



COMPARISON OF ANALYSIS MODELS OF HIGHWAY AND RAILWAY BRIDGES--LOADS AND LOAD COMBINATIONS

**By
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*A Thesis Submitted to the school of Graduate studies of Addis Ababa University in
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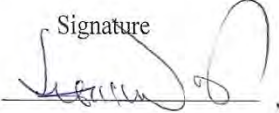
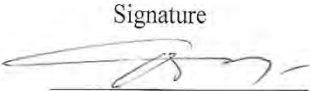

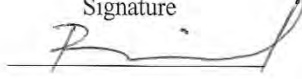
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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Abstract

Ethiopian engineers have good experience of highway bridge design. Currently railway bridges are necessary due to new railroad route projects in Ethiopia. It is absolutely necessary to bring the well developed knowledge of the engineers on Highway Bridge to Railway Bridge engineering because Railway Bridge needs careful design and detailing.

The work presented in this thesis compares the loading model and load combinations of Highway Bridge and railway bridges. Fatigue loads, serviceability requirements, their respective design philosophy, and related topics are briefly presented. Simply supported two lanes Highway Bridge (LRFD Method) and a double track Railway Bridge (ASD Method) of 30m span each are used as design examples. The design examples are neither complete nor optimum. The main sources of these loading conditions are AASHTO LRFD Specification and AREMA MRE 2010. Beside these codes, various theories and design books, thesis works, and journals on both types of bridges are used to enrich as well as disclose clearly the fundamental codes' intention, explanatory figures and diagrams.

The comparison mainly focuses on loads for fixed bridges, such loads as live load, centrifugal force, lateral force due to wind load, longitudinal force and impact.

Key words: Railway Bridge, Highway Bridge, design philosophy, loads, load combinations, fatigue

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Table of Contents

Abstract	i
Acknowledgement	i
List of symbols	vi
Abbrivation	x
List of tables	xiii
List of figures and charts	xiv
CHAPTER 1	vi
Introduction	1
1.1 Background	1
1.2 Aim and scope of the thesis	2
1.3 Lay out of the thesis	3
1.4 Notation	4
CHAPTER 2	5
Design philosophy	5
2.1 Highway Bridge	5
2.2 Railway Bridge	6
2.2.1 Reinforced concrete Railway Bridge	7
2.2.2 Steel Railway Bridges	7
CHAPTER 3	8
Loads on bridges	8
3.1 Permanent loads	8
3.1.1 Highway bridges	8
3.1.2 Railway bridges	8
3.2 Vehicular live load	9
3.2.1 Highway Bridge	9
3.2.2 Railway Bridge	11
3.3 Dynamic loads	13
3.3.1 Highway Bridge	15
3.3.2 Rail way bridges	15
3.3.2.1 Steel railway bridges	15
3.3.2.2 Concrete railway bridges	17
3.4 Centrifugal force	18

3.4.1	High way bridges	18
3.4.2	Railway Bridges	19
3.5	Longitudinal forces due to braking and traction.....	20
3.5.1	High Way Bridge.....	20
3.5.2	Railway bridges	21
3.6	Loads significant for Railway Bridge.....	22
3.6.1	Lateral forces from equipment	22
3.6.2	Lateral vibration loads	22
3.6.3	Forces from continuously welded rail (CWR)	22
3.6.4	Loads for stability check	22
3.6.5	Loads for overall lateral stability	23
3.6.6	Geometrical load	23
3.6.7	Resonance phenomena	23
3.7	Wind forces on bridges.....	24
3.7.1	Highway Bridge	24
3.7.1.1	Wind Pressure on Structures: <i>WS</i>	24
3.7.1.2	Wind Pressure on Vehicles: <i>WL</i>	25
3.7.1.3	Vertical Wind Pressure (effect of overturning)	25
3.7.2	Railway Bridge	25
3.7.2.1	Wind forces on loaded bridge.....	26
3.7.2.2	Wind forces on unloaded bridge	27
3.8	Distribution of live load.....	28
3.8.1	Highway Bridge.....	28
3.8.2	Railway Bridge	29
3.9	Multiple presences.....	32
3.9.1	Highway bridges.....	32
3.9.2	Railway Bridge	33
Chapter 4		33
Fatigue loads and serviceability requirements.....		33
4.1	Highway Bridge	34
4.1.1	Fatigue	34
4.1.2	Pedestrian loads	35
4.1.3	Deflection	35

4.2	Railway Bridge	35
4.2.1	Fatigue	35
4.2.2	Pedestrian load	37
4.2.3	Deflection	37
Chapter 5		39
Load combinations		39
5.1	Highway bridges.....	39
5.1.1	Load Factors and Load Combinations	39
5.2	Railway Bridge	41
5.2.1	Loading combinations for concrete bridges	41
5.2.1.1	Service Load Design	41
5.2.1.2	Load Factor Design.....	42
5.2.2	Loading combinations for steel bridges	42
Chapter 6		45
Conclusion and recommendation.....		45
Bibliography		46
Appendix A		48
Appendix B.....		68

List of Symbols

Symbols	Descriptions
A	detail category constant, cross-section area, axle load
A_s	area of steel
A	the length of uniform deceleration (highway bridge braking force)
b	a fraction Calculations using a braking length of 122 m and a speed of 90 km/hr.
b_c	of compression flange
b_e	exterior effective width of composite section
b_f	flange width
b_t	width of tension flange
C	ratio of the shear-buckling resistance to the shear specified minimum yield strength, Centrifugal force in percentage of the live load.
D	web depth, Degree of curve (Degrees based on 30 m chord)
D_{cp}	depth of the web in compression at the plastic moment
D_p	distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment
D_T	total depth of the composite section
d_o	transverse stiffener spacing (mm)
d	effective slab depth
E	modulus of elasticity of steel (MPa)
e_g	distance between the centers of gravity of the basic beam and deck (mm)
f_l	flange lateral bending stress (MPa); flange lateral bending stress due to the Service II loads (MPa)
F_y^f	specified minimum yield strength of a flange (MPa)
f_D	compression-flange stress caused by the factored permanent load applied before the concrete deck has hardened or is made composite.
f_{LL+IM}	compression-flange stress caused by the factored vehicular live load plus impact acting on the short-term composite section,
$(\Delta F)_n$	nominal fatigue resistance (MPa)

$(\Delta F)_{TH}$	constant amplitude fatigue threshold (MPa)
$f'c$	minimum specified 28-day compressive strength of concrete (MPa)
f_y	specified minimum yield strength of steel (MPa);
F_{yt}	specified minimum yield strength of a tension flange (MPa)
F_{yc}	specified minimum yield strength of a compression flange (MPa)
F_{yw}	specified minimum yield strength of a web (MPa)
f_f	flange stress due to the Service II loads calculated without consideration of flange lateral bending (MPa)
f	a constant , $f = 4/3$ is for combinations other than fatigue and $f = 1.0$ for fatigue;
g	acceleration due to gravity
I_{NA}	moment of inertia about neutral axis
I_{yc}	moment of inertia of the compression flange of a steel section about the vertical axis in the plane of the web (mm ⁴)
I_{yt}	moment of inertia of the tension flange of a steel section about the vertical axis in the plane of the web (mm ⁴)
k	shear-buckling coefficient for webs;
L	the span length
K_g	longitudinal stiffness parameter
M	multiple presence factors
M_g	load effect distribution factor
N	number of cycles of stress range
N	modular ratio
P	fraction of track traffic in a single lane
P_c	plastic force in the compression flange used to compute the plastic moment
P_s	plastic compressive force in the concrete deck used to compute the plastic moment
P_t	plastic force in the tension flange used to compute the plastic moment
P_w	plastic force in the web used to compute the plastic moment
Q_i	force effects from loads

R	radius of curvature of traffic lane in meters,
R_h	hybrid factor
R_n	nominal resistance
R_r	factored resistance
S_e	unsupported slab length between girders
S	elastic section modulus (mm^3)
S_f	elastic section modulus about the axis parallel with the web (mm^3)
t_s, h	slab thickness
t_f	thickness of flange
t_t	thickness of tension flange
t_c	thickness of compression flange
t_h	haunch thickness
t_w	web thickness
$UL+I$	the live-load effect plus allowance for dynamic loading
V	the highway design speed in (meters/second)
V_{fu}	the shear force due the fatigue truck and dead load of Special Fatigue Requirements for Webs
V_{cr}	shear-buckling resistance
V_p	plastic shear force
w_D	distributed dead load
W	wheel load
Φ	bar diameter, Resistance factors
Φ_f	resistance factor for flexure
Φ_v	resistance factor for shear
η_i	load modifier
η_D	ductility load modifier
η_R	redundancy load modifier
η_I	operational importance load modifier

γ_i	load factors
γ_p	load Factors for Permanent Loads
γ_{EQ}	the load factor for live load in Extreme Event Load Combination I
γ_{TG}	the load factor for temperature gradient,
γ_{SE}	the load factor for settlement
Δ	vertical deflection

Highway bridge load designations

<i>BR</i>	Vehicular braking force
<i>CE</i>	Vehicular centrifugal force
<i>CR</i>	Creep
<i>CT</i>	Vehicular collision force
<i>CV</i>	Vessel collision force
<i>EQ</i>	Earthquake
<i>FR</i>	Friction
<i>IC</i>	Ice Load
<i>IM</i>	Vehicular Dynamic Load Allowance
<i>LL</i>	Vehicular Live Load
<i>LS</i>	Live Load Surcharge
<i>PL</i>	Pedestrian Live Load
<i>SE</i>	Settlement
<i>SH</i>	Shrinkage
<i>TG</i>	Temperature Gradient
<i>TU</i>	Uniform Temperature
<i>WA</i>	Water Load and Stream Pressure
<i>WL</i>	Wind on Live Load
<i>WS</i>	Wind Load on Structure

Railway bridge notations

Symbol	Description
A	axle load
A_s	area of steel reinforcement
A_w	area of web
A	the transverse beam span(railway bridge load distribution)
c	distance from extreme fiber to neutral axis, (mm)
D	degree of curve (degrees based on 30 m chord), the effective beam spacing
d	distance from extreme compression fiber to centroid of tension reinforcement, (mm),The transverse beam spacing
d_b	depth from bottom of tie to bottom of ballast ,effective depth of slab for balanced design
E	actual superelevation in (mm)
$F_{V \text{ allowable}}$	allowable shear stress
F_{fat}	allowable fatigue stress
F_y	specified yield strength of reinforcement, (MPa)
f_s	tensile stress in reinforcement at service loads, (MPa).
f_f	stress range in steel reinforcement, (MPa).
f_{min}	algebraic minimum stress level, tension positive, compression negative,(M)Pa
f_v	shear stress
f_r	modulus of rupture of concrete, (MPa).
f'_c	specified compressive strength of concrete, (MPa)
f_c	extreme fiber compressive stress in concrete at service loads, (MPa), allowable service load Extreme fiber stress in compression $f_c = 0.40 f'_c$
f_b	total maximum stress on cross-section

$f\Delta$	various deflection criteria for railway bridge spans
g	acceleration due to gravity
h	web depth, The concrete slab thickness (mm)
I_b	the transverse beam moment of inertia, mm ⁴
I_v	vertical effects due to superstructure –vehicle interaction
I_f	the impact factor for Railway Bridge
j_b	factor for moment arm of tensile and compressive forces on cross-section
k_b	factor for depth of compression zone for balanced design
L	the span length
M_{cr}	cracking moment
N	lateral vibrational load
N	modular ratio = E_s/E_c
P	the portion of axle load on the transverse beam
Q	derailment load
R	radius of curvature of traffic lane in (meters),
RE	effects due to vehicle rocking
(r/h)	ratio of base radius to height of rolled-on transverse deformations; when the actual value is not known, use 0.3.
S	permissible speed in (km/hr), the axle spacing (mm)
S_{nc}	section of modulus about an axis parallel to web
V	the speed of the train in (km/hr)
V_T	top Notional lateral load
V_B	bottom Notional lateral load
W	wheel load
W_v	vertically distributed load from wind on live load.
Δ	vertical deflection

Railway bridge load designation

D, DL	Dead Load
F	Longitudinal Force due to Friction or Shear Resistance at Expansion Bearings
L, LL	Live Load
I	Impact
CF	Centrifugal Force
EQ	Earthquake (Seismic)
E	Earth Pressure
SF	Stream Flow Pressure
B	Buoyancy
ICE	Ice Pressure
W	Wind Load on Structure (can be and on train)
OF	Other Forces (Rib Shortening, Shrinkage, Temperature and/or Settlement of Supports)
WL	Wind Load on Live Load
LF	Longitudinal Force from Live Load
N	Lateral forces from equipment (noising)
LV	“Notional” lateral vibration load
LLT	Live load that creates a total stress increase of 33% over design stress
SL	Live load on leeward track of 1200 ib/ft without impact
f_{fat}	Allowable stress based on member loaded length and fatigue detail category
Q	Derailment Load
CWR	Continuously Welded Rail

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AREMA MRE	American Railway Engineering and Maintenance –Of-Way Association Manual for Railway Engineering
LRFD	Load Resistance and Factor Design
ERA	Ethiopian Road Authority
LFD	Load Factor Design
HL-93	Highway Loading Of 1993

DF	Distribution Factor
AAR	Association of American Railroads
<i>ADTT</i>	The number of trucks per day in one direction averaged over design life
ADTTsL	The number of trucks per day in a single-lane averaged over design life
SR fat	Fatigue Stress Range.

List of Tables

Table 3-1 load multipliers for live loads.....	11
Table 3-2dynamic load allowance	15
Table3-3 impact load for members receiving load from more than one track	17
Table 3-4Transverse Wind Pressure on Structures.....	24
Table 3-5 Wind Pressure on Vehicles with the skew angle.....	25
Table 3-6 multiple presence factors.....	30
Table 4-1 fraction of truck traffic in a single lane	34
Table 5-1 the load factors for various load combinations.	40
Table 5-2 Load Factors for permanent loads.....	38
Table 5-3 group loading combinations for service load design.....	39
Table 5-4 group loading combinations for load factor design.....	40
Table 5-5 Load combinations for steel railway superstructure design	41
Table A1 dead load effects.....	51
Table A2 Steel section properties.....	53
Table A3 Short term composite section properties (n=8).....	54
Table A4 Long term composite section properties (n=24).....	55
Table A5 Stress in bottom of exterior steel girder (tension) for positive flexure due to factored loading, strength I.....	56
Table A 6 Maximum flexural stress in the flange for exterior positive flexure, Service II.....	56
Table B 1 Loads on girders.....	70
Table B 2 Effects from other loads.....	71
Table B 3 applicable load combinations.....	72
Table B 4 Steel section properties.....	74
Table B 5 Composite steel and concrete section properties (shot term n=8).....	75
Table B 6 Section modulus (n=8).....	75
Table B 7 Composite steel and concrete section properties (shot term n=24).....	76

Table B 8 Section modulus (n=24).....	76
Table B 9 Composite section flexural stresses (unshored construction).....	77

List of figures and charts

Figure 3-1 (a) Open-deck through-plate girder (b) Ballasted-deck through-plate girder.....	9
Figure 3.2 design track.....	9
Figure 3.3 design tandem and design lane load.....	10
Figure 3.4 Application of Design Vehicular Live Loads with multispans for negative moment.....	11
Figure 3.5 Cooper E80 (EM 360) axle load diagram.....	11
Figure 3.6 an alternative live load for cooper EM 360 (E80).....	12
Figure 3.9 Centrifugal force for Highway Bridge.....	19
Figure 3.10 Centrifugal forces from a curved track of railway bridge.....	20
Figure 3.11 Derailment load for stability check.....	23
Figure 3.12 Longitudinal distribution to deck.....	30
Figure 3.13 Longitudinal distribution to transverse members.....	30
Figure 3.14 Lateral distribution to lateral members	31
Figure 3.15 Lateral distribution to deck	32
Figure A 1 Steel plate girder bridge design example: (a) General elevation, (b) plan view, and (c) cross-section.....	46
Figure A2 composite section for positive moment Section properties.....	53
Figure A3 Plastic neutral axis for positive moment section.....	57
Figure A4 Cross-frame locations.....	62
Figure A5 Wind load acting on bridge exterior.....	63
Figure B 1 deck plate girder bridge cross-section.....	66
Figure B 2 rocking effect of deck plate Girder Bridge.....	68
Figure B 3 composite section for positive flexure.....	74
Figure B 4 Location of cross-frame.....	79
Figure B 5 Wind loading on exterior girder.....	79
Chart No 1 load effects on highway and railway bridges for strength.....	12
Chart No 2 impact loads of railway and highway bridges for strength.....	16
Chart No 3 Lateral loads on support cross-frames of railway and highway bridges.....	26
Chart No 4 design fatigue and fatigue impact moments of highway and railway bridge.....	36

CHAPTER ONE

INTRODUCTION

1.1 Background

Bridge design is a combination of art and science. Conceptual design is usually the first step. The conceptual design process includes selection of bridge systems, materials, proportions, dimensions, foundations, esthetics, and consideration of the surrounding landscape and the environment. [18]

The final design process involves structural analysis, member detail design, and the preparation of construction drawings and specifications. Structural analysis commonly involves computer models, which use appropriate material properties, member discretization, boundary conditions, and loads. Members and connections joints are proportioned to carry all possible loads (permanent loads, vehicular live loads, wind loads, and earthquake loads), combined and factored in accordance with the requirements of applicable design standards and codes.[18]

Additionally, the types of live loads we used for design of highway bridges and railway bridges are different in many aspects. Accordingly, the ratio of live load to dead load is much higher for a railway bridge than for a similarly sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.[10] The design impact load on railway bridges is higher than on highway bridges. Since the bridge supports the track structure, the combination of track and bridge movement cannot exceed the tolerances in track standards. Interaction between the track and the bridge should be considered in designing and detailing.[18]

The dynamic response of railway bridge under train loads is one of the fundamental problems to be solved in railway bridge design and maintenance. Therefore great efforts have been constantly spent to the subject of dynamic interaction of vehicles and bridges. The base document for railway bridge design, construction, and inspection is the American railway engineering maintenance of way association (AREMA). These dynamic responses include resonance in high speed trains, rocking effect etc. [18]

The design of the bridge superstructure (or any other structural element for that matter) is based on a set of loading conditions which the component or element must withstand .the

design must take in to account a wide variety of loads which vary based on [11]

1. duration(permanent or temporary)
2. direction(vertical, longitudinal ,etc)
3. deformation (concrete creep, thermal expansion .etc.)
4. effect (shear, bending, torsion ,etc)

A number of differences exist between railroad and highway bridges [10]:

1. The ratio of live load to dead load is much higher than for a similarly sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.
2. The design impact load on railroad bridges is higher than on highway structures
3. Simple span structures are preferred over continuous structures for railroad bridges. many of the factors that make continuous spans attractive for highway structures are not as advantageous for railway use.
4. Interruptions in services are typically much more critical for railroads than for highway agencies. Therefore, constructability and maintainability without interruption to traffic are crucial for railroad bridges.
5. Since the bridges support the track structure, the combination of track and bridge movement cannot exceed the tolerances in track standards. Interaction between the track and the bridge should be considered in design and detailing.
6. Seismic performance of highway and railway bridges can vary significantly. railroad bridges have performed well
7. Railroad bridge owners typically expect a longer service life from their structures than Highway Bridge owners expect from theirs.

1.2 Aim and scope of the thesis

Ethiopian engineers have good experience of highway bridge designs. Currently the railway bridges are necessary due to new railroad projects in the country. It is absolutely necessary to bring the well developed knowledge of the engineers on Highway Bridge to Railway Bridge engineering because Railway Bridge needs careful design and detailing.

A variety of loads can be applied to a structure at the same time. For example a bridge may experience dead load, live load, impact, centrifugal force, wind, and stream flow simultaneously. The AREMA manual chapters on structure design recognize that it is unlikely that the maximum values of all loads will be applied concurrently to a structure. Load combination methods are given to develop maximum credible design forces on the structure. The objective of the thesis is to provide the bridge engineers with auxiliary manual material consists of comparison of load types as well as load combinations between highway and railway bridges as a quick reference in the process of design.

The comparison mainly focuses on loads which primarily affects the superstructure, such loads as Live load, Centrifugal force, Lateral force due to wind load and nosing of locomotives, longitudinal force and Impact between the railway and highway bridge. Loads which primarily affect substructures, such as earthquake load, water load are not included. AASHTO LRFD specification and AREMA MRE are the main sources of the loads on Highway Bridge and on Railway Bridge used in this thesis respectively.

1.3 Lay out of the thesis

Chapter two of the thesis focuses on design philosophy of the two types of bridges. Highway Bridges are currently based on LRFD method (evolving through some steps eventually to limit state design) which ERA bridge manual and AASHTO LRFD basis. the railway bridge (as allowed in AREMA MRE) is based on allowable stress and LFD method both can be used for concrete railway bridge . the former is used for design of railway steel brides.

The next chapter deals with loads on bridges. The loads mainly are those which primarily affect the superstructure of the bridges. Such loads as vehicular live load, dynamic allowances, centrifugal forces, longitudinal forces, wind forces and loads that are specific to either bridges. Beside of these comparisons of loads the chapter tries to show vehicular load distributions, multiple presences on both types of bridge.

Fatigue loads and serviceability requirements of both bridges are the main subject of chapter four, which includes vehicular fatigue loads and deflection limits of the bridges.

Chapter five deals with the load combinations of both types of bridges. Highway bridge load combinations are based on LRFD limit states (strength, service, fatigue and fracture, and extreme event limit states). Load combination for Concrete Railway bridges can be based on either allowable stress or LFD method, depending on the engineer, and that of steel rail way is based on allowable stress method.

Chapter six is the conclusion and recommendation part of the thesis. Appendices A and B are also included to show some necessary steps of design of both types of bridges to clarify the design criteria, load types and combinations.

1.4 Notation

The notations used in the thesis are from different references, mainly AASHTO LRFD specification and AREMA MRE and other reference books use similar notations as the codes. Load designations in both codes are used here directly to be consistent. Furthermore, whenever there is an expression of, for instance, [A 2.1.2.2] or [C 2.1.2.2], it refers to AASHTO LRFD specification reference or commentary respectively. Similarly if there is an expression of, for example, (ch.15, 1.3.10 b), it refers to the stated chapter and section on AREMA MRE.

CHAPTER TWO

DESIGN PHILOSOPHY

2.1 Highway Bridge

The design provisions of AASHTO Specifications and ERA Bridge manual currently employ the Load and Resistance Factor Design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance. The concepts of safety through redundancy and ductility and of protection against scour and collision are emphasized. Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics, Regardless of the type of analysis used, shall be satisfied for all specified force effects and combinations thereof.[4]

Limit States [4]

A limit state is a condition beyond which a bridge system or bridge component ceases to fulfill the function for which it is designed. The basic design expression in the AASHTO (2004b) LRFD Bridge Specifications that must be satisfied for all limit states, both global and local, is given as:

$$\eta_i \gamma_i Q_i \leq \phi R_n = R_r \dots \dots \dots \text{Eqn 2.1}$$

where Q_i is the force effect, R_n is the nominal resistance, γ_i is the statistically based load factor applied to the force effects, ϕ is the statistically based resistance factor applied to nominal resistance, and η_i is a load modification factor. For all nonstrength limit states, $\phi = 1.0$. This equation is the basis of LRFD methodology.

Service Limit State

The service limit state provides certain experience related provisions that cannot always be derived solely from strength or statistical considerations. The service limit state shall be taken as restrictions on stress, deformation, and crack width under regular service conditions.

Fatigue and Fracture Limit State

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge. The fatigue limit state shall be taken as restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles. The fracture limit state shall be taken as a set of material toughness requirements of the AASHTO Material Specifications.

Strength Limit State

Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained. Strength limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

Extreme Event Limit States

Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge. The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions.

Ductility, redundancy, and operational importance are significant aspects affecting the margin of safety of bridges. Ductility is important to the safety of a bridge. If ductility is present, overloaded portions of the structure can redistribute the load to other portions that have reserve strength. This redistribution is dependent on the ability of the overloaded component and its connections to develop inelastic deformations without failure.

Redundancy significantly affects the safety margin of a bridge structure. A statically indeterminate structure is redundant, that is, has more restraints than are necessary to satisfy equilibrium.

Bridges can be considered of operational importance if they are on the shortest path between residential areas and a hospital or school or provide access for police, fire, and rescue vehicles to homes, businesses, and industrial plants. Bridges can also be considered essential if they prevent along detour and save time and gasoline in getting to work and back home again.

In the event of an earthquake, it is important that all lifelines, such as bridges, remain open. Therefore, the following requirements apply to the extreme event limit state as well as to the strength limit state:

$$\eta I \geq 1.05 \text{ for a bridge of operational importance}$$

$$\eta I = 1.00 \text{ for typical bridges}$$

$$\eta I \geq 0.95 \text{ for relatively less important bridges}$$

For all other limit states:

$$\eta I = 1.00$$

2.2 Railway Bridge

Railway routes are well established and the construction of new railway routes is not common; thus, the majority of the Railway bridges built or rehabilitated are on existing routes and on existed right-of-way. In other words the railway industry first extends the life of existing bridges as long as economically justified. it is not uncommon for railway to evaluate an 80-or 90-year-old bridge. Estimate its remaining life, and then rehabilitate it sufficiently to extend its life for some economical period of time. The majority bridges by far are simple-span structures over streams or roadways. Complex bridges are generally associated with crossing major water ways or other significant topographical features.[10]

2.2.1 Reinforced concrete Railway Bridge

The design methods to be used for reinforced concrete bridges, Service Load Design or Load Factor Design (LFD), as directed by the Engineer. For reinforced concrete members designed with reference to service loads and allowable stresses, the service load stresses shall not exceed the values given AREMA MRE For reinforced concrete members designed with reference to load factors and strengths, the design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, and shear, shall be taken as the nominal strength calculated in accordance with the requirements and assumptions of Load Factor Design, multiplied by a strength reduction factor Φ .[2]

2.2.2 Steel Railway Bridges

Elastic analysis procedures are usually used for steel railroad bridges, and Chap. 15 of the AREMA *MRE* provides recommendations for allowable stress design (Service Load Design). The design service life of railroad bridges is generally considered to be about 80 years. [5]

CHAPTER THREE

LOADS ON BRIDGES

3.1 Permanent loads

3.1.1 Highway bridges

Permanent loads are those that remain on the bridge for an extended period of time, perhaps for the entire service life such loads include:[11]

- Dead load of structural components and nonstructural attachments (DC)
- Dead load of wearing surfaces and utilities (DW) and future overlays and planned widenings.
- Dead load of earth fills (EV)
- Earth pressure load (EH)
- Earth surcharge load (ES)
- Locked-in erection stresses (EL)
- Downdrag (DD)

3.1.2 Railway bridges

Superstructure dead load consists of the weight of the superstructure itself ,track, deck(open ballasted) ,utilities(conduits ,pipes and cables),walk ways(some engineers also include walk way live load as a component of super structure dead load.),permanent formwork, snow ,ice, and anticipated future dead loads(e.g. larger deck ties, increases in ballast depth, and additional utilities).

Open-deck through-plate girder, through-truss, and some deck truss spans usually contain floor systems comprised of longitudinal stringers and transverse floorbeams (Fig 3.1 (a)). Ballasted-deck through-plate girder spans generally have the concrete or steel plate decks supported on closely spaced transverse floorbeams framing into the main girder (Fig 3.1 (b)). In some cases, such as through-truss spans, stringers with less closely spaced transverse floorbeams are used. [5]

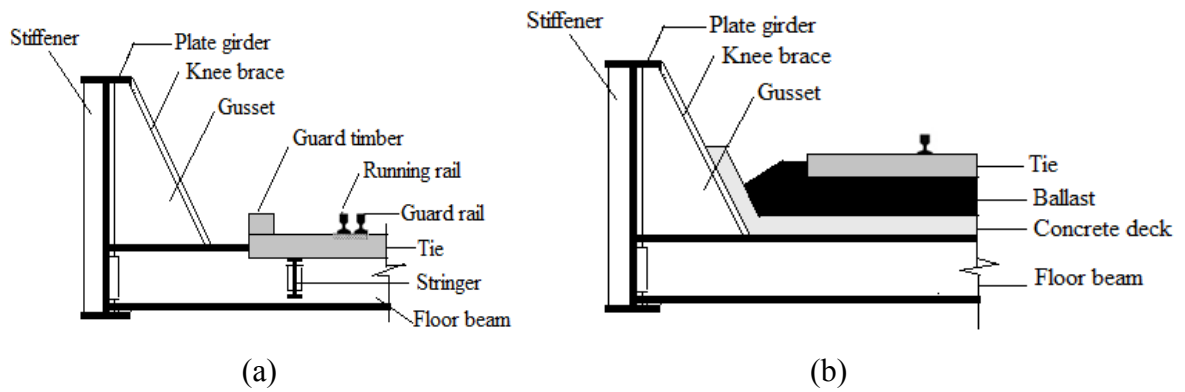


Figure3-1 (a) Open-deck through-plate girder (b) Ballasted-deck through-plate girder [12]

3.2 Vehicular live load

3.2.1 Highway Bridge

AASHTO LRFD specification recommends Vehicular live loading on the highways of bridges or incidental structures, designated HL-93, consist of a combination of the:[11]

- Design truck
- Design tandem
- Design lane

Design truck

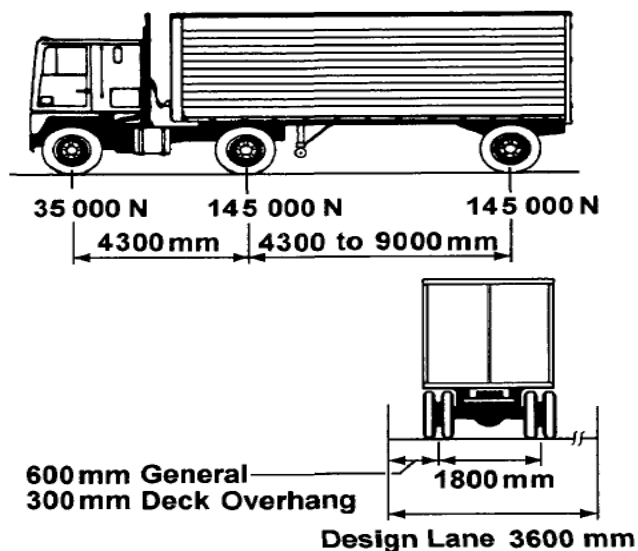


Figure 3-2 design truck [1]

The design truck (shown in fig 3-2) is a model load that consists of front axle weighs 35 kN, and the drive axle of 145 kN is located 4300 mm behind, and the rear trailer axle is also 145 kN and is positioned at a variable distance ranging between 4300 and 9000 mm.

Design tandem

The second configuration is the design tandem and consists of two axles weighing 110 kN each spaced at 1200 mm.

Design lane

The third load is the design lane load that consists of a uniformly distributed load of 9.3 N/mm and is assumed to occupy a region 3000 mm transversely. This load is the same as a uniform pressure of 3.1 kPa applied in a 3000-mm design lane.

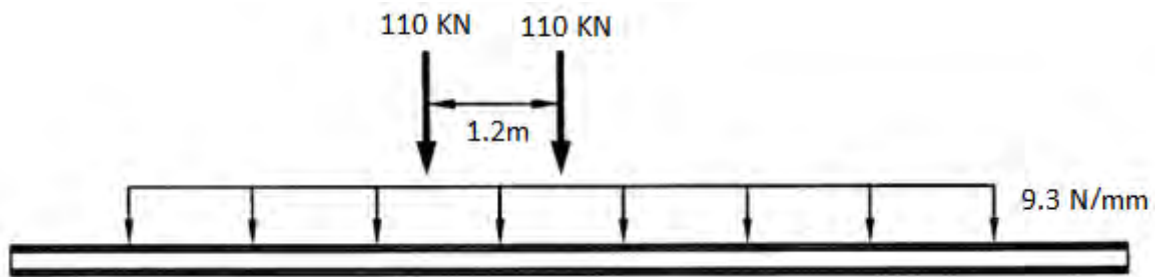


Figure 3-3 design tandem and design lane load [4]

Application of Design Vehicular Live Loads [A3.6.1.3.]

- The load effects of the design truck and the design tandem must each be *superimposed with* the load effects of the design lane load.
- For both negative moment (tension on top) between points of contra-flexure under a uniform load on all spans, and reaction at interior supports, 90 percent of the effect of two design trucks spaced a minimum of 15 000 mm between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 145-kN axles of each truck shall be taken as 4300 mm.

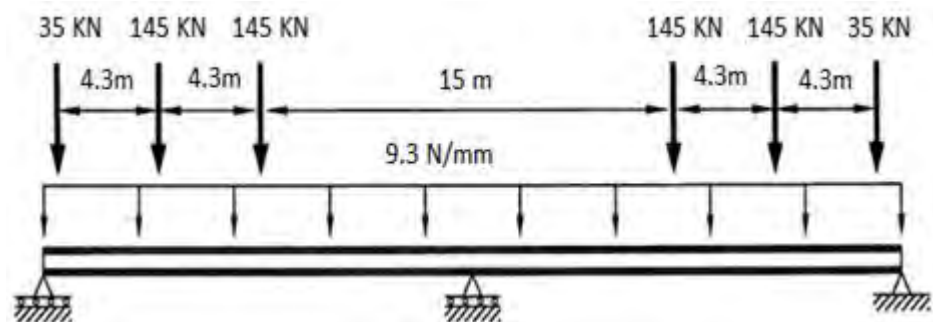


Figure 3-4 Application of Design Vehicular Live Loads with multi-spans for maximum negative moment [4]

▪ **Deck and Railing Loads [4]**

The deck must be designed for the load effect due to the design truck or the design tandem, whichever creates the most extreme effect. The two design vehicles should not be considered together in the same load case. The deck overhang, located outside the fascia girder and commonly referred to as the cantilever, is designed for the load effect of a uniform line load of 14.6 N/mm located 300 mm from the face of the curb or railing. The traffic barrier system and the deck overhang must sustain the infrequent event of a collision of a truck. The deck overhang and railing design is confirmed by crash testing as outlined in AASHTO [A13.7.2]. The appropriate *multiplier* to be used prior to superposition is shown in table 3.1

Table 3-1 load multipliers for live loads [4]

Live load Combination	Design Truck	Design Tandem	Two Design Trucks or Tandem with 15m Headway	Design Lane
1	1.0			1.0
2		1.0		1.0
3			0.9	0.9

3.2.2 Railway Bridge

The rail way bridge design live load recommended in AREMA (2010) is cooper’s E80 (EM 360) load, which is based on two consolidation –type steam locomotives with trailing cars represented by a uniformly distributed load. The maximum locomotive axle load is 360KN and freight equipment is represented by a uniform load of 120KN/m of track.[5]

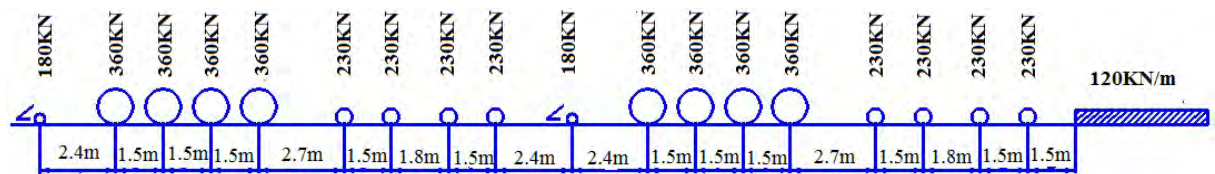


Figure 3-5 Cooper E80 (EM 360) axle load diagram [2]

An alternative live load, consisting of four 450 KN axles, is also recommended in order to represent the stress range effects of adjacent heavy rail cars on short spans.

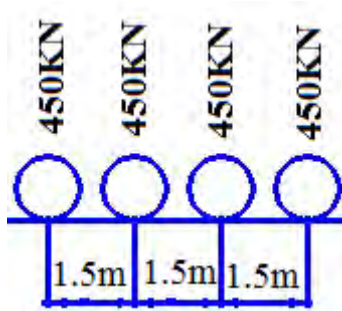


Figure 3-6 an alternative live load for cooper EM 360 [2]

From Appendix A the design highway distributed live load effects for strength are

$$\text{Moment, } (M_{LL+IM}) + M_{\text{Lane}} = 2910.5 \text{ KNm and}$$

$$\text{Shear force, } (V_{LL+IM}) + V_{\text{Lane}} = 450 \text{ KN.}$$

Similarly, from Appendix B, the design live load effects of Railway Bridge for strength are,

$$\text{Moment, } M_{LL+IM \text{ max +ve}} = 11,660 \text{ KNm, and}$$

$$\text{Shear force, } V_{LL+IM \text{ max +ve}} = 1736 \text{ KN.}$$

Based on rough estimation, it can be easily observed the bending moment on Highway Bridge is 25% of the bending moment on Railway Bridge. Similarly the shear force on the railway bridge is four times that of Highway Bridge.

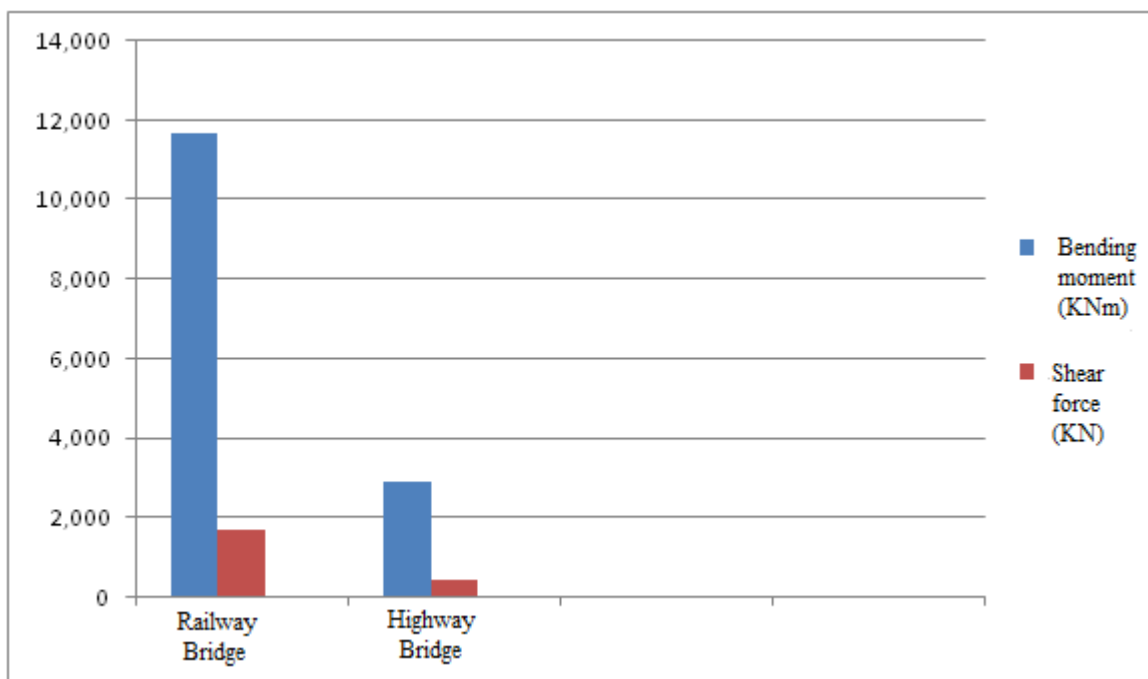


Chart No 1 load effects on highway and railway bridges for strength

In regard with Euro code the live load used for highway bridge design is based on a series of load models to be applied in different situations. The load models for vertical loads represent the following traffic effects:[20]

- a) Load Model 1 (LM1) : Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.
- b) Load Model 2 (LM2) : A single axle load applied on specific tyre contact areas which covers the dynamic effects of the normal traffic on short structural members.

NOTE 1 As an order of magnitude, LM2 can be predominant in the range of loaded lengths up to 3m to 7m.

NOTE 2 the use of LM2 may be further defined in the National Annex.

- c) Load Model 3 (LM3): A set of assemblies of axle loads representing special vehicles (e.g. for industrial transport) which can travel on routes permitted for abnormal loads. It is intended for general and local verifications.
- d) Load Model 4 (LM4): A crowd loading intended only for general verifications.

NOTE this crowd loading is particularly relevant for bridges located in or near towns if its effects are not obviously covered by Load Model 1.

(3) Load Models 1, 2 and 3, where relevant, should be taken into account for any type of design situation (e.g. for transient situations during repair works).

(4) Load Model 4 should be used only for some transient design situations. The details of Load Model 1 are illustrated in Figure 3.7.

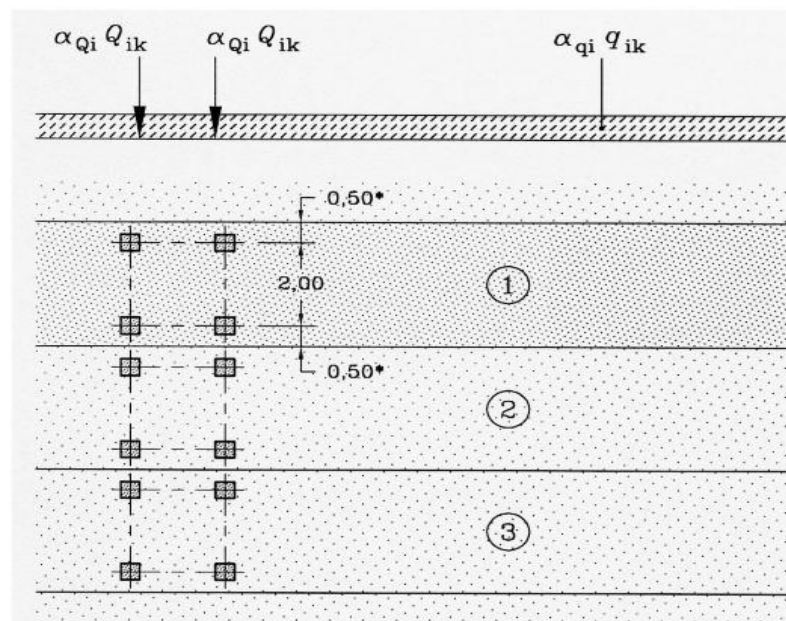
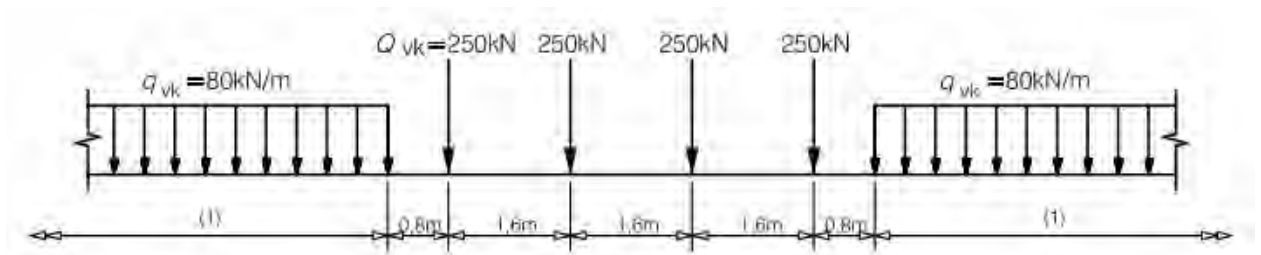


Figure 3.7 - Application of load Model 1 [20]

The vertical actions used for Railway Bridge according to euro code are defined by means of load models. Five models of railway loading are given:[20]

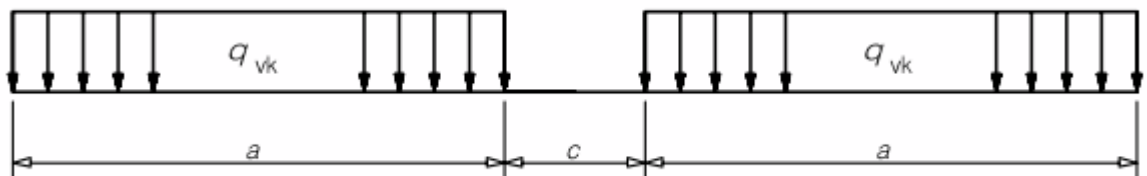
- Load Model 71 (and Load Model SW/0 for continuous bridges) to represent normal rail traffic on mainline railways,
- Load Model SW/2 to represent heavy loads,
- Load Model HSLM to represent the loading from passenger trains at speeds exceeding 200 km/h,
- Load Model “unloaded train” to represent the effect of an unloaded train.



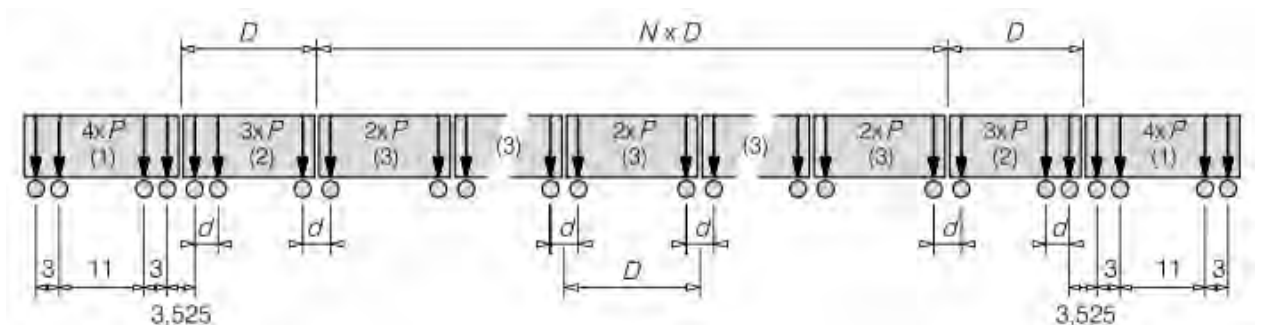
Key

(1) No limitation

(a) - Load Model 71 and characteristic values for vertical loads



(b) Load Models SW/0 and SW/2



(c) HSLM-A

Figure 3.8 railway bridge live load models [20]

3.3 Dynamic loads

3.3.1 Highway Bridge [1]

The dynamic load allowance (IM) is an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles. Dynamic effects due to moving vehicles may be attributed to two sources:

- Hammering effect is the dynamic response of the wheel assembly to riding surface discontinuities, such as deck joints, cracks, potholes, and delimitations, and
- Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

Table 3-2 dynamic load allowance (IM) [1]

Component	IM (%)
Deck joints-all limit states	75
All other components	
Fatigue and fracture limit states	15
All other limit states	33

All other components in Table 3.2 include girders, beams, bearings (except elastomeric bearings), and columns. These factors are to be applied to the static load as

$$UL+I = UL (1 + IM)$$

Where $UL+I$ is the live-load effect plus allowance for dynamic loading, UL is the live-load effect of live load, and IM is the fraction given in the Table 3.2.

3.3.2 Rail way bridges

3.3.2.1 Steel railway bridges [5]

The impact factor for railway bridge comprises the effects due to vehicle rocking, RE , and the vertical effects due to superstructure –vehicle interaction, I_v , or

$$I_f = RE + I_v$$

Rocking effect [5]

Railroad freight equipment will rock or sway in a lateral direction due to wind forces, rail profile variances, and equipment spring stiffness differences. rocking due to rail and equipment conditions will affect the magnitude of equipment axle loads and is considered as a dynamic increment of axle load by AREMA(2010).the Rocking effect ,RE, is determined for each member supporting the track as a percentage of the vertical live load. as recommended by AREMA(2010),the applied rocking effect is the force couple created by an upward force on one rail and downward force on the other rail of 20% of the design wheel load, or 0.2W, where W is the wheel load(1/2 axle load).

Vertical effects [5]

Superstructure vibration is induced by the moving load (locomotives and cars) suspension systems as the loads traverse a railway bridge with surface irregularities. AREMA (2010) recommended impact due to vertical effects for simply supported open deck steel bridges as

- Percentage of live load for rolling equipment without hammer blow (freight and passenger cars, and Locomotives other than steam):

- For L less than 24m: $40 - \frac{3L^2}{145}$

- For L 24m or more: $16 + \frac{183}{L-9}$

- Percentage of live load for steam locomotives with hammer blow:

- For beam spans, stringers, girders, floor beams, posts of deck truss spans carrying load from floor beam only, and floor beam hangers:

- For L less than 30m: $60 - \frac{L^2}{46.5}$

- For L 30m or more: $10 + \frac{548}{L-12}$

- For truss spans: $15 + \frac{1219}{L+7.62}$

For members receiving load from more than one track, the impact load shall be applied on the number of tracks designated in Table 3.3.

Table3-3 impact load for members receiving load from more than one track [2]

Span Length ,L	Impact
Loads Received From Two Tracks	
For L less than 53m	Full impact on two tracks
For L from 53m to 69m	Full impact on one track and a percentage of Full impact on the other as given by the formula, $450-6.56L$
For L greater than 69m	Full impact on one track and none on the other
Load Received From More than Two Tracks	
For all values of L	Full impact on any two tracks that creates the largest load effect

The impact load for ballasted deck steel bridges may be reduced to 90% of the total impact load determined for open deck bridges. Design impact load is the sum of the impact load due to rocking and vertical effects.[5]

3.3.2.2 Concrete railway bridges [2]

Impact forces, applied at the top of rail, shall be added to the axle loads specified. For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.), the impact shall be equal to the following percentages of the live load:

- For $L \leq 4$ meters $I = 60$
- For $4 \text{ meters} < L \leq 39 \text{ meters}$ $I = \frac{125}{\sqrt{L}}$
- For $L > 39$ meters $I = 20$, L is the span length in meters

For continuous structures, the impact value calculated for the shortest span shall be used throughout. For steam locomotives with hammer blow, the impact calculated shall be increased by 20%.

The impact load used for Highway Bridge, in regard with Appendix A

For strength =33%,

$$\begin{aligned} \text{Moment, } M_{LL+IM \text{ max +ve (WITH OUT LANE LOAD)}} &= 0.775 * 2037.6 * .33 (\text{Governing DF}) \\ &= 521 \text{ KNm} \end{aligned}$$

$$\begin{aligned} \text{Shear force, } V_{LL+IM \text{ max +ve (WITH OUT LANE LOAD)}} &= 0.849 * 294 * .33 (\text{Governing DF}) \\ &= 82.4 \text{ KN} \end{aligned}$$

And the impact load used for that of Railway Bridge from appendix B is

For strength =33 %,(from vertical and rocking effects)

$$M_{LL+IM \max +ve} = 8767 \text{KNm} * .33$$

$$= 2893 \text{KNm}$$

$$V_{LL+IM \max +ve} = 1335 \text{KN} * .33$$

$$= 441 \text{KN}$$

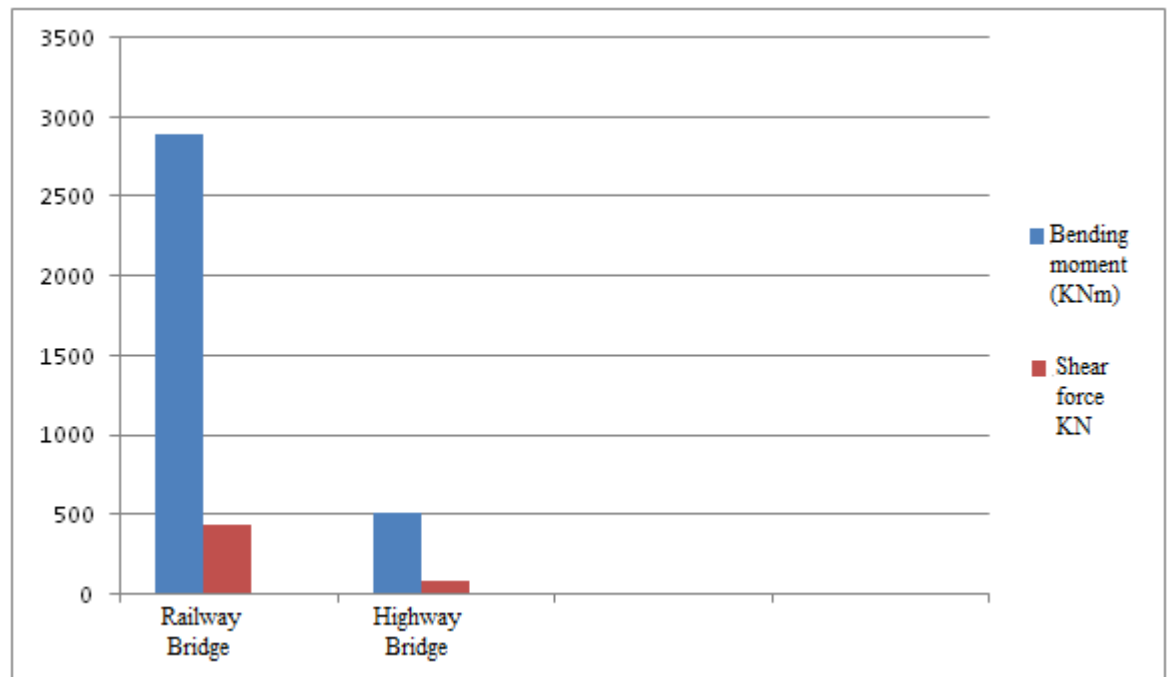


Chart No 2 impact loads of railway and highway bridges for strength

Though, coincidentally, both bridges have same percentage of impact load for strength, the increment due to the impact load is inevitably much higher on Railway Bridge. Thus, both the bending moment and shear force added by impact on Highway Bridge are about 18% of that of Railway Bridge.

3.4 Centrifugal force

3.4.1 High way bridges [4]

As a truck moves along a curvilinear path, the change in direction of the velocity causes a centrifugal acceleration in the radial direction. AASHTO [A3.6.3] gives an expression for the force on the truck directed toward the center of the curve (outward on the bridge) as

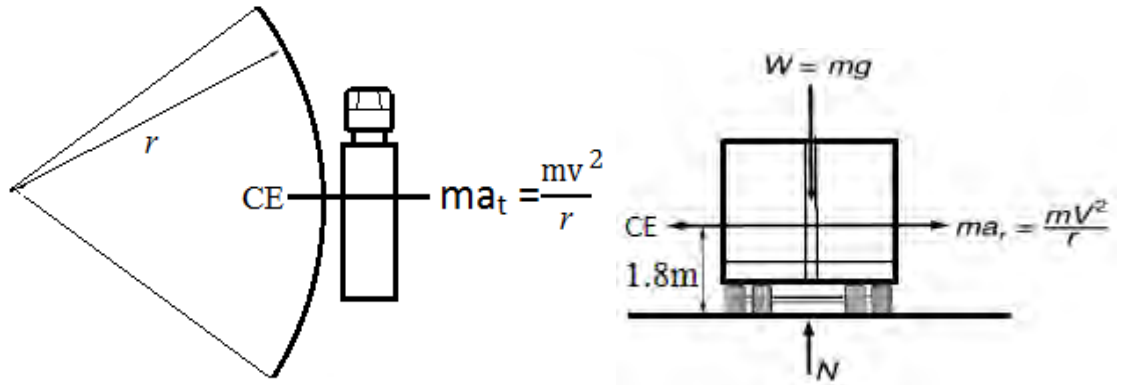


Figure 3-9 centrifugal force for Highway Bridge [4]

$$CE = CW$$

$$\text{Where } C = f \left(\frac{v^2}{Rg} \right)$$

- $f = 4/3$ for combinations other than fatigue and is $f = 1.0$ for fatigue;
- v the highway design speed in meters/second,
- R radius of curvature of traffic lane in meters, and
- CE applied at the assumed center of mass at a distance of 1800 mm above the deck surface.

Lane load is neglected in computing the centrifugal force, as the spacing of vehicles at high speed is assumed to be large, resulting in a low density of vehicles following and/or preceding the design truck. Centrifugal forces shall be applied horizontally at a distance 1800 mm above the roadway surface, and transformed as a couple to the girder.

3.4.2 Rail Way Bridge [5]

Centrifugal forces acting horizontally at the vehicle center of gravity (recommended as 2.4m above the top of the rails in AREMA, 2010) act on the moving live load as it traverse the curved track on the bridge. The centrifugal force corresponding to each axle load is

$$CF_A = \frac{m_A V^2}{R}$$

Where $m_A = A/g$; where A is the axle load , g is the acceleration due to gravity , V is the speed of the train in mph(Km/h),and R is the radius of curvature . The relationships between speed, degree of curve, centrifugal force and a superelevation which is 75 mm less than that required for zero resultant flange pressure between wheel and rail are expressed by the formulas:

$$C = 0.000452 S^2 D$$

$$E = 0.0068 S^2 D - 75$$

$$S = \sqrt{\left(\frac{E + 75}{0.0086D}\right)}$$

Where:

C = Centrifugal force in percentage of the live load

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

E = Actual super-elevation in mm

S = Permissible speed in km/hr

The centrifugal force is resisted by couple on the girders and the upward load on the intrados can be neglected conservatively.

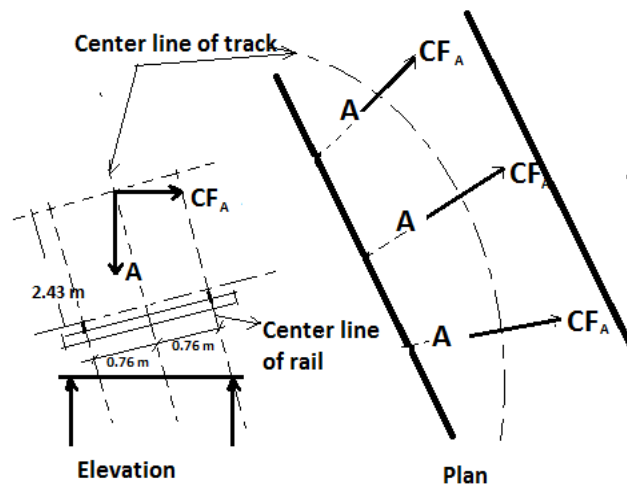


Figure 3-10 Centrifugal forces from a curved track of Railway Bridge [5]

3.5 Longitudinal forces due to braking and traction

3.5.1 High Way Bridge

Braking Force: BR [4]

Based on energy principles, and assuming uniform deceleration, the braking force determined as a fraction of vehicle weight is:

$$b = \frac{v^2}{2ga}$$

Where a is the length of uniform deceleration and b is the fraction. Calculations using a braking length of 122 m and a speed of 90 km/hr. (25 d sec.) yield $b = 0.26$ for a horizontal

force that will act for a period of about 10 seconds. The factor b applies to all lanes in one direction because all vehicles may have reacted within this time frame.

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem or,
- 5 percent of the design truck plus lane load or 5 percent of the design tandem plus lane load

These forces shall be assumed to act horizontally at a distance of 1800 mm above the roadway surface in either longitudinal direction to cause extreme force effects.

3.5.2 Railway bridges [5]

Longitudinal forces, due to train braking (acting at the center of gravity of the live load) and locomotive tractive effort (acting at the freight equipment draw bars or couplers), are considerable for modern railway freight equipment.

- The longitudinal force for E-80 loading shall be taken as the larger of:
 - Force due to braking, as prescribed by the following equation, acting 2500 mm above top of rail:

$$\text{Longitudinal braking force (KN)} = 200 + 17.5 L$$

- Force due to traction, as prescribed by the following equation, acting 900 mm above top of rail:

$$\text{Longitudinal traction force (kN)} = 200 \sqrt{L}$$

Where:

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

For design loads other than EM-360, these forces shall be scaled proportionally. The points of force application shall not be changed. The distribution of longitudinal forces for the design of span bracing, bearings, substructures, and foundations need careful consideration. The distribution and path of longitudinal forces between their point of application and the bridge supports depend on the arrangement, orientation, and relative stiffness of

- Bridge members in the load path
- Bearing type
- Substructure characteristics

3.6 Loads significant for Railway Bridge

3.6.1 Lateral forces from equipment [5]

In addition to the centrifugal lateral forces due to track curvature, lateral forces caused by track irregularities and the wheel rail interface geometry must be considered in the design of railway bridges.

A single moving concentrated lateral force equal to one-quarter of the weight of the heaviest axle of the specified live load, without impact, shall be applied at the base of rail in either direction and at any point along the span in addition to the other lateral forces specified. On spans supporting multiple tracks, the force shall be applied on one track only.

The magnitude of lateral forces is of particular importance regarding the design of span lateral and cross bracing and are considered applied directly to bracing members without producing lateral bending of supporting member flanges or chords

3.6.2 Lateral vibration loads [5]

In order to simplify the design procedures and ensure global rigidity of the superstructure. AREMA (2010) recommend a notional vibration load to be resisted by the lateral bracing. This design load is not combined with other loads and forces as it serves ensuring adequate lateral bracing (lateral stiffness to resist vibration from live load).therefore, it is to be applied to the lateral bracing as an alternative to the wind load on a loaded railway bridge. This notional load is to be taken as 2.965KN/m on the loaded chord or flange of the superstructure and 2.22KN/m on the unloaded chord or flange.

3.6.3 Forces from continuously welded rail (CWR) [5]

Continuously welded rail is used in modern track construction because it diminishes dynamic effects (no impact loads due to joints in the rail),provides a smoother ride and results in reduced rail maintenance and increased tie life. AREMA (2010) and many railway companies establish standard practice for anchoring CWR to long open deck steel span. In general the recommended practice is to use longitudinal rail anchors on approaches, and near fixed ends of spans, allowing some movement near expansions ends of spans.

3.6.4 Loads for stability check [5]

Events such as train derailments on bridges are relatively infrequent. However, particularly on long bridges, train derailments can occur and create overall stability of individual spans. AREMA (2010) recommends an eccentric derailment load be used to ensure stability of

spans. This derailment load, Q , is a single line of wheel loads, including impact, at a 1.5m eccentricity from the track center line. it is used as a load case for the design of cross frames and diaphragms in beam and girder spans requiring lateral bracing.

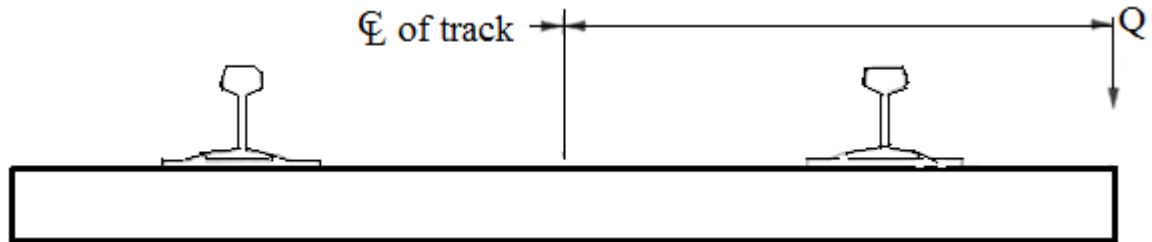


Figure 3-11 derailment load for stability check [2]

3.6.5 Loads for overall lateral stability [5]

The overall stability of the superstructure against wind, noising, and centrifugal forces must be also ensured. The stability of spans and towers should be calculated using a live load, without impact, of 17.8KN/m (represents a uniform load of empty rail cars).on multiple track bridges this live load should be placed on the most leeward track on the bridged.

3.6.6 Geometrical load [5]

The superelevated and curved track creates horizontal eccentricities based on the horizontal curve geometry (track curvature effect) and vertical superelevation (track shift effect).these eccentricities must be considered when determining the lateral distribution of live load forces(including dynamic forces) to members(stringers, floorbeams, and main girders or trusses).these effects are often negligible for short spans and shallow curvatures.

3.6.7 Resonance phenomena [8]

besides the effect of impact as an increase of static effects owing to the movement of rolling-stock, interference of their suspension systems with the rails and attachment to the structure itself which is particularly important at the wheel-rail interface and decreases drastically in railways with very good maintenance, High-speed railways can be affected by another phenomenon: the resonance of the structure. If the frequency of entry of the axles of the wagons into the structure, i.e. the excitation frequency, coincides with the primary vibration frequency of the structure, its deflections are gradually increased as the train passes over it, giving rise not only to greater stresses, but also to vertical accelerations of the deck, which, if they exceed a given value (0.35 g for ballasted deck 0.5g for unballasted track), can cause the wheels to leave the rails and the ballast to be shaken free from the deck, with the risk of detachment of the rails.

A draft has been worked out for a proposed revision of Eurocode clauses on dynamic effects including resonance. as for simply supported bridges the results of ERRI D214 showed that dynamic analysis is not required if the given limits of vertical accelerations of the deck (0.35 g for ballasted deck 0.5g for unballasted track) are satisfied. Moreover, resonance is unlikely for spans more than 40m.[17]

3.7 Wind forces on bridges

3.7.1 Highway Bridge [11]

Wind loading offers a complicated set of loading conditions which must be idealized in order to provide a workable design. Although the problem of modeling wind forces is a dynamic one, with winds acting over a given time interval, these forces can be approximated as a static load being uniformly distributed over the exposed regions of a bridge.

The loading on a bridge due to wind forces is specified by AASHTO based on an assumed wind velocity of 160 Km/h. With regard to the superstructure, wind forces are applied in a transverse and longitudinal direction at the center of gravity of the exposed region of the superstructure. AASHTO LRFD offers a set of wind loading values for truss and girder bridges based on the angle of attack (skew angle) of wind forces. Conventional slab on stringer bridges, however, with span lengths less than or equal to 38.1 m can utilize the following basic loading.

3.7.1.1 Wind Pressure on Structures: WS [1]

- Transverse loading

Table 3-4 Transverse Wind Pressure on Structures [1]

SUPER STRUCTURE COMPONENT	WIND WARD LOAD, MPa	LEEWARD LOAD, MPa
Trusses, columns, and arches	0.0024	0.0012
Beams	0.0024	NA
Large flat surfaces	0.0019	NA

- Longitudinal loading = 0.57 KN/m^2

The total wind loading shall not be taken less than 4.4 N/mm in the plane of a windward chord and 2.2 N/mm in the plane of a leeward chord on truss and arch components, and not less than 4.4 N/mm on beam or girder spans.

3.7.1.2 Wind Pressure on Vehicles: WL [1]

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles shall be represented by an interruptible, moving force of 1.46 N/mm acting normal to, and 1800 mm above, the roadway and shall be transmitted to the structure. When wind on vehicles is not taken as normal to the structure, the components of normal and parallel force applied to the live load may be taken as specified in Table 3.5 with the skew angle taken as referenced normal to the surface.

Table 3-5 Wind Pressure on Vehicles with the skew [1]

Skew Angle	Normal Component	Parallel Component
Degrees	N/mm	N/mm
0	1.46	0.00
15	1.28	0.18
30	1.20	0.35
45	0.96	0.47
60	0.50	0.55

- Longitudinal loading=0.58 KN/m²

The transverse and longitudinal loads are to be placed simultaneously for both the structure and live load.

3.7.1.3 Vertical Wind Pressure (effect of overturning) [1]

To account for the effect of overturning, AASHTO specifies A vertical upward wind force of 9.6×10^{-4} MPa times the width of the deck, including parapets and sidewalks, shall be considered to be a longitudinal line load. This force shall be applied only for limit states that do not involve wind on live load, and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge. This lineal force shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads. This load may govern where overturning of the bridge is investigated.

3.7.2 Railway Bridge

3.7.2.1 Wind forces on loaded bridge

In contrast to long span or flexible bridges (such as suspension or cable stayed bridges), ordinary bridges (such as those composed of beam, girder, truss, and arch spans) need not consider aerodynamic effects of the wind in design. However, the aerostatic effects of the wind on the superstructure and moving train must be considered, particularly in regard to lateral bracing design. [5]

In general, the wind force shall be considered as a moving load acting in any horizontal direction. As a minimum, the bridge shall be designed for laterally and longitudinally applied wind forces acting independently as follows:[2]

- Lateral wind loading on train
 - On the train, the lateral wind force shall be taken at 1.33 KN per linear meter applied normal to the train on one track at a distance of 2.4m above top of rail.
- Lateral Wind loading on bridge [5]
 - On the bridge, lateral wind pressure shall be taken at 1.48KN/m² (The AREMA (2010) design recommendations for wind load on a loaded steel railway bridge superstructure assume that the maximum wind velocity at which trains can safely operate (to avoid overturning of empty cars) will produce a wind pressure of 1.48KN/m² normal to the following surfaces:
 - a) For girder spans, 1.5 times the vertical projection of the span.(in order to account for the effects of paired or multiple girders, these wind pressures are to be applied to a surface 50% greater than the projected surface area of a girder span.)
 - b) For truss spans, the vertical projection of the span plus any portion of leeward trusses not shielded by the floor system.
 - c) For viaduct towers and bents, the vertical projection of all windward and leeward columns and bracing.
 - The lateral wind force on girder and truss spans, however, shall not be taken as less than 0.89 KN/m for the loaded chord or flange and 0.665KN/m for the unloaded chord or flange, neglecting the wind force on the floor system.
- Longitudinal wind loading [5]
 - The longitudinal wind force on spans shall be taken as:
 - a) For girder spans, 25 percent of the lateral wind force.

- b) For truss spans, 50 percent of the lateral wind force.
- c) For viaduct towers and bents, 1.48KN/m^2 on the vertical projection of all windward and leeward columns and bracing.

3.7.2.2 Wind forces on unloaded bridge

In general, the wind force shall be considered as moving load acting in any horizontal direction. As a minimum, the bridge shall be designed for laterally and longitudinally applied wind forces acting independently as follows:[5]

- Lateral wind loading [5]

- The lateral wind force on the unloaded bridge shall be taken as 2.5 KN /m^2 (The AREMA (2010) design recommendations for wind load on an unloaded railway bridge superstructure assume a maximum wind velocity corresponding to a typical hurricane event with a wind pressure of 2.5 KN /m^2) of surface as defined in Lateral Wind loading on loaded bridge above.

- Longitudinal wind loading [2]

- The longitudinal wind force on the unloaded spans shall be taken as:
 - a) For girder spans, 25 percent of the lateral wind force.
 - b) For truss spans, 50 percent of the lateral wind force.
 - c) For viaduct towers and bents, 2.5 KN /m^2 on the vertical projection of all windward and leeward columns and bracing.

Design loads for support cross-frame of Highway Bridge according To Appendix A are,

Loads on support cross frame (per cross frame)

$$P_{\text{cross frame}} = 45\text{KN}$$

Loads transmitted to bearings (per girder)

$$P_{\text{bearing}} = 34\text{KN}$$

Design loads for support cross-frame of Highway Bridge according To Appendix B are,

$$\text{Horizontal design load per girder} = 79\text{KN}$$

$$\text{Horizontal design load per cross-frame} = 105.6\text{KN}$$

The above values must be compared with the vertical derailment load check Q (using respective basic allowable stress increment).

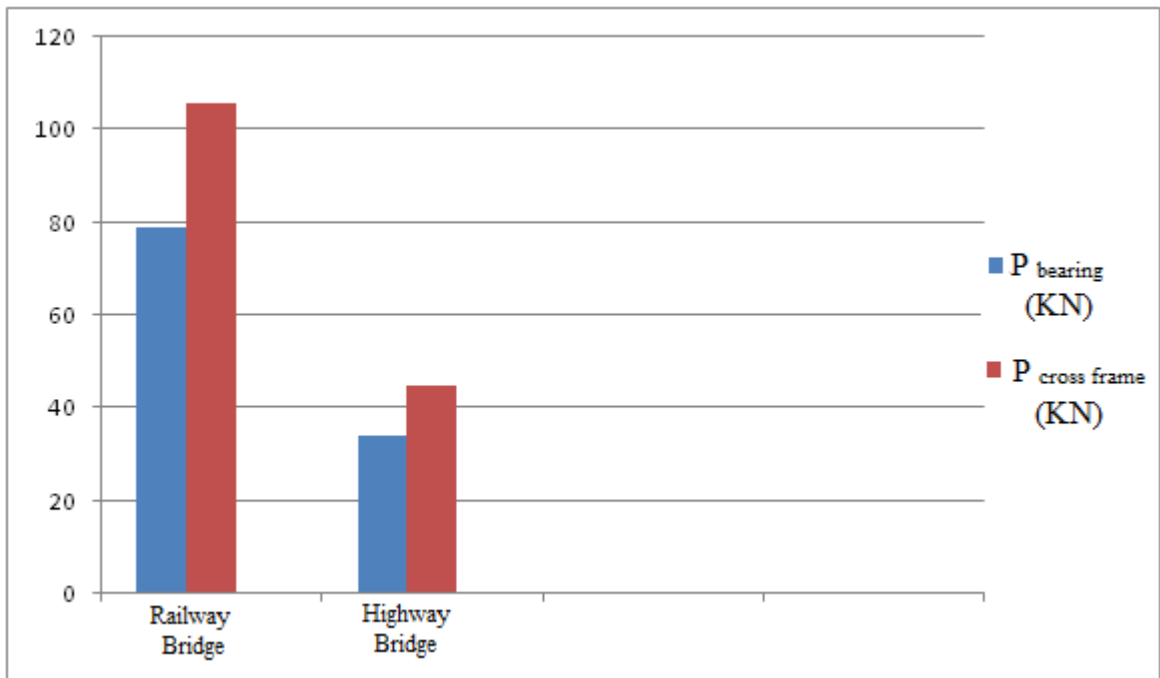


Chart No 3 Lateral loads on support cross-frames of railway and highway bridges

Lateral loads on cross-frames are mainly attributed to wind load. And wind load depends on the contact surface area of the bridge. Obviously the railway bridge has more surface area exposed to wind load due to its large cross-section to support heavy load. Rough observation reveals lateral Load on support cross frame (per cross frame) of highway bridge is about 40% of the railway bridge.

3.8 Distribution of live load

3.8.1 Highway Bridge

The following parameters determine how loads are distributed in a bridge superstructure [11]

- 1) Type of floor
- 2) Spacing between stringers
- 3) Spacing of secondary members
- 4) Stiffness of primary members
- 5) Stiffness of secondary members
- 6) Type of bracing employed
- 7) Size and position of loads

In order to simplify the computation of load distribution, AASHTO LRFD chose to utilize a distribution factor based on some of the above referenced criteria; type of floor, stringer spacing and stiffness, deck thickness and bridge span length. To account for the effects of

load distribution, a load distribution factor (DF) is computed and applied to live load bending moments. Distribution will also vary depending on whether longitudinal or transverse members are being analyzed. AASHTO LRFD offers tabulated and well organized distribution factors based on the above criteria.

3.8.2 Railway Bridge

Unlike highway loads, which may move laterally across the bridge deck, railway live loads are generally fixed in lateral position. However, they are a longitudinal series of large magnitude concentrated wheel loads, and longitudinal and lateral distribution to the deck and supporting members must be considered. [5]

- Distribution of live load for open deck steel bridges [5]

For open deck bridges, no longitudinal distribution is made and lateral distribution to supporting members is based on span cross-section geometry and type of lateral bracing system. Lateral bracing between longitudinal beams should be made with cross frames, or for spans with shallow beams .rolled beams, and/or close beam spacing, solid diaphragms.

The cross frames and diaphragms should not have a spacing exceeding 5.5m in some cases; AREMA (2010) recommends that diaphragms and cross bracings be fastened to the beam or girder flanges. When the lateral bracing system meets these criteria and is properly designed for the lateral forces, all beams or girders supporting the track are considered as equally loaded.

- Distribution of live load for ballasted deck steel bridges [5]

For ballasted deck bridges longitudinal and lateral distribution of live load to the deck is based on tests performed by the association of American railroads (AAR). Axle loads are distributed over a given width at a 2:1 (V/H ratio) distribution through ballast rock and the deck material. The longitudinal deck distribution width, $(0.91m+d_b)$, should not exceed either 1.52m, or the minimum axle spacing of the design load. The lateral deck distribution width, $(\text{length of tie} + d_b)$, should not exceed 4.3m or the distance between adjacent track centerlines or the width of the deck.

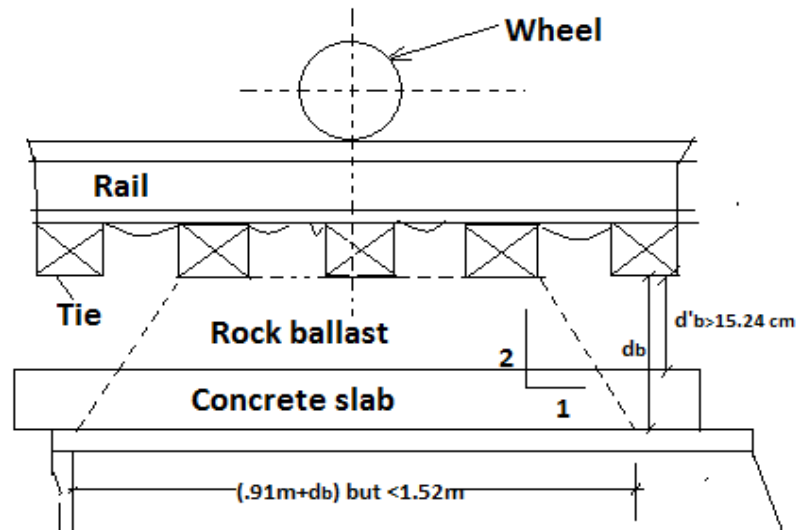


Figure 3-12 Longitudinal distribution to deck [5]

The longitudinal distribution of live load to members supporting the deck in the transverse direction is given in terms of an effective beam spacing, which is dependent on deck material, beam span, and spacing, and for concrete decks the stiffness of beams and deck, and the width of the deck.

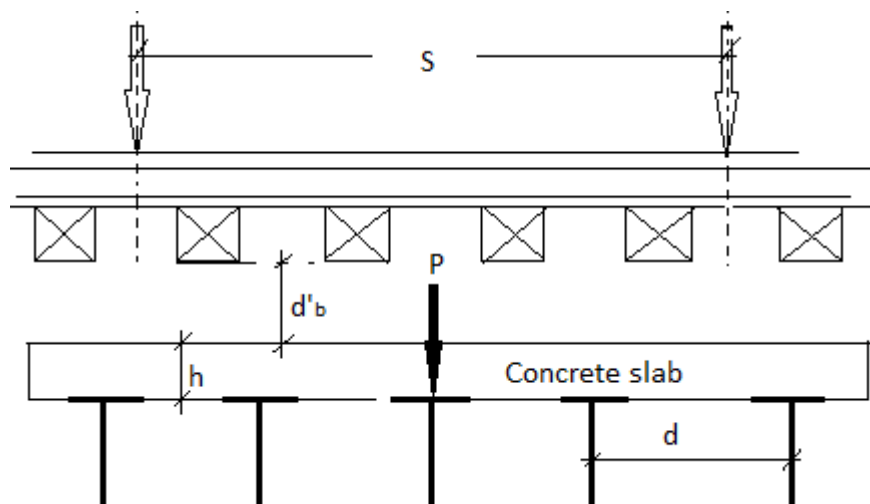


Figure 3-13 longitudinal distribution to transverse members [5]

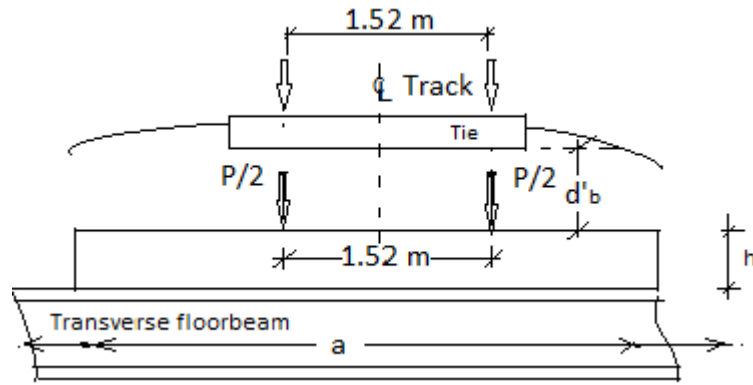


Figure 3-14 Lateral distribution to lateral members [5]

$$P = \frac{1.15AD}{S}$$

Where P is the portion of axle load on the transverse beam; A is the axle load; S is the axle spacing; D is the effective beam spacing, m, which

- 1) For bending moment calculations with a concrete deck, is

$$D = d \left(\frac{1}{1 + \left(\frac{d}{Ah}\right)} \right) \left(0.4 + \frac{1}{d} + \left(\sqrt{\frac{H}{12}} \right) \right)$$

But $D \leq d$ or s

- 2) For bending moment calculations with a steel plate deck, or for end shear in both concrete and steel decks, is

$$D = d$$

Where d is the transverse beam spacing $\leq S$, m (if $d > S$ then assume the width deck as simply supported between transverse beams). a is the transverse beam span, m, H is given by

$$H = \left(\frac{nI_b}{ah^3} \right)$$

Where

n is steel to concrete modular ratio

I_b is the transverse beam moment of inertia, mm^4

h is the concrete slab thickness, mm

No lateral distribution of load is made for transverse beams supporting ballasted decks.

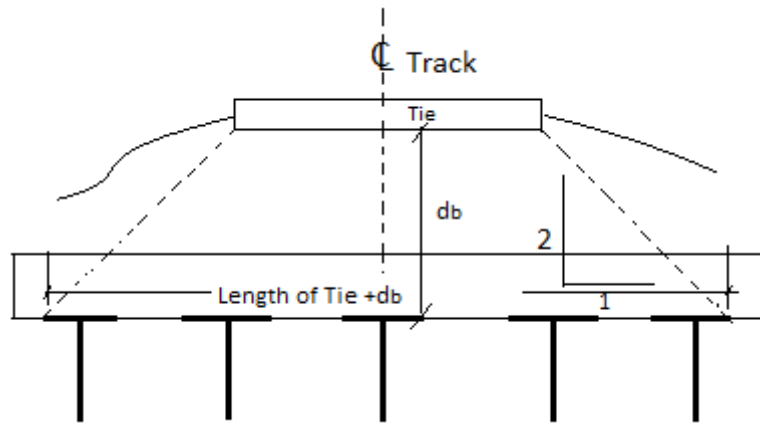


Figure 3-15 Lateral distribution to deck [5]

3.9 Multiple presences

3.9.1 Highway Bridges [4]

The multiple presence factors have been included in the approximate equations for distribution factors. the extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor to account for the probability of simultaneous lane occupation by the full HL-93 design live load.

The factors specified in Table 3.6 shall not be applied in conjunction with approximate load distribution factors, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges.

Table 3-6 multiple presence factors [1]

Number of Design Lane	Multiple Presence Factor m
1	1.20
2	1.00
3	0.85
More than 3	0.65

3.9.2 Railway Bridge [2]

For members receiving load from more than one track, the design live load on the tracks shall be as follows:

- For two tracks, full live load on two tracks.
- For three tracks, full live load on two tracks and one-half on the other track.
For four tracks, full live load on two tracks, one-half on one track, and one-quarter on the remaining one.
- For more than four tracks, as specified by the Engineer.
- The selection of the tracks for these loads shall be such as will produce the greatest live load stress in the member.

CHAPTER FOUR

FATIGUE LOADS AND SERVICIABILITY REQUIREMENTS

4.1 Highway Bridge

4.1.1 Fatigue [4]

Magnitude and Configuration

The strengths of various components of the bridge are sensitive to repeated stressing or fatigue. When the load is cyclic, the stress level that ultimately fractures the material can be significantly below the nominal yield strength. The fatigue strength is typically related to the range of live-load stress and the number of stress cycles under service load conditions.

The fatigue load shall be one design truck or axles with a constant spacing of 9000 mm between the 145 000-N axles and a load factor of 0.75 for finite load-induced fatigue life(Fatigue II) and 1.5 infinite load-induced fatigue life(Fatigue I). The dynamic load allowance shall be applied to the fatigue load. Since the fatigue and fracture limit state is defined in terms of accumulated stress-range cycles, specification of load alone is not adequate. Load should be specified along with the frequency of load occurrence. The frequency of the fatigue load shall be taken as the single-lane average daily truck traffic (*ADTT_{SL}*). This frequency shall be applied to all components of the bridge, even to those located under lanes that carry a lesser number of trucks.

In the absence of better information, the single-lane average daily truck traffic shall be taken as:

$$ADTT_{SL} = p \times ADTT$$

Where:

ADTT = the number of trucks per day in one direction averaged over the design life

ADTT_{SL} = the number of trucks per day in a single-lane averaged over the design life

P = taken as specified in Table 4.1

Table 4-1 fraction of truck traffic in a single lane, p [1]

Number of Lanes Available to Trucks	P
1	1.00
2	0.85
3 or more	0.80

Note that the number of stress–range cycles is not used in the structural analysis directly. The fatigue truck is applied in the same manner as the other vehicles and the range of extreme stress (actions) are used. The number of stress–range cycles is used to establish the available resistance.

4.1.2 Pedestrian loads [4]

The AASHTO [A3.6.1.6] pedestrian load is 3.6×10^{-3} MPa, which is applied to sidewalks that are integral with a roadway bridge. If the load is applied to a bridge restricted to pedestrian and/or bicycle traffic, then a 4.1×10^{-3} MPa live load is used.

4.1.3 Deflection [10]

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs and in metal bridges that could impair serviceability and durability, even if self-limiting and not a potential source of collapse. Bridges should be designed to avoid undesirable structural or psychological effects due to their deformations. AASHTO LRFD provides two alternative criteria for controlling the deflections.

Limiting computed deflection

Vehicular load, general	span length/800
Vehicular and/or pedestrian loads	span length/1000
Vehicular load on cantilever arms	span length/300
Vehicular and/or pedestrian loads on cantilever arms	span length/1000

Limiting span–to–depth ratios for superstructures with constant depth.

AASHTO recommends typical minimum depth for a given span length. Deflections of bridges can be estimated in two steps as instantaneous deflection and long time deflection. The live load portion of Load Combination Service I of Table 5.1 should be used, including the dynamic load allowance, IM;

4.2 Railway Bridge

4.2.1 Fatigue [5]

Fatigue resistance is a critical concern in design of steel structures. It is also a factor although of less significance in the design of concrete bridges. A fatigue design procedure, based on allowable stresses, impact values, number of cycle per train passage, fracture criticality of the member, and type of details, is applied to steel bridges. Fatigue can be the controlling design case for many new steel bridges.

The major factors governing fatigue strength at a particular location of a member or connection are the number of stress cycles, the magnitude of the stress range, and the relevant Fatigue Detail Category. The number of stress cycles, N, to be considered shall be selected from Table 4.2, unless traffic surveys or other considerations indicate otherwise, N depends on the span length in the case of longitudinal members, and on the number of tracks in the case of floorbeams, hangers, and certain truss members.

The stress range, SR, is defined as the algebraic difference between the maximum and minimum calculated stress due to dead load, live load, mean impact load, and centrifugal load. Where live load, impact load and centrifugal load result in compressive stresses and the dead load stress is compression, fatigue need not be considered. The stress range, SR, shall be computed on the basis of the effective net area or the effective gross area as defined in AREMA 2010. The stress range shall not exceed the allowable fatigue stress range. SRfat, [2]

Table 4-2 number of stress cycles, N [2]

Member Description	Span Length, L of Flexural Member or Truss or Load Condition	Constant Stress Cycles, N
Classification I		
Longitudinal flexural members and their connections. Truss chord members including end posts, and their connections	L > 30m	2,000,000
	L ≤ 30m	> 2,000,000
Classification II		
Floorbeams and their connections. Truss hangers and sub-diagonals that carry floorbeam reactions only, and their connections. Truss web members and their connections.	Two Tracks Loaded	2,000,000
	One Track Loaded	> 2,000,000

Note: This table is based on bridges designed for the live loading specified in [Article 1.3.13e](#).
For bridges designed for other live loadings see [Part 9, Commentary](#), Article 9.1.3.13.

Mean Impact Load shall be taken as the Table 4.3 percentages of the impact load specified

Table4-3 Mean Impact Load for fatigue [2]

Member	Percentage
Members with loaded lengths ≤ 3m and no load sharing	65%
Hangers	40%
Other Truss members	65%
Beams ,Stringers ,Girders and Floor Beams	35%

4.2.2 Pedestrian load [5]

Typical walkways for steel railway bridges consist of a steel grating or other system with nonslip surfaces. The walkway components are designed for a load of 4.25 KN/m² and maximum deflection of 1/160 of the walkway span length. Guardrails for pedestrian walkways are typically designed for railing and postloads of 0.9KN/m applied laterally or vertically at the location of maximum effect.

4.2.3 Deflection

Live load deflection control is a significant serviceability criterion. Track standards limit the amount of deflection in track under train passage. The deflection of the bridge under the live load accumulates with the deflection of the track structure itself. This total deflection can exceed the allowable limits if the bridge is not sufficiently stiff. Reinforced Concrete Bridge must be checked for crack control. Allowable stress limits are given for various service conditions for pre-stressed concrete members. [5]

Simple span deflection should be computed for the live load plus impact that produces the maximum bending moment at mid-span. The maximum deflection should not exceed 1/640 of the span length, center-to center of supports. The gross moment of inertia may be used for prismatic flexural members. Railroad companies may limit deflections to values less than 1/640 of the span length, based on their operating practices. Lateral deflection of spans shall be limited to 10 mm for tangent track as measured on a 19 meter chord. On curved track, lateral deflection shall be limited to 6 mm as measured on a 9.5 meter chord. Allowable lateral deflection for spans shall be calculated based on these limits taken in squared proportion to the span length under consideration.[12]

According to Appendix A, the design highway distributed live load effects for fatigue are

$$\text{Moment, } (M_{LL+IM})_{\text{fatigue}} = 1247 \text{ KNm, and}$$

$$\text{Shear force, } (V_{LL+IM})_{\text{fatigue}} = 198 \text{ KN.}$$

Similarly, from Appendix B, the design live load effects of Railway Bridge for fatigue are,

$$\text{Moment, } M_{\text{fatigue } LL+IM \text{ max +ve}} = 9779.54 \text{ KNm, and}$$

$$\text{Shear force, } V_{\text{fatigue } LL+IM \text{ max +ve}} = 1489.2 \text{ KN.}$$

Based on the above result, the fatigue moment used for highway bridge design is about 13 % of the fatigue moment used for railway bridge design.

The impact load used for Highway Bridge, in regard with Appendix A

For fatigue =15%

$$\begin{aligned} \text{Moment, } (M_{LL+IM})_{\text{fatigue}} &= 0.645 * 1679.4 * .15 \text{ (Governing DF)} \\ &= 162.4 \text{ KNm, and} \end{aligned}$$

$$\begin{aligned} \text{Shear force, } (V_{LL+IM})_{\text{fatigue}} &= 0.645 * 266 * .15 \text{ (Governing DF)} \\ &= 26 \text{ KN.} \end{aligned}$$

And the impact load used for that of Railway Bridge from appendix B is

For fatigue =11.55%

$$\begin{aligned} M_{\text{fatigue } LL+IM \text{ max +ve}} &= 8767 \text{ KNm} * .1155 \\ &= 1013 \text{ KNm} \end{aligned}$$

$$\begin{aligned} V_{\text{fatigue } LL+IM \text{ max +ve}} &= 1335 \text{ KN} * .1155 \\ &= 154 \text{ KN} \end{aligned}$$

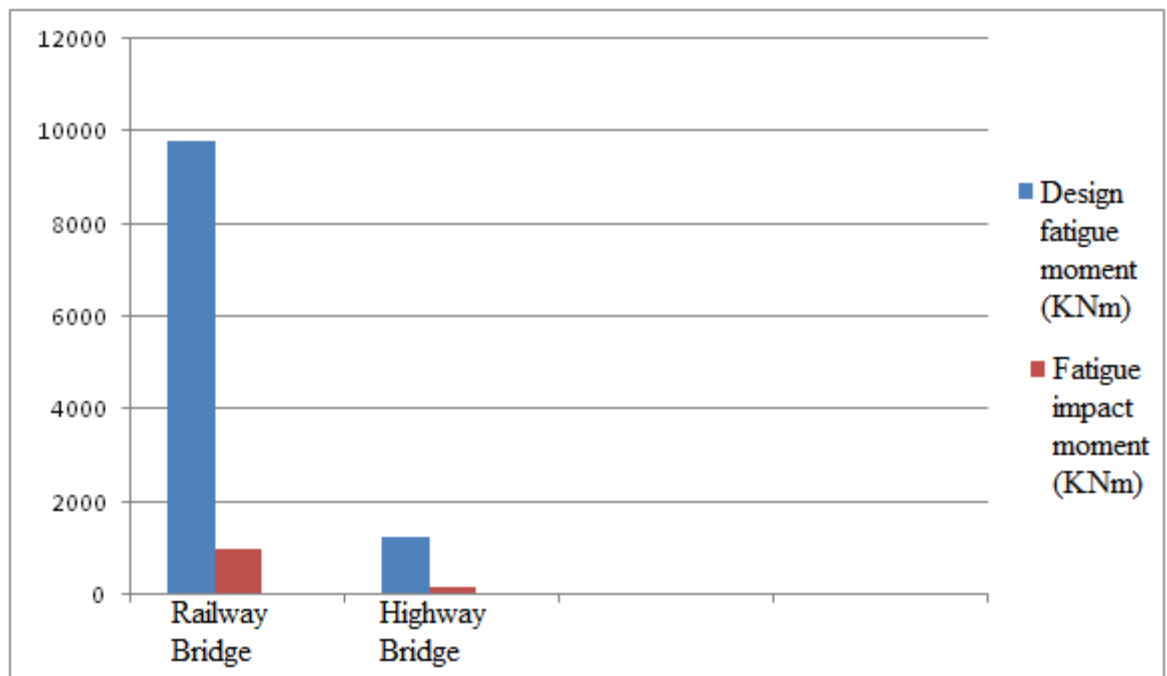


Chart No 4 design fatigue moment and fatigue impact moment of highway and Railway Bridge

The impact load percentage for Highway Bridge is 15% and for railway bridge is about 11.55%. However, due to heavy load of train, the bending moment of Highway Bridge due to impact is about 16% of the railway bridge,

CHAPTER FIVE

LOAD COMBINATIONS

5.1 Highway bridges

5.1.1 Load Factors and Load Combinations [4]

The load factors for various load combinations and permanent loads are given in Tables 5.1 and 5.2, respectively. Explanations of the different limit states are given in the sections that follow.

Service Limit State

The service limit state refers to restrictions on stresses, deflections, and crack widths of bridge components that occur under regular service conditions [A1.3.2.2]. For the service limit state, the resistance factors $\phi = 1.0$, and nearly all of the load factors γ_i are equal to 1.0. There are four different service limit state load combinations given in Table 3.1 to address different design situations [A3.4.1].

Fatigue and Fracture Limit State

The fatigue and fracture limit state refers to a set of restrictions on stress range caused by a design truck. The restrictions depend on the number of stress-range excursions expected to occur during the design life of the bridge [A1.3.2.3]. They are intended to limit crack growth under repetitive loads and to prevent fracture due to cumulative stress effects in steel elements, components, and connections. For the fatigue and fracture limit state, $\phi = 1.0$.

Strength Limit State

The strength limit state refers to providing sufficient strength or resistance to satisfy the inequality of Eq. 2.1 for the statistically significant load combinations that a bridge is expected to experience in its design life [A1.3.2.4]. Strength limit states include the evaluation of resistance to bending, shear, torsion, and axial load. The statistically determined resistance factor ϕ will usually be less than 1.0 and will have different values for different materials and strength limit states.

Extreme Event Limit State

The extreme event limit state refers to the structural survival of a bridge during a major earthquake or flood or when collided by a vessel, vehicle, or ice floe [A1.3.2.5]. The probability of these events occurring simultaneously is extremely low; therefore, they are specified to be applied separately. The recurrence interval of extreme events may be significantly greater than the design life of the bridge [C1.3.2.5]. Under these extreme

conditions, the structure is expected to undergo considerable inelastic deformation by which locked-in force effects due to TU, TG, CR, SH, and SE are expected.

Table 5-1 the load factors for various load combinations [1]

Load Combination Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time				
										EQ	BL	IC	CT	CV
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I— LL, IM & CE only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II— LL, IM & CE only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Table 5-2 Load Factors for Permanent Loads, γ_p [1]

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• AEP for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
o Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.5	0.9
o Thermoplastic culverts		1.3	0.9
o All others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

5.2 Railway Bridge

5.2.1 Loading combinations for concrete bridges [5]

A variety of loads can be applied to a structure at the same time .for example a bridge may experience dead load, live load, impact, centrifugal force, wind, and stream flow simultaneously. The AREMA manual chapters on structure design recognize that it is unlikely that the maximum values of all loads will be applied concurrently to a structure.

The following groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned for the group of loads that produce the most critical design condition. It is important to use the appropriate load combination method for each material and component in the bridge design.

5.2.1.1 Service Load Design [2]

The group loading combinations for SERVICE LOAD DESIGN are as shown in table 5.3

Table 5-3 group loading combinations for SERVICE LOAD DESIGN [2]

Group	Item	Allowable Percentage of Basic Unit Stress
I	$D + L + I + CF + E + B + SF$	100
II	$D + E + B + SF + W$	125
III	$\text{Group I} + 0.5W + WL + LF + F$	125
IV	$\text{Group I} + OF$	125
V	$\text{Group II} + OF$	140
VI	$\text{Group III} + OF$	140
VII	$\text{Group I} + ICE$	140
VIII	$\text{Group II} + ICE$	150

5.2.1.2 Load Factor Design. [2]

The group loading combinations for LOAD FACTOR DESIGN are as shown in table 5.4

Table 5-4 group loading combinations for LOAD FACTOR DESIGN [2]

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

5.2.2 Loading combinations for steel bridges [5]

Every component of substructure and superstructure should be proportioned to resist all combinations of forces applicable to the type of bridge and its site. Members subjected to stresses from dead, live, impact, and centrifugal loads should be designed for the basic allowable unit stress or the allowable fatigue stress, whichever governs.

With the exception of floorbeam hangers, members subjected to stresses from other lateral or longitudinal forces, as well as to dead, live, impact, and centrifugal loads, may be proportioned for 125% of the basic allowable unit stresses, without regard for fatigue. However, the section should not be smaller than that required to satisfy basic unit stresses or the allowable fatigue stress range when those lateral or longitudinal forces are not present. Components subject to stresses from wind loads only should be designed for the basic allowable stresses. Also, no increase in the basic allowable stresses in high-strength bolts should be taken for connections of members covered in this article. Elastic analysis procedures are usually used for steel railroad bridges.

Chap. 15 of the AREMA *MRE* provides recommendations for allowable stress design. The design service life of railroad bridges is generally considered to be about 80 years.

AREMA *MRE* does not provide explicit load combinations but incorporates combinations in various design recommendations (Sorgenfrei and Marianos, 2000). Table 5.5 outlines load combinations that apply to steel superstructure design found in various recommendations of AREMA. [5]

Table 5-5 Load combinations for steel railway superstructure design [5]

Load case	load combinations	members	F_L
A1	DL +LL+I+CF	All members	1.00
A2	DL+LLT+I+CF	Truss web members	1.33
B1	DL+LL+I+W+LF+N+CWR	All members, except floorbeam hangers and High strength bolts	1.25
B1A	DL+LL+I+W+LF+N+CWR	Floorbeam hangers and high strength bolts	1.00
B2	DL+LLT+I+W+LF+N+CWR	Truss web members, except floorbeam hangers	1.66
C	(LL+I) range	All members	f_{fat}
D1	SL+N+CF	Members resisting overall instability	1.50
D2	Q	Members resisting overall instability	1.50
E1	DL+EQ	All members	1.50
E2	DL+LL+I+CF+EQ	Members in long bridges only	1.50
F	W or LV	members loaded by wind only	1.00
G	DF	Cross frame, diaphragms, anchor rods	1.50
H1	DL	Members stressed during lifting or jacking	1.50
H2	DL	Members stressed during erection	1.25
H3	DL+W	Members stressed during erection	1.3

The Load combinations used in design example of Appendix A are (LRFD Method)

Strength I $U = \eta [1.25D1 + 1.50D2 + 1.25D3 + 1.75 (LL + IM)]$

Service I $U = \eta [DC + DW + (LL + IM)]$

Service II $U = \eta [DC + DW + 1.3(LL + IM)]$

Fatigue $U = \eta [0.75 (LL + IM)]$

The load combinations used in design example Appendix B are (ASD Method)

Load case	Load combination	Members	F_L
A1	DL+LL+I+CF	All members	1.00
B1	DL+LL+I+W+LF+N+CWR	All members (Except floorbeam hangers, high strength bolts)	1.25
C	(LL+I) _{range}	All members	f_{fat}
D2	Q	Members resisting stability	1.50

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

Based on the above discussion, it can be concluded that

- Bridge designers should be aware of the basic differences between design criteria of Railways Bridges and Highway Bridge.
- Intensity of Live Loads of Railways Bridges is quite larger than those used for the design of Highway Bridges. these Loads are repetitive and almost always maximum (because all trains are pulled by locomotives), resulting in important differences between the design criteria .This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.
- The design impact load on railroad bridges is higher than on highway structures. The dynamic response railway bridge under train loads is one of the fundamental problems to be solved in railway bridge design and maintenance. Interruptions in services are typically much more critical for railroads than for highway agencies. Therefore, constructability and maintainability without interruption to traffic are crucial for railroad bridges. Since the bridge supports the track structure, Interaction between the track and the bridge should be considered in designing and detailing.

Comparison of Loads which primarily affect substructures, such as earthquake load, water load can be area of further study.

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APPENDIX A

HIGHWAY BRIDGE DESIGN EXAMPLE

Design simply supported steel plate girder bridge Figure A1 with 30-m span for an HL-93 live load. Roadway width is 9860 mm curb to curb and carries an interstate highway. Allow for a future wearing surface of 75-mm thick bituminous overlay. Use $f_c = 30$ MPa and M270 Grade 345 steel .LRFD method.

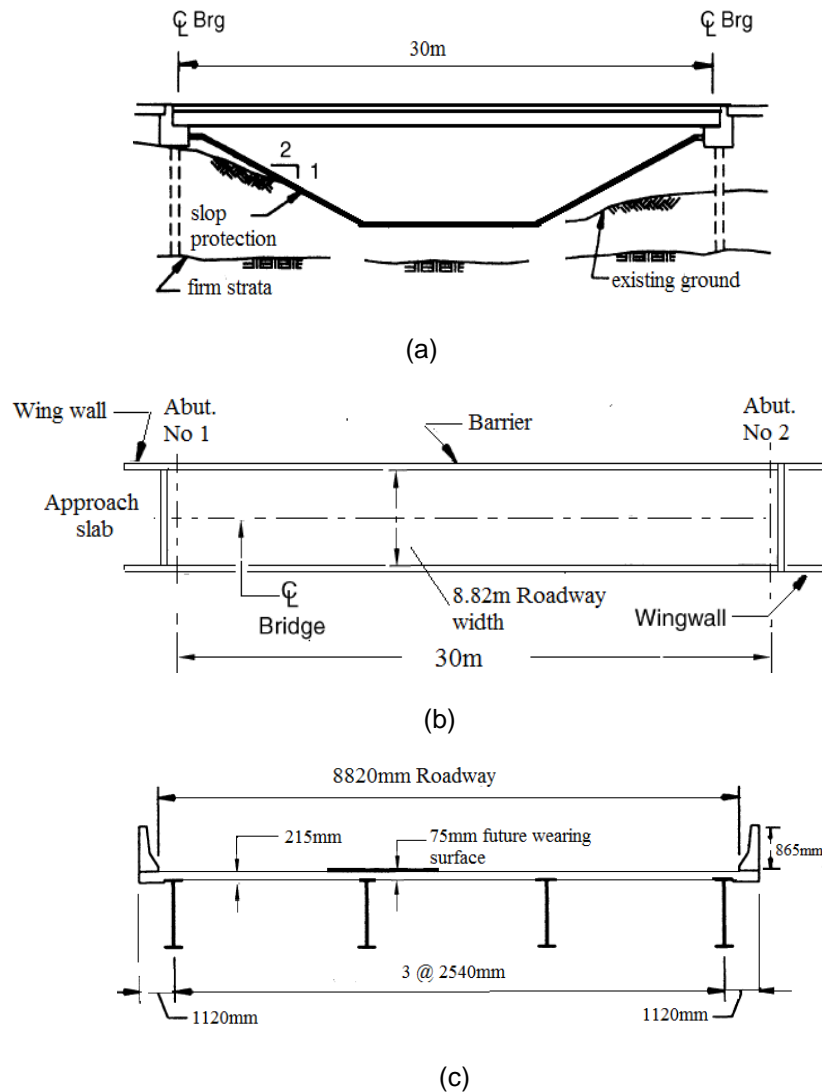


Fig.A 1 Steel plate girder bridge design example: (a) General elevation, (b) plan view, and (c) cross-section

- A. Develop General Section as shown in fig (Fig. A 1)
- B. Develop Typical Section as shown in fig (Fig. A 1 (c)) (The flanges and web of the plate girder are of the same material, so that $R_h = 1.0$.)
- C. Design Reinforced Concrete Deck

Empirical Design of Concrete Deck Slabs

1. Design Conditions [A9.7.2.4]

- Diaphragms are used at lines of support, YES
- Supporting components are made of steel and/or concrete, YES
- The deck is of uniform depth, YES
- The deck is fully CIP and water cured, YES
- $6.0 < \frac{Se}{h} = \frac{2130}{215} = 9.9 < 18.0$, OK
- Core depth = $215 - 10 - 25 = 180\text{mm} > 101\text{mm}$, OK
- Effective length [A9.7.2.3] = $2.13\text{m} < 4.1\text{m}$, OK
- Minimum slab depth = $178\text{mm} < 215\text{mm}$, OK
- Overhang = $1120\text{mm} > 5 * 215\text{mm} = 1075\text{mm}$, OK
- $f_c = 32\text{MPa} > 30\text{MPa}$, OK
- Deck must be made composite with girder, YES

2. Reinforcement Requirements [A9.7.2.5]

- Four layers of isotropic reinforcement, $f_y \geq 414\text{MPa}$
- Outer layers placed in direction of effective length
- Bottom layers: $\min A_s = 572\text{mm}^2/\text{m}$, $6\Phi 16$ at 180mm .
- Top layers: $\min A_s = 381\text{mm}^2/\text{m}$, $4\Phi 16$ at 300mm .
- Max spacing = 450mm .
- Straight bars only, hooks allowed, no truss bars

Design of overhang slab not included.

D. Select Resistance Factors [A6.5.4.2]

1. For flexure $\phi_f = 1.00$
2. For shear $\phi_v = 1.00$
3. For axial compression $\phi_c = 0.90$

E. Select Load Modifiers (for conventional bridge)

		Strength	Service, Fatigue	
1. Ductility	η_D	1.00	1.0 0	[A1.3.3]
2. Redundancy	η_R	1.00	1.0 0	[A1.3.4]
3. Importance	η_I	1.00	1.0 0	[A1.3.5]
$\eta = \eta_D \eta_R \eta_I$		1.00	1.00	

F. Determine Combinations and Factors

$$\text{Strength I } U = \eta [1.25D1 + 1.50D2 + 1.25D3 + 1.75 (LL + IM)]$$

$$\text{Service I } U = \eta [DC + DW + (LL + IM)]$$

$$\text{Service II } U = \eta [DC + DW + 1.3(LL + IM)]$$

$$\text{Fatigue } U = \eta [0.75 (LL + IM)]$$

G. Calculate Live-Load Force Effects

1. Dynamic Load Allowance [A3.6.2]

$$\text{Impact} = 33\%$$

$$\text{Fatigue and fracture} = 15\%$$

2. Distribution Factor for Moment [A4.6.2.2.2]

a. Interior beam

Design parameters are within the range of applicability of approximate method.

For preliminary design $\left(\frac{Kg}{Lt_s^3}\right)$ is taken as unity.

$$mg_M^{SI} = 0.521$$

$$mg_M^{MI} = 0.6386 \quad (\text{GOVERNS})$$

$$\text{For fatigue } mg_f = \frac{0.521}{1.2} = 0.426$$

b. Exterior beam

One design lane loaded

Use the lever rule

$$mg_M^{SE} = 1.2(0.645)$$

$$= 0.775 \quad (\text{governs})$$

Two or more design lanes loaded:

$$mg_M^{ME} = 0.6386$$

$$\text{For fatigue } mg_f = 0.645$$

Adjustment for skew not required. (Straight bridge) [A4.6.2.2.2e]

Distribution Factor for Shear [A4.6.2.2.3]

a. Interior Beams [A4.6.2.2.2a]

Parameters are within the range of applicability of the approximate method:

$$mg_v^{SI} = 0.694$$

$$mg_v^{ME} = 0.849 \text{ (governs)}$$

$$\text{For fatigue } mg_f = \frac{0.694}{1.2} = 0.5783$$

b. Exterior Beams [A4.6.2.2.2b]

For one design lane loaded, use the lever rule

$$mg_v^{SE} = 0.775 \text{ (governs)}$$

For two or more design lanes loaded,

$$mg_v^{ME} = 0.679$$

$$\text{For fatigue } mg_f = \frac{0.775}{1.2} = 0.645$$

No adjustment is required for skew.

3. Live load effect computation

$$M_{LL+IM} = mg [(M_{\text{TRACK OR TANDEM}}) (1 + \left(\frac{IM}{100}\right)) + M_{\text{LANE}}]$$

$$V_{LL+IM} = mg [(V_{\text{TRACK OR TANDEM}}) (1 + \left(\frac{IM}{100}\right)) + V_{\text{LANE}}]$$

Bending moment computation

Placing the resultant of axle loads equally spaced with the nearest axle load at the center to give maximum moment for design truck, which is governing. Similarly, placing axle loads of the design tandem equally from center to give maximum moment.

$$M_{\text{max of design truck}} = 2037.6 \text{ KNm (governing)}$$

$$M_{\text{max of tandem}} = 1584 \text{ KNm}$$

$$M_{\text{max lane load}} = 1046 \text{ KNm}$$

Shear force computation

$$V_{\text{max design truck}} = 294 \text{ KN (governing)}$$

$$V_{\text{max design tandem}} = 215 \text{ KN}$$

$$V_{\text{max lane load}} = 140 \text{ KN}$$

$$M_{LL+IM} = [(M_{\text{TRACK OR TANDEM}}) (1 + \left(\frac{IM}{100}\right)) + M_{\text{LANE}}]$$

$$= 2037.6 * 1.33 + 1046$$

$$= 3756 \text{ KNm}$$

$$V_{LL+IM} = [(V_{\text{TRACK OR TANDEM}})(1 + \left(\frac{IM}{100}\right)) + V_{\text{LANE}}]$$

$$= 294 * 1.33 + 140$$

$$= 531 \text{KN}$$

$$M_{LL+IM \text{ fatigue}} = 1679.4 * 1.15 = 1931 \text{KNm}$$

$$V_{LL+IM \text{ fatigue}} = 266 * 1.15 = 306 \text{KN}$$

4. Stiffness [A6.10.1.5]

The beam stiffness is modeled as prismatic and the same moment of inertia is used for the entire cross section.

5. Wind Effects [A4.6.2.7] Wind effects are considered later

H. Calculate Force Effects from Other Loads

Three separate dead loads are calculated

D1 = the dead load of the structural components and their attachments (acting on the non-composite section.)

D2 = the future wearing surface

D3 = barriers dead load (assumed to be distributed equally among the interior and exterior girders.)

Girder assumed cross-section = 30mm×400mm flange (both) and 10.5mm×1500mm web
Interior beam

D1

$$\text{Slab: } 2400 \times 9.81 \times 225 \times 2540 / 10^9 = 13.45 \text{ KN/m}$$

$$\text{Haunch: } 2400 \times 9.81 \times 305 \times 50 / 10^9 = 0.36 \text{ KN/m}$$

$$\text{Girder: } = 4 \text{KN/m (assumed)}$$

$$\text{Total } w_{D1} = 17.81 \text{KN/m}$$

D2 75-mm bituminous overlay (density 2250Kg/m³)

$$w_{D2} = \frac{2250 \times 9.81 \times 2540 \times 75}{10^9}$$

$$= 4.20 \text{ KN/m}$$

D3 Barriers, one-fourth share

$$w_{D3} = \frac{2(286937.5)(2400)(9.81)}{4 \times 10^9}$$

$$= 3.38 \text{ kN/m}$$

Exterior girder

D1

$$\text{Slab: } 2400 \times 9.81 \times 225 \times (1120+1270)/10^9 = 12.65 \text{ KN/m}$$

$$\text{Haunch: } 2400 \times 9.81 \times 305 \times 50/10^9 = 0.36 \text{ KN/m}$$

$$\text{Girder: } = 4 \text{ KN/m (assumed)}$$

$$\text{Total } w_{D1} = 17.01 \text{ KN/m}$$

D2 75-mm bituminous overlay (density 2250Kg/m³)

$$w_{D2} = \frac{2250 \times 9.81 \times (1270+600) \times 75}{10^9}$$

$$= 3.1 \text{ kN/m}$$

D3 Barriers, one-fourth share

$$w_{D3} = \frac{2(286937.5)(2400)(9.81)}{4 \times 10^9}$$

$$= 3.38 \text{ KN/m}$$

Computation of dead load effects

Table A1 dead load effects

	Load type	value (KN/m)	moment (KNm)	shear force (KN)
<i>Interior girder</i>	w _{D1}	17.81	2003.62	267
	w _{D2}	4.2	472.5	63
	w _{D3}	3.38	380.25	50.7
<i>Exterior girder</i>	w _{D1}	17.01	1914.2	255
	w _{D2}	3.1	349	47
	w _{D3}	3.38	380.25	50.7

Distributed live load effects (strength)

	$(M_{LL+IM})_{+Lane}$ (KNm)	$(V_{LL+IM})_{+Lane}$ (KN)
<i>Interior girder</i>	2398	450
<i>Exterior girder</i>	2910.5	410.75

Distributed Fatigue load effects

Maximum moment is computed placing the resultant of the axle loads and the nearest axle load equally spaced from center of span.

	$(M_{LL+IM})_{fatigue}$ (KN/m)	$(V_{LL+IM})_{fatigue}$ (KN)
<i>Interior girder</i>	822.25	177
<i>Exterior girder</i>	1247	198

1 Design Required Sections

- Girder assumed cross-section = 35mm×410mm flange (both) and 10.5mm×1500mm web
- Minimum thickness of steel is 8 mm [A6.7.3] OK
- The optional minimum depth requirement of [A2.5.2.6.3]
Min depth total composite section = 0.032L=960mm OK

Flexural Section Properties

Effective flange width [A4.6.2.6].

- Interior girder $b_i=2450\text{mm}$
- Exterior girder $b_e=2390\text{mm}$ (governing)

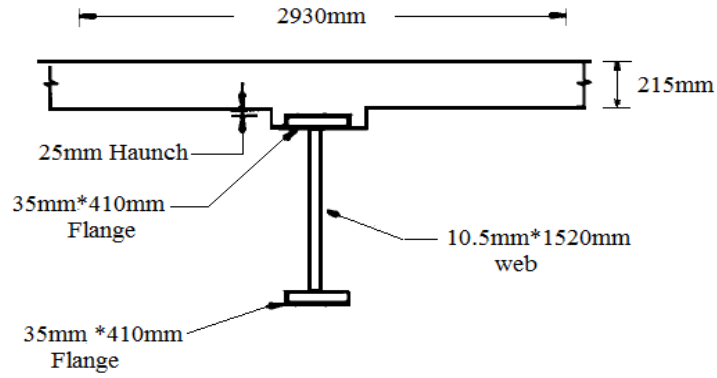


Fig. A2 composite section for positive moment Section properties

Table A2 Steel section properties

Component	A (mm ²)	y (mm)	Ay	Ay ²	I _o (mm ⁴)	I (mm ⁴)
Top flange (410×35)	14350	777.5	11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Web (1520*10.5)	15960	0	0	0	2.93*10 ⁹	2.93*10 ⁹
Bottom flange (410×35)	14350	- 777.5	-11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Total	43900		0			20.3*10 ⁹

$$y_c = \frac{\sum Ay}{\sum A} = 0$$

$$I_{NA} = I = 20.3 * 10^9$$

$$y_{top\ of\ steel} = \frac{D}{2} + t_f - y_c = \frac{1520}{2} + 35 - 0 = 795\text{mm}$$

$$y_{bottom\ of\ steel} = \frac{D}{2} + t_f + y_c = 795\text{mm}$$

$$S_{Top\ of\ steel} = \frac{I_{NA}}{y_{top\ of\ steel}} = \frac{20.3 * 10^9}{795} = 25.5 * 10^6\text{mm}^3$$

$$S_{Bottom\ of\ steel} = \frac{I_{NA}}{y_{bottom\ of\ steel}} = \frac{20.3 * 10^9}{795} = 25.5 * 10^6\text{mm}^3$$

Table A3 Short term composite section properties (n=8)

Component	A (mm ²)	y (mm)	Ay	Ay ²	I _o (mm ⁴)	I (mm ⁴)
Top flange (410×35)	14350	777.5	11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Web (1520*10.5)	15960	0	0	0	2.93*10 ⁹	2.93*10 ⁹
Bottom flange (410×35)	14350	- 777.5	-11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Concrete (n=8) (2390×215/8)	64231	927.5	59.6*10 ⁶	55.25*10 ⁹	247*10 ⁶	55.5*10 ⁹
Total	108131		59.6*10 ⁶			75.8*10 ⁹

$$y_c = \frac{\Sigma Ay}{\Sigma A} = \frac{59.6 \cdot 10^6}{108131} = 551 \text{ mm}$$

$$I_{NA} = I - (y_c \times \Sigma Ay) = 43 \cdot 10^9 \text{ mm}^4$$

$$y_{\text{top of steel}} = \frac{D}{2} + t_f - y_c = \frac{1520}{2} + 35 - 551 = 244 \text{ mm}$$

$$y_{\text{bottom of steel}} = \frac{D}{2} + t_f + y_c = 1346 \text{ mm}$$

$$y_{\text{top of concrete}} = \frac{D}{2} + t_f + \text{haunch} + t_s - y_c$$

$$= \frac{1520}{2} + 35 + 25 + 215 - 551 = 484 \text{ mm}$$

$$S_{\text{Top of steel}} = \frac{I_{NA}}{y_{\text{top of steel}}} = \frac{43 \cdot 10^9}{244} = 176 \cdot 10^6 \text{ mm}^3$$

$$S_{\text{Bottom of steel}} = \frac{I_{NA}}{y_{\text{bottom of steel}}} = \frac{43 \cdot 10^9}{1346} = 32 \cdot 10^6 \text{ mm}^3$$

$$S_{\text{Top of concrete}} = \frac{I_{NA}}{y_{\text{top of concrete}}} = \frac{43 \cdot 10^9}{484} = 89 \cdot 10^6 \text{ mm}^3$$

Table A4 Long term composite section properties (n=24)

Component	A (mm ²)	y (mm)	Ay	Ay ²	I _o (mm ⁴)	I (mm ⁴)
Top flange (410×35)	14350	777.5	11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Web (1520*10.5)	15960	0	0	0	2.93*10 ⁹	2.93*10 ⁹
Bottom flange (410×35)	14350	- 777.5	-11.157*10 ⁶	8.67*10 ⁹	1.465*10 ⁶	8.68*10 ⁹
Concrete (n=24) (2390×215/24)	21410	927.5	20*10 ⁶	18.4*10 ⁹	82.5*10 ⁶	18.5*10 ⁹
Total	108131		20*10 ⁶			39*10 ⁹

$$y_c = 304mm$$

$$I_{NA} = 32.75 * 10^9 mm^4$$

$$y_{top\ of\ steel} = 491mm$$

$$y_{bottom\ of\ steel} = 1099mm$$

$$y_{top\ of\ concrete} = 731mm$$

$$S_{Top\ of\ steel} = \frac{I_{NA}}{y_{top\ of\ steel}} = 66.3 * 10^6 mm^3$$

$$S_{Bottom\ of\ steel} = \frac{I_{NA}}{y_{bottom\ of\ steel}} = 30 * 10^6 mm^3$$

$$S_{Top\ of\ concrete} = \frac{I_{NA}}{y_{top\ of\ concrete}} = 44.8 * 10^6 mm^3$$

Table A5 Stress in bottom of exterior steel girder (tension) for positive flexure due to factored loading, strength I

Load	M_{D1} (kN m)	M_{D2} (kN m)	M_{D3} (kN m)	M_{LL+HM} (kN m)	S_b Steel (mm ³)	S_b Composite (mm ³)	Stress (MPa)
D1	1.25*2003				25.5*10 ⁶		98
D2		1.5*349				30*10 ⁶	17.55
D3			1.25*380			30*10 ⁶	15.95
LL+HM				1.75 (2910)		32*10 ⁶	159
TOTAL							290.5

Table A 6 Maximum flexural stress in the flange for exterior positive flexure, Service II

Load	M_{D1} (kN m)	M_{D2} (kN m)	M_{D3} (kN m)	M_{LL+HM} (kN m)	S_b Steel (mm ³)	S_b Composite (mm ³)	Stress (MPa)
D1	2003				25.5*10 ⁶		78.4
D2		349				30*10 ⁶	11.7
D3			380			30*10 ⁶	12.76
LL+HM				1.30 (2910)		32*10 ⁶	118
TOTAL							220.86

Member Proportions Check [A6.10.2.1]

Web shall be proportioned to meet the following requirement:

$$\left(\frac{D}{tw}\right) \leq 150 \quad , \quad \left(\frac{1520}{10.5}\right) = 144.7 \quad \text{OK}$$

And in [A6.10.2.2] states, that the flanges shall meet

$$\left(\frac{b_f}{2t_f}\right) \leq 12 \quad , \quad \left(\frac{410}{2*35}\right)=5.86 \quad OK$$

$$b_f \geq \frac{D}{6} \quad , \quad 410 \geq \frac{1520}{6}=253.3 \quad OK$$

$$t_f \geq 1.1t_w \quad , \quad 35 \geq 1.1*10.5=11.55 \quad OK$$

$$0.1 \leq \left(\frac{I_{yc}}{I_{yt}}\right)=1 \leq 10 \quad OK$$

Composite Section Stresses for Positive Moment [A6.10.1.1.1]

From [A6.10.5.1.4b] for sections in positive flexure,

Plastic Forces

Concrete slab $P_s = 0.85f_c a b_e$

$$= 0.85(30)(2392)a$$

$$= 60.945a \text{ kN (haunch and slab reinforcement are neglected)}$$

Tension flange = $P_t = F_{yt} b_t t_t$

$$= (345)(410)(35)$$

$$= 4950 \text{ kN}$$

Compression flange = $P_c = F_{yc} b_c t_c$

$$= (345)(410)(35)$$

$$= 4950 \text{ kN}$$

Web = $P_w = F_{yw} D t_w$

$$= (345)(1520)(10.5)$$

$$= 5506 \text{ kN}$$

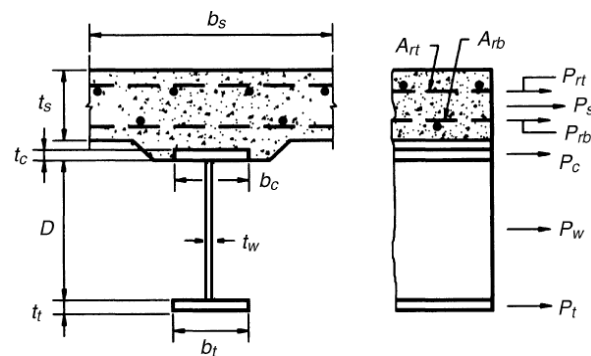


Fig.A3 Plastic neutral axis for positive moment section.

Plastic Neutral Axis (PNA): *Compression force = Tension force*

$$60.945a = 2*4950 + 5506, a = 252.78, \text{ PNA out of slab.}$$

Re-computing the PNA

$$0.85*30*2390*215 + 410*x*345 = 410*(35-x)*345 + 410*35*345 + 10.5*1520*345, x = 8\text{mm}$$

PNA is in the compression flange. Therefore D_{cp} is equal to zero. The web slenderness requirement is satisfied.

Plastic moment M_p

Taking moment about the PNA yields

$$M_p = 15833.5 \text{ KNm}$$

$$D_p = 248.14 \text{ mm} > 0.1 D_T = 0.1 * 1830 \text{ mm} = 183$$

$$D_p = 248 < 0.4 D_T = 0.4 * 1830 = 732 \text{ mm OK (ductile)}$$

$$\text{Then } M_n = M_p * \left(1.07 - 0.7 \left(\frac{D_p}{D_T} \right) \right)$$

$$= 15833.5 * \left(1.07 - 0.7 * \left(\frac{248.14}{1830} \right) \right)$$

$$= 15440 \text{ KNm}$$

$$M_n = 15440 \text{ KNm}$$

$$M_r = (\phi_f = 1.00) * M_n$$

$$= 15440 \text{ KNm}$$

Constructability check can be done.

Service limit state

a. Elastic Deformations [A6.10.4.1]

1. Optional Live-Load Deflection [A2.5.2.6.2]

$$\Delta_{\text{live load limit}} = \left(\frac{L}{800} \right) = \left(\frac{30000}{800} \right) = 37.5 \text{ mm}$$

$$mg \text{ deflection} = \left(\frac{2 \text{ lanes}}{4 \text{ girders}} \right) = 0.5$$

The live load used is the maximum of the deflection due to the

Design truck

Deflection resulting from 25% of design truck together with the design lane load .

Use moment of inertia of $43 \times 10^9 \text{ mm}^4$ (for $n=8$).

$$\Delta_{\text{max design truck}} = 21 \text{ mm (governing)}$$

$$\Delta_{\text{max 25\% design truck + lane load}} = 11.4 \text{ mm} + 0.25 \times 21 = 16.65 \text{ mm}$$

$$\Delta_{\text{max design truck}} * mg \text{ deflection} = 21 \text{ mm} * 0.5 = 10.5 \text{ mm} < 37.5 \text{ mm OK.}$$

2. Optional Criteria for Span-to-Depth Ratio [A2.5.2.6.3]

$$\text{Min depth total composite section} = 0.032L = 960 \text{ mm OK}$$

b. Permanent Deformations [A6.10.4.2]

$$f_f + \left(\frac{fl}{2} \right) \leq 0.95 R_h F_y f = 328 \text{ MPa}$$

$$fl = 22.3 \text{ MPa (calculated according to [C 6.10.3.4-2])}$$

$$\text{Service II } U = \eta [\text{DC} + \text{DW} + 1.3(\text{LL} + \text{IM})]$$

$$f_f = 1.0(f_{D1}) + 1.0(f_{D2}) + 1.0(f_{D3}) + 1.3(f_{LL+IM})$$

$$f_f = 220 \text{ MPa (Table A 6)} + \left(\frac{22.3 \text{ MPa}}{2} \right) < 328 \text{ MPa OK}$$

Fatigue and Fracture Limit State

a. Fatigue [A6.10.5.1] [A6.6.1]

Stress Cycles Assuming a rural interstate highway with 20,000 vehicles per lane per day,

$$\text{Fraction of trucks in traffic} = 0.20$$

$$\text{ADTT} = 0.20 \times \text{ADT} = 0.20(20\,000)(2 \text{ lanes}) = 8000 \text{ trucks/day}$$

$$\text{ADTTSL} = p \times \text{ADTT} = 0.85(8000) = 6800 \text{ trucks/day}$$

$$N = (365 \text{ days/year})(75 \text{ years})(6800 \text{ trucks/day}) = 186 \times 10^6 \text{ cycles}$$

Allowable Fatigue Stress Range—Category A

$$\text{For finite life, } (\Delta F)_n = \left(\frac{A}{N} \right)^{1/3} = \left(\frac{82 \times 10^{11}}{186 \times 10^6} \right)^{1/3} = 35.32 \text{ MPa}$$

$$\text{For infinite life, } \frac{1}{2} (\Delta F)_{TH} = \left(\frac{165}{2} \right) = 82.5 \text{ MPa} > 35.32 \text{ MPa}$$

$$(\Delta F)_n = 82.5 \text{ MPa}$$

The Maximum Stress Range

$$U = 0.75(LL + IM) = 0.75 * 1247 \text{KNm} = 935.25 \text{KNm}$$

$$f_{LL+IM} = \frac{U}{S_{\text{Bottom of steel}}} \text{ (COMPOSITE } n=8) = \frac{935.25 \text{KNm}}{32.0 * 10^6 \text{mm}^3} \\ = 30 \text{MPa} < 82.5 \text{MPa} \quad \text{OK}$$

c. Special Fatigue Requirements for Webs

$$V_{fu} = 1.0(V_{D1}) + 1.0(V_{D2}) + 1.0(V_{D3}) + 0.75(V_{LL+IM})$$

$$V_{fu} = 1.0(267 + 46.5 + 50.7) + 0.75 * (176) \text{ (interior girder governs.) KN.}$$

$$V_{fu} = 496.2 \text{KN} < \phi V_{cr} = 1405 \text{KN} \quad \text{OK}$$

Strength limit state

a. Composite Sections Flexure

$$\text{Strength 1 } Mu = \eta [1.25D1 + 1.50D2 + 1.25D3 + 1.75 (LL + IM)]$$

$$= \eta [1.25(2003) + 1.50(348.75) + 1.25(380) + 1.75 (2910)]$$

$$= 8596 \text{KNm}$$

$$\frac{1}{3} * f_t * S_{xt} = \frac{1}{3} * 43.2 \text{MPa} * 0.0299362 \text{m}^3 = 431 \text{KNm} \text{ (calculated according to} \\ \text{[C 6.10.3.4-2], [6.10.7.1.1] and [C4.6.2.7.1-2]).}$$

$$1. Mu = 8596 \text{KNm} + \frac{1}{3} * f_t * S_{xt} = 8596 \text{KNm} + 431 \text{KNm}$$

$$= 9027 \text{KNm}$$

$$Mu \leq (\phi f = 1.00) * M_n = 15,440 \text{KNm} \quad \text{OK}$$

2. The flange stress in the tension flange

$$= 98 \text{MPa} + 17.55 \text{MPa} + 15.95 \text{MPa} + 159 \text{MPa} \text{ (Table A 5)}$$

$$= 290.5 \text{MPa} < 345 \text{MPa} \quad \text{OK}$$

b. Shear design

Section shear capacity of unstiffened web

$$V_n = CV_p$$

$$V_p = 0.58 F_{yw} D_{tw} = \frac{0.58(345)(1520)(10.5)}{1000} = 3193.6 \text{ kN}$$

Computation of C, assuming $k=5.0$ for *unstiffened web*

$$\text{Since } \frac{D}{tw} = 144.7 > 1.40 \sqrt{\frac{Ek}{F_{yw}}} = 75.37$$

$$\text{Then } C = \left(\frac{1.57}{\left(\frac{D}{tw}\right)^2} \right) * \left(\frac{EK}{F_{yw}} \right)$$

$$= \left(\frac{1.57}{\left(\frac{1520}{10.5}\right)^2} \right) * \left(\frac{200000*5}{345} \right) = 0.216$$

$$V_n = CV_p = 0.216 * 3193.6 \text{KN} = 691 \text{KN} < V_u$$

$$\text{Strength 1 } V_u = \eta [1.25D_1 + 1.50D_2 + 1.25D_3 + 1.75 (LL + IM)]$$

$$V_u = 1.00 [1.25*267 + 1.25*50.7 + 1.5*63 + 1.75*450] \text{ KN} \\ = 1279.5 \text{KN}$$

Thus web stiffeners are required.

$$\text{Assume } \frac{D}{tw} > 1.40 \sqrt{\frac{Ek}{F_{yw}}}$$

$$\text{Then } C = \left(\frac{1.57}{\left(\frac{D}{tw}\right)^2} \right) * \left(\frac{EK}{F_{yw}} \right)$$

$$V_n = CV_p$$

$$= \left(\frac{1.57}{\left(\frac{1520}{10.5}\right)^2} \right) * \left(\frac{200000*K}{345} \right) * 3193.6 \text{KN}$$

Solving for K_{\min} ,

$$K_{\min} = \frac{V_n F_{yw}}{(1.57 * V_p * E) \left(\frac{D}{tw}\right)^2} \\ = \frac{1279.5 * 345}{(1.57 * 3193.6 * 200000) \left(\frac{1520}{10.5}\right)^2} \\ = 9.22$$

$$\text{and } K = 5 + \left(\frac{5}{\left(\frac{d_o}{D} \right)^2} \right), \quad K=9.22 \quad d_o=1654.52(\text{spacing of stiffener})$$

Check assumption for $d_o=1500\text{mm}$

Solving K for $d_o=1500\text{mm}$ gives 10.13

$$\frac{D}{t_w} = 144.7 > 1.40 \sqrt{\left(\frac{Ek}{F_{yw}} \right)} = 107.3 \quad \text{OK}$$

Place stiffeners 1500mm apart.

Then solving for C with $K=10.13$ yields 0.44

$$\begin{aligned} \phi V_n = V_{cr} &= \phi C V_p = 1.0 * 0.44 * 3193.6 \text{KN} \\ &= 1405 \text{KN} > 1275.9 \text{KN} \quad \text{OK} \end{aligned}$$

Similarly intermediate stiffener placing can be computed.

End, intermediate and bearing stiffeners as well as shear connectors can be designed.

Wind Load analysis

The wind load acts primarily on the exterior girders. In bridges with composite decks, the wind force acting on the upper half of the girder, deck, barrier, and vehicle is assumed to be transmitted directly to the deck. These forces are transferred to the supports through the deck acting as a horizontal diaphragm. The wind force acting on the lower half of the girder is transmitted directly to the bottom flange.

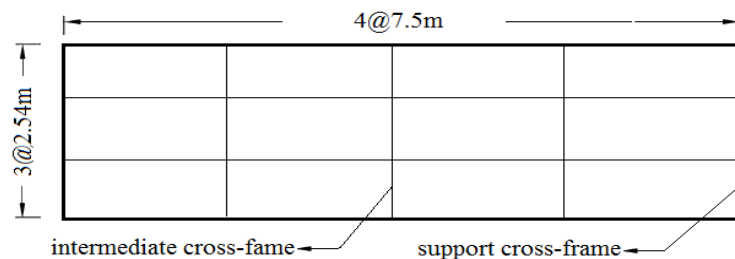


Fig.A4 Cross-frame locations.

$$\text{Wind load on bottom half } W = \left(\frac{\gamma p_B d}{2} \right) \quad [\text{A4.6.2.7}]$$

W = wind force per unit length applied to the flange

p_B = base horizontal wind pressure (MPa)

$p_B = 0.0024 \text{ MPa}$ [A3.8.1.2]

d = depth of the member (mm)

γ = load factor for the particular group loading combination (strength III applies = 1.4 which governing from that of strength 5)

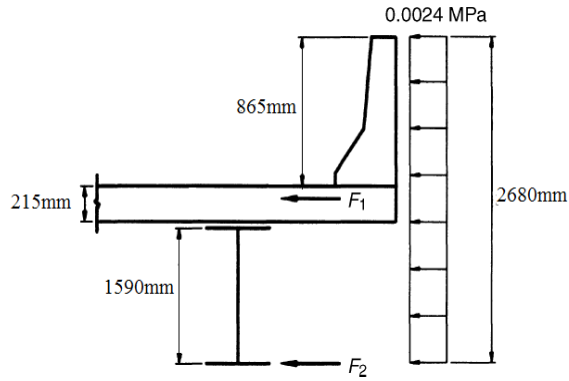


Fig A5 Wind load acting on bridge exterior.

$$W_{\text{bottom}} = F_2 = \left(\frac{1.4 \times 0.0024 \text{ MPa} \times 1590 \text{ mm}}{2} \right) = 2.67 \text{ kN/m}$$

$$M_{\text{max wind}} = \left(\frac{W L_b^2}{10} \right) \quad [\text{C 4.6.2.7.1-2}]$$

$$= \left(\frac{2.67 \times 7.5^2}{10} \right) = 15 \text{ kNm}, L_b = \text{unbraced length} = 7.5 \text{ m}$$

Section modulus of section

$$S_f = \frac{1}{6} (35) (410)^2 = 0.98 \times 10^6 \text{ mm}^3$$

$$f = \left(\frac{M_{\text{max wind}}}{S_f} \right) = \left(\frac{15 \text{ kNm}}{0.98 \times 10^6 \text{ mm}^3} \right) = 15.3 \text{ MPa}$$

$$W_{\text{top}} = F_1 = 1.4 \times 0.0024 \times 2680 - 2.67 = 6.34 \text{ MPa}$$

Loads on support cross frame (per cross frame)

$$P_{\text{cross frame}} = (F_2 + F_1) \times \frac{L}{2} \times \frac{1}{3} = (2.67 + 6.34) \times \frac{30}{2} \times \frac{1}{3}$$

$$= 45 \text{ kN}$$

Loads transmitted to bearings per girder

$$P_{\text{bearing}} = (F_B + F_T) \times \frac{L}{2} \times \frac{1}{4} = (2.67 + 6.34) \times \frac{30}{2} \times \frac{1}{4}$$

$$= 34 \text{ kN}$$

The provided section seems adequate.

Check assumption made in design

Longitudinal stiffness parameter K_g can be compute since For preliminary design $\left(\frac{Kg}{Lt_s^3}\right)$ is taken as unity.

Assumed beam weight =4KN/m (conservative),

$$n=8, I=20.2*10^9 \text{mm}^4, A=44660 \text{mm}^2$$

$$e_g = \frac{d}{2} + t_h + \frac{ts}{2} = \frac{1590}{2} + 25 + \frac{215}{2}$$

$$= 927.5 \text{mm}$$

$$K_g = n(I + Ae_g^2) = 8*(20.2*10^9 \text{mm}^4 + 44660*927.5^2 \text{mm}^4)$$

$$= 4.689*10^{11} \text{mm}^4$$

$$\frac{Kg}{Lt_s^3} = \left(\frac{4.689*10^{11} \text{mm}^4}{12*30000*215^3} \right) = 0.861$$

Re-compute distribution factors

Moment distribution for interior girder

$$m_{gm}^{SI} = 0.06 + \left(\frac{2540}{4300}\right)^{0.4} \left(\frac{2540}{30000}\right)^{0.3} \quad 0.816$$

$$= 0.375 \quad (0.512 \text{ primarily})$$

$$m_{gm}^{MI} = 0.075 + \left(\frac{2540}{4300}\right)^{0.6} \left(\frac{2540}{30000}\right)^{0.2} \quad (0.816)$$

$$= 0.5218 \quad (0.6386 \text{ primarily}) \quad (\text{governs})$$

Fatigue moment distribution for interior girder

$$m_{gm}^{SI \text{ fat}} = \frac{0.375}{1.2}$$

$$= 0.3126$$

No need to reevaluate for the other distribution factors..

$$M_{LL+IM} = [(M_{\text{TRACK OR TANDEM}}) \left(1 + \left(\frac{IM}{100}\right)\right) + M_{\text{LANE}}]$$

$$= 2037.6*1.33 + 1046$$

$$= 3756 \text{KNm}$$

$$V_{LL+IM} = [(V_{\text{TRACK OR TANDEM}}) \left(1 + \left(\frac{IM}{100}\right)\right) + V_{\text{LANE}}]$$

$$= 294*1.33 + 140$$

$$= 531 \text{KN}$$

$$M_{LL+IM \text{ fatigue}} = 1679.4 * 1.15 = 1931 \text{KNm}$$

$$V_{LL+IM \text{ fatigue}} = 266 * 1.15 = 306 \text{KN}$$

Re-Distributed live load effects (strength)

	$(M_{LL+IM})_{\text{Lane}}$ (KNm)	$(V_{LL+IM})_{\text{Lane}}$ (KN)
Interior girder	$0.5218 * 3756 = 1959$	450
Exterior girder	2910.5	410.75

Re-Distributed Fatigue load effects

	$(M_{LL+IM})_{\text{fatigue}}$ (KN/m)	$(V_{LL+IM})_{\text{fatigue}}$ (KN)
Interior girder	$0.3126 * 1931 = 603$	177
Exterior girder	1247	198

APPENDIX B

RAILWAY BRIDGE DESIGN EXAMPLE

Design simply supported ballasted deck plate girder bridge of Figure B1 with 30-m span for Cooper E80 load. Railroad width is 9860 mm from curb to curb which carries double track. Providing 170mm thick ballast and a standard tie 2.45m long and 205mm wide spaced at 610mm (ch.15,1.3.4.2.1).provide concrete deck thickness of 300mm (ch.15,1.3.4.2.2) Use $f'_c = 30$ MPa and M270 Grade 345 steel .ASD method.

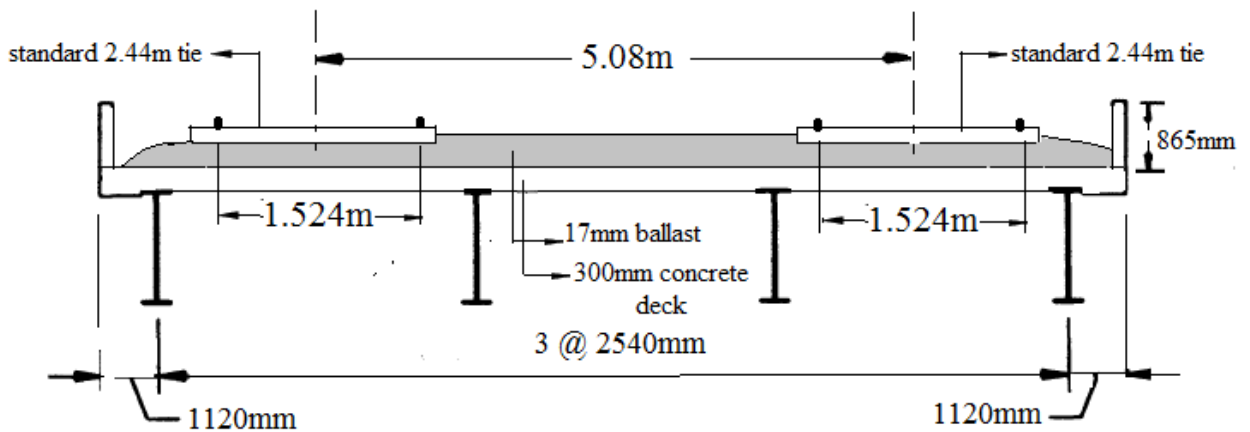


Fig. B 1 deck plate girder bridge cross-section

A. Develop General Section as shown in Fig B1

B. Design Reinforced Concrete Deck

Taking 1m Strip

Load computations

Dead load of ballast +tie +track

$$= 170\text{mm} \times 19\text{Kg/m}^3 \text{ (density of ballast including tie ch.15, 1.3.2a)} \times 9.81\text{m/s}^2$$

$$= 3.168\text{KN/m} + 0.64\text{KN/m} \text{ (allowance for track weight)}$$

$$= 3.8\text{KN/m}$$

Concrete slab dead load

$$= 300\text{mm} \times 2400\text{Kg/m}^3 \text{ (density of concrete)} \times 9.81\text{m/s}^2$$

$$= 7\text{KN/m}$$

Curb dead load (treated as concentrated load)

$$= 300\text{mm} \times 680\text{mm} \times 2400\text{Kg/m}^3 \times 9.81\text{m/s}^2$$

$$= 4.8\text{KN}$$

Live load

According to (ch.15, 1.7.9.2 b) and (ch.15, 1.15.1), the governing live load for 30m span length is that of Cooper E80 (EM360). then the maximum axle load to be used for design of slab is 360KN which distributed according to (ch.15, 1.3.4.2.2) to a length of $L_{tie} + L_{bottom\ of\ tie\ to\ top\ of\ girder} = 2.934m$ to yield 123KN/m laterally distributed load. And applying two full live loads for double track (ch.15, 1.3.3 d).

Analysis of load effects (taking 1m wide slab)

$$DL\ M_{max +ve} = 1.36\text{KNm}$$

$$DL\ M_{max -ve} = -5.26\text{KNm (along with the LL } M_{max -ve} \text{),}$$
$$\&-1.55\text{KNm (at face of support)}$$

$$LL\ M_{max +ve} = 82.65\text{KNm}$$

$$LL\ M_{max -ve} = -32.6\text{KNm}$$

Impact load

According to (ch.15, 1.7.9.2 b), the impact load is applied that of steel bridges, for $L < 53m$ full impact for double track.

Vertical effects (ch.15, 1.3.5 c)

The train is assumed to be modern railroad equipment (diesel locomotives and modern freight cars)

$$\text{Vertical effect} = 16 + \left(\frac{183}{L-9} \right) \quad \text{for } L > 24m, L=30m$$
$$= 16 + \left(\frac{183}{30-9} \right)$$
$$= 24.57\%$$

Rocking effect (ch.15, 1.3.5 d)

RE is calculated from loads applied as a vertical force couple, each being 20 percent of the wheel load without impact, acting downward on one rail and upward on the other on equally loaded girders and yields

$$\left(\frac{0.2W * 1.524}{2.54m} \right) = 12.12\%$$

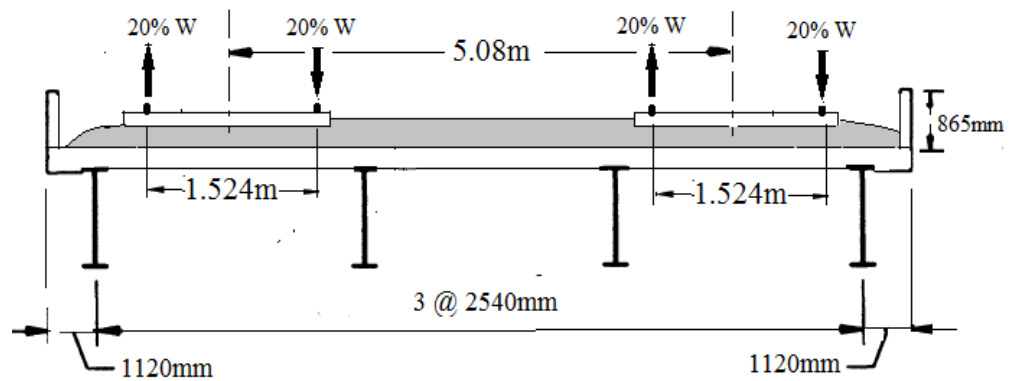


Fig. B 2 rocking effect of deck plate Girder Bridge

Thus, impact load = Vertical effects+ Rocking effect

$$=24.57\%+12.12\%$$

$$=36.69\%$$

Impact load for ballasted deck can be reduced to 90% of the impact calculated (ch.15, 1.3.5 b). The Design impact load in 90 %*(36.69%) =33%. Accordingly,

$$LL + IM M_{\max +ve} = 82.65 \text{ KNm} * 1.33$$

$$= 110 \text{ KNm}$$

$$LL + IM M_{\max -ve} = 32.6 \text{ KNm} * 1.33$$

$$= -43.4 \text{ KNm}$$

Load combination applicable for design of concrete slab (ch.8, 2.2.4b)

Group loading combination 1 is applicable

Group 1 D+L+I+CF+E+B+SF

Can be reduced to group 1 D+L+I

$$\text{Design moment } (DL+LL+IM)_{+ve} = 110 \text{ KNm} + 1.36 \text{ KNm}$$

$$= 111.36 \text{ KNm}$$

$$\text{Design moment } (DL+LL+IM)_{-ve} = -43.4 \text{ KNm} - 1.55 \text{ KNm}$$

$$= -45 \text{ KNm}$$

- Concrete slab thickness = 300mm
- Width of slab = 1000mm
- $d = 300 \text{ mm} - 40(8, 2.6.1 \text{ minimum cover}) - 10(\text{use } \Phi 20)$
= 250mm

For balanced design of concrete slab

$$k_b = \left(\frac{1}{1 + \left(\frac{f_s}{n f_c} \right)} \right) = 0.361$$

$$j_b = 1 - \left(\frac{k_b}{3} \right) = 1 - \left(\frac{0.361}{3} \right) = 0.879$$

$$d_b = \sqrt{\left(\frac{2M}{f_c b j k} \right)} = \sqrt{\left(\frac{2 * 111.36 \text{KNm}}{12 \text{MPa} * 1 \text{m} * 0.879 * 0.361} \right)} = 231 \text{mm}$$

$$d_b = 231 \text{mm} < d = 250 \text{mm}$$

Use $d = 250 \text{mm}$

$$f_c = \left(\frac{2M}{(k d)(j d)b} \right) = \left(\frac{2 * 111.36 \text{KNm}}{0.361 * 0.25 * 0.879 * 0.25 * 1} \right)$$

$$= 11.2 \text{MPa} < 12 \text{MPa OK}$$

$$A_s = \left(\frac{M}{f_s * j * d} \right) = \left(\frac{111.36 \text{KNm}}{170 \text{MPa} * 0.879 * 0.25 \text{m}} \right)$$

$$= 2985.5 \text{mm}^2/\text{m}$$

Use 10 $\Phi 20$

[ch.8, 2.7a] states that at any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a design moment strength ΦM_n at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete [ch.8, 2.26.1a].

$$M_{cr} = S_{nc} * f_r = \left(\frac{b h^2}{6} \right) * 0.62 \sqrt{f'_c}$$

$$= \left(\frac{1 * 0.3^2}{6} \right) * 0.62 \sqrt{30} = 51 \text{KNm}$$

$$1.2 M_{cr} = 61.1 \text{KNm} < M_n = 117.3 \text{KNm OK}$$

$$M_n = A_s (f_s * j * d) = 3140 \text{mm}^2 * (170 \text{MPa} * 0.879 * 0.25 \text{m})$$

$$= 117.3 \text{KNm}$$

Check for fatigue

Mean Impact load for fatigue (ch.15, 1.3.13) is taken as 35% of the impact load computed, which is,

$$\text{Fatigue}_{IM} = 35\% (33\%)$$

$$= 11.55\%$$

$$\text{Fatigue}_{LL + IM M_{\max +ve}} = 82.65 \text{ KNm} * 1.1155$$

$$= 92.2 \text{ KNm}$$

The fatigue stress limit according to (ch.8, 2.26.2 b) is

$$f_f = 145 - 0.33f_{\min} + 55(r/h) ,$$

$$f_{\min} = 0 \text{ for this case.}$$

$$r/h = 0.3$$

$$f_f = 145 - 0 + 0.3 * 55 = 161.5 \text{ MPa}$$

$$f_{LL + IM M_{\max +ve}} = \left(\frac{M}{A_S * j * d} \right) = \left(\frac{92.2 \text{ KNm}}{3140 \text{ mm}^2 * 0.879 * 250 \text{ mm}} \right)$$

$$= 134 \text{ MPa} < 161.5 \text{ MPa OK}$$

The provided section seems to be OK.

Girder design

The girders can be treated as equally loaded. That is two girders per track (ch.15, 1.3.4.2.4a).

Table B 1 Loads on girders

Design forces	Exterior girder	interior girder
DL of ballast +tie	7.57KN/m	8.05KN/m
DL of track (per girder)	1.5KN/m	1.5 KN/m
DL of concrete slab	16.88KN/m	17.94KN/m
DL of curb	1.2KN/m	1.2 KN/m

Live load	Cooper E80 (governing)	Cooper E80
-----------	------------------------	------------

From (ch.15, 1.15.1, Table ch.15-1-15) Cooper E80 governs for L=30m (wheel load per rail, per girder in this case). And the effects are,

$$M_{\max +ve} = 8767 \text{KNm}$$

$$V_{@end} = 1335 \text{KN}$$

$$M_{@ \text{quarter points}} = 6882 \text{KNm}$$

$$V_{@center E80} = 350 \text{KN}$$

$$V_{@center E100} = 374 \text{KN}$$

And the impact load, as already determined =33%

$$M_{LL+IM \max +ve} = 8767 \text{KNm} * 1.33$$

$$= 11,660 \text{KNm}$$

$$V_{LL+IM \max +ve} = 1335 \text{KN} * 1.33$$

$$= 1736 \text{KN}$$

Table B 2 Effects from other loads

Load type (KNm)	Exterior girder		Interior girder	
	Shear (KN)	moment (KNm)	shear (KN)	moment
DL ballast + tie	113.55	852	120.75	905.6
DL track	22.5	168.75	22.5	168.5
DL curb	21	157.5	21	157.5
DL deck	253.2	1899	269	2018
Assumed girder weight	186.45	1398.4	186.45	1398.4

Fatigue loads

Mean impact for fatigue is 11.55% (calculated earlier)

$$\begin{aligned} M \text{ fatigue}_{LL+IM \max +ve} &= 8767 \text{KNm} * 1.1155 \\ &= \mathbf{9779.54 \text{KNm}} \end{aligned}$$

$$\begin{aligned} V \text{ fatigue}_{LL+IM \max +ve} &= 1335 \text{KN} * 1.1155 \\ &= \mathbf{1489.2 \text{KN}} \end{aligned}$$

Effective width determination

In regard with (ch.15, 1.7.9.2c) the effective width of interior girder is=2.54m

In regard with (ch.15, 1.7.9.2d) the effective width of exterior girder is=2.39m

Applicable load combinations

According to [5] section 4.47 the applicable load combinations are

Table A-3 applicable load combinations

Load case	Load combination	Members	F _L
A1	DL+LL+I+CF	All members	1.00
B1	DL+LL+I+W+LF+N+CWR	All members (Except floorbeam hangers, high strength bolts)	1.25
C	(LL+I) _{range}	All members	<i>f_{fat}</i>
D2	Q	Members resisting stability	1.50

Load combinations uses for girder design are A1, B1 and C, the rest applies for design of cross-frames.

Comparing load combinations A1 and B1

Combination A1 V=3565.5KN M=16307KNm

Combination B1 DL+LL+I+W+LF+N+CWR

W_L (wind load on live load) = 4.4 kN/m wind on live load acting @ 2.45m above the rail (ch.15, 1.3.7a).

Vertical distributed load on exterior girder can be calculated as

$$= \left(\frac{4.4 \text{ kN/m} * 2.45 \text{ m}}{3 * 2.54 \text{ m}} \right) = 1.4 \text{ kN/m}$$

Moment from $W_V = 158 \text{ kNm}$ on exterior girder

Shear from $W_V = 21 \text{ kN}$ on exterior girder

Longitudinal wind load (which produce shear force and longitudinal horizontal force)

- **For loaded bridge** = 25% of lateral wind force (0.00144 MPa) (ch.15, 1.3.7d)

Longitudinal wind = 0.25 * 8.635 kN/m

= 2.15 kN/m acting @ 2.45m above rail

$$\text{Shear force} = \left(\frac{2.15 \text{ kN/m} * 30 \text{ m} * 2.45 \text{ m}}{30 \text{ m}} \right)$$

= 5.3 kN (additive with shear force computed Shear

from WV to compare with the shear force for unloaded longitudinal calculated below)

Total wind shear force on girder = 5.3 kN + 21 kN (governs)

= 26.3 kN > 8.8 kN

- **For unloaded bridge** = 25% of lateral wind force (0.0024 MPa) (ch.15, 1.3.8b)

Longitudinal wind = 0.25 * 14.33 kN/m

= 3.6 kN/m @ 2.45m above rail

$$\text{Shear force} = \left(\frac{3.6 \text{ kN/m} * 30 \text{ m} * 2.45 \text{ m}}{30 \text{ m}} \right)$$

= 8.8 kN

LF (vertical shear from longitudinal force)

$LF_{\text{BRAKING}} = 200 + 17.5 L = 734 \text{ kN}$ acting at 5.74m from rail

$$LF_{\text{BRAKING}} \text{ shear} = \left(\frac{734 \text{ kN} * 5.74 \text{ m}}{30 \text{ m}} \right) = 140 \text{ kN}$$

$LF_{\text{TRACTION}} = 200 \sqrt{L} = 1112 \text{ kN}$ acting at 4.214m from rail

$$LF_{\text{TRACTION}} = \left(\frac{1112 \text{ kN} * 4.214 \text{ m}}{30 \text{ m}} \right) = 156 \text{ kN (governs)}$$

Combination B1

$$V=3565.5+156+26.3=3747.8\text{KN}$$

$$M=16307+158=16465\text{KNm}$$

But for this combination 125% basic allowable stress can be used, therefore

$$V = \frac{3747.8\text{KN}}{1.25} = 2998\text{KN} < 3565.5\text{KN} \quad \text{and}$$

$$M = \frac{16465\text{KNm}}{1.25} = 13172\text{KNm} < 16307\text{KNm}$$

Thus combination A1 governs the strength design.

Section properties

The preliminary section assumed has 2840mm*20mm web and 600mm*80mm for both flanges, $F_y=345\text{MPa}$.

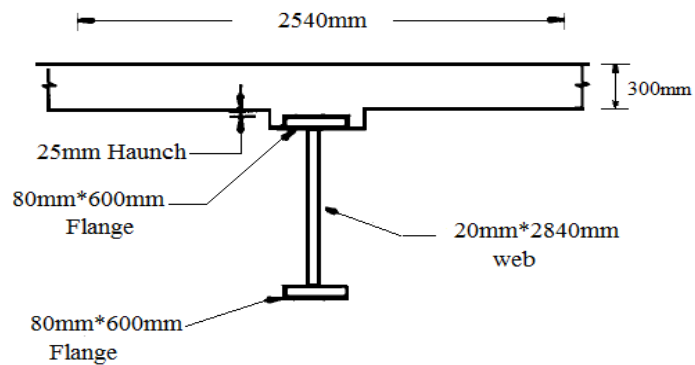


Fig. B 3 composite section for positive flexure

Table B 4 Steel section properties

Component	A (mm ²)	y _{bottom} (mm)	Ay _b (mm ³)	y _b -y	A(y _b -y ²)(mm ⁴)	I _O (mm ⁴)
Top flange (410×35)	48000	2960	1.421*10 ⁸	1460	1.02*10 ¹¹	25.6*10 ⁶
Web (1520*10.5)	56800	1500	85.2*10 ⁶	0	0	3.817*10 ¹⁰
Bottom flange (410×35)	48000	40	1.92*10 ⁶	-1460	1.02*10 ¹¹	25.6*10 ⁶
Total	152800		229.2*10 ⁶		2.0462*10 ¹¹	38228.8*10 ⁶

$$I_g = A (y_b - y^2)(\text{mm}^4) + I_o (\text{mm}^4) = 2.42848 * 10^{11} \text{mm}^4$$

$$S_{\text{top of steel}} = \frac{I_g}{y} = \frac{2.42848 * 10^{11} \text{mm}^4}{1500 \text{mm}} = 0.16189 * 10^9 \text{mm}^3$$

$$S_{\text{bottom of steel}} = 90\% \text{ of } I_g \text{ (account for connection holes in tension)} = 0.1457 * 10^9 \text{mm}^3$$

Table B 5 Composite steel and concrete section properties (shot term n=8)

Component	A (mm ²)	y _{bottom} (mm)	Ay _b (mm ³)	y _b -y	A(y _b -y ²)(mm ⁴)	I _o (mm ⁴)
Steel section	152800	1500	229.2*10 ⁶	-633.44	6.13*10 ¹⁰	2.42848*10 ¹¹
Concrete slab (n=8)	95250	3150	300*10 ⁶	1016.55	9.843*10 ¹⁰	7.14375*10 ⁸
Total	248050		529*10 ⁶		15.973*10 ¹⁰	2.4356*10 ¹¹

$$I_g = A (y_b - y^2)(\text{mm}^4) + I_o (\text{mm}^4) = 4.0329 * 10^{11} \text{mm}^4$$

$$I_{\text{net}} = 0.9 I_g = 3.6296 * 10^{11} \text{mm}^4$$

$$y_c = \frac{\sum Ay_b}{\sum A} = \frac{529 * 10^6 \text{mm}^3}{248050 \text{mm}^2} = 2133.44 \text{mm}$$

Table B6 Section modulus (n=8)

Location	n	c (mm)	I (I _g or I _{net}) (mm ⁴)	ns (gross or net)(mm ³)
Top concrete	8	1166.56	4.0329*10 ¹¹	2.7656*10 ⁹
Bottom concrete	8	866.56	4.0329*10 ¹¹	3.7231*10 ⁹
Top steel	1	866.56	4.0329*10 ¹¹	0.4654*10 ⁹
Bottom steel	1	2133.44	3.629*10 ¹¹	0.17*10 ⁹

Table B 7 Composite steel and concrete section properties (shot term n=24)

Component	A (mm ²)	y _{bottom} (mm)	Ay _b (mm ³)	y _b -y	A(y _b -y ²)(mm ⁴)	I _O (mm ⁴)
Steel section	152800	1500	229.2*10 ⁶	-283.8	1.23*10 ¹⁰	2.43*10 ¹¹
Concrete slab (n=24)	31750	3150	100*10 ⁶	1366.2	5.926*10 ¹⁰	2.38*10 ⁸
Total	184550		329*10 ⁶		7.136*10 ¹⁰	2.4356*10 ¹¹

$$I_g = A (y_b - y^2)(\text{mm}^4) + I_O (\text{mm}^4) = 3.1447 * 10^{11} \text{mm}^4$$

$$I_{\text{net}} = 0.9I_g = 2.83 * 10^{11} \text{mm}^4$$

$$y_c = \frac{\sum Ay_b}{\sum A} = \frac{329 * 10^6 \text{ mm}^3}{184550 \text{ mm}^2} = 1783.8 \text{ mm}$$

Table B8 Section modulus (n=24)

Location	n	c (mm)	I (I _g or I _{net}) (mm ⁴)	ns (gross or net)(mm ³)
Top concrete	24	1516.2	3.1447*10 ¹¹	4.978*10 ⁹
Bottom concrete	24	1216.2	3.1447*10 ¹¹	6.2*10 ⁹
Top steel	1	1216.2	3.1447*10 ¹¹	0.25865*10 ⁹
Bottom steel	1	1783.8	2.83*10 ¹¹	0.1586*10 ⁹

Table B 9 Composite section flexural stresses (unshored construction)

Location	DL1 _{non composite} (MPa)	DL2 _{composite (n=24)} (MPa)	LL+I _{n=8} (MPa)	LL+I _{(Range) n=8} (MPa)	Max stress (MPa)
Top concrete	-----	0.247	4.21	3.536	4.457
Bottom concrete	-----	0.198	3.13	2.626	3.328
Top steel	21.1	4.76	25	21	50.91
Bottom steel	23.44	7.76	68.6	57.52	100

$$\text{Shear stress} = f_v \text{ total} = \frac{V_{\text{TOTAL}}}{A_w}$$

$$= \frac{(120.75 + 22.5 + 21 + 1735.5 + 261 + 186.45) \text{KN}}{(20 * 2840 * 10^{-6}) \text{m}^2} = 41.32 \text{MPa}$$

Flexure and shear design

The stresses computed above must be cross checked with the allowable stresses prescribed by the code (AREMA MRE).

Allowable stresses (ch.15, 1.4.1)

$$F_{\text{concrete allowable}} = 0.4f_c = 0.4 * 30 \text{MPa} = 12 \text{MPa} > 4.457 \text{MPa} \quad \text{OK}$$

$$F_{\text{steel allowable}} = 0.55f_y = 0.55 * 345 \text{MPa} = 189 \text{MPa} > 100 \text{MPa} \quad \text{OK}$$

$$F_{\text{fat}} (\text{taking category B for } L=30\text{m}) = 110 \text{MPa} > 57.5 \text{MPa} \quad \text{OK}$$

$$F_v \text{ allowable} = 0.35 f_y = 0.35 * 345 \text{MPa} = 120 \text{MPa} > 41.32 \text{MPa} \quad \text{OK}$$

Detailed design of girder

Detailed Web plate design

Flexural buckling

$$\frac{h}{tw} = \frac{2840}{20} = 142 < 4.18 * \sqrt{\frac{E}{f_c}} = 4.18 * \sqrt{\frac{200000}{51}} = 261$$

Therefore no longitudinal stiffeners are required.

Shear buckling

$$h=2840\text{mm} > 2.12 * t_w * \sqrt{\frac{E}{f_y}} = 2.12 * 20 * \sqrt{\frac{200000}{345}} = 1020.87\text{mm}$$

Therefore web transverse stiffeners are required.

Combining bending and shear

$$f_b = 100\text{MPa} < (0.75 - 1.05 * \left(\frac{f_v}{f_y}\right)) * F_y < (0.75 - 1.05 \left(\frac{41.32}{345}\right)) * 345\text{MPa}$$

$$< 215\text{MPa} \text{ or } < 0.55 F_y = 189\text{MPa} \quad \text{OK}$$

Flange to web connection can be done.

Shear connector can be designed

Serviceability –deflection criteria

The required section gross moment of inertia for various deflection criteria, $f\Delta$, (ref [5] section 5.3.2.2.1) is

$$I_g > \left(\frac{M_{LL} + IM L f \Delta}{1934} \right), \quad \text{for } f\Delta = 640$$

$$I_{g \text{ Required}} > \left(\frac{8600 \text{Kip-ft} * 100 \text{ft} * (f\Delta = 640)}{1934} \right) = 284595 \text{in}^4$$

$$I_{g \text{ Provided}} = 968918 \text{in}^4 \gg I_{g \text{ Required}} = 284595 \text{in}^4$$

$$\text{and } \Delta_{LL+I} = \left(\frac{0.104 M_{LL} + I L^2}{EI} \right)$$

$$\Delta_{LL+I} = \left(\frac{0.104 * 8600 \text{kip-ft} * 100^2 \text{ft}^2}{29000 * 968908} \right) = 0.55 \text{in} = 14 \text{mm}$$

$$\text{And } \Delta_{\text{LIMIT}} = \left(\frac{L}{640} \right) = \left(\frac{30000}{640} \right) = 46.8 \text{mm} > 14 \text{mm} \quad \text{OK}$$

$$\Delta_{DL1} = \left(\frac{0.104 M_{DL1} L^2}{EI} \right) = \left(\frac{0.104 * 2520 \text{kip-ft} * 100^2 \text{ft}^2}{29000 * 583447 \text{in}^4} \right)$$

$$= 0.2676 \text{in}$$

$$\Delta_{DL2} = \left(\frac{0.104 M_{DL2} L^2}{EI} \right) = \left(\frac{0.104 * 908 \text{kip-ft} * 100^2 \text{ft}^2}{29000 * 755517 \text{in}^4} \right)$$

$$= 0.07446 \text{in}$$

$$\Delta_{DL1} + \Delta_{DL2} = 0.342 \text{ in} = 8.69 \text{ mm}$$

10mm cambered may be provided for dead load deflection. (ch.15, 1.2.10)

Girder design for erection (unshored construction) can be done.

Cross frame design

Applicable load combinations

Combinations D2 and F are applicable.

- Loads on cross-frame

According to (ch.15,1.11.4 g) , Where poured-in-place concrete decking is used in ballasted deck construction, cross frames or diaphragms shall be used at intervals not exceeding 7.32m.then provide cross-framed at interval of 6m.

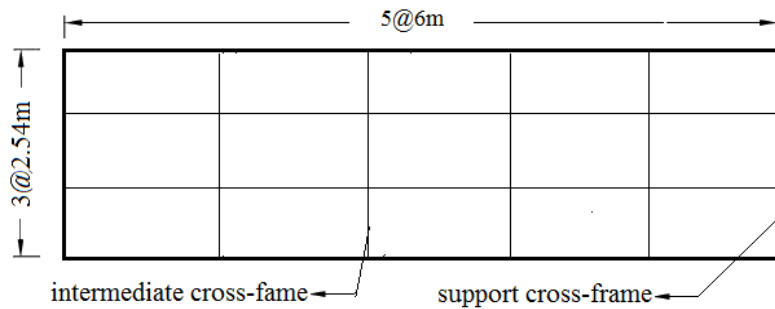


Fig. B 3 Location of cross-frame

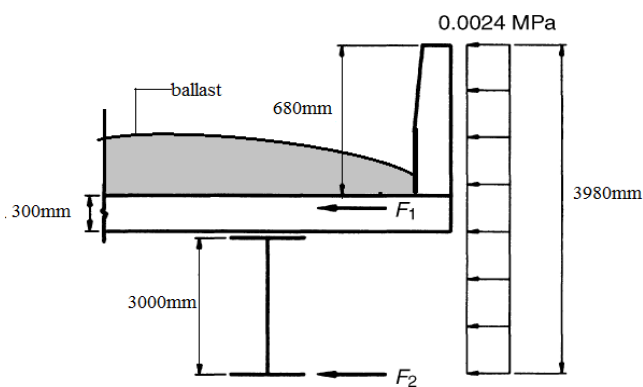


Fig. B 4 Wind loading on exterior girder

Wind pressure on loaded bridge =0.00144MPa [ch.15, 1.3.7 b)

Wind pressure on unloaded bridge =0.0024MPa (ch.15, 1.3.8 a)

Wind load on moving train =4.4KN/m (ch.15, 1.3.7a)

Notional lateral load = 2.93KN/m for loaded chord, 2.2KN/m for unloaded chord (ch.15, 1.3.7 c)

Lateral vibrational load =25% of axle load (ch.15, 1.3.9)

Derailment load = Cooper E80 wheel load (ch.15, 1.3.10 b)

Computation of lateral load design forces

- Unloaded bridge

$$w_{\text{top}} = (.68 + .3 + 1.5) * 0.0024 \text{MPa} * 1.5 = 8.928 \text{KN/m}$$

$$w_{\text{bottom}} = 1.5 * 0.0024 \text{MPa} * 1.5 = 5.4 \text{KN/m}$$

- Loaded bridge

$$W_{\text{top}} = (.68 + .3 + 1.5) \text{m} * 0.00144 \text{MPa} * 1.5 + 4.4 \text{KN/m}$$

$$= 9.795 \text{KN/m} > V_T = 2.93 \text{KN/m}$$

$$W_{\text{bottom}} = 1.5 * 0.00144 \text{MPa} * 1.5 = 3.24 \text{KN/m} > V_B = 2.2 \text{KN/m}$$

Notional vibrational load

$$V_T = 2.93 \text{KN/m}$$

$$V_B = 2.2 \text{KN/m}$$

Thus, the design wind load becomes

$$W_{\text{top}} = 9.795 \text{KN/m}$$

$$W_{\text{bottom}} = 5.4 \text{KN/m}$$

Lateral locomotive force

$$N = 25\% \text{ of Cooper E80} = 0.25 * 360 \text{KN} = 90 \text{KN}$$

Derailment load

Q = Cooper E80 wheel load at 1.524m eccentricity from center line of track with impact

Support cross frame design loads

- Wind load

$$W_T = 9.795 \text{ kN/m} \times 15 \text{ m} = 146.926 \text{ kN}$$

$$W_B = 5.4 \text{ kN/m} \times 15 \text{ m} = 81 \text{ kN}$$

$$N = 90 \text{ kN}$$

- Horizontal design loads

$$H_{\text{TOP}} = 146.925 \text{ kN} + 90 \text{ kN} = 235.925 \text{ kN}$$

$$H_{\text{BOTTOM}} = 81 \text{ kN}$$

- Stability check load (vertical load)

$$Q = 1.33 \times \text{Wheel load} = \frac{1.33 \times 360 \text{ kN}}{2} = 239.4 \text{ kN},$$

Since 1.5 times the allowable stress can be applied, the load can be reduced to $239.4 \text{ kN} / 1.5 = 160 \text{ kN}$ to compare with the above load computed.

- Horizontal design loads transferred to bearings

$$H_{\text{TOP}} + H_{\text{BOTTOM}} = 146.625 + 90 + 81 = 317 \text{ kN}$$

$$\text{Horizontal design load per girder} = \frac{317 \text{ kN}}{4} = 79 \text{ kN}$$

$$\text{Horizontal design load per cross-frame} = \frac{317 \text{ kN}}{3} = 105.6 \text{ kN}$$