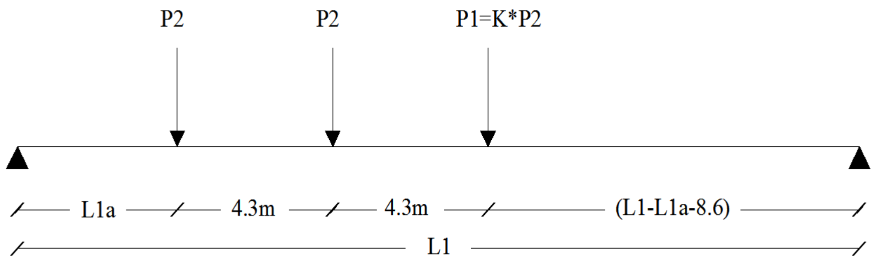

Figure 4.13 Truck Moving to the Right

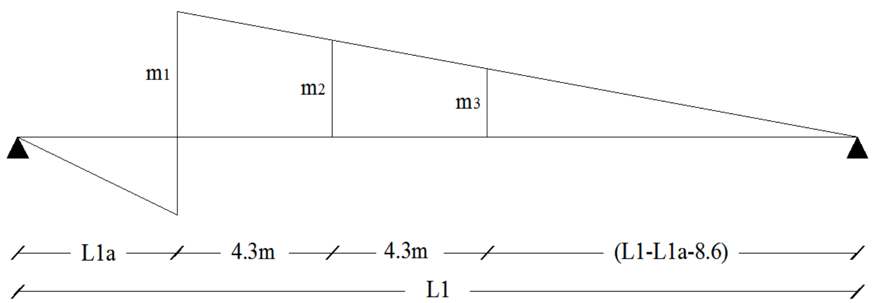

Figure 4.13.1 IL for Shear Force at “L_{1a}” Distance from End Support

Influence line coefficients for shear force

$$m_1 = (L_1 - L_{1a})/L_1$$
$$m_2 = (L_1 - L_{1a} - 4.3)/L_1$$
$$m_3 = (L_1 - L_{1a} - 8.6)/L_1$$
$$V_{trl}\ (L_{1a}) = P_2 * (m_1 + m_2 + (K * m_3)) + (0.5 * W * (L_1 - L_{1a}) * m_1)$$
$$= (P_2/L_1) * [(L_1 - L_{1a}) + (L_1 - L_{1a} - 4.3) + (K * (L_1 - L_{1a} - 8.6))] + (0.5 * W * (L_1 - L_{1a})^2 / L_1)$$

Where:

V_{trl} (L_{1a}) = Shear force at a distance L_{1a} due to truck and lane loads

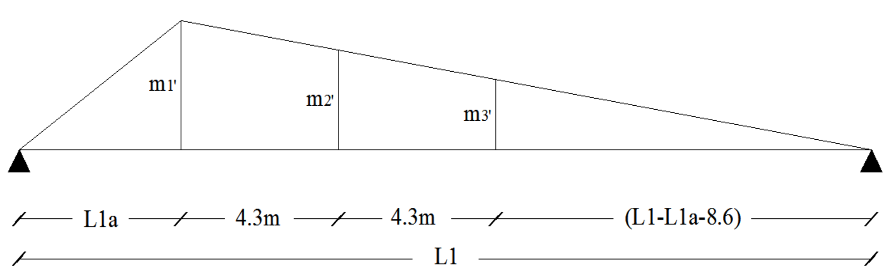

Figure 4.13.2 IL for Bending Moment at “L_{1a}” Distance from End Support

Influence line coefficients for bending moment

$$m_1' = L_{1a} * (L_1 - L_{1a})/L_1$$
$$m_2' = L_{1a} * (L_1 - L_{1a} - 4.3)/L_1$$
$$m_3' = L_{1a} * (L_1 - L_{1a} - 8.6)/L_1$$
$$M_{trl1}(L_{1a}) = P_2 * (m_1' + m_2' + (K * m_3')) + (0.5 * W * L_1 * m_1')$$
$$= (P_2 * L_{1a}/L_1) * [(L_1 - L_{1a}) + (L_1 - L_{1a} - 4.3) + (K * (L_1 - L_{1a} - 8.6))] + (0.5 * W * L_{1a} * (L_1 - L_{1a}))$$

Where:

$M_{trl1}(L_{1a})$ = Bending moment at a distance L_{1a} due to truck and lane load

If the value in bracket is negative, then it is taken as zero. It implies that the wheel is out of the span.

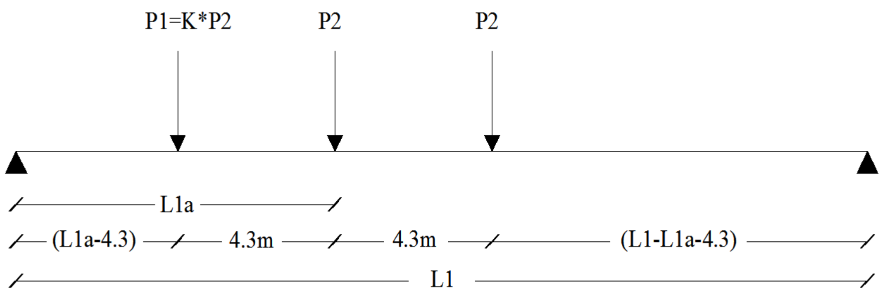

Figure 4.14 Truck Moving to the Left

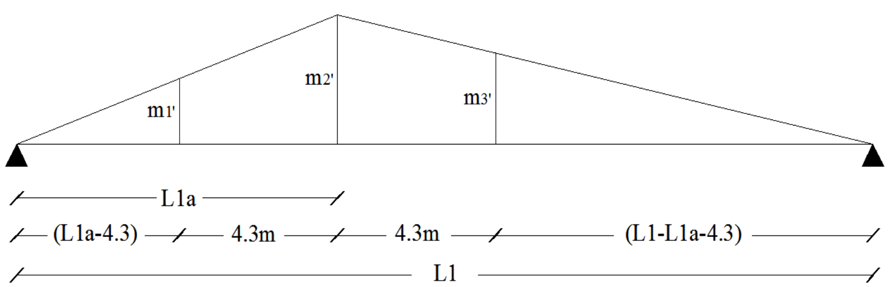

Figure 4.14.1 IL for Bending Moment at "L_{1a}" Distance from End Support

Influence line coefficients for bending moment

$$m_1' = (L_{1a} - 4.3) * (L_1 - L_{1a})/L_1$$
$$m_2' = L_{1a} * (L_1 - L_{1a})/L_1$$
$$m_3' = L_{1a} * (L_1 - L_{1a} - 4.3)/L_1$$
$$M_{trl2}(L_{1a}) = P_2 * (m_1' + m_{2'} + (K * m_3')) + (0.5 * W * L_{1a} * (L_1/(L_{1a} - 4.3)) * m_1')$$
$$= (P_2 * /L_1) * [((L_{1a} - 4.3) * (L_1 - L_{1a})) + (L_{1a} * ((L_1 - L_{1a}) + (K * (L_1 - L_{1a} - 4.3))))] +$$
$$(0.5 * W * L_{1a} * (L_1 - L_{1a}))$$

Where:

$M_{trl2}(L_{1a})$ = Bending moment at a distance L_{1a} due to truck and lane load

If the value in bracket is negative, then it is taken as zero. It implies that the wheel is out of the span.

2. <u>Shear Force and Bending Moment for Tandem and Lane Load</u>

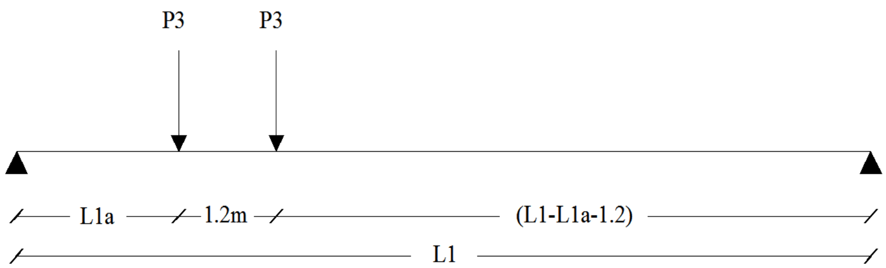

Figure 4.15 Tandem Load

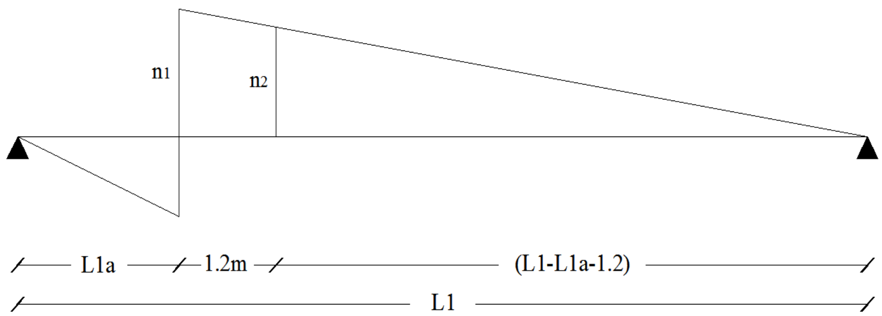

Figure 4.15.1 IL for Shear Force at “L_{1a}” Distance from End Support

Influence line coefficients for shear force

$$n_1 = (L_1 - L_{1a})/L_1$$
$$n_2 = (L_1 - L_{1a} - 1.2)/L_1$$
$$V_{tml}(L_{1a}) = P_3 * (n_1 + n_2) + (0.5 * W * (L_1 - L_{1a}) * n_1)$$
$$= (P_3/L_1) * ((L_1 - L_{1a}) + (L_1 - L_{1a} - 1.2)) + (0.5 * W * (L_1 - L_{1a})^2 / L_1)$$

Where:

V_{tml} (L_{1a}) = Shear force at a distance L_{1a} due to tandem and lane load

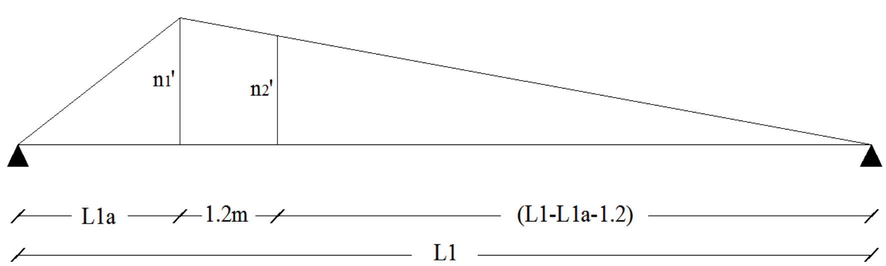

Figure 4.15.2 IL for Bending Moment at “L_{1a}” Distance from End Support

Influence line coefficients for bending moment

$$n_1' = L_{1a} * (L_1 - L_{1a})/L_1$$
$$n_2' = L_{1a} * (L_1 - L_{1a} - 1.2)/L_1$$

$$M_{tml}(L_{1a}) = P_3*(n_1' + n_2') + (0.5*W*L_1*n_1')$$
$$= (P_3*L_{1a}/L_1)*((L_1 - L_{1a}) + (L_1 - L_{1a} - 1.2)) + (0.5*W*L_{1a}*(L_1 - L_{1a}))$$

Where:

M_{tml} (L_{1a}) = Bending moment at a distance L_{1a} due to tandem and lane load

If the value in bracket is negative, then it is taken as zero. It implies that the wheel is out of the span.

In the above derivation of concentrated and distributed live load due to truck, tandem, and lane loads, L_{1a} is the location of the rear/middle wheel from the support and it is given by the equation:

$$L_{1a} = (0.01 + ((a_{11} - 1)*0.1))*L_1$$

Where:

a_{11} = 1, 2, 3, 4, 5, and 6

L_1 = Center-to-center length of the bridge under consideration

The effects of forces due to concentrated and distributed live loads are calculated at all distances L_{1a}.

The computation stops at mid span plus 1m, 2m, and 3m for L_1 lesser or equal to 14m, 24m, and 36m respectively. The allowance is provided to consider the occurrence of the absolute maximum moment Somewhere in the mid span.

Among the above calculated results of live load effects, the maximum effects are the one which govern the design.

4.3.4 Design Procedures

Design of Deck Slab

Factored Design Moment and Shear

In the strength I limit state, the load modifier η_i is 1.0 and the load factors for permanent loads γ_p is taken at its maximum value if the force effects are additive and at its minimum value if it subtract from the dominant force effect. The factor for the live load force effects is 1.75 [7].

$$R_{200} = (1.25*(R_{200d} + R_{200o} + R_{200b})) + (1.5*R_{200w}) + (1.75*1.33*R_{200el}) \quad \ldots\ldots(4.5-1)$$
$$M_{200} = 0.8*[(1.25*(M_{200o} + M_{200b})) + (1.5*M_{200w}) + (1.75*1.33*M_{200ol})] \ldots\ldots(4.5-2)$$
$$M_{204} = (1.25*M_{204d}) + (0.9*(M_{204o} + M_{204b})) + (1.5*M_{204w}) + (1.75*1.33*M_{204pl}) \quad \ldots\ldots(4.5-3)$$
$$M_{300} = 0.8*[(1.25*M_{300d}) + (0.9*(M_{300o} + M_{300b})) + (1.5*M_{300w}) + (1.75*1.33*M_{300nl})] \ldots(4.5-4)$$

Where:

R_{200} is the maximum reaction on the exterior girder (kN)

M_{200} is the maximum overhang moment at the face of support (kN-m /m)

M_{204} is the maximum span/ positive moment (kN-m /m)

M_{300} is the maximum support/ negative moment at the face of support (kN-m /m)

Unfactored Design Moment and Shear

In the service I limit state, the load modifier η_i is 1.0 and the load factors for dead and live load are 1.0.

$$M_{204s} = M_{204d} + M_{204o} + M_{204b} + M_{204w} + (1.33 * M_{204pl}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.6-1)$$

$$M_{300s} = M_{300d} + M_{300o} + M_{300b} + M_{300w} + (1.33 * M_{300nl}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.6-2)$$

Where:

M_{204s} = maximum positive moment for service limit state investigation (kN-m/m)

M_{300s} = maximum negative moment for service limit state investigation (kN-m/m)

Checking the Adequacy of the Section

The section is checked for the maximum design moment whether the initial thickness of the deck slab under consideration is sufficed or not.

For diameter of bars less than 20 mm

$$t_{ic} = \sqrt{((Max(M_{204}, M_{300}) * 10^3 * F_c)/(7.796 * (F_c - 6.389)))} + 25 + (0.5 * dd)$$

For diameter of bars greater or equal to 20 mm

$$t_{ic} = \sqrt{((Max(M_{204}, M_{300}) * 10^3 * F_c)/(6.889 * (F_c - 5.645)))} + 25 + (0.5 * dd)$$

If $t_{ic} < t_s$, then the assumed depth suffice. Otherwise, the section has to be revised until the depth requirement is satisfied.

Reinforcement Computation

The effective concrete depth for positive and negative bending is different because of different cover requirements.

For reinforcement computation, negative moment may be taken at face of support. The webs are b_{min} wide. A minimum web width of the girders is 200mm [3].

$$B_{cs} = Max((1.5 * db), (1.5 * 25.4), 38)$$

$$C_{cbs} = B_{cs} + db$$

Web width for T-girder bridge

$$b_{11} = 2 * (40 + dst + (0.5 * db)) + (3 * C_{cbs}) \qquad \text{for } L1 < 12m$$

$$b_{11} = 2 * (40 + dst + (0.5 * db)) + (3 * C_{cbs}) + (2 * db) \qquad \text{for } L1 \geq 12m$$

Web width for Box-girder bridge

$$b_{11} = 2*(25 + dst + (0.5*db)) + C_{cbs} \qquad \text{for } L1 < 12m$$

$$b_{11} = 2*(25 + dst + (0.5*db)) + C_{cbs} + (2*db) \qquad \text{for } L1 \geq 12m$$

$$b_{min} = Max(b_{11}, 200) \quad (mm) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.7)$$

Where:

B_{cs} = clear distance between parallel bars in a layer (mm)

C_{cbs} = center to center distance between parallel bars in a layer (mm)

b_{min} = the minimum web width (mm)

1. **Positive Moment Reinforcement**

Using dd bars, the effective depth becomes:

$$d_p = t_s - 25 - (0.5*dd)$$

$$F_r = 0.63*\sqrt{(0.8*F_c)}$$

$$I_{cra} = (1/12)*10^3*t_s^{\ 3}$$

$$Y_t = 0.5*t_s$$

$$M_{cr} = (1/10^6)*(F_r*I_{cra}/Y_t)$$

$$M_{up} = Max(M_{204}, Min((1.2*M_{cr}), ((1.33*M_{204}))))$$

$$A_{s1} = (M_{up}*10^6)/(0.828*F_y*d_p) \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.8)$$

$$M_{np} = (R_f*A_{s1}*F_y*(d_p - (0.5*(A_{s1}*F_y)/(0.85*(0.8*F_c)*10^3))))/10^6$$

$$S_i = Min((10^3*((0.25*\pi*dd^2)/A_{s1})), (1.5*t_s), 450)$$

$$A_{pp} = (250*\pi*dd^2)/S_i$$

$$A_{dp} = (A_{pp}*F_y)/(0.85*(0.8*F_c)*10^3)$$

Where:

d_p = effective depth for positive reinforcement (mm)

M_{cr} = cracking moment (kN-m)

M_{up} = factored design positive moment (kN-m/m)

A_{s1} = area of positive moment reinforcing bars (mm^2/m)

M_{np} = section capacity for positive moment (kN-m/m)

S_i = spacing of main reinforcing bars (mm)

A_{pp} = area of positive moment reinforcing bars provided (mm^2/m)

A_{dp} = depth of concrete under compression (mm)

2. Negative Moment Reinforcement

Using dd bars, the effective depth becomes:

$$d_n = t_s - 60 - (0.5 * dd)$$
$$M_{un} = Max(M_{300}, Min((1.2 * M_{cr}), ((1.33 * M_{300})))$$
$$A_{s2} = (M_{un} * 10^6)/(0.828 * F_y * d_n) \quad \ldots\ldots\ldots (4.9)$$
$$M_{nn} = (R_f * A_{s2} * F_y * (d_n - (0.5 * (A_{s2} * F_y)/(0.85 * (0.8 * F_c) * 10^3))))/10^6$$
$$S_e = Min((10^3 * ((0.25 * \pi * dd^2)/A_{s2})), (1.5 * t_s), 450)$$
$$A_{pn} = (250 * \pi * dd^2)/S_e$$
$$A_{dn} = (A_{pn} * F_y)/(0.85 * (0.8 * F_c) * 10^3)$$

Where:

d_n = effective depth for negative reinforcement (mm)

M_{un} = factored design negative moment (kN-m/m)

A_{s2} = area of negative moment reinforcing bars (mm^2/m)

M_{nn} = section capacity for negative moment (kN-m/m)

S_e = spacing of main reinforcing bars (mm)

A_{pn} = area of negative moment reinforcing bars provided (mm^2/m)

A_{dn} = depth of concrete under compression (mm)

3. Distribution Reinforcement

Secondary reinforcement is placed in the bottom of the slab to distribute wheel loads in the longitudinal direction of the bridge to the primary reinforcement in the transverse direction. The required area is a percentage of the primary positive moment reinforcement [7].

$$S_x = (G_s * 10^3) - b_{min}$$
$$P_e = Min((3840/\sqrt{(S_x)}), 67)$$
$$A_{di} = (P_e * A_{pp})/100$$
$$S_{di} = Min((\pi * dd1^2 * 250/A_{di}), 250)$$

Where:

S_x = clear spacing between girders (mm)

P_e = percentage of distribution reinforcement

A_{di} = area of longitudinal reinforcing bars provided (mm^2/m)

S_{di} = spacing of distribution reinforcing bars (mm)

dd1 = diameter of distribution reinforcing bars (mm)

4. **Shirnkage and Temperature Reinforcement**

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes [3].

The area of reinforcing bars in each direction shall satisfy:

$$A_{st} = 0.5 * 0.11 * ((10^3 * t_s)/F_y)$$

$$S_{st} = \mathrm{Min}((10^3 * ((0.25 * \pi * d1t^2)/A_{st})), (3 * t_s), 450)$$

Where:

A_{st} = area of temperature reinforcing bars (mm^2/m)

S_{st} = spacing of temperature reinforcing bars (mm)

d1t = diameter of temperature reinforcing bars (mm)

Investigation of Serviceability Limit State

1. **Durability**

It is assumed that concrete materials and construction procedures provide adequate concrete cover, non-reactive aggregates, thorough consolidation, adequate cement content, low water/cement ratio, thorough curing, and air-entrained concrete.

Cover for unprotected main reinforcing steel deck surface subjected to tire wear: 60mm

Bottom of cast-in-place deck slab: 25mm

2. **Control of Cracking**

Flexural cracking is controlled by limiting the bar spacing in the reinforcement closest to the tension face under service load stress.

Elastic-cracked transformed section analysis required to check crack control. The modular ratio n = E_s/E_c transforms the section properties of steel to equivalent properties of concrete [3].

The equivalent concrete area for positive and negative reinforcements are n*A_{pp} and n*A_{pn}, respectively.

Check Positive Moment Reinforcement

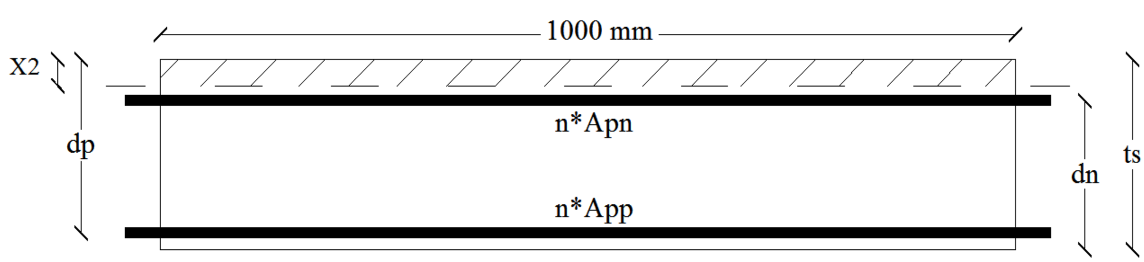

Figure 4.16 Positive Moment Cracked Section

$$n = E_s/E_c$$

For $X_2 \le (t_s - d_n)$

$$X_2 = ((-1*n*(A_{pp} + A_{pn})) + ((n*(A_{pp} + A_{pn}))^2 - 2000*n*((A_{pn}*(d_n - t_s)) - (A_{pp}*d_p)))^{0.5})/10^3$$

$$I_{cr} = (333.33*X_2^{\,3}) + (n*A_{pn}*(t_s - d_n - X_2)^2) + (n*A_{pp}*(d_p - X_2)^2)$$

For $X_2 > (t_s - d_n)$

$$X_2 = ((-1*((n-1)*A_{pn} + (n*A_{pp}))) + (((n-1)*A_{pn} + (n*A_{pp}))^2 - 2000*((n-1)*((A_{pn}*d_n) - (A_{pn}*t_s)) - n*A_{pp}*d_p))^{0.5})/10^3$$

$$I_{cr} = (333.33*X_2^{\,3}) + ((n-1)*A_{pn}*(X_2 + d_n - t_s)^2) + (n*A_{pp}*(d_p - X_2)^2)$$

$$F_s = (M_{204s}*(d_p - X_2)*n*10^6)/I_{cr}$$

$$B_{sp} = 1.0 + ((t_s - d_p)/(0.7*d_p))$$

$$S_{maxp} = ((123000*Y_{ec})/(B_{sp}*F_s)) - (2*(t_s - d_p))$$

Where:

n = modular steel ratio

X_2 = neutral axis depth (mm)

I_{cr} = moment of inertia of transformed cracked section (mm^4/mm)

F_s = tensile stress in the bottom reinforcement at the service limit state (MPa)

S_{maxp} = maximum spacing of positive moment reinforcement for crack control (mm)

If $S_i > S_{maxp}$, then the spacing of reinforcing bars has to be reduced to provide distribution of reinforcement for controlling flexural cracks.

Check Negative Moment Reinforcement

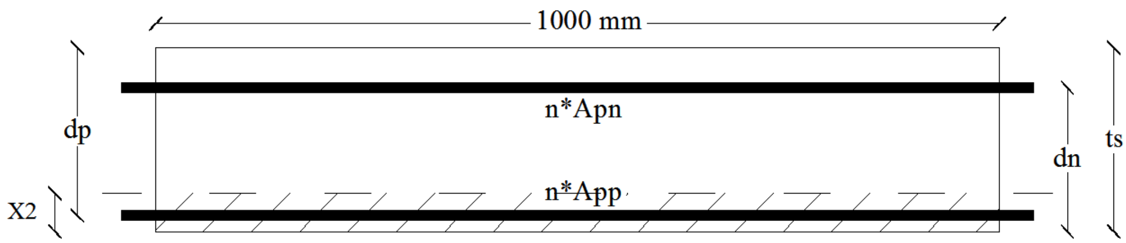

Figure 4.17 Negative Moment Cracked Section

For $X_3 \le (t_s - d_p)$

$$X_3 = ((-1*n*(A_{pp} + A_{pn})) + ((n*(A_{pp} + A_{pn}))^2 + 2000*n*((A_{pp}*(t_s - d_p)) + (A_{pn}*d_n)))^{0.5})/10^3$$

$$I_{cre} = (333.33*X_3^{\,3}) + (n*A_{pn}*(d_n - X_3)^2) + (n*A_{pp}*(t_s - d_p - X_3)^2)$$

For $X_3 > (t_s - d_p)$

$$X_3 = ((-1*((n-1)*A_{pp} + (n*A_{pn}))) + (((n-1)*A_{pp} + (n*A_{pn}))^2 - 2000*((n-1)*((A_{pp}*d_p) -$$

$$(A_{pp} * t_s)) - n * A_{pn} * d_n))^{0.5}) / 10^3$$

$$I_{cre} = (333.33 * X_3^{\,3}) + (n * A_{pn} * (d_n - X_3)^2) + ((n-1) * A_{pp} * (X_3 + d_p - t_s)^2)$$

$$F_{se} = ((0.8 * M_{300s}) * (d_n - X_3) * n * 10^6) / I_{cre}$$

$$Bs_n = 1.0 + ((t_s - d_n - 12) / (0.7 * (d_n - 12)))$$

$$S_{maxn} = ((123000 * Y_{ec}) / (B_{sn} * F_{se})) - (2 * (t_s - d_n))$$

Where:

X_3 = neutral axis depth (mm)

I_{cre} = moment of inertia of transformed cracked section (mm^4/mm)

F_{se} = tensile stress in the top reinforcement at the service limit state (MPa)

S_{maxn} = maximum spacing of negative moment reinforcement for crack control (mm)

If $S_e > S_{maxn}$, then the spacing of reinforcing bars has to be reduced to provide distribution of reinforcement for controlling flexural cracks.

3. Fatigue Limit State

Fatigue need not be investigated for concrete decks in multigirder applications [3].

Design of Deck Overhang

Reinforcement Computation

Using dd bars, the effective depth becomes:

$$d_{od} = t_s - 50 - (0.5 * dd)$$

$$R_3 = (0.8 * F_c / (1.176 * F_y)) * (1 - (1 - (2.352 * \mathrm{Max}(M_{200}, \mathrm{Min}((1.2 * M_{cr}), (1.33 * M_{200}))) * 10^3) /$$

$$(R_f * d_{od}^{\,2} * (0.8 * F_c)))^{0.5})$$

$$A_{s3} = R_3 * d_{od} * 10^3$$

$$S_{e3} = \mathrm{Min}((10^3 * ((0.25 * \pi * dd^2) / A_{s3})), (1.5 * t_s), 450)$$

$$A_{p3} = (250 * \pi * dd^2) / S_{e3}$$

Where:

d_{od} = effective depth for overhang main reinforcement (mm)

R_3 = steel reinforcement ratio

A_{s3} = area of overhang moment reinforcing bars (mm^2/m)

S_{e3} = spacing of overhang main reinforcing bars (mm)

A_{p3} = area of overhang moment reinforcing bars provided (mm^2/m)

Design of Longitudinal Girders

Factored Design Moment and Shear

The load factors and load combinations are according to AASHTO LRFD Bridge Design Specifications, 2005, section 3.4. The load combination to be used for design is strength I limit state.

a) Interior Girder

At a distance L_{1a} from the support, MD (factored design moment) is given by the equation:

$$MD(L_{1a}) = 1.25 * M_{dl}(L_{1a}) + 1.75 * [Max(M_{trl}(L_{1a}), M_{tml}(L_{1a}))] \ldots\ldots\ldots\ldots\ldots (4.10)$$

Mu, the maximum factored design moment is the one which is maximum of MD (L_{1a}).

Similarly, at a distance L_{1a} from the support, VD (factored design shear) is given by the equation:

$$VD(L_{1a}) = 1.25 * V_{dl}(L_{1a}) + 1.75 * [Max(V_{trl}(L_{1a}), V_{tml}(L_{1a}))] \ldots\ldots\ldots\ldots\ldots (4.11)$$

Vu, the maximum factored design shear is the one which is maximum of VD (L_{1a}).

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain the design moment and shear.

Unfactored Design Moment and Shear

In the service I limit state, the load modifier η_i is 1.0 and the load factors for dead and live load are 1.0.

a) Interior Girder

At a distance L_{1a} from the support, MDu (unfactored design moment) is given by the equation:

$$MDu(L_{1a}) = M_{dl}(L_{1a}) + Max(M_{trl}(L_{1a}), M_{tml}(L_{1a}))$$

Mus, the maximum unfactored design moment is the one which is maximum of MDu (L_{1a}).

Similarly, at a distance L_{1a} from the support, VDu (unfactored design shear) is given by the equation:

$$VDu(L_{1a}) = V_{dl}(L_{1a}) + Max(V_{trl}(L_{1a}), V_{tml}(L_{1a}))$$

Vus, the maximum unfactored design shear is the one which is maximum of VDu (L_{1a}).

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain the design moment and shear.

Reinforcement Computation

1. **Main Reinforcing Bars**

   a) Interior Girder

Using db bars, the effective depth (d_{ppr}) at L_{1a} distance from support becomes:

$$d_{ppr}\,(nb) = D_w - d_{pp}\,(nb)$$

$$A_{si}\,(nb) = nb * (0.25 * \pi * db^2)$$

$$R_8\,(nb) = A_{si}\,(nb) / (b_{ii} * d_{ppr}\,(nb))$$

$$A_{ig}\,(nb) = (A_{si}\,(nb) * F_y) / (0.85 * (0.8 * F_c) * b_{ii})$$

If A_{ig} (nb) ≤ ts, then a rectangular beam analysis will be considered otherwise it is a T-beam.

$$M_{ni}\,(nb) = (R_f * A_{si}\,(nb) * F_y * (d_{ppr}\,(nb) - (0.5 * A_{ig}\,(nb))))/10^6$$

For T-beam analysis, A_{ig} (nb) > ts

$$A_{sfi} = (0.85 * (0.8 * F_c) * (b_{ii} - b_{min}) * t_s)/F_y$$

$$M_{ni1}\,(nb) = (A_{sf}i * F_y * (d_{ppr}\,(nb) - (0.5 * t_s)))/10^6$$

$$A_{sc}\,(nb) = A_{si}\,(nb) - A_{sfi}$$

$$A_{xg}\,(nb) = A_{sc}\,(nb) * F_y / (0.85 * (0.8 * F_c) * b_{min})$$

$$M_{ni2}\,(nb) = (A_{sc}\,(nb) * F_y * (d_{ppr}\,(nb) - (0.5 * A_{xg}\,(nb))))/10^6$$

$$M_{ni}\,(nb) = R_f * (M_{ni1}\,(nb) + M_{ni2}\,(nb))$$

All the above parameters are computed at every L_{1a} distances to curtail bars at these distances.

Where:

d_{ppr} (nb) = effective depth of interior girder at the n^{th} bar iteration (mm)

A_{si} (nb) = area of reinforcement at the n^{th} bar iteration (mm^2)

R_8 (nb) = steel reinforcement ratio at the n^{th} bar iteration

A_{ig} (nb) = compression depth of concrete in the slab at the n^{th} bar iteration (mm)

A_{sfi} = area of steel required to balance the longitudinal compression in the overhang (mm^2)

A_{sc} (nb) = area of steel required to balance the longitudinal compression in the web (mm^2)

A_{xg} (nb) = compression depth of concrete in the web at the n^{th} bar iteration (mm)

M_{ni} (nb) = section capacity of the girder for flexure (kN-m/m)

   b) Exterior Girder

Such a similar approach is carried out to exterior girder.

2. Skin Reinforcement

For relatively deep flexural members, reinforcement should also be distributed in the vertical faces in the tension region to control cracking in the web. If the effective depth exceeds 900mm, longitudinal skin reinforcements are to be uniformly distributed over a height of d/2 nearest the tensile reinforcement [3].

Area of skin reinforcement required on each side face shall satisfy:

$$A_{ski} = \mathrm{Min}[(0.001*(d_{ppi} - 760)), (0.25*A_{s70})]$$

$$S_{kio} = \mathrm{Min}((10^3 *((0.25*\pi*dsk^2)/A_{ski})), (d_{ppi}/6), 300)$$

Where:

A_{ski} = area of skin reinforcing bars (mm^2/m)

S_{kio} = spacing of skin reinforcing bars (mm)

dsk = diameter of skin reinforcing bars (mm)

Investigation of Serviceability Limit State

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

1. Durability

Concrete cover for unprotected main reinforcing steel in cast-in-place girder: 50mm

2. Control of Cracking

To control flexural cracking of the concrete, tension reinforcements shall be well distributed with in maximum flexural zone.

a) Interior Girder

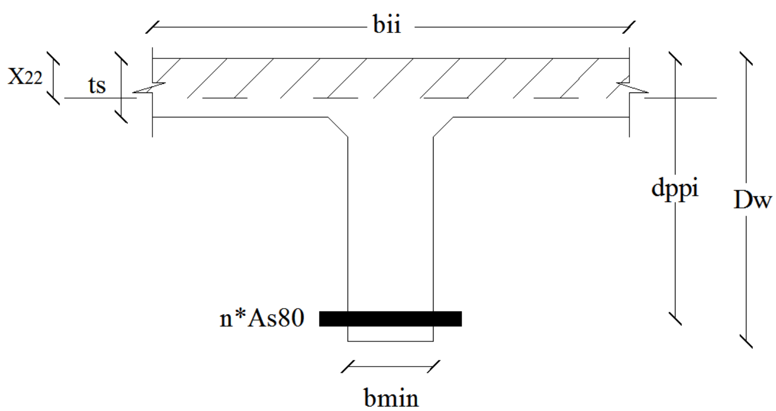

Figure 4.18 Elastic-Cracked Transformed Section

The equivalent concrete area is $n*A_{s80}$.

$$A_{s80} = (Mus*10^6)/(0.6*F_y*0.875*d_{ppi})$$

$$n_{ri} = INT(A_{s80}/(0.25*\pi*db^2)$$

$$\text{For } X_{22} \le t_s$$

$$X_{22} = ((-1*n*A_{s80}) + ((n*A_{s80})^2 + (2*(b_{ii}*n*A_{s80}*d_{ppi})))^{0.5})/b_{ii}$$

$$I_{cr1} = (b_{ii}*X_{22}{}^3)/3 + (n*A_{s80}*(d_{ppi} - X_{22})^2)$$

$$\text{For } X_{22} > t_s$$

$$X_{22} = ((-1*((b_{ii}*t_s) - (b_{min}*t_s) + (n*A_{s80}))) + (((b_{ii}*t_s) - (b_{min}*t_s) + (n*A_{s80}))^2 - (4*(0.5*b_{min})*(0.5*((b_{min}*t_s{}^2) - (b_{ii}*t_s{}^2)) - (n*A_{s80}*d_{ppi}))))^{0.5}) / (2*(0.5*b_{min}))$$

$$I_{cr1} = ((b_{ii}*t_s{}^3)/12) + ((b_{ii}*t_s)*(X_{22} - (0.5*t_s))^2) + ((b_{min}*(X_{22} - t_s)^3)/12) + ((b_{min}*(X_{22} - t_s))*(0.5*(X_{22} - t_s))^2) + n*A_{s80}*(d_{ppi} - X_{22})^2$$

$$F_{s1} = (Mus*(d_{ppi} - X_{22})*n*10^6)/I_{cr1}$$

$$B_{si} = 1.0 + (d_{p1}/(0.7*(D_w - d_{p1})))$$

$$S_{maxi} = ((123000*Y_{ec})/(B_{si}*F_{s1})) - (2*d_{p1})$$

Where:

X_{22} = neutral axis depth from top extreme fiber (mm)

I_{cr1} = moment of inertia of transformed cracked section (mm^4/mm)

F_{s1} = tensile stress in main reinforcing bars at the service limit state (MPa)

S_{maxi} = maximum spacing of main reinforcing bars for crack control (mm)

Components shall be so proportioned that the spacing of main reinforcing bars at service limit state, S_{maxi} doesn't exceed. If the spacing of main reinforcing bar in a layer (C_{cbs}) greater than the spacing limit (S_{maxi}) the section has cracked.

b) Exterior Girder

A similar procedure is carried for exterior girder for flexural crack control.

3. Deformations

Service-load deformations may not be a potential source of collapse mechanisms but usually cause some undesirable effects, such as the deterioration of wearing surfaces and local buckling in concrete slab which could impair serviceability and durability [8].

Deflection and camber calculations shall consider dead load, live load, erection loads, concrete creep and shrinkage, and steel relaxation [3].

Dead Load Camber

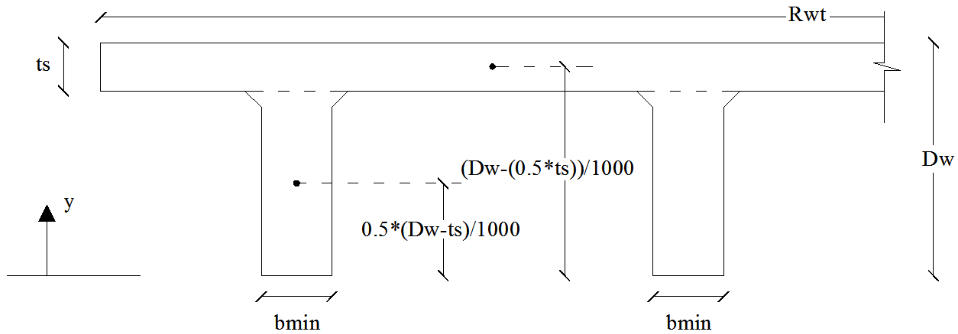

Figure 4.19 Uncracked / Gross cross section

For simplicity of calculations, the slab surface is assumed level, i.e. without crossfall. The center of gravity is calculated from bottom of girder.

$$Y_{sg} = (((R_{wt} * (t_s/10^3)) * ((D_w - (0.5 * t_s))/10^3)) + (((N_g * b_{min} * (D_w - t_s))/10^6) * ((0.5 * (D_w - t_s))/10^3)))/((R_{wt} * (t_s/10^3)) + ((N_g * b_{min} * (D_w - t_s))/10^6))$$

$$I_{gt} = ((1/12) * R_{wt} * (t_s/10^3)^3) + ((R_{wt} * (t_s/10^3)) * (ABS(((D_w - (0.5 * t_s))/10^3) - Y_{sg}))^2) + ((N_g/12) * (b_{min}/10^3) * ((D_w - t_s)/10^3)^3) + (((N_g * b_{min} * (D_w - t_s))/10^6) * (ABS(((0.5 * (D_w - t_s))/10^3) - Y_{sg}))^2)$$

$$M_{crt} = (F_r * I_{gt} * 10^3)/Y_{sg}$$

For $L_1 \leq 12$m

$$M_{ad} = (((2 * D_{lge}) + ((N_g - 2) * D_{lgi})) * L_1^2)/8 + ((2 * D_{Le}) + D_{Li}) * L_1)/4$$

For $L_1 > 12$m

$$M_{ad} = (((2 * D_{lge}) + ((N_g - 2) * D_{lgi})) * L_1^2)/8 + (((2 * D_{Le}) + (2 * D_{Li})) * L_1)/3$$

$$Y_{st} = (((R_{wt} * (t_s/10^3)) * ((D_w - (0.5 * t_s))/10^3)) + ((((2 * b_{min} * (K_{fed} - t_s))/10^6) + (((N_g - 2) * b_{min} * (K_{fid} - t_s))/10^6)) * ((D_W - t_s - (0.5 * (K_{fid} - t_s)))/10^3)) + (((((2 * A_{s60}) + ((N_g - 2) * A_{s70})) * n)/10^6) * ((D_w - d_{ppi})/10^3)))/((R_{wt} * (t_s/10^3)) + (((2 * b_{min} * (K_{fed} - t_s))/10^6) + (((N_g - 2) * b_{min} * (K_{fid} - t_s))/10^6)) + ((((2 * A_{s60}) + ((N_g - 2) * A_{s70})) * n)/10^6))$$

$$I_{crd} = ((1/12) * R_{wt} * (t_s/10^3)^3) + ((R_{wt} * (t_s/10^3)) * (ABS(((D_w - (0.5 * t_s))/10^3) - Y_{st}))^2) + ((N_g/12) * (b_{min}/10^3) * ((K_{fid} - t_s)/10^3)^3) + ((((2 * b_{min} * (K_{fed} - t_s))/10^6) + (((N_g - 2) * b_{min} * (K_{fid} - t_s))/10^6)) * (ABS(((D_w - t_s - (0.5 * (K_{fid} - t_s)))/10^3) - Y_{st}))^2) + ((((2 * A_{s60}) + ((N_g - 2) * A_{s70})) * n)/10^6) * (ABS(((D_w - d_{ppi})/10^3) - Y_{st}))^2$$

$$I_{eff} = Min(((((M_{crt}/M_{ad})^3) * I_{gt}) + ((1 - ((M_{crt}/M_{ad})^3)) * I_{crd})), I_{gt})$$

Where:

Y_{sg} = centroid of the cross section measured from bottom of girder (m)

I_{gt} = gross moment of inertia of about the centroid (m^4)

M_{crt} = cracking moment (kN-m)

M_{ad} = maximum moment in the component at stage for which deformation is computed (kN-m)

Y_{st} = centroid of transformed cross section measured from bottom of girder (m)

I_{crd} = moment of inertia of transformed cracked section (m^4)

I_{eff} = effective moment of inertia (m^4)

Immediate (instantaneous) deflection may be computed taking the moment of inertia as either the effective moment of inertia, or the gross moment of inertia. It is assumed to have maximum value where the moment is maximum in a component at stage deformation is computed.

$$D_{ld1} = (5*((2*D_{lge}) + ((N_g - 2)*D_{lgi}))*L_1^{\ 4})/(384*(E_c*10^3)*I_{eff})$$

$$D_{ld2} = (((2*D_{Le}) + ((N_g - 2)*D_{Li}))*L_1^{\ 3})/(48*(E_c*10^3)*I_{eff})$$

$$D_{ld} = D_{ld1} + D_{ld2}$$

Where:

D_{ld1} = dead load deflection due to girder and deck slab (mm)

D_{ld2} = deflection due to diaphragm dead load (mm)

D_{ld} = maximum dead load deflection (mm)

The long-term deflection due to creep and shrinkage may be taken as the immediate deflection (D_{ld}) multiplied the following factor:

- If the instantaneous deflection is based on I_{gt}: 4.0
- If the instantaneous deflection is based on I_{eff}: $3.0-1.2*(A_s'/A_s) \geq 1.6$

Live Load Deflection

When calculating the vehicular deflection, it should be taken as the larger of that resulting from the design truck alone or that resulting from 25% of the design truck taken together with the design lane load. All of the design lanes should be loaded and all of the girders may be assumed to deflect equally in supporting the load [7].

a) **Deflection due to truck load**

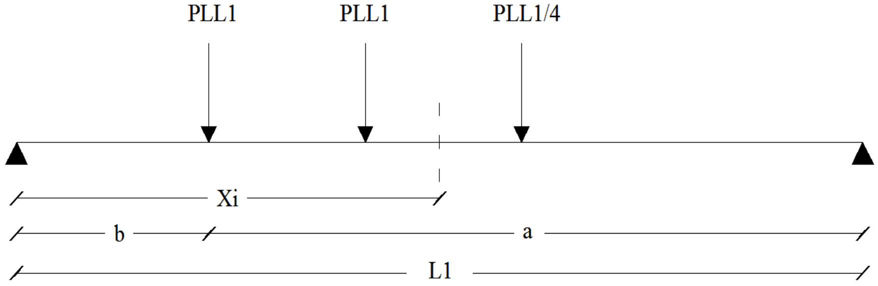

Figure 4.20 Truck Load Deflection

$$P_{ll1} = 1.33 * N_L * 145 * m$$

Where:

P_{ll1} = design truck load (kN)

The maximum deflection of the bridge due to truck load occurs at a wheel load position where moment is maximum. Thus, the deflection at the point of maximum moment due to each truck axle load at a distance "b" from the left support is given by:

$$d_{xi} = ((P_{ll1} * b * X_i)/(6 * E_c * I_c * L_1)) * [L_1^2 - b^2 - X_i^2]$$

where:

d_{xi} = deflection due to each truck load

Using the method of superposition, the total live load deflection due to truck load (d_{eft}) is the sum of each deflections, d_{xi}'s.

b) **Deflection due to lane load**

$$w_{l1} = 1.33 * N_L * 9.3 * m$$

$$d_{efl} = (5 * w_{l1} * L_1^4)/(384 * E_c * I_{eff} * 10^3)$$

Where:

w_{l1} = design lane load

d_{efl} = deflection due to lane load

Deflection of the bridge due to live load will be maximum of deflection from 25% of truck load plus lane load and truck load alone. Hence, compare the value obtained with the allowable limit.

$$d_{eftl} = (0.25 * d_{eft}) + d_{efl}$$

$$d_{lmax} = \mathrm{Max}(d_{eft}, d_{eftl})$$

$$d_{max} = (L_1 * 10^3)/800$$

Where:

d_{eft}, d_{eftl} = maximum deflection from truck load alone and 25% of truck load plus lane load (mm)

d_{lmax} = maximum live load deflection of the bridge (mm)

d_{max} = maximum allowable live load deflection of the bridge (mm)

Investigation of Fatigue Limit State

Fatigue limit states are used to limit stress in steel reinforcements to control concrete crack growth under repetitive truck loading in order to prevent early fracture failure before the design service life of a bridge [8].

According to AASHTO LRFD Bridge Design Specifications, 2005, the fatigue load:

- One design truck with constant spacing of 9000 mm between 145000-N axles.
- Dynamic load allowance: IM = 15%
- Distribution factor for one traffic lane shall be used
- Multiple presence factor of 1.2 shall be removed

Influence Lines for Bending Moments Due to Truck Load

Design Truck Load: P_{1f} = 1.15*18.125; P_2 = 1.15*72.5, one line of wheel truck load of 18.125 and 72.5 kN, respectively

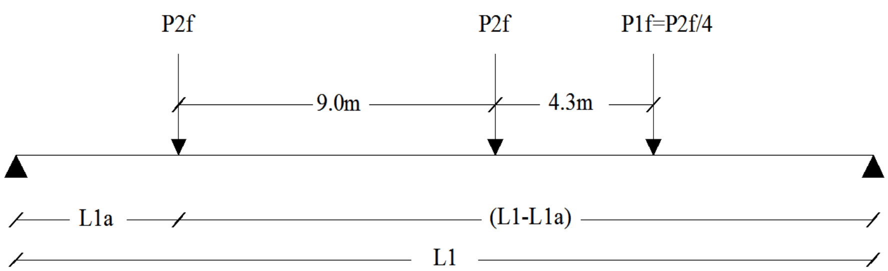

Figure 4.21 Truck Moving to the Right

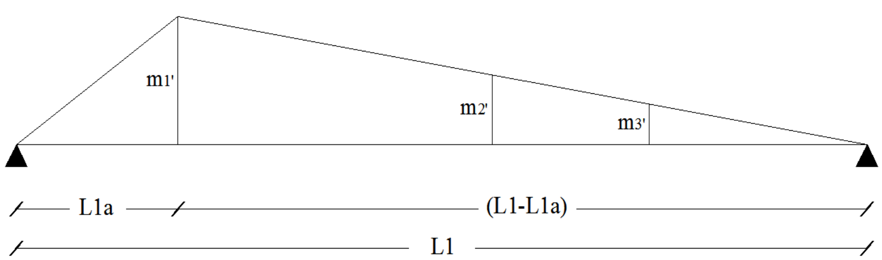

Figure 4.21.1 IL for Bending Moment at "L_{1a}" Distance from End Support

Influence line coefficients for bending moment

$$m_1' = L_{1a} * (L_1 - L_{1a})/L_1$$

$$m_2' = L_{1a} * (L_1 - L_{1a} - 9.0)/L_1$$

$$m_3' = L_{1a} * (L_1 - L_{1a} - 13.3)/L_1$$

$$M_{trf1}(L_{1a}) = P_{2f} * (m_1' + m_2' + ((P_{1f} / P_{2f}) * m_3'))$$

$$= (P_{2f} * L_{1a}/L_1) * [(L_1 - L_{1a}) + (L_1 - L_{1a} - 9.0) + ((P_{1f} / P_{2f}) * (L_1 - L_{1a} - 13.3))] * Mgi1$$

Where:

$M_{trf1}(L_{1a})$ = Bending moment at a distance L_{1a} due to truck load

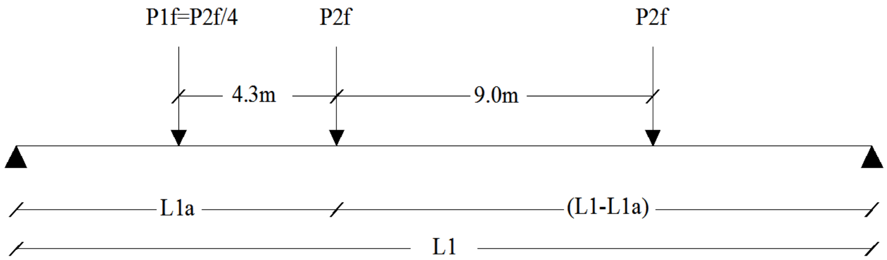

Figure 4.22 Truck Moving to the Left

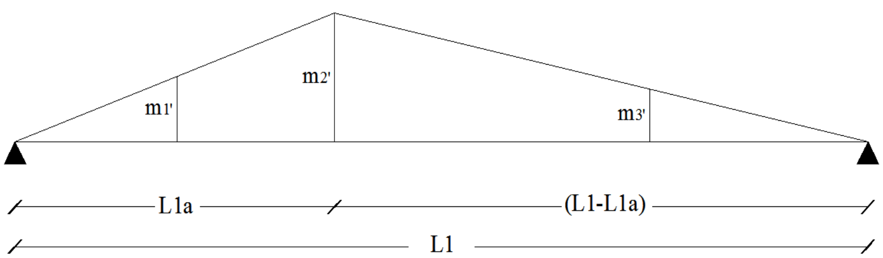

Figure 4.22.1 IL for Bending Moment at "L_{1a}" Distance from End Support

Influence line coefficients for bending moment

$$m_1' = (L_{1a} - 4.3) * (L_1 - L_{1a})/L_1$$

$$m_2' = L_{1a} * (L_1 - L_{1a})/L_1$$

$$m_3' = L_{1a} * (L_1 - L_{1a} - 9.0)/L_1$$

$$M_{trf2}(L1a) = P_{2f} * (m_1' + m_2' + ((P_{1f} / P_{2f}) * m_3'))$$

$$= (P_{2f}/L_1) * [((L_{1a} - 4.3) * (L_1 - L_{1a})) + (L_{1a} * ((L_1 - L_{1a}) + ((P_{1f} / P_{2f}) * (L_1 - L_{1a} - 9.0))))] * Mgi1$$

Where:

$M_{trf2}(L_{1a})$ = Bending moment at a distance L_{1a} due to truck load

Maximum of the two live load bending moments is used for fatigue stress computation

$$M_{trf}(L_{1a}) = Max(M_{trf1}(L_{1a}), M_{trf2}(L_{1a}))$$

$$K_{fi} = ((R_{70} * n) + (0.5 * (t_s/d_{ppi})^2))/((R_{70} * n) + (t_s/d_{ppi}))$$

$$K_{fid} = K_{fi} * d_{ppi}$$

For $K_{fid} < t_s$

$$j_{fi} = 1 - (K_{fi}/3)$$

For $K_{fid} \geq t_s$

$$j_{fi} = (6 - (6 * (t_s/d_{ppi})) + (2 * (t_s/d_{ppi})^2) - ((t_s/d_{ppi})^3 / (2 * R_{70} * n)))/(6 - (3 * (t_s/d_{ppi})))$$

$$F_{smin} = (M_{dlm} * 10^6)/(A_{s70} * j_{fi} * d_{ppi})$$

$$F_{smax} = ((M_{dlm} + M_{trfi}) * 10^6)/(A_{s70} * j_{fi} * d_{ppi})$$

$$F_{sran} = F_{smax} - F_{smin}$$

$$F_{fr} = 145 - (0.33 * F_{smin}) + 55 * (r/h)$$

Where:

F_{smin} = minimum fatigue stress due to dead load (N/mm^2)

F_{smax} = maximum fatigue stress due to total load (N/mm^2)

F_{sran} = fatigue stress range (N/mm^2)

F_{fr} = limit for fatigue stress range (N/mm^2)

3. Shear Reinforcement

a) Interior Girder

$$d_{v1} = \mathrm{Max}(((D_w - d_{p1}) - ((A_{s7(1)} * F_y)/(0.588 * \sqrt{(0.8 * f_c)} * b_{ii}))), (0.9 * (D_w - d_{p1})), (0.72 * D_w))$$

$$V_{cc1} = (0.083 * 2 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3$$

$$V_{dv1} = (1.1236 * VD(0)) * (((0.5 * L_1) - (0.5 * W_{Rs}) - (d_{v1}/10^3))/(0.5 * L_1)$$

$$V_{si1} = \mathrm{ABS}((V_{dv1}/R_f) - V_{cc1})$$

$$S_{sm1} = (0.5 * \pi * dst^2 * F_y)/(0.083 * b_{min} * \sqrt{(0.8 * f_c)})$$

For $V_{dv1} < ((0.125 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3)$

$$S_{sm2} = \mathrm{Min}((0.8 * d_{v1}), 600)$$

For $V_{dv1} \geq ((0.125 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3)$

$$S_{sm2} = \mathrm{Min}((0.4 * d_{v1}), 300)$$

$$S_{sh1} = (0.5 * \pi * dst^2 * F_y * d_{v1})/(V_{si1} * 10^3)$$

$$S_{rip} = \mathrm{Min}(S_{sh1}, S_{sm1}, S_{sm2})$$

Where:

d_{v1} = effective shear depth

V_{cc1} = the nominal shear strength provided by the concrete

V_{dv1} = design maximum factored shear force at L_{1a} distance from center of support

V_{si1} = the nominal shear strength provided by the shear reinforcement

S_{rip} = spacing of shear reinforcing bars provided

b) Exterior Girder

Such a similar approach is carried out to exterior girder.

The source program for Analysis and Design of T-Girder Bridge is developed on the basis of the procedure outlined above.

4.4 Analysis of Internal Forces and Design of Box-Girder Bridges

4.4.1 Design Loads

All types of load taken in to account in the analysis of T-girder bridge are also considered here. The only additional load considered in the analysis of box-girder bridge is self-weight of the bottom slab.

4.4.2 Load Distribution

The permanent load is distributed to the girders by assigning to each all loads from superstructure elements within half the distance to the adjacent girder where as the live load is distributed using load distribution tables and the “lever rule”.

4.4.3 Analysis Procedures

Analysis of Deck Slab

An approximate analysis of strips perpendicular to girders is considered acceptable.

The extreme positive moment in any deck panel between girders shall be taken to apply to all positive moment regions. Similarly, the extreme negative moment over any girder shall be taken to apply to all negative moment regions [3].

A similar procedure as did for T-Girder Bridge is investigated here.

Analysis of Bottom Slab

Thickness of bottom slab (t_{sb}) [3]:

$$t_{sb} = Max[140, (((G_s * 10^3) - b_{min})/16), (80 + (2 * db) + dst + Max(25.4, db))] \ldots\ldots\ldots\ldots (4.12)$$

where:

t_{sb} = thickness of bottom slab (mm)

G_s = girder spacing (m)

db= diameter of girder main reinforcing bars (mm)

Analysis of Longitudinal Girders

Structural Depth

The minimum structural depth for simple span I-beams is given as follows:

$$D_w = 0.06 * L_1 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4.13)$$

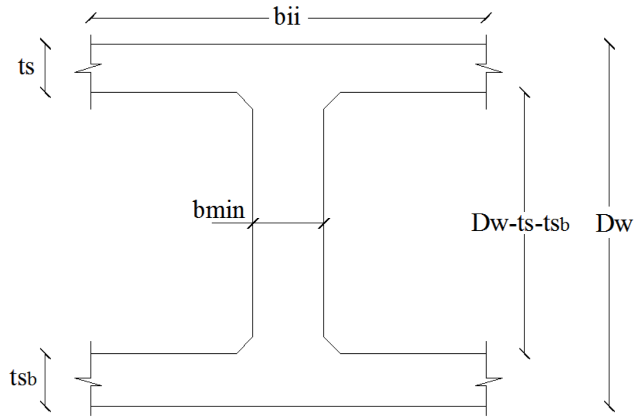

Figure 4.23 Cross Section of an Interior Girder

Effective Flange Width

It depends on effective span length, which is defined as the distance between points of permanent load inflection for continuous beams [7].

a) Interior girder

$$b_{ii} = \mathrm{Min}((0.25*(10^3 * L_1)), ((12*t_s) + b_{min}), (10^3 * G_s)) \ldots\ldots\ldots\ldots\ldots(4.14)$$

b) Exterior girder

$$b_{eii} = (0.5*b_{ii}) + \mathrm{Min}((0.125*(10^3 * L_1)), ((6*t_s) + (0.5*b_{min})), (10^3 * (C_x + C_w) - (0.5*b_{min}))) \ldots(4.15)$$

Dead Load Force Effects

a) Interior girder

$$D_{lsbi} = b_{ii} * t_{sb} * \gamma_c * 9.81*10^{-9}$$

$$D_{lw} = b_{min} * (D_w - t_s - t_{sb}) * \gamma_c * 9.81*10^{-9}$$

$$D_{lgi} = D_{lsi} + D_{lsbi} + D_{lw} + D_{lwi} + D_{rp} + 0.50 \text{ kN/m}$$

$$D_{Le} = (25*0.5*((G_s*10^3) - b_{min})*10^{-3})*((D_t*(D_w - t_s - t_{sb})*10^{-6}) + (3*0.5*0.1*0.1))$$

$$D_{Li} = 2*(25*0.5*((G_s*10^3) - b_{min})*10^{-3})*((D_t*(D_w - t_s - t_{sb})*10^{-6}) + (4*0.5*0.1*0.1))$$

Where:

D_{lsbi} is dead load from bottom slab (kN/m)

D_{lw} is dead load from web (kN/m)

D_{lgi} is the total dead load on interior girder (kN/m)

D_{Le} is diaphragm dead load on exterior girder (kN)

D_{Li} is diaphragm dead load on interior girder (kN)

b) Exterior girder

$$D_{lsbe} = (0.5*(b_{eii} + b_{min}))*t_{sb}*\gamma_c*9.81*10^{-9}$$
$$D_{lc} = (10^3*C_w)*(10^3*C_d)*\gamma_c*9.81*10^{-9}$$
$$D_{loh} = (((10^3*C_w)*t_{oh}) + (((10^3*C_x) - (0.5*b_{min}))*t_s))*\gamma_c*9.81*10^{-9}$$
$$D_{lge} = D_{lse} + D_{lsbe} + D_{lw} + D_{lwe} + D_{lc} + D_{loh} + D_{rp} + 0.375 \text{ kN/m}$$

Where:

D_{lsbe} is dead load from bottom slab (kN/m)

D_{lc} is dead load from curb (kN/m)

D_{loh} is dead load from overhang slab (kN/m)

D_{lge} is the total dead load on exterior girder (kN/m)

The bending moment and shear force due to dead loads at every distance L_{1a} from the support is computed for both interior and exterior girders.

$$L_{1a} = (0.01 + ((a_{11} - 1)*0.1))*L_1$$

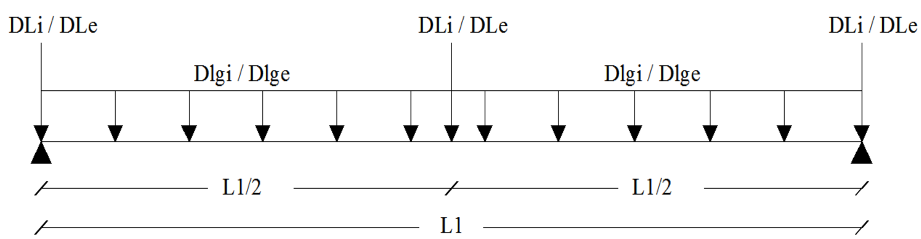

Figure 4.24 Dead-Load Placement for Interior/Exterior Girder

a) Interior girder

For $L_{1a} \le L_1/2$

$$M_{dl}(L_{1a}) = 0.5*L_{1a}*((D_{lgi}*(L_1 - L_{1a})) + D_{Li})$$
$$V_{dl}(L1a) = D_{lgi}*((0.5*L_1) - L_{1a}) + (0.5*D_{Li})$$

For $L_{1a} > L_1/2$

$$M_{dl}(L_{1a}) = 0.5*(D_{lgi}*(L_{1a}^{\ 2} - (L_1*L_{1a})) + D_{Li}*(L_{1a} - L_1))$$
$$V_{dl}(L_{1a}) = (0.5*D_{Li}) + D_{lgi}*(L_{1a} - (0.5*L_1))$$

Where:

$M_{dl}(L_{1a})$ = Moment due to dead load at L_{1a} distance from support for interior girder.

$V_{dl}(L_{1a})$ = Shear due to dead load at L_{1a} distance from support for interior girder.

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain dead load moment and shear

Distribution Factors for Live-Load Moment and Shear

Distribution Factors for Moment

1. Interior beams with concrete decks

One design lane loaded

$$Mgi1 = 1.75 + ((G_s * 10^3)/1100) * (300/(L_1 * 10^3))^{0.35} * (1/N_c)^{0.45}$$

Two or more design lane loaded

$$Mgi2 = (13/N_c)^{0.3} * ((G_s * 10^3)/430) * (1/(L_1 * 10^3))^{0.25}$$

$$Mgi = 2 * (Max(Mgi1, Mgi2))$$

Where:

Mgi is distribution factor for moment of interior beams

2. Exterior beams with concrete decks

One design lane loaded-lever rule

$$d_1 = 600 - ((C_x * 10^3) + (0.5 * b_{min}))$$

$$d_2 = (G_s * 10^3) - d_1 - 1800$$

$$Mge1 = 1.2 * (d_2 + ((G_s * 10^3) - d_1))/(G_s * 10^3)$$

Two or more design lane loaded

$$We = (0.5 * G_s * 10^3) + ((C_w * 10^3) + (C_x * 10^3) - (0.5 * b_{min}))$$

$$Mge2 = We/4300$$

$$Mge = Max(Mge1, (2 * Mge2))$$

Where:

Mge is distribution factor for moment of exterior beams

Distribution Factors for Shear

1. Interior beams

One design lane loaded

$$Mgvi1 = ((G_s * 10^3)/2900)^{0.6} * (D_w/(L_1 * 10^3))^{0.1}$$

Two or more design lane loaded

$$Mgvi2 = ((G_s * 10^3)/2200)^{0.9} * (D_w/(L_1 * 10^3))^{0.1}$$

$$Mgvi = 2 * (Max(Mgvi1, Mgvi2))$$

Where:

Mgvi is distribution factor for shear of interior beams

2. Exterior beams

One design lane loaded-lever rule

$$Mgve1 = 1.2 * (d_2 + ((G_s * 10^3) - d_1))/ (G_s * 10^3)$$

Two or more design lane loaded

$$ev = 0.64 + ((C_w + C_x - (0.5 * (b_{min} / 10^3)) - P_d)/3.8)$$

$$Mgve2 = ev * Mgvi2$$

$$Mgve = Max(Mgve1, (2 * Mgve2))$$

Where:

Mgve is distribution factor for shear of exterior beams

Influence Lines for Shear Forces and Bending Moments Due to Live Loads

The influence lines for shear force and moment for truck, tandem, and lane loads of a Box-Girder bridge is similar to that of T-Girder bridge. Moreover, the influence line coefficients, the equation for shear force and moments are the same.

The effects of forces due to concentrated and distributed live loads are calculated at all distances L_{1a}.
The computation stops at mid span plus 1m, 2m, and 3m for L_1 greater or equal to 14m, 18m, and 26m respectively. The allowance is provided to consider the occurrence of the absolute maximum moment Somewhere in the mid span.

Among the above calculated results of live load effects, the maximum effects are the one which govern the design.

4.4.4 Design Procedures

Design of Deck Slab

The design of deck slab for Box-Girder bridge is similar to that of a T-Girder bridge. Positive moment, negative moment, distribution, and temperature reinforcements are provided on the basis of the analysis output.

After providing all the required reinforcements, investigation of service limit state proceeds to check the design section is safe against durability and cracking as did for T-Girder bridge.

Design of Bottom Slab

A uniformly distributed reinforcement of 0.4% of the flange area shall be placed in the bottom slab parallel to the girder span, either in single or double layers [3].

$$A_{sbl} = (0.4/100)/\,(t_{sb} * 10^3)$$
$$S_{a3} = Min((1000 * ((0.25 * \pi * dbs^2)/A_{sbl})), 450)$$
$$A_{sp3} = (250 * \pi * dbs^2)/S_{a3}$$

Where:

A_{sbl} = area of reinforcement in the longitudinal direction (mm^2/m)

S_{a3} = spacing of reinforcement in the longitudinal direction (mm)

A_{sp3} = area of provided reinforcement in the longitudinal direction (mm^2/m)

A uniformly distributed reinforcement of 0.5% of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with maximum spacing of 450 mm [3].

$$A_{sbt} = (0.5/100)/\,(t_{sb} * 10^3)$$
$$S_{b3} = 2 * (Min((1000 * ((0.25 * \pi * dbs^2)/A_{sbt})), 450))$$
$$A_{sp4} = (250 * \pi * dbs^2)/S_{b3}$$

Where:

A_{sbt} = area of reinforcement in the transversal direction (mm^2/m)

S_{b3} = spacing of top and bottom reinforcement in the transversal direction (mm)

A_{sp4} = area of provided reinforcement in the transversal direction (mm^2/m)

Design of Longitudinal Girders

Factored Design Moment and Shear

a) Interior Girder

At a distance L_{1a} from the support, MD (factored design moment) and VD (factored design shear) are given by the equations:

$$MD(L_{1a}) = 1.25 * M_{dl}(L_{1a}) + 1.75 * [Max(M_{trl}(L_{1a}), M_{tml}(L_{1a}))] \quad \ldots\ldots\ldots\ldots\ldots(4.16)$$

Mu, the maximum factored design moment is the one which is maximum of MD (L_{1a}).

$$VD(L_{1a}) = 1.25 * V_{dl}(L_{1a}) + 1.75 * [Max(V_{trl}(L_{1a}), V_{tml}(L_{1a}))] \quad \ldots\ldots\ldots\ldots\ldots(4.17)$$

Vu, the maximum factored design shear is the one which is maximum of VD (L_{1a}).

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain the design moment and shear.

Unfactored Design Moment and Shear

a) Interior Girder

At a distance L_{1a} from the support, MDu (unfactored design moment) and VDu (unfactored design shear) are given by the equations:

$$MDu(L_{1a}) = M_{dl}(L_{1a}) + Max(M_{trl}(L_{1a}), M_{tml}(L_{1a}))$$

Mus, the maximum unfactored design moment is the one which is maximum of MDu (L_{1a}).

$$VDu(L_{1a}) = V_{dl}(L_{1a}) + Max(V_{trl}(L_{1a}), V_{tml}(L_{1a}))$$

Vus, the maximum unfactored design shear is the one which is maximum of VDu (L_{1a}).

b) Exterior Girder

A similar procedure is carried for exterior girder to obtain the design moment and shear.

Reinforcement Computation

1. **Main Reinforcing Bars**

a) Interior Girder

Using db bars, the effective depth (d_{ppr}) at L_{1a} distance from support becomes:

$$d_{ppr}(nb) = D_w - d_{pp}(nb)$$

$$A_{si}(nb) = nb * (0.25 * \pi * db^2)$$

$$R_8(nb) = A_{si}(nb) / (b_{ii} * d_{ppr}(nb))$$

$$A_{ig}(nb) = (A_{si}(nb) * F_y) / (0.85 * (0.8 * F_c) * b_{ii})$$

If A_{ig} (nb) $\leq$ ts, then a rectangular beam analysis will be considered otherwise it is a T-beam.

$$M_{ni}(nb) = (R_f * A_{si}(nb) * F_y * (d_{ppr}(nb) - (0.5 * A_{ig}(nb)))) / 10^6$$

For T-beam analysis, A_{ig} (nb) > ts

$$A_{sfi} = (0.85 * (0.8 * F_c) * (b_{ii} - b_{min}) * t_s) / F_y$$

$$M_{ni1}(nb) = (A_{sf}i * F_y * (d_{ppr}(nb) - (0.5 * t_s))) / 10^6$$

$$A_{sc}(nb) = A_{si}(nb) - A_{sfi}$$

$$A_{xg}(nb) = A_{sc}(nb) * F_y / (0.85 * (0.8 * F_c) * b_{min})$$

$$M_{ni2}(nb) = (A_{sc}(nb) * F_y * (d_{ppr}(nb) - (0.5 * A_{xg}(nb)))) / 10^6$$

$$M_{ni}(nb) = R_f * (M_{ni1}(nb) + M_{ni2}(nb))$$

All the above parameters are computed at every L_{1a} distances to curtail bars at these distances.

Where:

d_{ppr} (nb) = effective depth of interior girder at the n^{th} bar iteration (mm)

A_{si} (nb) = area of reinforcement at the n^{th} bar iteration (mm^2)

R_8 (nb) = steel reinforcement ratio at the n^{th} bar iteration

A_{ig} (nb) = compression depth of concrete in the slab at the n^{th} bar iteration (mm)

A_{sfi} = area of steel required to balance the longitudinal compression in the overhang (mm^2)

A_{sc} (nb) = area of steel required to balance the longitudinal compression in the web (mm^2)

A_{xg} (nb) = compression depth of concrete in the web at the n^{th} bar iteration (mm)

M_{ni} (nb) = section capacity of the girder for flexure (kN-m/m)

b) Exterior Girder

Such a similar approach is carried out to exterior girder.

2. Skin Reinforcement

Longitudinal skin reinforcement required if the effective depth > 900 mm [3].

Investigation of Serviceability Limit State

1. Durability

Concrete cover for unprotected main reinforcing steel in cast-in-place girder: 50mm

2. Control of Cracking

Cracking is controlled by limiting the spacing (C_{cbs}) in the reinforcement under service loads [3]. A similar procedure as has been for T-girder bridge is also considered here too.

3. Deformations

Dead Load Camber

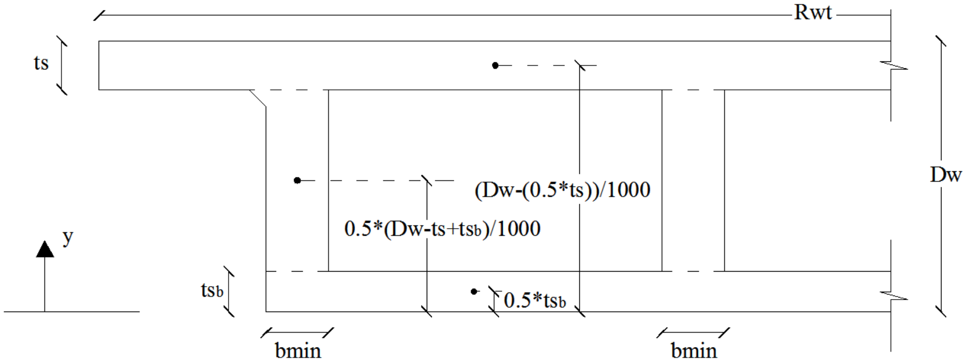

Figure 4.25 Uncracked / Gross cross section

For simplicity of calculations, the slab surface is assumed level, i.e. without crossfall. The center of gravity is calculated from bottom of girder.

$$\begin{aligned}Y_{sg} = {} & (((R_{wt} * (t_s/10^3)) * ((D_w - (0.5 * t_s))/10^3)) + (((N_g * b_{min} * (D_w - t_s - t_{sb}))/10^6) * \\ & ((0.5 * (D_w - t_s + t_{sb}))/10^3))) + (((N_g - 1) * (G_s/10^3) + (b_{min}/10^3)) * (t_{sb}/10^3)) * \\ & (0.5 * (t_{sb}/10^3))/((R_{wt} * (t_s/10^3)) + ((N_g * b_{min} * (D_w - t_s))/10^6) + (((N_g - 1) * (G_s/10^3) + \\ & (b_{min}/10^3)) * (t_{sb}/10^3)))\end{aligned}$$

$$\begin{aligned}I_{gt} = {} & ((1/12) * R_{wt} * (t_s/10^3)^3) + ((R_{wt} * (t_s/10^3)) * (ABS(((D_w - (0.5 * t_s))/10^3) - Y_{sg}))^2) + \\ & ((N_g/12) * (b_{min}/10^3) * ((D_w - t_s - t_{sb})/10^3)^3) + (((N_g * b_{min} * (D_w - t_s - t_{sb}))/10^6) * \\ & (ABS(((0.5 * (D_w - t_s + t_{sb}))/10^3) - Y_{sg}))^2) + ((1/12) * ((N_g - 1) * (G_s/10^3) + (b_{min}/10^3)) * \\ & (t_{sb}/10^3)^3) + (((N_g - 1) * (G_s/10^3) + (b_{min}/10^3)) * (t_{sb}/10^3) * (ABS((0.5 * t_{sb}/10^3) - Y_{sg}))^2)\end{aligned}$$

$$M_{crt} = (F_r * I_{gt} * 10^3)/Y_{sg}$$

$$M_{ad} = (((2 * D_{lge}) + ((N_g - 2) * D_{lgi})) * L_1^2)/8 + (((2 * D_{Le}) + (2 * D_{Li})) * L_1)/4$$

$$\begin{aligned}Y_{st} = {} & (((R_{wt} * (t_s/10^3)) * ((D_w - (0.5 * t_s))/10^3)) + ((((2 * b_{min} * (K_{fed} - t_s))/10^6) + (((N_g - 2) * \\ & b_{min} * (K_{fid} - t_s))/10^6)) * ((D_W - t_s - (0.5 * (K_{fid} - t_s)))/10^3)) + (((((2 * A_{s60}) + ((N_g - 2) * \\ & A_{s70})) * n)/10^6) * ((D_w - d_{ppi})/10^3)))/ ((R_{wt} * (t_s/10^3)) + (((2 * b_{min} * (K_{fed} - t_s))/10^6) + \\ & (((Ng - 2) * bmin * (Kfid - ts))/106)) + ((((2 * As60) + ((Ng - 2) * As70)) * n)/106))\end{aligned}$$

$$\begin{aligned}I_{crd} = {} & ((1/12) * R_{wt} * (t_s/10^3)^3) + ((R_{wt} * (t_s/10^3)) * (ABS(((D_w - (0.5 * t_s))/10^3) - Y_{st}))^2) + \\ & ((N_g/12) * (b_{min}/10^3) * ((K_{fid} - t_s)/10^3)^3) + ((((2 * b_{min} * (K_{fed} - t_s))/10^6) + (((N_g - 2) * \\ & b_{min} * (K_{fid} - t_s))/10^6)) * (ABS(((D_w - t_s - (0.5 * (K_{fid} - t_s)))/10^3) - Y_{st}))^2) + ((((2 * A_{s60}) + \\ & ((N_g - 2) * A_{s70})) * n)/10^6) * (ABS(((D_w - d_{ppi})/10^3) - Y_{st}))^2\end{aligned}$$

$$I_{eff} = Min(((((M_{crt}/M_{ad})^3) * I_{gt}) + ((1 - ((M_{crt}/M_{ad})^3)) * I_{crd})), I_{gt})$$

Where:

Y_{sg} = centroid of the cross section measured from bottom of girder (m)

I_{gt} = gross moment of inertia of about the centroid (m^4)

M_{crt} = cracking moment (kN-m)

M_{ad} = maximum moment in the component at stage for which deformation is computed (kN-m)

Y_{st} = centroid of transformed cross section measured from bottom of girder (m)

I_{crd} = moment of inertia of transformed cracked section (m^4)

I_{eff} = effective moment of inertia (m^4)

Similarly as did for T-girder bridge the same procedure is carried out for computing the immediate and long term deflections.

Live Load Deflection

Vehicular deflection resulting from the larger of the design truck alone or that resulting from 25% of the design truck taken together with the design lane load is compared with the permissible limit.

Investigation of Fatigue Limit State

Design for the fatigue limit state involves limiting the live load stress range produced by the fatigue truck to a value suitable for the number of stress range repetitions expected over the life of the bridge [7].

Fatigue loading consists of one design truck with a constant spacing of 9 m between the 145-kN axles.

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

3. **Shear Reinforcement**

a) **Interior Girder**

$$d_{v1} = \mathrm{Max}(((D_w - d_{p1}) - ((A_{s7(1)} * F_y)/(0.588 * \sqrt{(0.8 * f_c)} * b_{ii}))), (0.9 * (D_w - d_{p1})), (0.72 * D_w))$$

$$V_{cc1} = (0.083 * 2 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3$$

$$V_{dv1} = (1.1236 * VD(0)) * (((0.5 * L_1) - (0.5 * W_{Rs}) - (d_{v1}/10^3))/(0.5 * L_1)$$

$$V_{si1} = ABS((V_{dv1}/R_f) - V_{cc1})$$

$$S_{sm1} = (0.5 * \pi * dst^2 * F_y)/ (0.083 * b_{min} * \sqrt{(0.8 * f_c)})$$

$$\text{For } V_{dv1} < ((0.125 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3)$$

$$S_{sm2} = \mathrm{Min}((0.8 * d_{v1}), 600)$$

$$\text{For } Vdv1 \geq ((0.125 * \sqrt{(0.8 * f_c)} * b_{min} * d_{v1})/10^3)$$

$$S_{sm2} = Min((0.4 * d_{v1}), 300)$$
$$S_{sh1} = (0.5 * \pi * dst^2 * F_y * d_{v1}) / (V_{si1} * 10^3)$$
$$S_{rip} = Min(S_{sh1}, S_{sm1}, S_{sm2})$$

where:

d_{v1} = effective shear depth

V_{cc1} = the nominal shear strength provided by the concrete

V_{dv1} = design maximum factored shear force at L_{1a} distance from center of support

V_{si1} = the nominal shear strength provided by the shear reinforcement

S_{rip} = spacing of shear reinforcing bars provided

b) Exterior Girder

Such a similar approach is carried out to exterior girder.

The source program for Analysis and Design of Box-Girder Bridge is also prepared on the basis of the procedure outlined above.

5 Computer Program for Demarcating T- or Box- Girder Bridge

5.1 Basis of Demarcation

It is being shown that minimum material content alone does not necessarily give the best value or most economical solution in overall terms. Issues such as constructability, repeatability, simplicity, esthetics, maintainability, speed of construction must all be taken in to account.

Nevertheless, for selecting bridge type of the same material, cost comparison will be a useful parameter to select a better optimum. In this regard, the materials used for the construction of the bridges such as total volume of concrete, total amount of reinforcing bars, area of asphalt concrete, area of formwork, and steel pipes for posts and railing or their associated costs are the only costs taken in to consideration. The cost of materials (construction cost) will be used in the computer program, to select and identify the economical span range of the two types of bridges.

5.2 Computer Program for Analysis &Design of T- and Box-Girder Bridges

The developed computer program, which makes use of "FORTRAN 95" programming language software, for comparative study of the analysis and design of reinforced concrete T- and Box- girder bridges takes in to account both theoretical and practical considerations. Moreover, the program is developed following the recently prepared bridge design manual AASHTO LRFD Bridge Design Specifications, 2005 and also reference is made to ERA Bridge Design Manual, 2002.

The program is easily understandable, simple to use, easy to amend or modify, less time taking and effort demanding, and deliver outputs with high degree of accuracy. Furthermore, it is verified for different span length and deck width configurations for both types of bridges and gives satisfactory, comprehensive, and reliable output.

5.3 Algorithm / Flow Chart

5.3.1 Algorithm / Flow Chart for T-Girder Bridge

Design Input Data

The input data consists of;

- Reinforcement steel grade for diameter of bars greater or equal to 20mm and lesser,
- Cubic compressive strength of concrete,
- Concrete density,
- Bituminous density, and
- Modulus of elasticity of steel

Moreover, input data includes initial dimension of the bridge to be designed such as:

- Clear span,
- Roadway width,
- Curb width,
- Curb depth,
- Beam seat width,
- Bitumen thickness,
- Railing dimensions, and
- Post spacing and dimensions

Furthermore, deck slab reinforcements and main, distribution, temperature, skin, and shear reinforcing bars for longitudinal girders are the input data. Material costs are also taken as an input data for cost analysis.

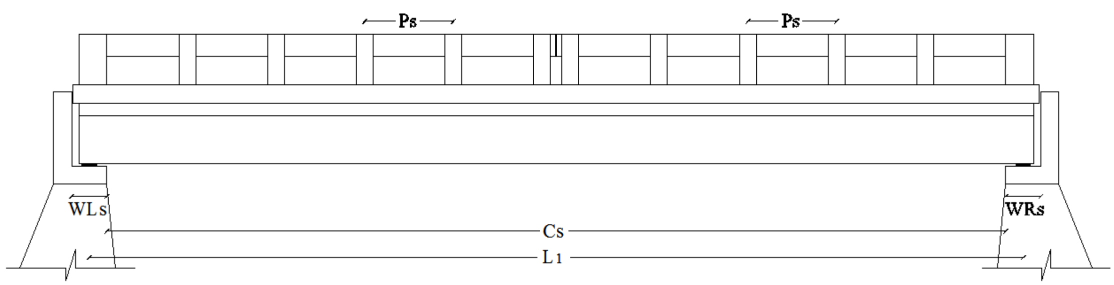

Figure 5.1 Longitudinal Section of a T-/ Box- Girder Bridge

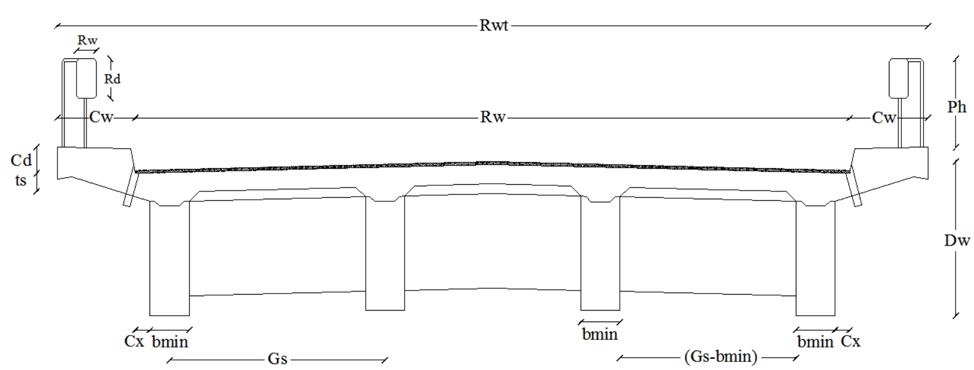

Figure 5.2 T-Girder Bridge Cross Section

Analysis

Given the above input data of material properties and dimension of the bridge to be designed, the following will be computed:

- The thickness of the deck slab is determined from equation (4.1).
- Structural analysis of the deck slab involves taking a continuous strip perpendicular to the girders. The dead load force effects are obtained by analyzing the continuous beam loaded in figure 4.2, 4.3, 4.4, and 4.5 for dead load force effects due to deck slab, future wearing surface, overhang, and barrier loads, respectively. The critical live load effects are obtained by varying the position of the wheel loads as shown in the figure 4.6, 4.7, 4.8, and 4.9.
- Equation (4.5-1), (4.5-2), (4.5-3), and (4.5-4) are used to determine the critical effects due to dead and live loads. Appropriate load factors and load combinations are applied for the selected limit state design from tables provided in AASHTO LRFD Bridge Design Specifications, 2005.

Design

The deck slab is designed for both the critical positive and negative moments obtained from equations (4.5-3) and (4.5-4). The positive moment reinforcement for the deck slab are calculated on the basis of equation (4.8) and negative moment reinforcement computation; equation (4.9) is used. The adequacy of the section for flexure and the assurance of ductility is also checked.

Distribution and temperature reinforcements may be taken as a percentage of the main reinforcements required for positive moment.

The unfactored design moments are determined from equations (4.6-1) and (4.6-2) to investigate the design section for service limit state. If the section is safe against durability and cracking, then the section is sufficed and the design of deck slab is completed.

Once the task of analyzing and designing of deck slab, overhang and curb are accomplished, then the analysis and design of both interior and exterior longitudinal girders proceeds. The minimum structural depth, effective flange width, and minimum web thickness for both interior and exterior girders are determined from equations (4.2), (4.3), (4.4), and (4.7) respectively. The live load force effects due to vehicular loads and lane load is obtained and distributed for the girders with the appropriate distribution factors for shear and moment.

The loading and distribution of dead load for maximum force effects is shown in figure 4.11 and 4.12. Influence lines will be used to determine load positions for maximum live load force effects and magnitude of these effects is as shown from figure 4.13 to figure 4.15.2. Here again load factors, combinations, and modifiers are applied.

The moments and shears obtained from the analysis are factored and combined using equation (4.10) and (4.11) to give the critical force effects for designing the longitudinal girders. Service limit state is investigated to check the sufficiency of the section.

The designed section is investigated for service and fatigue limit state and if the section is safe against durability, cracking, deflection, and fatigue, then the section is sufficed and the design is completed.

Total amount of materials used

After examining and investigating the adequacy of the section for different limit states, the total amount of materials used are obtained by determining all reinforcing bars (Grade 300 & Grade 420), volume of concrete, area of asphalt coat, area of formwork (Class F_1 and F_2 surface finishes for concealed and exposed faces, respectively), and weight of steel pipes.

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

The detail of the above algorithm is shown in a flow chart in Annex II

5.3.2 Algorithm / Flow Chart for Box-Girder Bridge

Design Input Data

The input data consists of: reinforcement steel grade for diameter of bars greater or equal to 20mm and lesser, cubic compressive strength of concrete, concrete density, bituminous density, and modulus of elasticity of steel.

Moreover, input data includes initial dimension of the bridge to be designed such as: clear span, roadway width, curb width, curb depth, beam seat width, bitumen thickness, railing dimensions, and post spacing and dimensions.

Furthermore, deck and bottom slab reinforcements and main, distribution, temperature, skin, and shear reinforcing bars for longitudinal girders are the input data. Material costs are also taken as an input data for cost analysis.

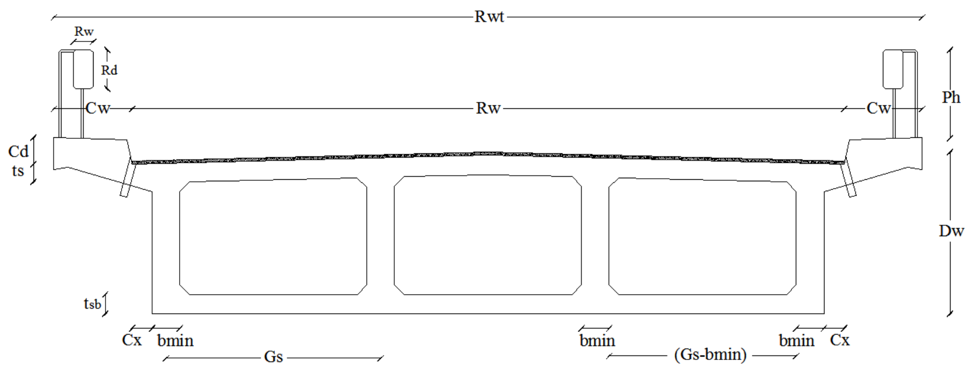

Figure 5.3 Box-Girder Bridge Cross Section

Analysis

Having the material properties and bridge dimensions as an input data, the thickness of the deck slab and bottom slab are computed using equation (4.1) & (4.12) respectively. In addition to this, the followings are computed:

- The dead load force effects are obtained by analyzing the continuous beam strip loaded in figure 4.2, 4.3, 4.4, and 4.5 for dead load force effects due to deck slab, future wearing surface, overhang, and barrier loads. The critical live load effects are obtained by varying the position of the wheel loads as shown in the figure 4.6, 4.7, 4.8, and 4.9.
- Appropriate load factors and load combinations are applied for the selected limit state design from tables provided in AASHTO LRFD Bridge Design Specifications, 2005. Then the critical force effects due to dead and live loads are obtained from equations (4.5-1), (4.5-2), (4.5-3), and (4.5-4)

Design

The positive moment reinforcement for the deck slab are calculated on the basis of equation (4.8) and negative moment reinforcement computation; equation (4.9) is used. The adequacy of the section for flexure and the assurance of ductility is also checked.

Distribution and temperature reinforcements of the deck slab may be taken as a percentage of the main reinforcements required for positive moment.

The reinforcements of the bottom slab in both longitudinal and transversal direction taken as the percentage of bottom flange area.

The unfactored design moments are determined from equations (4.6-1) & (4.6-2) to investigate the design section for service limit state. If the section is safe against durability and cracking, then the section is sufficed and the design of deck slab is completed.

Once the task of analyzing and designing of deck slab, bottom slab, overhang, and curb accomplished, then the analysis and design of both interior and exterior longitudinal girders proceeds. The minimum web thickness, structural depth, and effective flange width for both interior and exterior girders are determined from equations (4.7), (4.13), (4.14), and (4.15) respectively. The live load force effects due to vehicular loads and lane load is obtained and distributed for the girders with the appropriate distribution factors for shear and moment.

The loading and distribution of dead load for maximum force effects is shown in figure 4.24. Influence lines will be used to determine load positions for maximum live load force effects and magnitude of these effects is as shown from figure 4.13 to figure 4.15.2. Here again load factors, combinations, and modifiers are applied.

The critical force effects obtained from equation (4.16) & (4.17) are used to design the longitudinal girders. Service and fatigue limit states are investigated to check the sufficiency of the section. If the section is adequate, then the construction materials are calculated in a similar way did for T-girder bridge and then the total cost for the bridge is determined.

The detail of the above algorithm is shown in a flow chart in Annex II

6 Results and Discussions

6.1 Demonstrative Design Examples

Demonstrative design examples showing the whole process of computation involved in the program, for 24 m span T-and Box-girder bridges, are attached in the Annex III. The summary of necessary outputs obtained from the developed program for a 20m, 22m, and 24m span T- and Box-girder bridges are presented in Table 6.1 and Table 6.2, respectively.

6.2 Application of the Program and Results

6.2.1 Applications and Limitations of the Program

Applications

- The developed computer program will be applicable for Analysis and Design of T- and Box-Girder Bridges for any span length and also perform the selection of the bridge type either T- or Box-Girder on the basis of economy.
- The program shall benefit clients, consulting firms and contractors working in road infrastructure sector in the checking and design review works and simplifies to look matters of optimal solution. Moreover, the department may use this for future research in line with upgrading the program to handle many other aspects of bridge design.

Limitations

While developing the program, the following are only taken in to account

- The cost analysis of bridges considers only construction costs.
- Only the superstructure cost of the bridge is considered for economical analysis since it is assumed that the substructures are almost the same for both T- and Box- girder bridges having the same span length.
- The program is developed for single span bridges

Table 6.1 Summary of Outputs for 20, 22, and 24m of T-Girder Bridges

Span Length (m)	Deck Slab Thick. (mm)	Girder Spacing (m)	Girder Dimension		Reinforcements								Reinforcement Quantity (ton)		Volume of Concrete (m^3)	Area of Formwork (m^2)	
			Web Width (mm)	Depth (mm)	Deck Slab				Longitudinal Girder								
					Positive Rein.	Negative Rein.	Distribution Rein.	Temp. Rein.	Interior Girder		Exterior Girder						
									Main Rein.	Shear Rein.	Main Rein.	Shear Rein.	Grade 420	Grade 300		Class F1	Class F2
20	180	2.2	450	1500	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ12 mm c/c 140mm L=22120mm	ϕ12 mm c/c 440mm L=22120mm	Φ32 mm No. = 12 L=Var.	ϕ12 mm c/c Var. L=3820mm	Φ32 mm No. = 12 L=Var.	ϕ12 mm c/c Var. L=3820mm	5.94.	12.88	114.37	40.9	537.0
22	180	2.2	450	1600	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ12 mm c/c 140mm L=26420mm	ϕ12 mm c/c 440mm L=26420mm	Φ32 mm No. = 13 L=Var.	ϕ12 mm c/c Var. L=4020mm	Φ32 mm No. = 13 L=Var.	ϕ12 mm c/c Var. L=4020mm	7.20.	14.35	128.69	43.0	603.7
24	180	2.2	450	1800	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ12 mm c/c 140mm L=28700mm	ϕ12 mm c/c 440mm L=28700mm	Φ32 mm No. = 14 L=Var.	ϕ12 mm c/c Var. L=4420mm	Φ32 mm No. = 15 L=Var.	ϕ12 mm c/c Var. L=4420mm	8.08.	16.04	149.29	47.3	700.8

Table 6.2 Summary of Outputs for 20, 22, and 24m of Box-Girder Bridges

Span Length (m)	Deck Slab Thick. (mm)	Girder Spacing (m)	Girder Dimension		Reinforcements								Reinforcement Quantity (ton)		Volume of Concrete (m^3)	Area of Formwork (m^2)	
			Web Width (mm)	Depth (mm)	Deck Slab		Bottom Slab		Longitudinal Girder								
									Interior Girder		Exterior Girder						
					Positive Rein.	Negative Rein.	Longitudinal Direction	Transversal Direction	Main Rein.	Shear Rein.	Main Rein.	Shear Rein.	Grade 420	Grade 300		Class F1	Class F2
20	180	2.2	260	1300	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ16 mm c/c 260mm L=22150mm	ϕ16 mm c/c 420mm L=7010mm	Φ32 mm No. = 11 L=Var.	ϕ12 mm c/c Var. L=3020mm	Φ32 mm No. = 9 L=Var.	ϕ12 mm c/c Var. L=3020mm	5.76.	14.61	108.6	299	291
22	180	2.2	260	1400	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ16 mm c/c 260mm L=26450mm	ϕ16 mm c/c 420mm L=7010mm	Φ32 mm No. = 13 L=Var.	ϕ12 mm c/c Var. L=3220mm	Φ32 mm No. = 11 L=Var.	ϕ12 mm c/c Var. L=3220mm	7.47.	16.33	120.87	340	321
24	180	2.2	260	1500	ϕ16 mm c/c 170mm L=7800mm	ϕ16 mm c/c 140mm L=7800mm	ϕ16 mm c/c 260mm L=28750mm	ϕ16 mm c/c 420mm L=7010mm	Φ32 mm No. = 14 L=Var.	ϕ12 mm c/c Var. L=3420mm	Φ32 mm No. = 13 L=Var.	ϕ12 mm c/c Var. L=3420mm	8.31.	17.75	133.53	384	353

6.2.2 Results

The quantity of materials obtained from the output of the program and current materials market prices are used for demarcating the economical span of the two types of bridges (T- and Box- Girder Bridges).

For economical span demarcation, recent values of construction costs of materials including overhead cost are used here. Table 6.3 below shows the cost for different materials.

Table 6.3 Unit Price for Construction Materials

No.	Type of material	Unit	Rate (Birr)
1	High yield stress steel bars (Grade 420)	ton	46,420
2	Mild steel bars (Grade 300)	ton	42,240
3	Cast in situ concrete	m^3	3,900
4	Formworks to provide class F_1 finish	m^2	465
5	Formworks to provide class F_2 finish	m^2	615
6	Asphalt concrete	m^2	223
7	Steel pipes	ton	56,000
8	Elastomeric bearing	No.	14,500

An output of the results of the two types of bridges for span ranges from 18 to 28m and their total associated costs of the superstructure are summarized and tabulated in Table 6.4 below.

Table 6.4 Total Superstructure Costs for Different Span Lengths

Sr No.	Span length (m)	Total Superstructure cost (Birr)	
		T-Girder Bridge	Box-Girder Bridge
1	18	1,553,442.63	1,601,670.50
2	19	1,655,673.88	1,684,289.25
3	20	1,765,555.88	1,776,223.13
4	21	1,901,350.25	1,909,196.88
5	22	1,987,032.50	2,017,131.38
6	23	2,112,304.50	2,125,283.50
7	24	2,244,821.50	2,208,436.50
8	25	2,342,358.50	2,314,515.50
9	26	2,479,165.50	2,420,385.25
10	27	2,639,653.50	2,544,612.00
11	28	2,750,544.75	2,655,358.00

Figure 6.1 shows a relationship between span length and total cost of superstructure for span length of a bridge ranging from 18 to 28m.

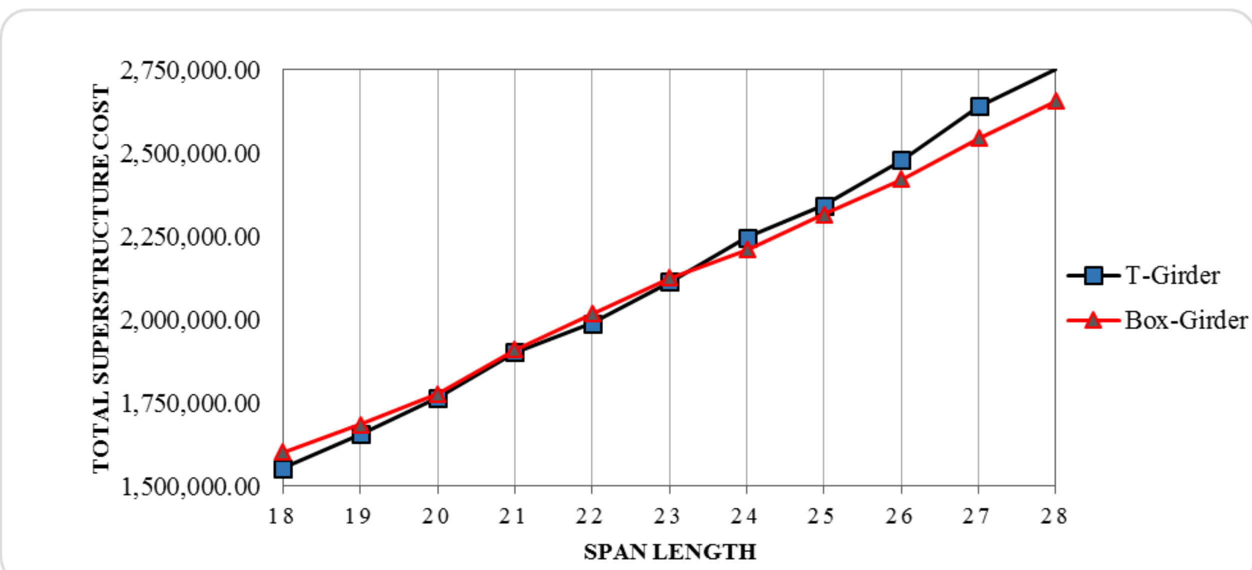

Figure 6.1 Span Length vs Total Superstructure Cost

6.3 Discussions

- Per the observation made on the summarized output data from Table 6.1 & 6.2, the area of formwork of a T-Girder bridge for class F_2 finish is much larger than those in Box-Girder bridges due to larger area of structural components' exposed faces. Hence, it gives considerable contribution for increment of the total cost of a T-Girder bridge.

- Again from Table 6.1 & 6.2, the total cost of a T-Girder bridge is also governed by its' larger volume of concrete than in a Box-Girder bridge, which is directly related to larger structural depth and width of girders.

- As one can easily understand from the tabulated output data and graphical description, the economical span that demarcates the two types of bridges (T- & Box- girder) at time of completion of the study is 23m.

- Cost of construction differs from time to time due to the variability in price of construction materials, skilled & unskilled workers, and other associated factors. For these reason, the economical span that demarcates the two types of bridges can vary with time accordingly.

7 Conclusions and Recommendations

7.1 Conclusions

This study has attempted to address the selection problem of the two types of bridges (T- & Box-Girder bridges). Per the condition survey made, it is observed that some designers even have not adopted the standard as in the case of the 25.8m span T-Girder. Moreover, inconsistency in selection of the bridge type based on provided span length have been noticed while conducting the site visit for this study. Thus, identifying a clear demarcation of span length for selecting the more appropriate bridge type (T- / Box-Girder), on the basis of economy, happens to be found indispensable.

Per the observation of the result from Table 6.4 and Figure 6.1 of cost comparison for superstructures of the two types of bridges using the prevailing cost, it is found that a span of 23m is the demarcation span for selecting T-Girder or Box-Girder bridge. Thus, up to 23m T-Girder bridge is economical whereas beyond 23m Box-Girder bridge is economical.

Since exposed faces of structural components of a T-Girder bridge is much larger than those in Box-Girder bridges, it is observed that the area of formwork for class F_2 finish is much bigger, hence, it gives considerable contribution for increment of the total cost of a T-Girder bridge. Furthermore, the total cost of a T-Girder bridge is also governed by the total volume of concrete, which is higher than those in Box-girder bridge due to larger structural depth and width of girders.

In selection of bridge types, not only the cost of construction is taken into account but also the cost of future expenditures during the projected service life of the bridge should be considered. Constructed bridges have to be inspected and maintained as soon as possible prior to damage.

Regional factors, such as availability of construction materials, fabrication, location, transportation, erection constraints, inspection, maintenance, repair, and/or replacement shall be considered. Lower first cost does not necessarily lead to lower total cost.

7.2 Recommendations

From the study that has been conducted, the followings recommendations are drawn from the result:

- The economical span that demarcates the two types of bridges (T- & Box- Girder bridges) is 23m. Accordingly, T-Girder bridge is economical up to 23m and that of the Box-Girder bridge is economical beyond 23m.

- As the developed program is easy to apply, clients, consultants, and contractors working in road development sector can make use of the program for analysis, design, and cost estimate of superstructures of T- and Box-Girder bridges, having an appropriate consultation with the department and the developer.

- The department can upgrade the developed program for continuous spans T-Girder and Box-Girder bridges. Besides, the economical span range of reinforced concrete Box-Girder bridge can further be demarcated with that of prestressed concrete Box-Girder bridge following similar procedures as did for T- & Box-Girder bridges.

References

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ANNEXES

ANNEX I

Influence Functions for Deck Analysis

INFLUENCE FUNCTIONS FOR DECK ANALYSIS

To facilitate the analysis of deck slab for uniform and concentrated (line) loads, influence functions were developed for a deck with five interior bays and two cantilevers. The width (S) of the interior bays are assumed to be the same and the cantilever are assumed to be of length (L). The required ordinates and areas are given underneath.

Location		M_{200}	M_{204}	M_{205}	M_{300}	$R_{200}{}^{b}$
C	100	-1.0000	-0.4920	-0.3650	0.2700	1 + 1.270L/S
A	101	-0.9000	-0.4428	-0.3285	0.2430	1 + 1.143L/S
N	102	-0.8000	-0.3936	-0.2920	0.2160	1 + 1.016L/S
T	103	-0.7000	-0.3444	-0.2555	0.1890	1 + 0.889L/S
I	104	-0.6000	-0.2952	-0.2190	0.1620	1 + 0.762L/S
L	105	-0.5000	-0.2460	-0.1825	0.1350	1 + 0.635L/S
E	106	-0.4000	-0.1968	-0.1460	0.1080	1 + 0.508L/S
V	107	-0.3000	-0.1476	-0.1095	0.0810	1 + 0.381L/S
E	108	-0.2000	-0.0984	-0.0730	0.0540	1 + 0.254L/S
R	109	-0.1000	-0.0492	-0.0365	0.0270	1 + 0.127L/S
110 or 200		0.0000	0.0000	0.0000	0.0000	1.0000
201		0.0000	0.0494	0.0367	-0.0265	0.8735
202		0.0000	0.0994	0.0743	-0.0514	0.7486
203		0.0000	0.1508	0.1134	-0.0731	0.6269
204		0.0000	0.2040	0.1150	-0.0900	0.5100
205		0.0000	0.1598	0.1998	-0.1004	0.3996
206		0.0000	0.1189	0.1486	-0.1029	0.2971
207		0.0000	0.0818	0.1022	-0.0954	0.2044
208		0.0000	0.0491	0.0614	-0.0771	0.1229
209		0.0000	0.0217	0.0271	-0.0458	0.0542

Location	M_{200}	M_{204}	M_{205}	M_{300}	R_{200}[b]
210 or 300	0.0000	0.0000	0.0000	0.0000	0.0000
301	0.0000	-0.0155	-0.0194	-0.0387	-0.0387
302	0.0000	-0.0254	-0.0317	-0.0634	-0.0634
303	0.0000	-0.0305	-0.0381	-0.0761	-0.0761
304	0.0000	-0.0315	-0.0394	-0.0789	-0.0789
305	0.0000	-0.0295	-0.0368	-0.0737	-0.0737
306	0.0000	-0.0250	-0.0313	-0.0626	-0.0626
307	0.0000	-0.0191	-0.0238	-0.0476	-0.0476
308	0.0000	-0.0123	-0.0154	-0.0309	-0.0309
309	0.0000	-0.0057	-0.0072	-0.0143	-0.0143
310 or 400	0.0000	0.0000	0.0000	0.0000	0.0000
401	0.0000	0.0042	0.0052	0.0104	0.0104
402	0.0000	0.0069	0.0086	0.0171	0.0171
403	0.0000	0.0083	0.0103	0.0206	0.0206
404	0.0000	0.0086	0.0107	0.0214	0.0214
405	0.0000	0.0080	0.0100	0.0201	0.0201
406	0.0000	0.0069	0.0086	0.0171	0.0171
407	0.0000	0.0053	0.0066	0.0131	0.0131
408	0.0000	0.0034	0.0043	0.0086	0.0086
409	0.0000	0.0016	0.0020	0.0040	0.0040
410 or 500	0.0000	0.0000	0.0000	0.0000	0.0000
501	0.0000	-0.0012	-0.0015	-0.0031	-0.0031
502	0.0000	-0.0021	-0.0026	-0.0051	-0.0051
503	0.0000	-0.0026	-0.0032	-0.0064	-0.0064
504	0.0000	-0.0027	-0.0034	-0.0069	-0.0069

Location	M_{200}	M_{204}	M_{205}	M_{300}	R_{200} [b]
505	0.0000	-0.0027	-0.0033	-0.0067	-0.0067
506	0.0000	-0.0024	-0.0030	-0.0060	-0.0060
507	0.0000	-0.0020	-0.0024	-0.0049	-0.0049
508	0.0000	-0.0014	-0.0017	-0.0034	-0.0034
509	0.0000	-0.0007	-0.0009	-0.0018	-0.0018
510	0.0000	0.0000	0.0000	0.0000	0.0000
Area + (w/o cantilever) [c]	0.0000	0.0986	0.0982	0.0134	0.4464
Area - (w/o cantilever) [c]	0.0000	-0.0214	-0.0268	-0.1205	-0.0536
Area Net (w/o cantilever) [c]	0.0000	0.0772	0.0714	-0.1071	0.3928
Area + (cantilever) [d]	0.0000	0.0000	0.0000	0.1350	1 + 0.635L/S
Area - (cantilever) [d]	-0.5000	-0.2460	-0.1825	0.0000	0.0000
Area Net- (cantilever) [d]	-0.5000	-0.2460	-0.1825	0.1350	1 + 0.635L/S

[a] Multiply coefficients by the span length where the load is applied, that is, L on cantilever and S in the other spans.

[b] Do not multiply by the cantilever span length; use formulas or values given.

[c] Multiply moment area coefficient by S^2, reaction area coefficient by S.

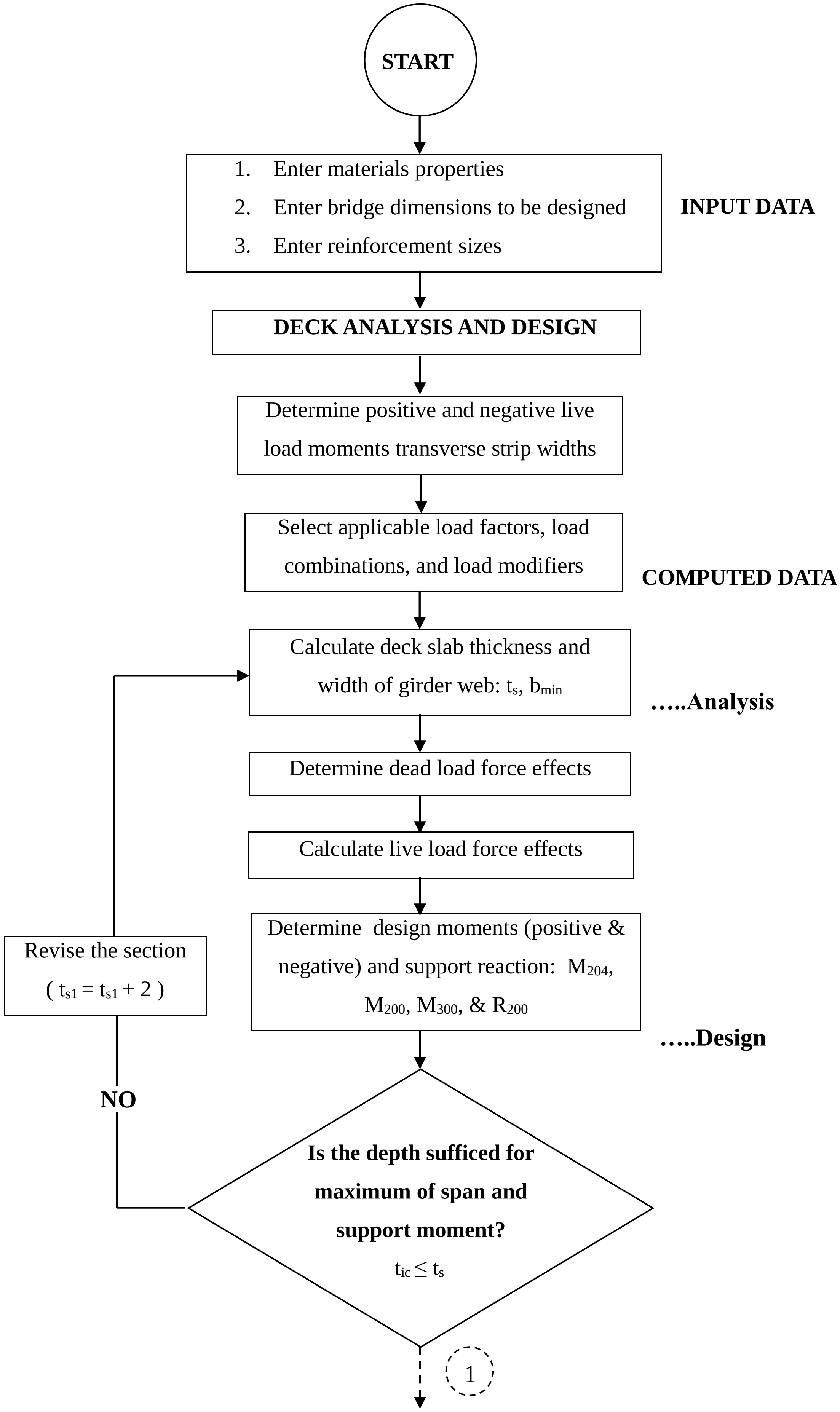
[d] Multiply moment area coefficient by L^2, reaction area coefficient by L.

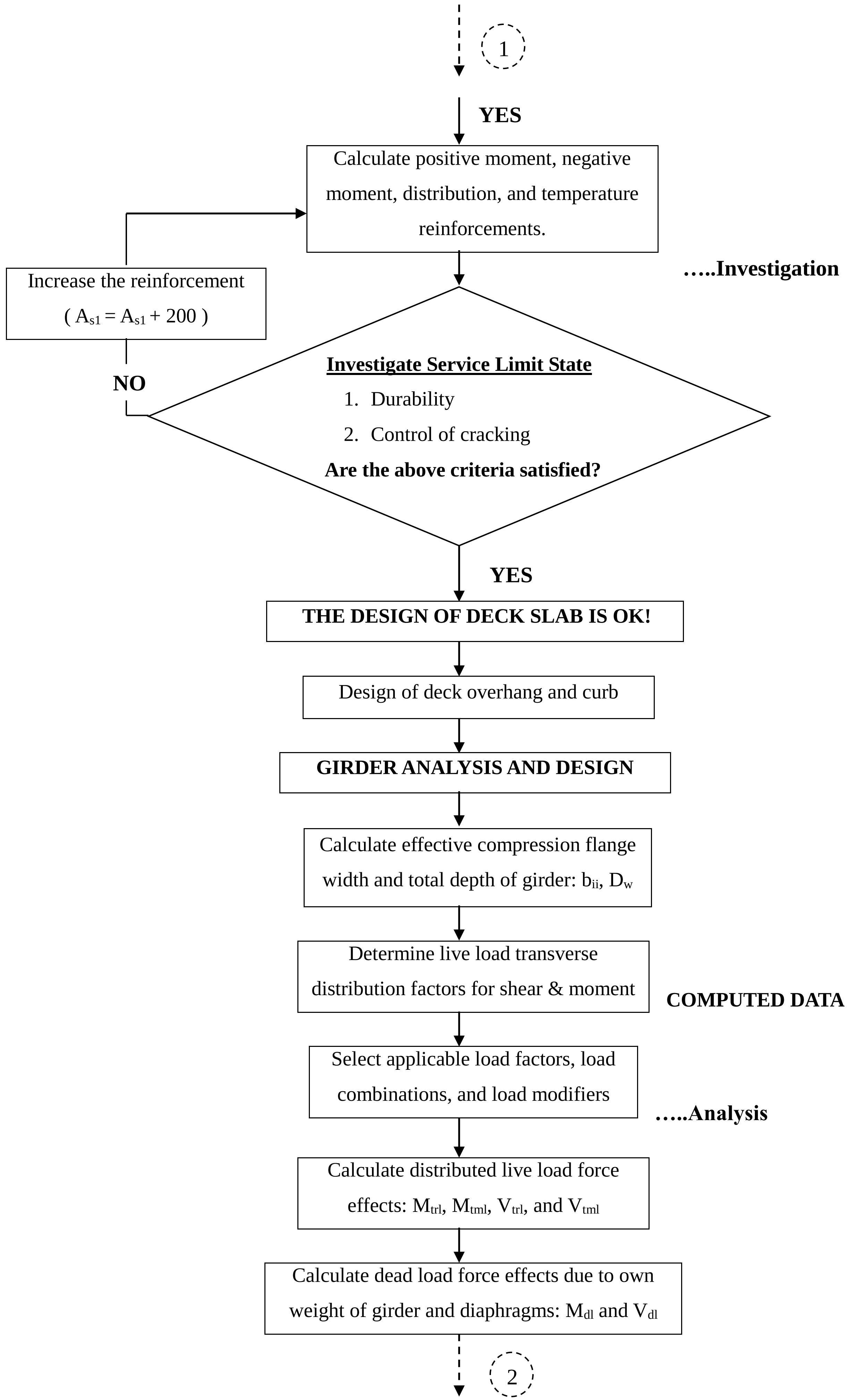
ANNEX II

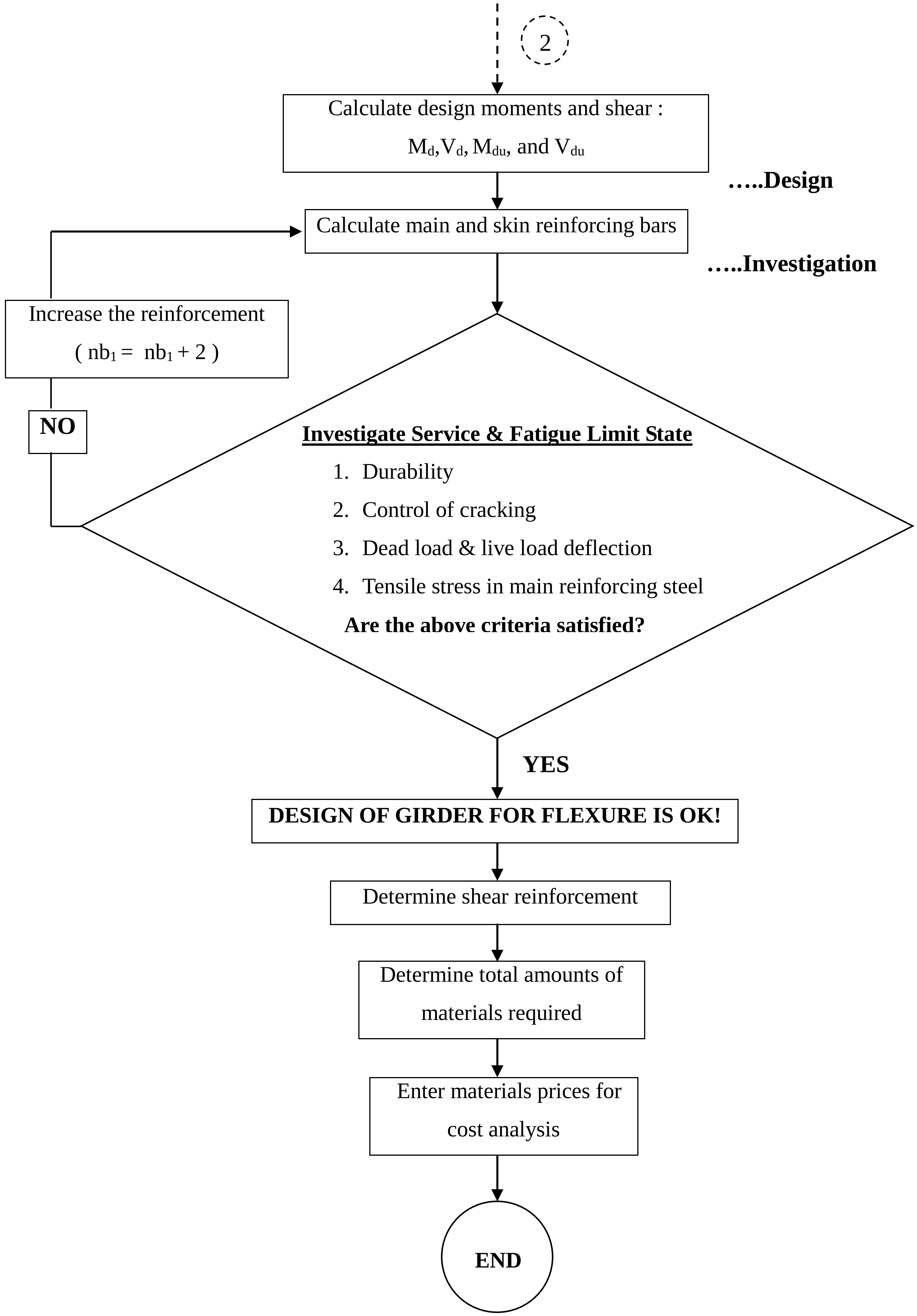
Flow Charts for T-Girder &

Box-Girder Bridges

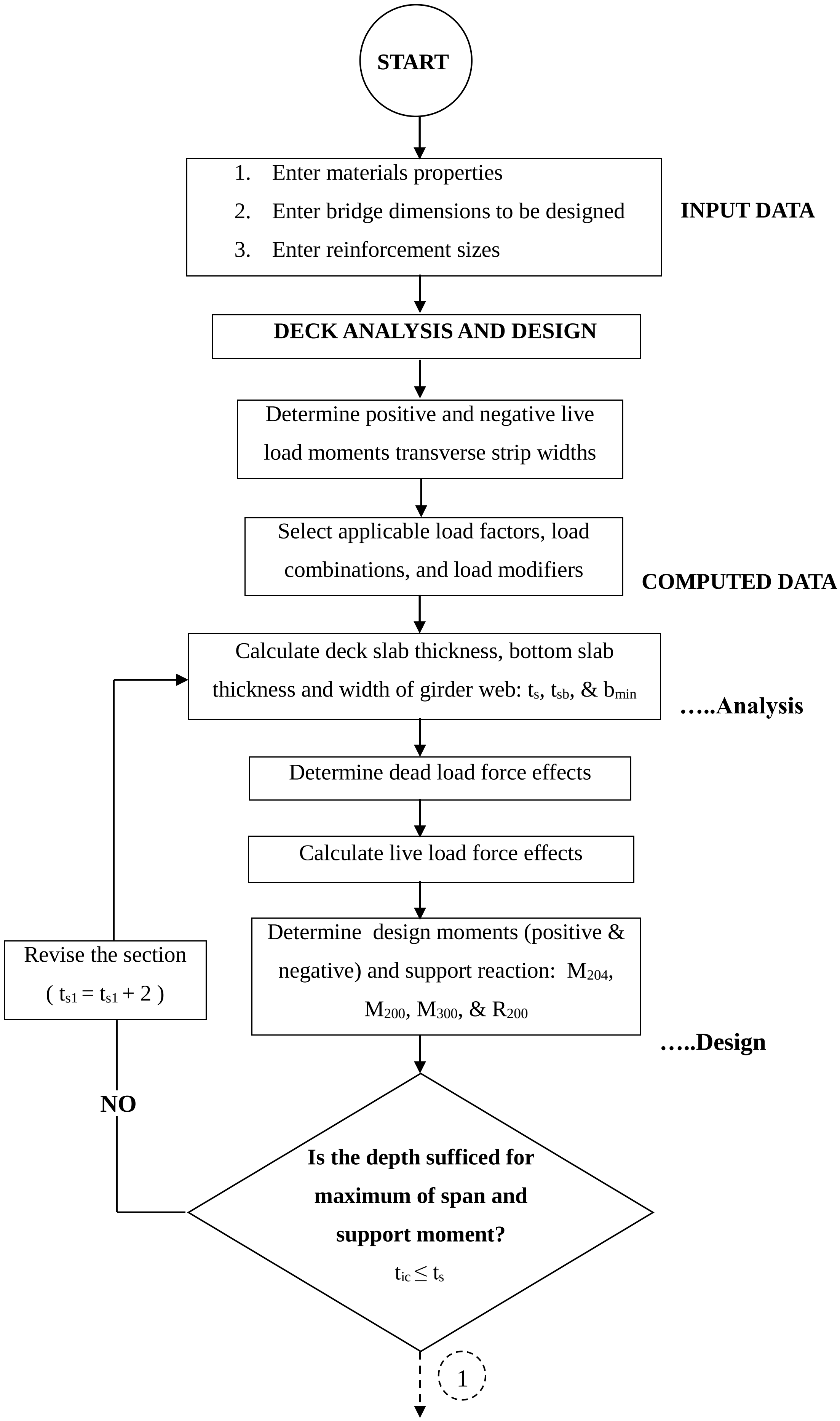
FLOW CHART FOR T-GIRDER BRIDGE

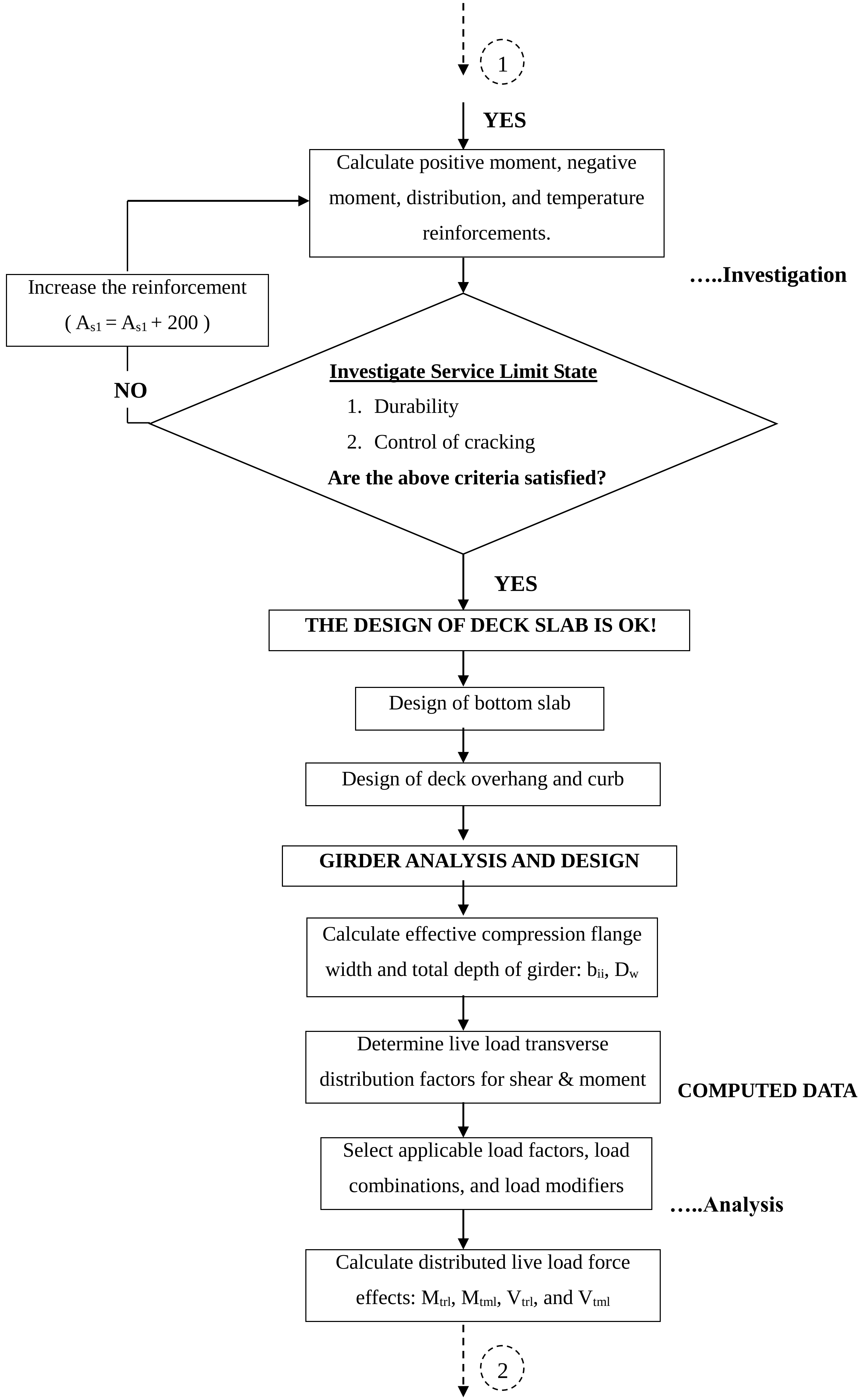


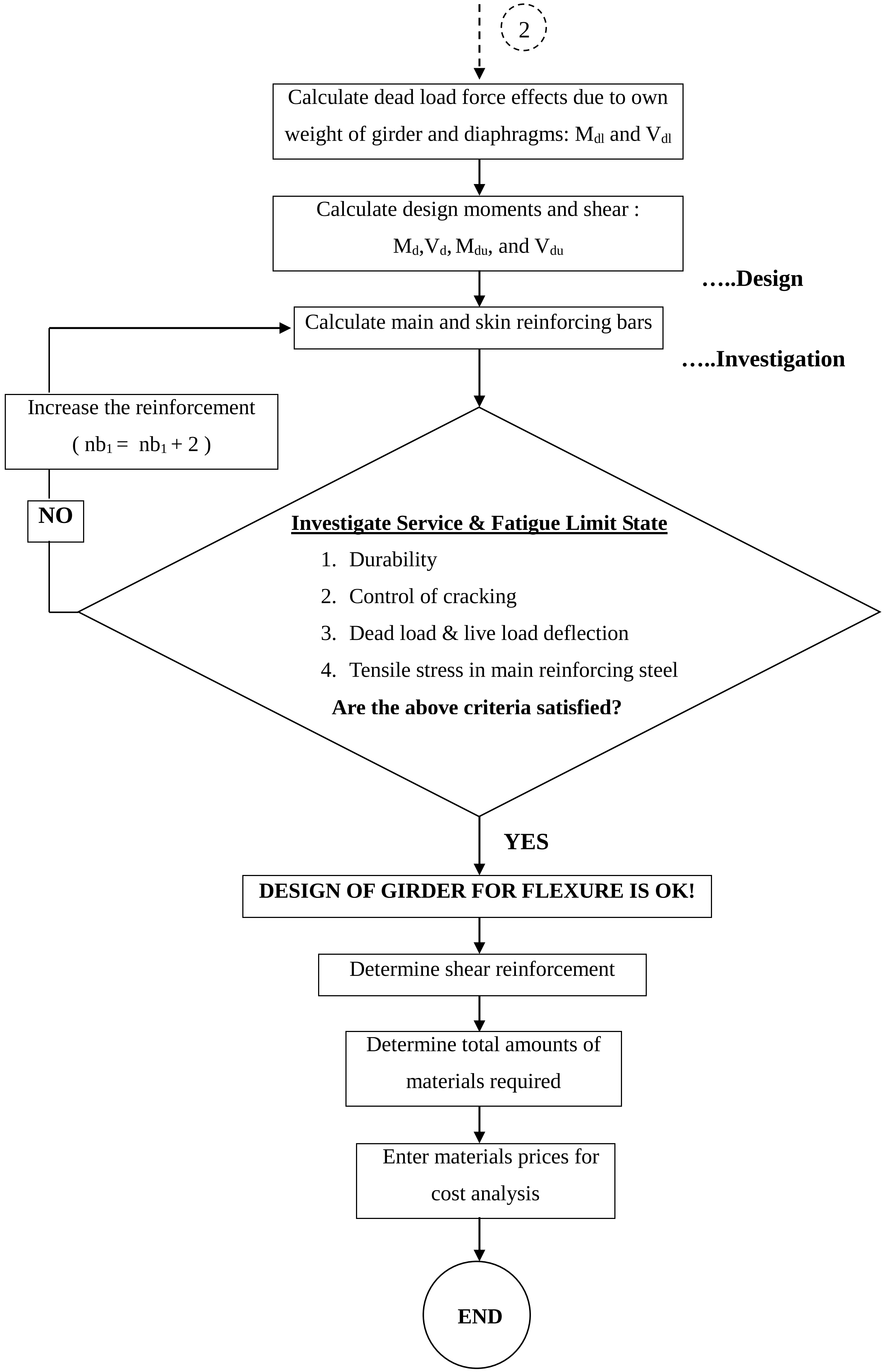

1
YES
Calculate positive moment, negative moment, distribution, and temperature reinforcements.
…..Investigation
Increase the reinforcement
($A_{s1} = A_{s1} + 200$)
NO
Investigate Service Limit State
1. Durability
2. Control of cracking
Are the above criteria satisfied?
YES
THE DESIGN OF DECK SLAB IS OK!
Design of deck overhang and curb
GIRDER ANALYSIS AND DESIGN
Calculate effective compression flange width and total depth of girder: b_{ii}, D_w
Determine live load transverse distribution factors for shear & moment
COMPUTED DATA
Select applicable load factors, load combinations, and load modifiers
…..Analysis
Calculate distributed live load force effects: M_{trl}, M_{tml}, V_{trl}, and V_{tml}
Calculate dead load force effects due to own weight of girder and diaphragms: M_{dl} and V_{dl}
2


2
Calculate design moments and shear :
M_d, V_d, M_{du}, and V_{du}
…..Design
Calculate main and skin reinforcing bars
…..Investigation
Increase the reinforcement
($nb_1 = nb_1 + 2$)
NO
Investigate Service & Fatigue Limit State
1. Durability
2. Control of cracking
3. Dead load & live load deflection
4. Tensile stress in main reinforcing steel
Are the above criteria satisfied?
YES
DESIGN OF GIRDER FOR FLEXURE IS OK!
Determine shear reinforcement
Determine total amounts of materials required
Enter materials prices for cost analysis
END

FLOW CHART FOR BOX-GIRDER BRIDGE




1
YES
Calculate positive moment, negative moment, distribution, and temperature reinforcements.
…..Investigation
Increase the reinforcement
($A_{s1} = A_{s1} + 200$)
NO
Investigate Service Limit State
1. Durability
2. Control of cracking
Are the above criteria satisfied?
YES
THE DESIGN OF DECK SLAB IS OK!
Design of bottom slab
Design of deck overhang and curb
GIRDER ANALYSIS AND DESIGN
Calculate effective compression flange width and total depth of girder: b_{ii}, D_w
Determine live load transverse distribution factors for shear & moment
COMPUTED DATA
Select applicable load factors, load combinations, and load modifiers
…..Analysis
Calculate distributed live load force effects: M_{trl}, M_{tml}, V_{trl}, and V_{tml}
2


2
Calculate dead load force effects due to own weight of girder and diaphragms: M_{dl} and V_{dl}
Calculate design moments and shear : M_d,V_d, M_{du}, and V_{du}
…..Design
Calculate main and skin reinforcing bars
…..Investigation
Increase the reinforcement ($nb_1 = nb_1 + 2$)
NO
Investigate Service & Fatigue Limit State
1. Durability
2. Control of cracking
3. Dead load & live load deflection
4. Tensile stress in main reinforcing steel
Are the above criteria satisfied?
YES
DESIGN OF GIRDER FOR FLEXURE IS OK!
Determine shear reinforcement
Determine total amounts of materials required
Enter materials prices for cost analysis
END

ANNEX III

Design Outputs of T-Girder & Box-Girder Bridges

ANALYSIS, DESIGN & COST ESTIMATE OF R.C. T-GIRDER BRIDGE

1. Enter Material Properties

Reinforcement steel grade for diameter of bars ≥ 20mm, Fy1, in Mpa = 420

Reinforcement steel grade for diameter of bars < 20mm, Fy2, in Mpa = 300

Concrete grade, Fc, in Mpa = 30

Concrete density, Yc, in Kg/m^3 =2500

Bituminous density, Yb, in Kg/m^3 = 2250

Modulus of elasticity of steel, Es, in Gpa = 200

2. Enter Bridge Span and Support Dimension

Clear span of the bridge, Cs, in m = 24

Road way width, Rw, in m = 7.32

Additional curb width, Cw, in m = 0.8

Curb depth, Cd, in m = 0.25

Beam seat width, WRs, in m = 0.5

Bituminous thickness, bt, in mm = 50

Railing depth of concrete, Rd, in m = 0.4

Railing width of concrete, Rww, in m = 0.2

Steel railing pipe diameter, Rds, in mm = 0

Post depth of concrete, Pd, in m = 0.25

Post width of concrete, Pw, in m = 0.3

Steel post pipe diameter, Pds, in mm = 0

Post height, Ph, in m = 0.55

Post spacing, Ps, in m = 2

Grider spacing, Gs, in m = 2.2

3. Enter Reinforcement Sizes

Diameter of girder main reinforcing bars, db, used in mm = 32

Diameter of deck main reinforcing bars, dd, used in mm = 16

Diameter of distribution reinforcements, dd1, used in mm = 12

Diameter of temprature & shrinkage reinforcements, d1t, used in mm = 12

Diameter of skin reinforcements, dsk, used in mm = 16

Diameter of shear reinforcements, dst, used in mm = 12

DESIGN OF T-GIRDER BRIDGE HAVING A CLEAR SPAN OF 24 m AND A ROADWAY WIDTH OF 7.32 m

C/C Support Length = 24.50 m

Total No. of Posts = 26

Ec = 26.332 Gpa

No. of Girders = 4

Distance b/n center of exterior girder & inner face of curb = 0.360 m

Thic. of Deck Slab = 180.00 mm, Width of Web = 450.00 mm

DESIGN OF DECK SLAB

Design Moments

Maximum Span Moment = 42.395 Kn-m/m

Maximum Support Moment = 38.317 Kn-m/m

Reinforcements

Positive Moment Reinforcements

As provided = 1182.72 mm^2 Use 148   dia.16 mm Rebars

C/C Spacing = 170 mm   Length = 7800.0 mm

Compression depth = 17.393 mm < 0.35*dp = 51.450 mm

Ductility requirement is satisfied!

Design mom. = 42.395 Kn-m/m < Section capacity = 43.406 Kn-m/m

The section is adequate for flexure!

Negative Moment Reinforcements

As provided = 1436.16 mm^2 Use 180   dia.16 mm Rebars

C/C Spacing = 140 mm   Length = 7800.0 mm

Compression depth = 21.120 mm < 0.35*dn = 39.200 mm

Ductility requirement is satisfied!

Design mom. = 38.317 Kn-m/m < Section capacity = 39.116 Kn-m/m

The section is adequate for flexure!

Distribution Reinforcement Transversal (bottom)

As provided = 792.42 mm^2 Use 53   dia.12 mm Rebars

C/C Spacing = 140 mm   Length = 28720.0 mm

Shirnkage and Tempature Reinforcement Transversal (top)

As provided = 33.00 mm^2 Use 17   dia.12 mm Rebars

C/C Spacing = 440 mm   Length = 28720.0 mm

DESIGN OF DECK OVERHANG

Design Moment

Maximum Overhang Moment = 65.925 Kn-m/m

Transversal Reinforcement

As provided = 2513.27 mm^2 Use 312   dia.16 mm Rebars

C/C Spacing = 80 mm   Length = 1950.0 mm both sides

Shirnkage and Tempature Reinforcement (top & bottom)

As provided = 19.14 mm^2 Use 5   dia.12 mm Rebars

C/C Spacing = 260 mm   Length = 28720.0 mm both sides

Curb Reinforcement

As provided = 609.28 mm^2 Use 76   dia.16 mm Rebars

C/C Spacing = 330 mm   Length = 1950.0 mm

ANALYSIS OF LONGITUDINAL GIRDERS

INTERIOR GIRDER

Effective flange width = 2200 mm

Total depth of girder = 1800 mm

Truck Load + Lane Load

Mtr (0) = 0.00000,	Vtr (0) = 469.753
Mtr (2) = 889.680,	Vtr (2) = 415.086
Mtr (5) = 1900.61,	Vtr (5) = 337.464
Mtr (7) = 2358.83,	Vtr (7) = 288.634
Mtr (10) = 2722.57,	Vtr (10) = 219.767
Mtr (12) = 2816.92,	Vtr (12) = 176.774

Tandem Load + Lane Load

Mtm (0) = 0.00000,	Vtm (0) = 394.632
Mtm (2) = 752.033,	Vtm (2) = 348.834
Mtm (5) = 1625.59,	Vtm (5) = 284.516
Mtm (7) = 2038.30,	Vtm (7) = 244.555
Mtm (10) = 2402.87,	Vtm (10) = 188.992
Mtm (12) = 2476.25,	Vtm (12) = 154.868

Dead Load Moment

Mdl (0) = 0.00000,	Vdl (0) = 402.361
Mdl (2) = 741.637,	Vdl (2) = 339.276
Mdl (5) = 1617.53,	Vdl (5) = 244.650
Mdl (7) = 2043.74,	Vdl (7) = 181.565
Mdl (10) = 2678.04,	Vdl (10) = 70.9699
Mdl (12) = 2756.90,	Vdl (12) = 7.88555

Design Moment & Shear, (Factored)

MD (0) = 0.00000,	VD (0) = 1325.02
MD (2) = 2483.99,	VD (2) = 1150.50
MD (5) = 5347.97,	VD (5) = 896.374
MD (7) = 6682.63,	VD (7) = 732.066
MD (10) = 8112.05,	VD (10) = 473.305
MD (12) = 8375.73,	VD (12) = 319.212

Maximum Design Moment & Shear, (Factored)

Mu = 8375.73 Kn-m/m,   Vu = 1325.02 Kn/m

Design Moment & Shear, (Unfactored)

MDu (0) = 0.00000, VDu (0) = 872.114

MDu (2) = 1631.32, VDu (2) = 754.363

MDu (5) = 3518.13, VDu (5) = 582.114

MDu (7) = 4402.57, VDu (7) = 470.199

MDu (10) = 5400.62, VDu (10) = 290.737

MDu (12) = 5573.82, VDu (12) = 184.660

Maximum Design Moment & Shear, (Unfactored)

Mus = 5573.82 Kn-m/m, Vus = 872.114 Kn/m

EXTERIOR GIRDER

Effective flange width = 2035 mm

Truck Load + Lane Load

Mtre (0) = 0.00000, Vtre (0) = 428.250

Mtre (2) = 902.708, Vtre (2) = 378.413

Mtre (5) = 1928.44, Vtre (5) = 307.649

Mtre (7) = 2393.37, Vtre (7) = 263.133

Mtre (10) = 2762.44, Vtre (10) = 200.351

Mtre (12) = 2858.17, Vtre (12) = 161.156

Tandem Load + Lane Load

Mtme (0) = 0.00000, Vtme (0) = 359.766

Mtme (2) = 763.045, Vtme (2) = 318.014

Mtme (5) = 1649.39, Vtme (5) = 259.378

Mtme (7) = 2068.15, Vtme (7) = 222.948

Mtme (10) = 2438.05, Vtme (10) = 172.294

Mtme (12) = 2512.51, Vtme (12) = 141.185

Dead Load Moment

Mdle (0) = 0.00000, Vdle (0) = 430.747

Mdle (2) = 792.472, Vdle (2) = 361.725

Mdle (5) = 1722.35, Vdle (5) = 258.191

Mdle (7) = 2169.71, Vdle (7) = 189.168

Mdle (10) = 2697.68, Vdle (10) = 77.6503

Mdle (12) = 2783.96, Vdle (12) = 8.62781

Design Moment & Shear, (Factored)

MDe (0) = 0.00000, VDe (0) = 1287.87

MDe (2) = 2570.33, VDe (2) = 1114.38

MDe (5) = 5527.70, VDe (5) = 861.124

MDe (7) = 6900.53, VDe (7) = 696.943

MDe (10) = 8206.37, VDe (10) = 447.676

MDe (12) = 8481.74, VDe (12) = 292.808

Maximum Design Moment & Shear, (Factored)

Mue = 8481.74 Kn-m/m, Vue = 1287.87 Kn/m

Design Moment & Shear, (Unfactored)

MDue (0) = 0.00000, VDue (0) = 858.997

MDue (2) = 1695.18, VDue (2) = 740.138

MDue (5) = 3650.78, VDue (5) = 565.840

MDue (7) = 4563.08, VDue (7) = 452.302

MDue (10) = 5460.12, VDue (10) = 278.001

MDue (12) = 5642.13, VDue (12) = 169.784

Maximum Design Moment & Shear, (Unfactored)

Muse = 5642.13 Kn-m/m, Vuse = 858.997 Kn/m

DESIGN OF LONGITUDINAL GIRDER FOR FLEXURE

REINFORCEMENT, INTERIOR GIRDER

As provided = 4288.73	No. of bars = 4.00000	No. of added bars = 4.00000	Length = 31091.0
As provided = 8356.32	No. of bars = 9.00000	No. of added bars = 5.00000	Length = 19049.0
As provided = 9636.98	No. of bars = 10.0000	No. of added bars = 1.00000	Length = 15049.0
As provided = 12138.1	No. of bars = 14.0000	No. of added bars = 4.00000	Length = 7257.0
As provided = 12138.1	No. of bars = 14.0000	No. of added bars = 0.00000	Length = 3257.0

Serviceability Requirement

Fatigue Stress Limit

Fatigue stress range = 25.203 Mpa < Fatigue stress limit = 115.732 Mpa

The fatigue stress range is within the permissible limit!

Crack Control

Bar spacing in a layer = 80.000 mm < Spacing limit = 238.230 mm

Width of flexural crack is within the permissible limit!

Skin Reinforcement

As provided = 970.00 mm^2 Use 5   dia.16 mm Rebars

C/C Spacing = 200 mm   Length = 25764.0 mm on each face of girder

REINFORCEMENT, EXTERIOR GIRDER

As provided = 4288.68   No. of bars = 4.00000   No. of added bars = 4.00000   Length = 31091.0

As provided = 8355.60   No. of bars = 9.00000   No. of added bars = 5.00000   Length = 19049.0

As provided = 10914.2   No. of bars = 12.0000   No. of added bars = 3.00000   Length = 15049.0

As provided = 12135.1   No. of bars = 14.0000   No. of added bars = 2.00000   Length = 7257.0

As provided = 13353.9   No. of bars = 15.0000   No. of added bars = 1.00000   Length = 3257.0

Serviceability Requirement

Fatigue stress limit

Fatigue stress range = 46.643 Mpa < Fatigue stress limit = 119.517 Mpa

The fatigue stress range is within the permissible limit!

Crack Control

Bar spacing in a layer = 80.000 mm < Spacing limit = 237.388 mm

Width of flexural crack is within the permissible limit!

Skin Reinforcement

As provided = 970.00 mm^2 Use 5   dia.16 mm Rebars

C/C Spacing = 200 mm   Length = 25764.0 mm on each face of girder

DESIGN OF LONGITUDINAL GIRDER FOR SHEAR

REINFORCEMENT, INTERIOR GIRDER

Use 24 dia. 12.0000 mm shear reinf. c/c 210.000 mm

For a distance of 4.90000 m from both ends Length = 4420.00 mm

Use 11 dia. 12.0000 mm shear reinf. c/c 230.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

Use 9 dia. 12.0000 mm shear reinf. c/c 300.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

Use 9 dia. 12.0000 mm shear reinf. c/c 300.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

REINFORCEMENT, EXTERIOR GIRDER

Use 22 dia. 12.0000 mm shear reinf. c/c 230.000 mm

For a distance of 4.90000 m from both ends Length = 4420.00 mm

Use 10 dia. 12.0000 mm shear reinf. c/c 250.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

Use 9 dia. 12.0000 mm shear reinf. c/c 300.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

Use 9 dia. 12.0000 mm shear reinf. c/c 300.000 mm

For a distance of 2.45000 m from both ends Length = 4420.00 mm

Serviceability Requirement/Deflection

Provide a camber of 91.5 mm at middle span of the bridge

Computed LL deflection = 11.809 mm < Maximum allowable LL deflection = 30.625 mm

Satisfies live load deflection requirement!

Diaphragm Reinforcement

Use 2 dia. 24 mm bars both at top and bottom

Provide 22 Length 3560.0 mm dia. 12.0 mm Rebars stirrups

Provide 8 Length 6970.0 mm dia. 16.0 mm skin rein. bars on each face

Abutment Head Reinforcement

Provide 59 Length 2040.0 mm dia. 12.0 mm Rebars stirrups

Provide 5 Length 8840.0 mm dia. 12.0 mm longitudinal bars (top/bottom)

Abutment Backwall Reinforcement

Provide 59 Length 4580.0 mm dia. 12.0 mm Rebars stirrups

Provide 14 Length 8840.0 mm dia. 12.0 mm longitudinal bars on each face

T-GIRDER BRIDGE OF 24.0000 m CLEAR SPAN & 7.32000 m ROADWAY WIDTH

Total amount of materials used are:

1) Reinforcement bars ≥ 20 mm = 8.08 ton

2) Reinforcement bars < 20 mm = 16.04 ton

3) Volume of concrete = 149.29 m^3

4) Area of wearing surface = 183.00 m^2

5) Area of formwork (class F_1 finish) = 47.31 m^2

6) Area of formwork (class F_2 finish) = 700.81 m^2

7) Total wt. of steel pipe = 0.000 ton

8) Total no. of elastomeric bearings = 8.0

4. Enter Current Cost in Birr

Unit price for rein. bars >= 20 mm dia./ton = 46420

Unit price for rein. bars < 20mm dia./ton = 42240

Unit price for concrete/m^3 = 3900

Unit price for asphalt wearing surface/m^2 = 223

Unit price for formwork (class F_1 finish)/m^2 = 465

Unit price for formwork (class F_2 finish)/m^2 = 615

Unit price for steel pipes/ton = 56000

Unit price for elastomeric bearings/pcs = 14500

Total cost for comparison = 2,244,821.50 Birr

====================THE END====================

ANALYSIS, DESIGN & COST ESTIMATE OF R.C. BOX-GIRDER BRIDGE

1. Enter Material Properties

Reinforcement steel grade for diameter of bars ≥ 20mm, Fy1, in Mpa = 420

Reinforcement steel grade for diameter of bars < 20mm, Fy2, in Mpa = 300

Concrete grade, Fc, in Mpa = 30

Concrete density, Yc, in Kg/m^3 =2500

Bituminous density, Yb, in Kg/m^3 = 2250

Modulus of elasticity of steel, Es, in Gpa = 200

2. Enter Bridge Span and Support Dimension

Clear span of the bridge, Cs, in m = 24

Road way width, Rw, in m = 7.32

Additional curb width, Cw, in m = 0.8

Curb depth, Cd, in m = 0.25

Beam seat width, WRs, in m = 0.5

Bituminous thickness, bt, in mm = 50

Railing depth of concrete, Rd, in m = 0.4

Railing width of concrete, Rww, in m = 0.2

Steel railing pipe diameter, Rds, in mm = 0

Post depth of concrete, Pd, in m = 0.25

Post width of concrete, Pw, in m = 0.3

Steel post pipe diameter, Pds, in mm = 0

Post height, Ph, in m = 0.55

Post spacing, Ps, in m = 2

Grider spacing, Gs, in m = 2.2

3. Enter Reinforcement Sizes

Diameter of girder main reinforcing bars, db, used in mm = 32

Diameter of deck main reinforcing bars, dd, used in mm = 16

Diameter of bottom slab reinforcing bars, dbs, used in mm = 16

Diameter of distribution reinforcements, dd1, used in mm = 12

Diameter of temprature & shrinkage reinforcements, d1t, used in mm = 12

Diameter of skin reinforcements, dsk, used in mm = 16

Diameter of shear reinforcements, dst, used in mm = 12

DESIGN OF BOX-GIRDER BRIDGE HAVING A CLEAR SPAN OF 24 m AND A ROADWAY WIDTH OF 7.32 m

C/C Support Length = 24.50 m

Total No. of Posts = 26

Ec = 26.332 Gpa

No. of Girders = 4

Distance b/n center of exterior girder & inner face of curb = 0.360 m

Thic. of Deck Slab = 180.00 mm, Width of Web = 260.00 mm

Thic. of Bottom Slab = 188.00 mm

DESIGN OF DECK SLAB

Design Moments

Maximum Span Moment = 42.395 Kn-m/m

Maximum Support Moment = 38.317 Kn-m/m

Reinforcements

Positive Moment Reinforcements

As provided = 1182.72 mm^2 Use 148 dia.16 mm Rebars

C/C Spacing = 170 mm Length = 7800.0 mm

Compression depth = 17.393 mm < 0.35*dp = 51.450 mm

Ductility requirement is satisfied!

Design mom. = 42.395 Kn-m/m < Section capacity = 43.406 Kn-m/m

The section is adequate for flexure!

Negative Moment Reinforcements

As provided = 1436.16 mm^2 Use 180 dia.16 mm Rebars

C/C Spacing = 140 mm Length = 7800.0 mm

Compression depth = 21.120 mm < 0.35*dn = 39.200 mm

Ductility requirement is satisfied!

Design mom. = 38.317 Kn-m/m < Section capacity = 39.116 Kn-m/m

The section is adequate for flexure!

Distribution Reinforcement Transversal (bottom)

As provided = 792.42 mm^2 Use 53 dia.12 mm Rebars

C/C Spacing = 140 mm Length = 28720.0 mm

Shirnkage and Tempature Reinforcement Transversal (top)

As provided = 33.00 mm^2 Use 17 dia.12 mm Rebars

C/C Spacing = 440 mm Length = 28720.0 mm

BOTTOM SLAB REINFORCEMENT BETWEEN GIRDERS

Longitudinal Direction

As provided = 773.31 mm^2 Use 4   dia.16 mm Rebars

C/C Spacing = 260 mm   Length = 28750.0 mm both sides

Transversal Direction

As provided = 478.72 mm^2 Use 60   dia.16 mm Rebars

C/C Spacing = 420 mm   Length = 7010.0 mm both sides

DESIGN OF DECK OVERHANG

Design Moment

Maximum Overhang Moment = 65.925 Kn-m/m

Transversal Reinforcement

As provided = 2513.27 mm^2 Use 312   dia.16 mm Rebars

C/C Spacing = 80 mm   Length = 1950.0 mm both sides

Shirnkage and Tempature Reinforcement (top & bottom)

As provided = 19.14 mm^2 Use 5   dia.12 mm Rebars

C/C Spacing = 260 mm   Length = 28720.0 mm both sides

Curb Reinforcement

As provided = 609.28 mm^2 Use 76   dia.16 mm Rebars

C/C Spacing = 330 mm   Length = 1950.0 mm

ANALYSIS OF LONGITUDINAL GIRDERS

INTERIOR GIRDER

Effective flange width = 2200 mm

Total depth of girder = 1500 mm

Truck Load + Lane Load

Mtr (0) = 0.00000,	Vtr (0) = 462.091
Mtr (2) = 707.248,	Vtr (2) = 408.316
Mtr (5) = 1510.88,	Vtr (5) = 331.960
Mtr (7) = 1875.14,	Vtr (7) = 283.927
Mtr (10) = 2164.30,	Vtr (10) = 216.183
Mtr (12) = 2239.30,	Vtr (12) = 173.891

Tandem Load + Lane Load

Mtm (0) = 0.00000,	Vtm (0) = 388.195
Mtm (2) = 597.826,	Vtm (2) = 343.145
Mtm (5) = 1292.26,	Vtm (5) = 279.875
Mtm (7) = 1620.34,	Vtm (7) = 240.566
Mtm (10) = 1910.15,	Vtm (10) = 185.909
Mtm (12) = 1968.49,	Vtm (12) = 152.342

Dead Load Moment

Mdl (0) = 0.00000,	Vdl (0) = 390.469
Mdl (2) = 718.388,	Vdl (2) = 327.919
Mdl (5) = 1561.41,	Vdl (5) = 234.093
Mdl (7) = 1967.04,	Vdl (7) = 171.543

Mdl (10) = 2340.93, Vdl (10) = 77.7170

Mdl (12) = 2433.81, Vdl (12) = 15.1666

Design Moment & Shear, (Factored)

MD (0) = 0.00000, VD (0) = 1296.75

MD (2) = 2135.67, VD (2) = 1124.45

MD (5) = 4595.80, VD (5) = 873.546

MD (7) = 5740.30, VD (7) = 711.300

MD (10) = 6713.68, VD (10) = 475.466

MD (12) = 6961.04, VD (12) = 323.268

Maximum Design Moment & Shear, (Factored)

Mu = 6961.04 Kn-m/m, Vu = 1296.75 Kn/m

Design Moment & Shear, (Unfactored)

MDu (0) = 0.00000, VDu (0) = 852.560

MDu (2) = 1425.64, VDu (2) = 736.235

MDu (5) = 3072.29, VDu (5) = 566.053

MDu (7) = 3842.18, VDu (7) = 455.469

MDu (10) = 4505.23, VDu (10) = 293.900

MDu (12) = 4673.11, VDu (12) = 189.058

Maximum Design Moment & Shear, (Unfactored)

Mus = 4673.11 Kn-m/m, Vus = 852.560 Kn/m

EXTERIOR GIRDER

Effective flange width = 2130 mm

Truck Load + Lane Load

Mtre (0) = 0.00000,	Vtre (0) = 390.589
Mtre (2) = 602.541,	Vtre (2) = 345.135
Mtre (5) = 1287.20,	Vtre (5) = 280.594
Mtre (7) = 1597.53,	Vtre (7) = 239.993
Mtre (10) = 1843.88,	Vtre (10) = 182.731
Mtre (12) = 1907.77,	Vtre (12) = 146.984

Tandem Load + Lane Load

Mtme (0) = 0.00000,	Vtme (0) = 328.127
Mtme (2) = 509.318,	Vtme (2) = 290.048
Mtme (5) = 1100.94,	Vtme (5) = 236.568
Mtme (7) = 1380.45,	Vtme (7) = 203.342
Mtme (10) = 1627.36,	Vtme (10) = 157.142
Mtme (12) = 1677.06,	Vtme (12) = 128.769

Dead Load Moment

Mdle (0) = 0.00000,	Vdle (0) = 364.980
Mdle (2) = 670.961,	Vdle (2) = 305.981
Mdle (5) = 1456.16,	Vdle (5) = 217.483
Mdle (7) = 1832.13,	Vdle (7) = 158.485
Mdle (10) = 2174.83,	Vdle (10) = 69.9867
Mdle (12) = 2255.81,	Vdle (12) = 10.9881

Design Moment & Shear, (Factored)

MDe (0) = 0.00000,	VDe (0) = 1139.76

MDe (2) = 1893.15,   VDe (2) = 986.462

MDe (5) = 4072.79,   VDe (5) = 762.893

MDe (7) = 5085.84,   VDe (7) = 618.093

MDe (10) = 5945.32,   VDe (10) = 407.263

MDe (12) = 6158.36,   VDe (12) = 270.957

Maximum Design Moment & Shear, (Factored)

Mue = 6158.36 Kn-m/m,   Vue = 1139.76 Kn/m

Design Moment & Shear, (Unfactored)

MDue (0) = 0.00000,   VDue (0) = 755.569

MDue (2) = 1273.50,   VDue (2) = 651.116

MDue (5) = 2743.35,   VDue (5) = 498.077

MDue (7) = 3429.66,   VDue (7) = 398.477

MDue (10) = 4018.71,   VDue (10) = 252.718

MDue (12) = 4163.58,   VDue (12) = 157.972

Maximum Design Moment & Shear, (Unfactored)

Muse = 4163.58 Kn-m/m,   Vuse = 755.569 Kn/m

DESIGN OF LONGITUDINAL GIRDER FOR FLEXURE

REINFORCEMENT, INTERIOR GIRDER

As provided = 7238.22   No. of bars = 7.00000   No. of added bars = 7.00000   Length = 31091.0

As provided = 8618.20   No. of bars = 9.00000   No. of added bars = 2.00000   Length = 19049.0

As provided = 9997.48   No. of bars = 11.0000   No. of added bars = 2.00000   Length = 15049.0

As provided = 11376.0   No. of bars = 13.0000   No. of added bars = 2.00000   Length = 7257.0

As provided = 12753.9   No. of bars = 14.0000   No. of added bars = 1.00000   Length = 3257.0

Serviceability Requirement

Fatigue Stress Limit

Fatigue stress range = 23.895 Mpa < Fatigue stress limit = 116.666 Mpa

The fatigue stress range is within the permissible limit!

Crack Control

Bar spacing in a layer = 80.000 mm < Spacing limit = 233.239 mm

Width of flexural crack is within the permissible limit!

Skin Reinforcement

As provided = 738.00 mm^2 Use 4   dia.16 mm Rebars

C/C Spacing = 240 mm   Length = 25764.0 mm on each face of girder

REINFORCEMENT, EXTERIOR GIRDER

As provided = 7238.22   No. of bars = 7.00000   No. of added bars = 7.00000   Length = 31091.0

As provided = 7238.22   No. of bars = 7.00000   No. of added bars = 0.00000   Length = 19049.0

As provided = 8618.13   No. of bars = 9.00000   No. of added bars = 2.00000   Length = 15049.0

As provided = 9997.32   No. of bars = 11.0000   No. of added bars = 2.00000   Length = 7257.0

As provided = 11375.8   No. of bars = 13.0000   No. of added bars = 2.00000   Length = 3257.0

Serviceability Requirement

Fatigue stress limit

Fatigue stress range = 49.321 Mpa < Fatigue stress limit = 114.888 Mpa

The fatigue stress range is within the permissible limit!

Crack Control

Bar spacing in a layer = 80.000 mm < Spacing limit = 233.605 mm

Width of flexural crack is within the permissible limit!

Skin Reinforcement

As provided = 738.00 mm^2 Use 4 dia.16 mm Rebars

C/C Spacing = 240 mm Length = 25764.0 mm on each face of girder

DESIGN OF LONGITUDINAL GIRDER FOR SHEAR

REINFORCEMENT, INTERIOR GIRDER

Use 46 dia. 12.0000 mm shear reinf. c/c 80.000 mm

For a distance of 4.90000 m from both ends Length = 3420.00 mm

Use 28 dia. 12.0000 mm shear reinf. c/c 90.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

Use 19 dia. 12.0000 mm shear reinf. c/c 130.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

Use 11 dia. 12.0000 mm shear reinf. c/c 240.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

REINFORCEMENT, EXTERIOR GIRDER

Use 50 dia. 12.0000 mm shear reinf. c/c 100.000 mm

For a distance of 4.90000 m from both ends Length = 3420.00 mm

Use 23 dia. 12.0000 mm shear reinf. c/c 110.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

Use 17 dia. 12.0000 mm shear reinf. c/c 150.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

Use 9 dia. 12.0000 mm shear reinf. c/c 300.000 mm

For a distance of 2.45000 m from both ends Length = 3420.00 mm

Serviceability Requirement/Deflection

Provide a camber of 192.5 mm at middle span of the bridge

Computed LL deflection = 27.018 mm < Maximum allowable LL deflection = 30.625 mm

Satisfies live load deflection requirement!

Diaphragm Reinforcement

Use 2 dia. 24 mm bars both at top and bottom

Provide 24 Length 3360.0 mm dia. 12.0 mm Rebars stirrups

Provide 6 Length 6780.0 mm dia. 16.0 mm skin rein. bars on each face

Abutment Head Reinforcement

Provide 59 Length 2040.0 mm dia. 12.0 mm Rebars stirrups

Provide 5 Length 8840.0 mm dia. 12.0 mm longitudinal bars (top/bottom)

Abutment Backwall Reinforcement

Provide 59 Length 3980.0 mm dia. 12.0 mm Rebars stirrups

Provide 12 Length 8840.0 mm dia. 12.0 mm longitudinal bars on each face

BOX-GIRDER BRIDGE OF 24.0000 m CLEAR SPAN & 7.32000 m ROADWAY WIDTH

Total amount of materials used are:

1) Reinforcement bars >= 20 mm = 8.31 ton

2) Reinforcement bars < 20 mm = 17.75 ton

3) Volume of concrete = 133.53 m^3

4) Area of wearing surface = 183.00 m^2

5) Area of formwork (class F_1 finish) = 384.24 m^2

6) Area of formwork (class F_2 finish) = 352.71 m^2

7) Total wt. of steel pipe = 0.000 ton

8) Total no. of elastomeric bearings = 8.0

4. Enter Current Cost in Birr

Unit price for rein. bars ≥ 20 mm dia./ton = 46420

Unit price for rein. bars < 20mm dia./ton = 42240

Unit price for concrete/m^3 = 3900

Unit price for asphalt wearing surface/m^2 = 223

Unit price for formwork (class F_1 finish)/m^2 = 465

Unit price for formwork (class F_2 finish)/m^2 = 615

Unit price for steel pipes/ton = 56000

Unit price for elastomeric bearings/pcs = 14500

Total cost for comparison = 2,208,436.50 Birr

===================THE END===================