



Bridge design

Part 2: Design loads



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Australian Standard[®]

Bridge design

Part 2: Design loads

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PREFACE

This Standard was prepared by the Standards Australia Committee BD-090, Bridge Design, to supersede AS 5100.2—2004.

This Standard is also designated as Austroads publication AP-G51.2-17.

The objectives of the AS(AS/NZS) 5100 series are to provide nationally acceptable requirements for—

- (a) the design of road, rail, pedestrian and cyclist path bridges;
- (b) the specific application of concrete, steel, timber and composite construction, which embody principles that may be applied to other materials in association with relevant standards;
- (c) the assessment of the load capacity of existing bridges; and
- (d) the strengthening and rehabilitation of existing bridges.

The objective of this Part (AS 5100.2) is to specify minimum design loads and load effects for road, rail, pedestrian and cyclist path bridges, and other associated structures.

The requirements of the AS(AS/NZS) 5100 series are based on the principles of structural mechanics and knowledge of material properties, for both the conceptual and detailed design, to achieve acceptable probabilities that the bridge or associated structure being designed will not become unfit for use during its design life.

Significant differences between this Standard and AS 5100.2—2004 are the following:

- (i) Changes and clarifications to the provision for collision loads from rail traffic.
- (ii) Changes to dynamic load allowance for rail traffic load effects.
- (iii) Addition to provisions for bridge collision from waterway traffic.
- (iv) Updated bridge traffic barrier loads to more closely reflect vehicles currently using the road network. Barrier test levels and minimum effect heights were adopted from the AASHTO *Manual for Assessing Safety Hardware* (MASH 2009) which replaced NCHRP Report 350 (1993).
- (v) Earthquake design procedures for bridges rewritten to align with the current earthquake loading Standard AS 1170.4—2007, *Structural design actions*, Part 4: *Earthquake actions in Australia*. New displacement-based earthquake design procedures were included.
- (vi) Improvement to serviceability and fatigue limit states for road signs and lighting structures.
- (vii) Expansion of water flow forces to include impact from large moving objects during flood events.
- (viii) Addition of light rail vehicles.

Other differences between this Standard and AS 5100.2—2004 are the following:

- (A) Improved pedestrian and cyclist path barrier loads.
- (B) Expanded dynamic loads for pedestrian and cyclist path bridges.
- (C) New table for unfactored vertical pressure due to design rail traffic loads.
- (D) Inclusion of super-t girders in the calculation of bridge thermal effects.
- (E) Clarification of loads and load factors for construction loads.
- (F) Addition of protective screen design for wind load and robustness.

(G) New fire effect load case.

A number of new or revised appendices have been added to this edition of the Standard, which provide additional information and guidance as follows:

- (1) Update to special performance level bridge barrier loads.
- (2) New alternative force-based earthquake design procedures.
- (3) Bending moment and shear force for SM1600 and 300LA loads for simply supported spans.
- (4) A summary of load factors and load combinations.

In line with Standards Australia editorial policy, the words ‘shall’ and ‘may’ are used consistently throughout this Standard to indicate, respectively, a mandatory provision and an acceptable or permissible alternative.

Statements expressed in mandatory terms in Notes to Tables are deemed to be requirements of this Standard.

The term ‘informative’ has been used in this Standard to define the application of the appendix to which it applies. An ‘informative’ appendix is only for information and guidance.

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STANDARDS AUSTRALIA

Australian Standard Bridge design

Part 2: Design loads

1 SCOPE AND GENERAL

1.1 Scope

This Standard sets out minimum design loads, forces and load effects for road, rail, pedestrian and cyclist path bridges, and other associated structures.

1.2 General

Structures shall be proportioned for the design loads, forces and load effects in accordance with Clauses 6 to 26, as appropriate.

NOTE: If the relevant authority approves, the designer may vary any of the loads set out in this Standard, provided the provisions of AS 5100.1 are complied with.

The design loads and forces shall be considered as acting in combinations as set out in Clause 23.

NOTE: A summary of load factors is tabulated in Appendix D.

Each individual bridge shall be assessed to ascertain whether any other loads, forces or load effects are applicable for that particular design. The magnitude of these additional forces or load effects and their combination with other loads shall be consistent with the principles set out in AS 5100.1.

On the front sheet of the bridge drawings, the following details relating to design loads shall be shown, where relevant:

- (a) The Standard used.
- (b) Any significant variation to the minimum design loads as set out in this Standard.
- (c) Traffic load, e.g. 300LA and SM1600, including lateral position, if critical, and the number of design lanes.
- (d) Design traffic speed.
- (e) Fatigue criteria, including number of cycles and route factor.
- (f) Pedestrian loads, both horizontal and vertical.
- (g) Collision load on the structure (e.g. substructure and superstructure where applicable) or alternative load paths provided.
- (h) Design wind speeds.
- (i) Flood data, e.g. design velocities, levels, debris, and the like.
- (j) Earthquake criteria.
- (k) Differential settlements and mining subsidence effects allowed for in the design.
- (l) Foundation data where not shown elsewhere.
- (m) Barrier performance level.
- (n) The construction loads, methods and sequence, and any other specific limitations.

(o) Fire effects.

NOTE: Wave action is not included in this Standard. Where the bridge is subject to wave action, refer to specialist literature.

1.3 Special studies

Where changes are made to a part or all of the design processes detailed in this Standard or new information or methods are introduced, they should be established by special studies.

NOTE: For information on special studies refer to AS 5100.1 Appendix B

2 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

NOTE: Documents referenced for informative purposes are listed in the Bibliography.

AS

1170 Minimum design loads on structures

1170.4—2007 Part 4: Earthquake actions

1530 Methods for fire tests on building materials, components and structures

1530.4 Part 4: Fire-resistance test of elements of construction

1657 Fixed platforms, walkways, stairways, and ladders—Design, construction and installation

5100 Bridge design

5100.1 Part 1: Scope and general principles

5100.3 Part 3: Foundations and soil-supporting structures

5100.4 Part 4: Bearings and deck joints

5100.5 Part 5: Concrete

5100.9 Part 9: Timber bridges

AS/NZS

1170 Structural design actions

1170.2 Part 2: Wind actions

5100 Bridge design

5100.6 Part 6: Steel and composite construction

Austroads

Guide to Road Design

Guide to Traffic Management Part 3: Traffic Studies and Analysis

AASHTO

LRFD Bridge Design Specifications

Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

Manual for Assessing Safety Hardware (MASH)

3 DEFINITIONS

For the purposes of this Standard, the definitions given in AS 5100.1 and those below apply.

3.1 Air space developments

A structure built over a railway or road to support overhead offices, shops, public space or accommodation.

3.2 Cantilever sign structure

A sign structure supported at one end only.

3.3 External developments

A public or private structure adjacent to the road or rail corridor.

3.4 High-mast light poles

Light poles with an overall height exceeding 13 m.

3.5 Portal sign structure

A sign structure comprising one or more horizontal or sloped members supported by at least two vertical members.

NOTE: The members may be trusses.

3.6 Rail

Rail traffic includes rail freight trains, rail passenger trains, electrified trains, light rail traffic, trams and cane rail traffic.

3.7 Track category

3.7.1 Heavy haul freight (HHF)

Freight rail transport carrying axle loads over 25 t.

3.7.2 Main line freight (MLF)

Freight rail transport carrying axle loads up to 25 t.

3.7.3 Branch line freight (BLF)

Freight rail transport carrying axle loads below 25 t.

3.7.4 Light rail (LR)

Passenger only rail transport with axle loads not exceeding 15 t, including trams.

3.8 Underground rail

A rail track that is continuously enclosed above, below and both sides by structure and/or ground for a length of 80 m or greater, or as otherwise specified by the relevant rail authority.

4 NOTATION

Unless a contrary intention is given, the following applies:

- (a) Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.
- (b) The dimensional units for length and stress in all expressions or equations are to be taken as millimetres (mm), Newtons (N) and megapascals (MPa) respectively, unless specifically noted otherwise.
- (c) An asterisk (*) placed after a symbol as a superscript denotes a design action effect due to the design load for the ultimate limit state (ULS).

Symbol	Definition
A	axle load
ALF_i	accompanying lane factor for the i^{th} lane where ' i ' equals 2 or more
A_d	wetted area of the pier normal to the water flow, equal to the thickness of the pier normal to the direction of the water flow multiplied by the height of the water flow
A_{deb}	projected area of debris

A_L	wetted area, equal to the width of the pier parallel to the direction of the water flow multiplied by the height of the water flow; <i>or</i>
	plan deck area of the superstructure
A_p	bridge area in plan
A_s	net wetted area of the superstructure, including any railings or barriers, projected on a plane normal to the water flow
A_t	area of the structure for calculation of wind load
a	maximum vertical acceleration
B	length of the bearing seat transverse to the bridge longitudinal axis
b	width between external barriers ignoring internal kerbs, median barriers and medians; <i>or</i>
	overall width of the bridge between outer faces of barriers
$C(T_f)$	design action coefficient for elastic horizontal earthquake response
C_d	drag coefficient
$C_d(T_f)$	design action coefficient
$C_h(T)$	acceleration spectral shape factor
$C_h(T_f)$	acceleration spectral shape factor at fundamental natural period of vibration of the bridge frame in the direction considered (longitudinal or transverse)
C_L	lift coefficient
C_m	moment coefficient
C_s	side force coefficient (which depends on the angle between the water flow direction and the plane containing the pier)
C_T	base number of load cycles
C_1	coefficient dependent on the end fixity conditions
D	total depth of superstructure
D_c	section depth in the direction considered
d	depth of the superstructure, including solid barrier, if applicable; <i>or</i>
	depth below underside of sleepers; <i>or</i>
	thickness of the slab over a box cell
d_{bl}	diameter of longitudinal reinforcement steel
d_{sp}	wetted depth of the superstructure (including any railings or barriers) projected on a plane normal to the water flow
d_{ss}	wetted depth of the solid superstructure (excluding any railings but including solid barriers) projected on a plane normal to the water flow
d_{wgs}	vertical distance from the girder soffit to the flood water surface upstream of the bridge
F_{BM}	braking force applied by multiple vehicles
F_{BS}	braking force applied by a single vehicle
F_c	centrifugal force
F_d	design drag force

F_i	design transverse force at bridge frame mass location m_i proportional to $m_i \phi_i$
F_F	bridge frame horizontal earthquake force
F_L	ultimate longitudinal or transverse inward load on a barrier; <i>or</i> design lift force
F_r	Froude number
F_T	ultimate transverse outward load on a barrier
F_V	ultimate vertical downward load on a barrier
F_{Lu}^*	ultimate design lift force
f_0	fundamental frequency of vibration
f_{sy}	characteristic yield strength of flexural reinforcement
f_{sye}	expected yield strength of flexural reinforcement
$f_{sy,t}$	characteristic yield strength of lateral reinforcement
f_{ul}	characteristic ultimate strength of flexural reinforcement
f^*	fatigue design stress range
f'_c	characteristic compressive (cylinder) strength of concrete at 28 days
f'_{cc}	confined compressive strength of concrete, which may be taken as $1.5f'_c$ if not calculated by rational analysis
G	dead load
G_b	rail ballast and track loads
G_e	soil and groundwater loads on retaining walls and buried structures
G_s	superimposed dead load
g	acceleration due to gravity
H	pier height between the centre of plastic hinges at the top and bottom of the pier in double bending; <i>or</i> pier height from the centre of the plastic hinge to the point of contraflexure at top or bottom of the pier in single bending; <i>or</i> height from the base to the intersection of the uppermost principal horizontal members
h	depth of fill cover height of the post above the base plate
h_d	average height of piers supporting the superstructure length L_d
K_i	individual pier longitudinal or transverse stiffness, expressed as force per unit longitudinal or transverse displacement at the location of mass i
k	coefficient
k_e	equivalent effective stiffness of the bridge frame
k_p	probability factor
L	effective span; <i>or</i> loaded length; <i>or</i> span of member between posts; <i>or</i>

	length from the intersection of the principal members to the tip of the cantilever arm of a cantilever sign structure
L_{α}	characteristic length
L_{bs}	minimum overlap length measured normal to the face of an abutment or pier
L_c	distance from the centre of the plastic hinge to the point of contraflexure in the pier
L_d	length of the superstructure to the next expansion joint
L_f	span of main girders, trusses or stringers, or cross-girder spacing for cross-girders
L_g	distance of wheel load to the track centre-line
L_L	vehicle contact length for longitudinal collision load on a barrier
L_{LF}	total length of the bridge
L_m	mean length of main girders over n continuous spans
L_{max}	largest of the values $L_1, L_2, \dots, L_n$
L_p	plastic hinge length
L_{sp}	strain penetration length for reinforced concrete piers
L_T	vehicle contact length for transverse collision load on a barrier
L_v	distance between centres of axle groups
	vehicle contact length for vertical loads on a barrier
L_1, L_2, L_n	span lengths of a continuous structure
L_{α}	characteristic length
M_g	design superstructure moment due to water flow
M_s	shielding multiplier
M_{Lu}^*	ultimate moment due to water flow and/or debris loading, as applicable
m_e	effective mass of the bridge frame
m_i	one of the n individual masses representing the bridge frame
N^*	pile ultimate axial force
N	duration of construction, in years
n	number of standard design lanes; <i>or</i> effective number of stress cycles; <i>or</i> number of continuous main girder spans
n_T	number of equivalent stress cycles of amplitude (f^*) per train
P	annual probability of exceedance
PS	prestress effect
P_r	proximity ratio
Q	traffic loading from road, rail, pedestrian, cyclist path or maintenance traffic
R	return period
R_{ξ}	damping modifier

r	radius of curve
S_r	relative submergence
T	period of vibration; <i>or</i>
	temperature
T_f	fundamental natural period of the bridge; <i>or</i>
	bridge frame fundamental natural period in the direction considered
t	thickness of the deck
V	design traffic speed; <i>or</i>
	velocity of water flow
V_i	shear force of the i^{th} element of the bridge
	seismic shear force at the top of pier or abutment component i
V_s	design wind speed for serviceability limit state (SLS)
V_u	design wind speed for ultimate limit state (ULS)
W_{BM}	load due to multiple lanes of the M1600 moving traffic load for the length under consideration
W_{BS}	load due to a single lane of the M1600 moving traffic load for the length under consideration
W_c	load due to multiple standard design lanes of the M1600 moving traffic load for the length under consideration
W_i	one of the n individual weights representing the bridge frame
W_t	serviceability design transverse wind load
W_t^*	ultimate design transverse wind load
W_v	serviceability design vertical wind load
W_v^*	ultimate design vertical wind load
y	average flow depth; <i>or</i>
	static displacement due to design pedestrian load; <i>or</i>
	variable distance taken in determining temperature change at $T(y)$
y_{gs}	average vertical distance from the girder soffit to the bed assuming no scour at the span under consideration
Z	design seismic hazard factor; <i>or</i>
	bearing layout modulus
α	dynamic load allowance (DLA)
δ	logarithmic decrement of decay of vibration
δ_b	displacement capacity at superstructure resulting from the pier-cap bearing deformation
δ_c	displacement capacity of the first bridge frame pier or abutment to reach displacement capacity
δ_d	ductile displacement capacity
δ_f	displacement capacity at superstructure resulting from foundation deformation

δ_i	transverse displacement capacity of a bridge pier or abutment
δ_p	inelastic displacement capacity
δ_y	yield displacement capacity
ΔF_i	transverse displacement at bridge frame mass location m_i due to the application of F_i
$\Delta_b, \Delta_f, \Delta_s$	bearing, foundation and pier structural displacement, respectively
Δ_d	ductile seismic displacement demand
Δ_e	corner-period elastic seismic displacement demand
$\Delta_e(T)$	elastic seismic displacement demand for horizontal earthquake response
$\Delta_e(T_f)$	elastic seismic displacement demand for the bridge frame fundamental natural period in the direction considered
$\Delta_h(T)$	elastic seismic displacement spectral shape factor
Δ_i	ductile seismic displacement demand at the top of pier or abutment component i
Δ_k	characteristic horizontal seismic displacement demand of the bridge frame
Δ_L	longitudinal seismic displacement at the abutment
ε_{sd}	tensile strain limit in flexural reinforcement; <i>or</i> tensile strain in reinforcing steel
ε_{sul}	strain at maximum stress of flexural reinforcement
ε_{sut}	strain at maximum stress of lateral reinforcement
ε_y	strain at the expected yield strength of the flexural reinforcement (or structural steel)
γ_g	load factor for dead load
γ_{gb}	load factor for rail ballast and track loads
γ_{ge}	load factor for soil and groundwater loads
γ_{gs}	load factor for superimposed dead load
γ_Q	load factor for traffic load
γ_{WF}	ultimate load factor for forces resulting from water flow
μ	design ductility factor
μ_d	displacement ductility
ξ_b, ξ_f, ξ_s	bearing, foundation and pier structural damping, respectively
ξ_e	bridge frame equivalent viscous damping ratio corresponding to the design ductility level of response
ξ_i	elastic viscous damping
θ	super elevation of the road
θ_p	plastic rotation capacity at the plastic hinge
θ_w	angle between the direction of the water flow and the transverse centre-line of the pier
ρ_s	volumetric ratio of lateral (transverse) reinforcement

ϕ_c	value of normalised fundamental displacement mode shape at the first bridge frame pier or abutment to reach displacement capacity
ϕ_e	characteristic value of the fundamental mode shape of the bridge frame
ϕ_i	value of normalized fundamental displacement mode shape at pier or abutment
ϕ_{ls}	ductile curvature corresponding to the strain limit at the relevant design performance level
ϕ_y	yield curvature
ψ	dynamic response factor

5 MATTERS FOR RESOLUTION BEFORE DESIGN COMMENCES

The matters for resolution listed below shall, where relevant, be confirmed as accepted by the relevant authority before commencing the design process.

- 1 Approval to vary any of the loads set out in this Standard, provided the provisions of AS 5100.1 are complied with (Clause 1.2).
- 2 Design loads and factors for road bridges carrying rail traffic (Clause 7.4).
- 3 Load factors for centrifugal and braking loads from heavy load platforms, when applicable (Clause 7.10).
- 4 Approval of an analytical procedure for the distribution of road traffic loads through fill (Clause 7.12).
- 5 Design load for a pedestrian or cyclist path bridge that is also used for maintenance, inspection or emergency vehicle access (Clause 8.1).
- 6 Design loads for rail bridges carrying cane rail traffic and/or other special applications (Clause 9.1).
- 7 Approval to use the rational method for braking and traction forces (Clause 9.7.2 and 9.7.2.3).
- 8 Bridge-specific design parameters to be used in applying the rational method for braking and traction forces (Clause 9.7.2.3).
- 9 Approval of a risk analysis for road bridges designed with an alternative load path under collision load (Clause 11.1).
- 10 Approval of a risk analysis for bridge supports located between 10 m and 20 m from the centre-line of a rail track (Clause 11.4.2.4).
- 11 Approval of a dynamic collision analysis (Clause 11.4.4.2).
- 12 Recommendation of the type of vessel, weight of vessel and speed of impact on a bridge for collision from waterway traffic, and approval of the proposed design vessel and speed (Clause 11.6).
- 13 Approval of the minimum equivalent static ship impact force applicable to piers in navigable waterways (Clause 11.6).
- 14 Specification of the ultimate design load, load distribution length and minimum effective height for special barrier performance levels (Clause 12.2.2 and 12.2.3).
- 15 Approval of a load transfer mechanism across a movement joint in a rigid barrier (Clause 12.4.2).
- 16 Approval of a detailed dynamic analysis (Clause 13.2.3).
- 17 Approval of a vibration assessment of a rail bridge, when required (Clause 13.3).

- 18 Bridge earthquake design category classification (Clause 15.4.1).
- 19 Approval of large items for flood impact (Clause 16.7.3).
- 20 Construction design load criteria for other types of bridge construction (Table 22.2.2).
- 21 Approval of the average recurrence interval for wind load on noise barriers and protection screens (Clause 25.3.2).

6 DEAD LOADS (G)

6.1 General

The nominal dead load shall be calculated from the dimensions shown on the drawings and the mean value of the weight per unit volume of the materials. A value based on the densities of the materials, the percentage of reinforcement and other appropriate factors shall be adopted. Wherever possible, design densities shall be based on measurements of the materials to be used.

NOTES:

- 1 The weights per cubic metre given in Tables 6.1(A) and 6.1(B) may be used in calculating the nominal dead load unless a more precise determination has been made. Where a range of values is given, calculations should be performed using the extremes of the expected range and the most critical case used for design.
- 2 For other materials, refer to AS 1170.1.

Selecting a high value of density may be conservative when considering some limit states, but may not be conservative when considering stability, stresses at transfer of prestress and the like. If insufficient information is available to accurately assess the mean weight per unit volume, calculations shall be performed using a range of values and the most critical case shall be used for the design. The density of reinforced concrete shall take account of variations in aggregate density and shall allow for the mass of included reinforcement.

To ensure that the structure satisfies minimum strength and stability criteria, an ultimate load combination comprising only dead load, superimposed dead load, rail ballast and track load and soil and groundwater load shall be considered. The load factors for this load combination shall be in accordance with Clause 23.2.

TABLE 6.1(A)
WEIGHT FOR NOMINATED DEAD LOADS

Material	Weight per cubic metre kN/m ³
Aluminium alloy	26.7
Bituminous wearing surface, asphalt	22.0
Ballast for railways	19.0
Compacted earth filling	22.0 (see Note 1)
Compacted gravel, road metal	19.0–23.0
Concrete, unreinforced (add 0.6 for each 1% by volume of steel reinforcement and tendons)	See Table 6.1(B)
Masonry	23.5
Neoprene	11.3
Sand, fine (dry)	15.5–17.5
Sand, coarse (dry)	18.0–19.5
Sand (saturated)	22.5
Steel and other ferrous metals	77.0
Timber, softwood	7.0 (see Note 2)
Timber, hardwood	11.0 (see Note 2)
Water, fresh	9.8
Water, salt	10.0

NOTES:

- 1 A value of 22 kN/m³ for unit weight of fill is used for normal fill material; however, the unit weight of soils of volcanic origin (e.g. pumice sand at 14 kN/m³) may be significantly lower and the unit weight of soils of oxide origin (e.g. iron ore at 27 kN/m³) may be significantly higher.
- 2 For the unit weight of particular timber species, refer to AS 5100.9.

TABLE 6.1(B)
WEIGHT PER CUBIC METRE OF UNREINFORCED CONCRETE

Typical coarse aggregates		Density of coarse aggregates kg/m ³	Cement content kg/m ³	Weight per cubic metre kN/m ³
Melbourne basalt, Sydney basalt	Adelaide quartzite, Brisbane gravel, Perth granite, Sydney gravel	2500	450	24.0
			330	22.5
		2700	450	24.5
			330	23.0
	Hobart dolerite	2900	450	25.5
			330	24.0
		3100	450	26.0
			330	25.0

NOTE: The values given in the Table apply to normal concrete, have no added air and the accuracy is approximately ± 0.5 kN/m³.

6.2 Dead load of structure

Dead load shall be considered as the weight of the parts of the structure that are structural elements and any non-structural elements that are considered unlikely to vary during construction and use of the structure, such as barriers and kerbs of steel or concrete.

To obtain the design dead loads for ULS and SLS, the nominal dead load shall be multiplied by the appropriate load factor for dead load (γ_g) given in Table 6.2. The appropriate value of γ_g shall be applied to the dead load of all parts of the structure.

TABLE 6.2
LOAD FACTORS FOR DEAD LOAD OF STRUCTURE (γ_g)

Type of material	ULSs where dead load		SLSs
	Reduces safety	Increases safety	
Steel	1.10	0.90	1.0
Concrete	1.20	0.85	1.0
Concrete at transfer of prestress	1.15	0.90	N/A
Timber	1.25	0.80	1.0

NOTE: For precast members, where—

- (a) appropriate control and monitoring are exercised over dimensions and mass; and
- (b) the value for reinforced concrete density is based upon the measured density of the concrete to be used and accurate estimates of reinforcement inclusion,

the relevant authority may approve a reduction of load factor to not less than 1.1 for ULSs for the cases where the dead load reduces safety.

6.3 Superimposed dead load (G_s)

Superimposed dead load shall be considered as the weight of all materials forming the loads on the structure which are not structural elements and which vary during construction and use of the structure.

NOTE: Examples of superimposed dead load include surfacing material, footway filling, pipes, conduits, cables and other utility services, and additional concrete to compensate for the hog of prestressed beams.

If a separate wearing surface is to be placed when the bridge is constructed or if placement of a separate wearing surface is anticipated in the future, allowance shall be made for its weight in the superimposed dead load.

The design of superimposed dead loads for ultimate and SLSs shall be obtained by applying the appropriate load factor (γ_{gs}), given in Table 6.3, to the nominal superimposed dead loads on the structure.

For special cases, and subject to approval of the relevant authority, the values of γ_{gs} to be applied to the nominal superimposed dead load may be reduced to an amount not less than those given in Item (b) of Table 6.3, provided the nominal superimposed dead load is not exceeded during the life of the bridge.

TABLE 6.3
LOAD FACTORS FOR SUPERIMPOSED DEAD LOAD (γ_{gs})

Type of structure	Type of load	ULSs where G_s		SLSs
		Reduces safety	Increases safety	
(a) All structures, except for Item (b)	Permanent	2.0	0.7	1.3
	Removable	2.0	0.0	1.3
(b) <i>Special cases</i> Where specified by the relevant authority for major structures and superimposed dead loads are controlled.	Permanent	1.4	0.8	1.0
	Removable	1.4	0.0	1.0

6.4 Soil and groundwater loads on retaining walls and buried structures (G_e)

The design of foundations and soil-supporting structures shall be carried out in accordance with this Standard and AS 5100.3. The soil and groundwater loads shall be factored by the load factor (γ_{ge}) given in Table 6.4.

TABLE 6.4
LOAD FACTORS FOR SOIL AND GROUNDWATER LOADS (γ_{ge})

Type of soil	ULSs where soil		SLSs
	Reduces safety	Increases safety	
Controlled fill with regular testing of soil density	1.25	0.85	1.0
All other fills and in situ soils	1.5	0.7	1.2
Groundwater	1.0	1.0	1.0

NOTE: Variation in water levels shall be taken into account by using design levels based on a return period of 1000 years for the ULS or 100 years for the SLS.

6.5 Rail ballast and track loads (G_b)

Rail ballast and track shall be considered as superimposed dead loads. The design loads for the ULS and SLS shall be obtained by applying the appropriate load factor (γ_{gb}) given in Table 6.5 to the nominal ballast and track loads.

For bridges where it is possible to fill with ballast to a much greater depth than normally specified, the maximum amount of ballast possible on the bridge shall also be determined and the nominal amount of ballast shall be taken as not less than 0.7 times that maximum amount.

TABLE 6.5
LOAD FACTORS FOR RAIL BALLAST AND TRACK LOADS (γ_{gb})

Type of structure	Type of load	ULSs where G_{bs}		SLSs
		Reduces safety	Increases safety	
All structures	Ballast and track	1.7	0.7	1.3
	Transom track	1.4	0.9	1.2

7 ROAD TRAFFIC (Q)

7.1 General

Road traffic load is the load resulting from the passage of vehicles, either singly or in groups, or pedestrians. The magnitude, direction and positioning of loads in this Standard produce effects in structures that approximate the effects of vehicles or groups of vehicles. The load models are not intended to be the same as actual vehicles.

All road bridges shall be designed to resist the following:

- (a) The traffic loads specified in this Standard, which approximate the effects induced by moving traffic, stationary queues of traffic and pedestrian traffic.
- (b) The most adverse effects induced by the following loading elements, combinations of these elements and their corresponding load factors:
 - (i) W80 wheel load.
 - (ii) A160 axle load.
 - (iii) M1600 moving traffic load.
 - (iv) M1600 moving tri-axle group load.
 - (v) S1600 stationary traffic load.
 - (vi) HLP320 or HLP400, if specified by the relevant authority.
 - (vii) Dynamic load allowance (α).
 - (viii) Number and position of traffic lanes.
 - (ix) Accompanying lane factors (ALF).
 - (x) Centrifugal forces (F_c).
 - (xi) Braking forces (F_{BS} , F_{BM}).
 - (xii) Fatigue load.
 - (xiii) Pedestrian load.

7.2 SM1600 loads

7.2.1 General

The abbreviation SM1600 represents the design loads W80, A160, M1600, M1600 tri-axle group and S1600 traffic design loads.

NOTE: Appendix C tabulates values of bending moment and shear force due to SM1600 loading.

7.2.2 W80 wheel load

The W80 wheel load models an individual heavy wheel load.

The W80 wheel load shall consist of an 80 kN load uniformly distributed over a contact area of 400 mm $\times$ 250 mm. It shall be applied anywhere on the roadway surface and to all structural elements for which the critical load is a single wheel load.

7.2.3 A160 axle load

The A160 axle load models an individual heavy axle.

The A160 axle load shall be positioned laterally within a 3.2 m standard design lane, as shown in Figure 7.2.3.

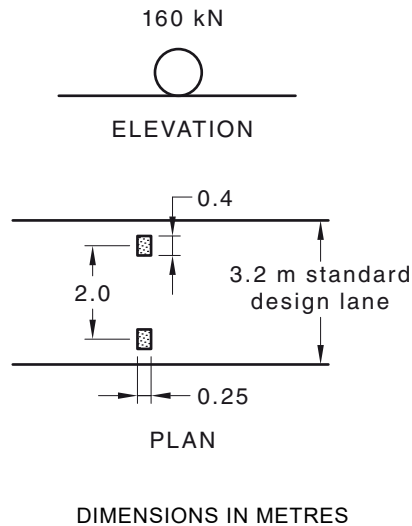


FIGURE 7.2.3 A160 AXLE LOAD

7.2.4 M1600 moving traffic load

The M1600 moving traffic load models the loads applied by a moving stream of traffic.

The M1600 load shall be positioned laterally within a 3.2 m standard design lane as shown in Figure 7.2.4.

The moving traffic load shall consist of a uniformly distributed load together with the axle group loads as shown in Figure 7.2.4. The uniformly distributed component of the M1600 moving traffic load shall be continuous under the axle group loads and shall be considered as uniformly distributed over the width of a 3.2 m standard design lane.

The uniformly distributed component of the M1600 moving traffic load shall be continuous or discontinuous and of any length as may be necessary to produce the most adverse effects. Likewise, the axle group loads position and variable spacing shall be determined such as to produce the most adverse effects.

Where a single tri-axle group from the M1600 moving traffic load (including the UDL component) is considered, the dynamic load allowance (α) shall be as given in Table 7.7.2. The UDL component shall be continuous or discontinuous and of any length as may be necessary to produce the most adverse effects.

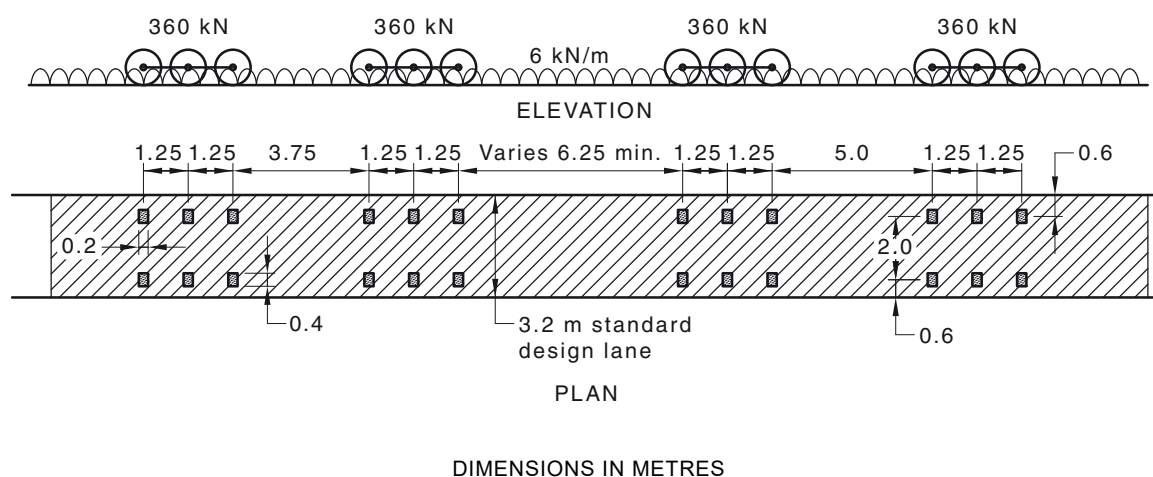


FIGURE 7.2.4 M1600 MOVING TRAFFIC LOAD

7.2.5 S1600 stationary traffic load

The S1600 stationary traffic load models the loads applied by a stationary queue of traffic.

The S1600 stationary traffic load shall consist of a uniformly distributed load together with axle group loads as shown in Figure 7.2.5. The uniformly distributed component of the S1600 stationary traffic load shall be continuous under the axle group loads and shall be considered as uniformly distributed over the width of a 3.2 m standard design lane. The S1600 stationary load shall be positioned laterally within a 3.2 m standard design lane as shown in Figure 7.2.5.

The uniformly distributed component of the S1600 stationary traffic load shall be continuous or discontinuous and of any length as may be necessary to produce the most adverse effects. Likewise, the axle group load position and variable spacing shall be determined such as to produce the most adverse effects.

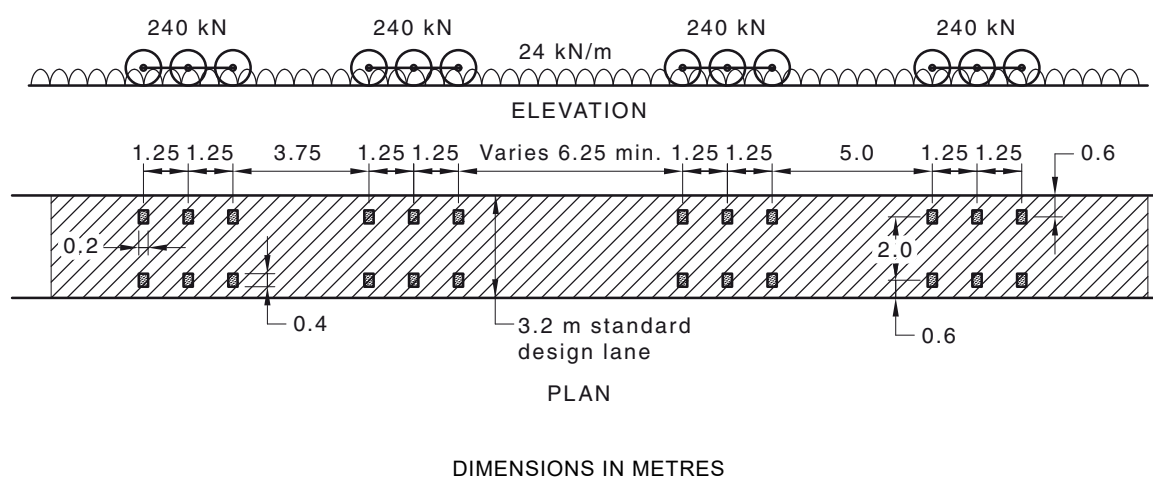


FIGURE 7.2.5 S1600 STATIONARY TRAFFIC LOAD

7.3 Heavy load platform

The heavy load platform design load HLP 320 or HLP 400, or an alternative platform design load, may be specified by the relevant authority. Details of HLP 320 and HLP 400 are given in Figure 7.3.

The HLP 320 and HLP 400 heavy load platform shall be positioned within two standard design lanes. The heavy load platform shall be positioned up to 1.0 m laterally from the centre of two standard design lanes. The two standard design lanes shall be positioned laterally to create the worst effect unless otherwise specified by the relevant authority.

For bridges with three or more standard design lanes, the unobstructed standard design lanes shall be loaded with half of either M1600 moving traffic load or the S1600 stationary traffic load, to create the worst effect, unless the relevant authority specifies otherwise. Accompanying lane factor for S1600 and M1600 is not applicable to this Clause.

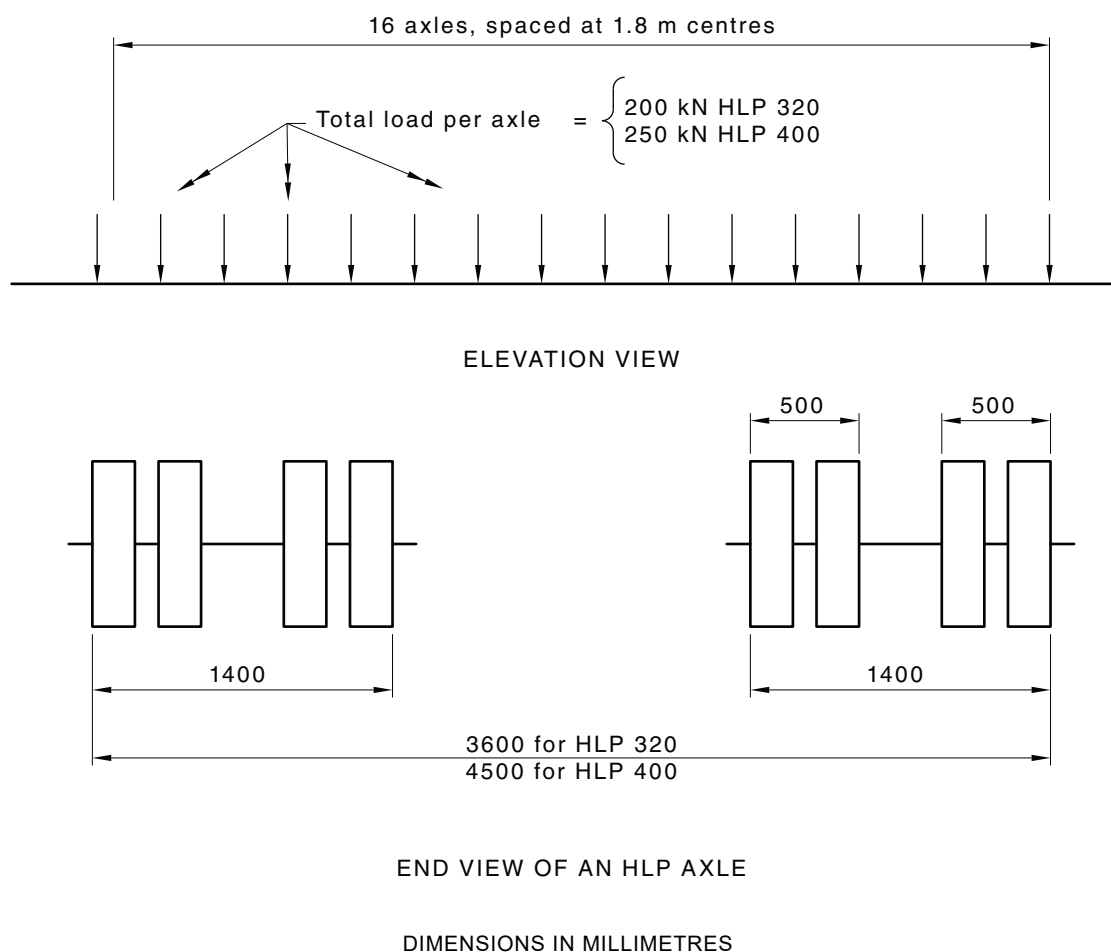


FIGURE 7.3 LATERAL SPACING OF DUAL WHEELS ALONG AN AXLE FOR HLP LOADS

7.4 Rail traffic

Where road bridges are to carry rail traffic, the operating authority for the utility shall be consulted to determine the appropriate design loads and load factors.

7.5 Standard design lanes

The A160, M1600 and S1600 loadings shall be assumed to occupy one standard design lane of 3.2 m width. The number of standard design lanes shall be as follows:

$$n = \frac{b}{3.2} \text{ (rounded down to next integer)} \quad \dots 7.5$$

where

n = number of standard design lanes

b = width between external barriers ignoring internal kerbs, median barriers and medians

The full width between external barriers on the bridge shall be designed for road traffic to allow for future changes to lane markings, unless otherwise specified by the relevant authority.

These standard design lanes shall be positioned laterally on the bridge to produce the most adverse effects.

7.6 Accompanying lane factors

If more than one lane is loaded, the A160, M1600, S1600, M1600 tri-axle load or light rail loading applied to the additional lanes shall be multiplied by the accompanying lane factors given in Table 7.6.

The number of standard design lanes loaded and the load patterning (standard design lane numbering) shall be selected to produce the most adverse effects.

For bridges that support vehicle and pedestrian, cyclist path or maintenance traffic, the accompanying lane factors shall be applied to both the vehicle and pedestrian, cyclist path or maintenance traffic. The pedestrian, cyclist path or maintenance load shall be considered as one standard design lane.

For bridges supporting both road and light rail traffic, each light rail track shall be considered as one standard design lane.

TABLE 7.6
ACCOMPANYING LANE FACTORS

Standard design lane number (n)	Accompanying lane factor (ALF_i)
1 lane loaded	1.0
2 lanes loaded	1.0 for first lane; and 0.8 for second lane
3 or more lanes loaded	1.0 for first lane; 0.8 for second lane; and 0.4 for third and subsequent lanes

NOTES:

- 1 First lane—the loaded lane giving the largest effect.
- 2 Second lane—the loaded lane giving the second largest effect.
- 3 Third lane—the loaded lane giving the third largest effect.

7.7 Dynamic load allowance

7.7.1 General

The dynamic load allowance (α) set out in this Clause specifies an increase in the traffic load resulting from the interaction of moving vehicles and the bridge structure. It shall be described in terms of the static equivalent of the dynamic and vibratory effects. For design purposes, α shall be specified as a proportion of the traffic load and shall be applied as specified in Clause 7.7.2. The dynamic load allowance applies to both the ULS and SLS.

The dynamic load allowance models the dynamic effects of vehicles moving over bridges with typical road profile irregularities.

7.7.2 Magnitude

The design action is equal to $(1 + \alpha) \times$ the load factor $\times$ the action under consideration.

The value of α for the appropriate loading shall be as given in Table 7.7.2.

For deck joints, the values for α specified in AS 5100.4 shall be used.

TABLE 7.7.2
DYNAMIC LOAD ALLOWANCE (α)

Loading	Dynamic load allowance (α)
W80 wheel load	0.4
A160 axle load	0.4
M1600 tri-axle group (see Note 1)	0.35
M1600 (see Note 1)	0.30
S1600 (see Note 1)	0.0
Heavy load platform (see Note 2)	0.1
Centrifugal force, braking force or pedestrian load	0.0

NOTES:

- 1 Including the UDL component of the traffic load.
- 2 A heavy load platform travels at a maximum speed of 10 km/h. A higher dynamic load allowance (α) may apply where this speed is exceeded.

7.7.3 Application

The dynamic load allowance (α) shall be applied to all parts of the structure extending down to the ground level.

For parts of the structure below the ground level, the dynamic load allowance to be applied to each part shall be—

- (a) the ground level value for a cover depth of zero;
- (b) zero for a cover depth of 2 m or more; or
- (c) a linear interpolation between depths of zero and 2 m.

Consideration shall be made of the depth of possible scour when determining ground level.

For buried structures such as pipes, culverts and soil-steel structures, the dynamic load allowance to be applied to the entire structure shall be—

- (i) the ground level value for a cover depth of zero;
- (ii) 0.1 for a cover depth of 2 m or more for loads excluding S1600. For S1600 loads, the dynamic load allowance is zero; or
- (iii) a linear interpolation between depths of zero and 2 m.

7.7.4 Dynamic load reversal

Consideration shall be given to the reversal of the dynamic response to traffic load. (Vibrations may continue and slowly decay after passing of traffic.) In particular, the minimum reaction on bearings shall take into consideration any reduction that may occur as a result of dynamic effects.

7.8 Horizontal forces

7.8.1 Centrifugal forces

For bridges on horizontal curves, allowance shall be made for the centrifugal effects of traffic load on all parts of the structure. The bridge shall be designed to resist the most adverse co-existing effects induced by the M1600 moving traffic load and the centrifugal force (F_c), in kilonewtons.

The centrifugal force (F_c) shall be assumed to act at deck level and shall be applied in accordance with the distribution of load for the M1600 moving traffic load. The centrifugal force (F_c) shall be calculated as follows:

$$F_c = \frac{V^2}{rg} W_c \quad \dots 7.8.1(1)$$

$$\leq (0.35 + \theta) W_c \quad \dots 7.8.1(2)$$

where

V = design traffic speed, in metres per second

r = radius of curve, in metres

g = acceleration due to gravity (9.81 m/s²)

W_c = load due to multiple standard design lanes of the M1600 moving traffic load for the length under consideration, in kilonewtons

No dynamic load allowance shall be considered.

Accompanying lane factors shall be applied, i.e.—

$$= \sum_{i=1}^n ALF_i \times M1600_i \quad \dots 7.8.1(3)$$

where

n = number of standard design lanes

ALF_i = accompanying lane factor (see Table 7.6)

θ = super elevation of the road, expressed as a ratio (e.g. 4% superelevation is expressed as 0.04 and adverse super elevation is negative)

7.8.2 Braking forces

Braking effects of traffic shall be considered as a longitudinal force. Braking forces shall be applied in either direction. The restraint system shall be designed to resist the most adverse co-existing effects induced by the braking force and the M1600 traffic load. The braking force shall be applied in accordance with the distribution of the M1600 traffic load.

The braking force shall be assumed to act at the road surface.

The number and lateral position of the standard design lanes on the bridge shall be selected to produce the most adverse effects.

The most adverse effects from the following scenarios shall be considered:

- (a) *Single standard design lane* The braking force (F_{BS}) applied in a single standard design lane shall be calculated as follows:

$$F_{BS} = 0.45 W_{BS} \quad \dots 7.8.2(1)$$

$$200 \text{ kN} < F_{BS} < 720 \text{ kN}$$

where

W_{BS} = load due to a single lane of the M1600 moving traffic load for the length under consideration, in kilonewtons, up to a maximum of 1600 kN

No dynamic load allowance shall be included.

F_{BS} shall be applied to any lane of a multi-lane bridge to produce the most adverse effects.

- (b) *Multiple standard design lanes* The total braking force (F_{BM}) applied in multiple standard design lanes shall be calculated as follows:

$$F_{BM} = 0.15W_{BM} \quad \dots 7.8.2(2)$$

$$200 \text{ kN} < F_{BM} < 1200 \text{ kN}$$

where

W_{BM} = load due to multiple standard design lanes of the M1600 moving traffic load for the length under consideration, in kilonewtons, up to a maximum of 8000 kN

No dynamic load allowance shall be included.

Accompanying lane factors shall be applied, that is—

$$= \sum_{i=1}^n ALF_i \times M1600_i \quad \dots 7.8.2(3)$$

where

n = number of standard design lanes

ALF_i = accompanying lane factor for i^{th} lane where 'i' equals 2 or more (see Table 7.6)

The number of standard design lanes to be included shall be limited to those likely to carry traffic in a single direction, unless otherwise specified by the relevant authority.

When assessing the effects of longitudinal forces on bridge bearings and substructures, the friction or shear displacement characteristics of expansion bearings and the stiffness of the substructure shall be taken into account.

7.9 Fatigue load effects

The fatigue design traffic load effects shall be determined from 70% of the effects of a single A160 axle or 70% of a single M1600 moving traffic load, without UDL, whichever is more severe. In both cases, a load factor of 1.0 shall be used and the load effects shall be increased by the dynamic load allowance (α).

The single A160 axle load or M1600 moving traffic load, without UDL, shall be placed within any design traffic lane to maximize the fatigue effects for the component under consideration.

Unless otherwise specified by the relevant authority, the number of fatigue stress cycles to be used for the calculation of the fatigue capacity of the structural element under consideration shall be as follows:

- For the fatigue design load of $0.70 \times (\text{A160 axle load}) \times (1 + \alpha)$:
(current number of heavy vehicles per lane per day) $\times 4 \times 10^4 \times (\text{route factor})$.
- For the fatigue design load of $0.70 \times (\text{M1600 moving traffic load without UDL}) \times (1 + \alpha)$:
(current number of heavy vehicles per lane per day) $\times 2 \times 10^4 (L^{-0.5}) \times (\text{route factor})$.

where L is the effective span, in metres, and is defined as follows:

- For positive bending moments, L is the actual span in which the bending moment is being considered.
- For negative moment over interior supports, L is the average of the adjacent spans.
- For end shear, L is the actual span.
- For reactions, L is the sum of the adjacent spans.
- For cross-girders, L is twice the longitudinal spacing of the cross-girders.

Unless otherwise specified by the relevant authority, the route factor shall be—

- (A) for principal interstate freeways and highways.....1.0;
- (B) for urban freeways.....0.7;
- (C) for other rural routes.....0.5; and
- (D) for urban roads other than freeways 0.3.

On interstate and other rural routes where there are two or more lanes in one direction, the number of heavy vehicles per lane per day shall be the total of the heavy vehicles travelling in that direction. On urban routes where there are two or more lanes in one direction, the number of heavy vehicles per lane per day shall be 65% of the total number of heavy vehicles in that direction.

The fatigue design traffic load effects and relevant stress cycles shall be applied to each design lane independently.

A fatigue stress cycle shall be taken to be the maximum peak to peak stress from the passage of the relevant fatigue design load.

Heavy vehicles shall be as defined by the Austroads *Guide to Traffic Management*, Part 3: *Traffic Studies and Analysis*, Table A8, i.e., Classes 3 to 12.

The current number of heavy vehicles shall be based on the year the bridge is to be put into service.

This Clause does not apply to fatigue design of roadway deck joints.

7.10 Load factors

For ultimate, serviceability and fatigue limit state design loads, the load factors for design road traffic loads shall be as given in Table 7.10(A).

The load factor to be applied in calculating the design centrifugal and braking forces, excluding HLP loads, shall be as given in Table 7.10(B). The HLP load factors shall be specified by the relevant authority.

Each of the design horizontal forces due to road traffic load shall be applied separately but in combination with the coexisting M1600 traffic load and such load cases or any combination thereof shall be considered as a single vehicular traffic load specified in Clause 23.1.4.

TABLE 7.10(A)
LOAD FACTORS FOR DESIGN ROAD TRAFFIC LOADS (γ_Q)

Loading	Limit state		
	Ultimate	Serviceability	Fatigue
W80 wheel	1.8	1.0	0.0
A160 axle	1.8	1.0	1.0 (see Note 1)
M1600	1.8	1.0	1.0 (see Notes 1 and 2)
S1600	1.8	1.0	0.0
Heavy load platform (HLP)	1.5	1.0	0.0
Half of SM1600 traffic load in unobstructed lanes when applied in conjunction with HLP loading	1.8	1.0	0.0

NOTES:

- 1 70% of traffic load.
- 2 Excludes uniformly distributed load.

TABLE 7.10(B)
LOAD FACTORS FOR DESIGN
CENTRIFUGAL AND BRAKING FORCES (γ_Q)

Force	Limit state	
	Ultimate	Serviceability
Centrifugal force	1.8	1.0
Braking force	1.8	1.0

7.11 Deflection of superstructure

The deflection limits of a road bridge under traffic for the SLS shall be appropriate to the structure and its intended use, the nature of the loading and the elements supported by it.

Notwithstanding this requirement, the deflection for the SLS under traffic load plus dynamic load allowance shall be not greater than 1/600 of the span or 1/300 of the cantilever projection, as applicable.

The traffic load to be used for calculating deflection shall be one M1600 moving traffic load, without UDL, including dynamic load allowance, placed longitudinally in each design lane to produce the maximum deflection, taking into account the accompanying lane factors.

NOTE: In calculating the deflection, the following assumptions may be made:

- (a) The deflection of the bridge may be averaged across all beams.
- (b) The design cross-section of the bridge may include continuous portions of permanent road furniture contributing to stiffness, provided adequate connection is included to ensure composite action with the bridge deck.

In addition, road traffic bridges shall be designed so that—

- (a) deflections do not infringe on clearance diagrams;
- (b) hog deflection does not exceed 1/300 of the span; and
- (c) no sag deflection occurs under permanent loads.

7.12 Distribution of road traffic loads through fill

For all types of roadway pavements and fill above buried structures, the distribution of SM1600 design loads, with the factors and allowances applied in accordance with this Standard, shall be as specified below, unless calculated otherwise by an analytical modelling procedure approved by the relevant authority. This requirement shall apply to all types of roadway pavements.

SM1600 design wheel loads shall be distributed through the fill cover over the structure, from the imprint of the rectangular wheel contact area at the road surface to a rectangular distribution area on the surface of the structure, proportioned in accordance with the wheel contact area dimensions.

The length of the sides of the distribution rectangle shall be determined as follows:

- (a) For depths of fill cover from 0 to 200 mm—sides of distribution rectangle = sides of wheel contact rectangle + $0.5 h$, where h is the depth of fill cover in millimetres.
- (b) For depths of fill cover greater than 200 mm—sides of distribution rectangle = sides of wheel contact rectangles + 100 mm + $1.2 \times (h - 200)$.

Where distribution areas from several wheel loads overlap, the total load may be considered to be evenly distributed on the surface over the total area of distribution.

The uniformly distributed component of the SM1600 design load shall be applied with no longitudinal distribution. Transverse distribution shall be as for wheel loads.

The total width of transverse distribution shall not exceed the total width of the structure supporting the fill.

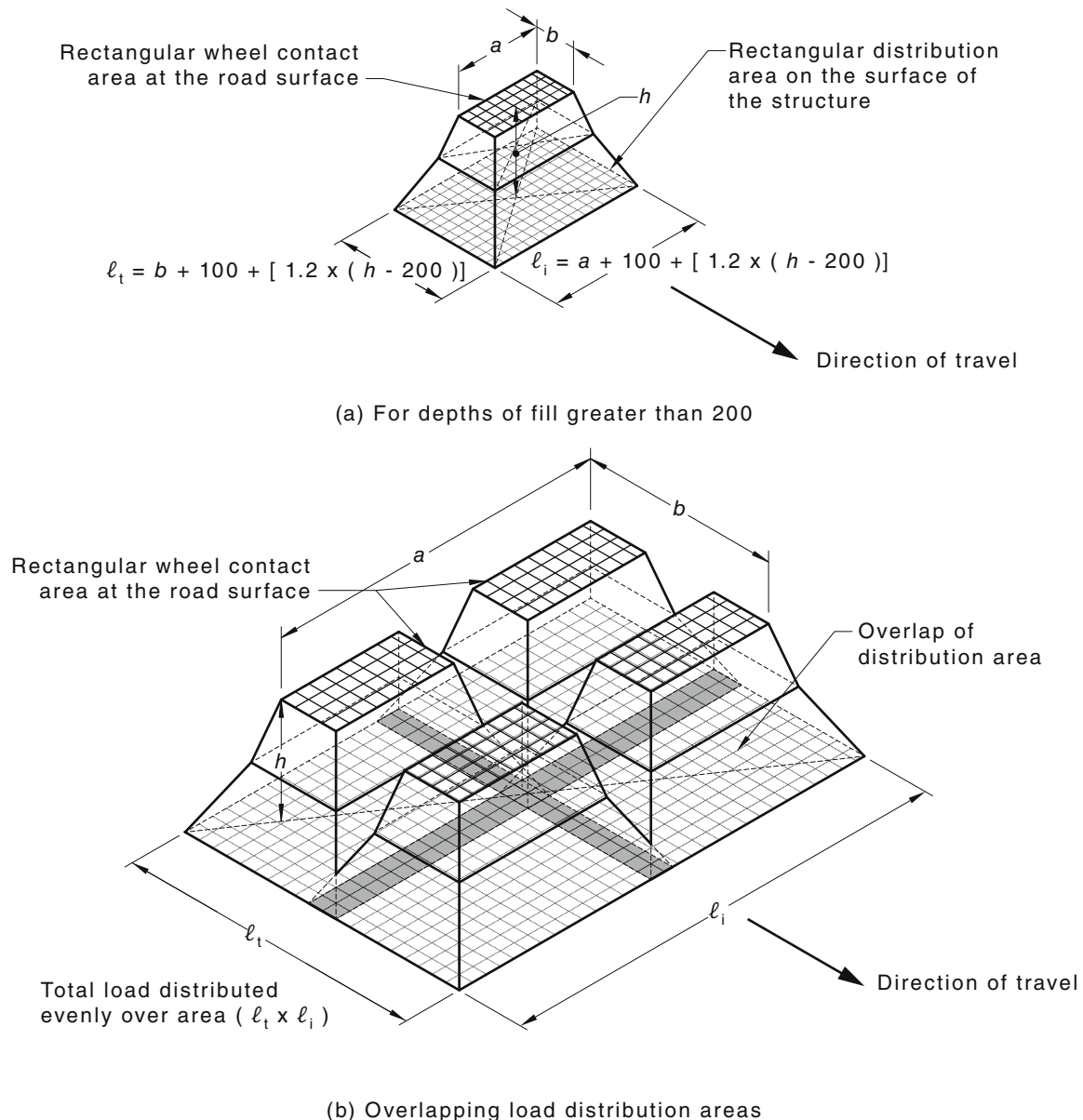


FIGURE 7.12 WHEEL LOADS THROUGH FILL

8 PEDESTRIAN, CYCLIST PATH AND MAINTENANCE TRAFFIC (Q)

8.1 Pedestrian and cyclist path loads

Pedestrian, cyclist path or combined (that is, shared path) bridges, ramps and stairs, independent or part of a road and/or rail traffic bridges that provide public access shall be designed for the following, unless otherwise specified by the relevant authority:

- The load per square metre of loaded area shall be as shown in Figure 8.1. The loaded area shall be the area related to the structural element under consideration.
- Where the relevant authority specifies that the bridge be designed for crowd loading, such as for special events, a minimum design load of 5 kPa shall be used.

- (c) Where it is possible for a light vehicle not exceeding 4.5 t to access the pedestrian or cyclist path bridge, the bridge shall be designed to carry a concentrated load of at least 20 kN on an area of 200 mm × 200 mm, with no dynamic load allowance.
- (d) Stairs treads and landings shall be designed for the more severe of the following live loads:
 - (i) Uniformly distributed load of 5.0 kPa.
or
 - (ii) Point load of 4.5 kN.
or
 - (iii) Line load of 2.2 kN/m.
- (e) Where it is required for maintenance, inspection or emergency vehicles to access the bridge, the loading shall be specified by the relevant authority.

NOTES:

- 1 For barrier loading, see Clause 12.5.
- 2 The dynamic load allowance (α) is 0.0 (see Table 7.7.2).

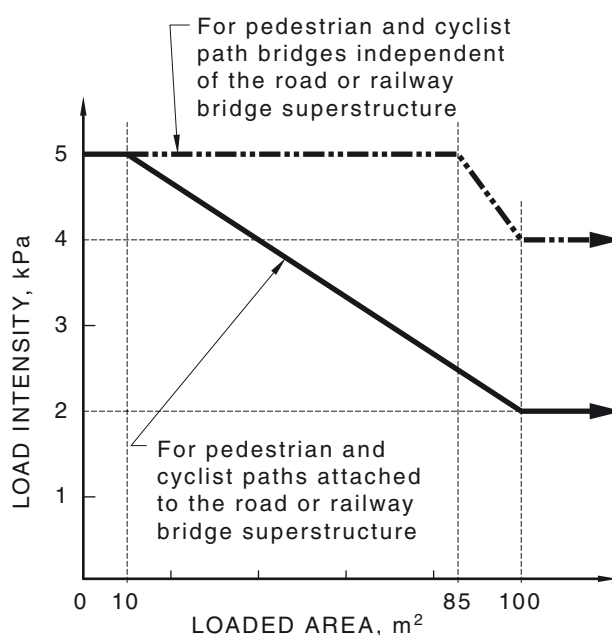


FIGURE 8.1 PEDESTRIAN AND CYCLIST PATH LOADS

8.2 Maintenance load on service walkways not intended for public use

For rail bridges fitted with a service walkway or service platform, the minimum load shall be 5 kPa, unless otherwise specified by the relevant authority.

For all other structures fitted with a service walkway or service platform, loads shall be determined in accordance with AS 1657, unless otherwise specified by the relevant authority.

8.3 Load factors

For ULS and SLS design loads, the load factors for design pedestrian, cyclist path and maintenance loads shall be as given in Table 8.3.

TABLE 8.3
LOAD FACTORS FOR DESIGN PEDESTRIAN, CYCLIST PATH
AND MAINTENANCE TRAFFIC LOADS (γ_Q)

Loading	Limit state	
	Ultimate	Serviceability
Pedestrian, cyclist path and maintenance load	1.5	1.0

8.4 Deflection

For pedestrian and cyclist path bridges, the deflection under the SLS traffic load shall be not greater than 1/600 of the span or 1/300 of the cantilever projection, as applicable.

The traffic load to be used for calculating deflection shall be in accordance with Clause 8.1.

In addition, pedestrian and cyclist path bridges shall be designed so that—

- (a) deflections under all SLS load cases do not infringe on clearance diagrams;
- (b) deflections under permanent effects do not permit ponding of water on the bridge;
- (c) where a hog deflection under permanent effects applies, it does not exceed 1/300 of the span; and
- (d) no sag deflection occurs under permanent loads.

9 RAIL TRAFFIC (Q)

9.1 General

Rail bridges shall be designed for the loads specified in this Clause (9), unless otherwise specified by the rail authority. Bridges carrying cane rail traffic and/or other special applications shall be designed for loads specified by the relevant authority.

9.2 300LA design rail traffic load

The 300LA load consists of a number of simulated axle groups with four axles, each having a load of 300 kN, and axle spacings of 1.7 m, 1.1 m and 1.7 m. To simulate coupled locomotives, a 360 kN axle load shall be added 2 m in front of the first simulated axle group, as shown in Figure 9.2(A). The spacing between the centres of each simulated axle group shall vary between 12 m and 20 m to give maximum effect in the member under consideration, as shown in Figure 9.2(B).

NOTE: Appendix C tabulates values of bending moment and shear force due to 300LA loading.

The position of the loads and the number of axle groups shall be selected to give maximum load effects in the member under consideration.

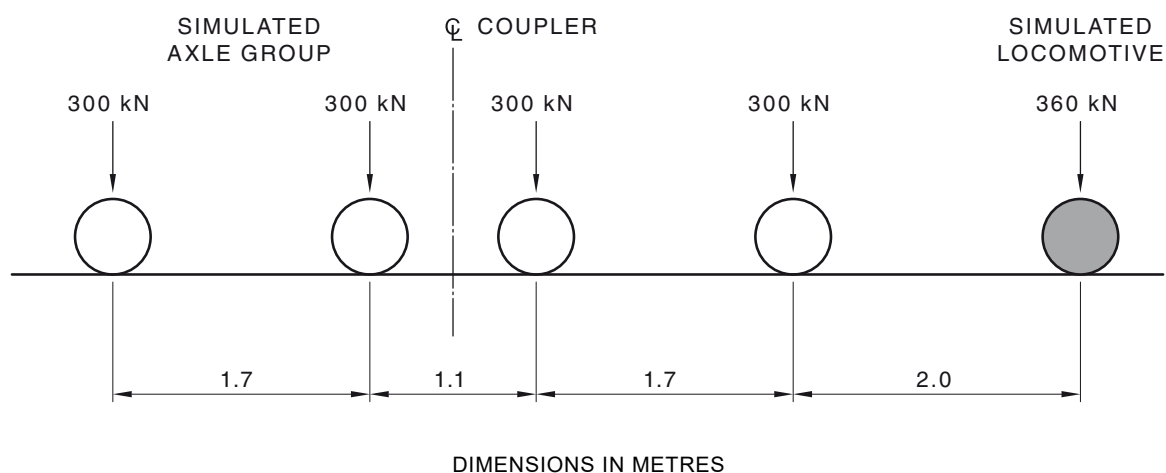


FIGURE 9.2(A) 300LA RAIL TRAFFIC LOADS—AXLE LOADS

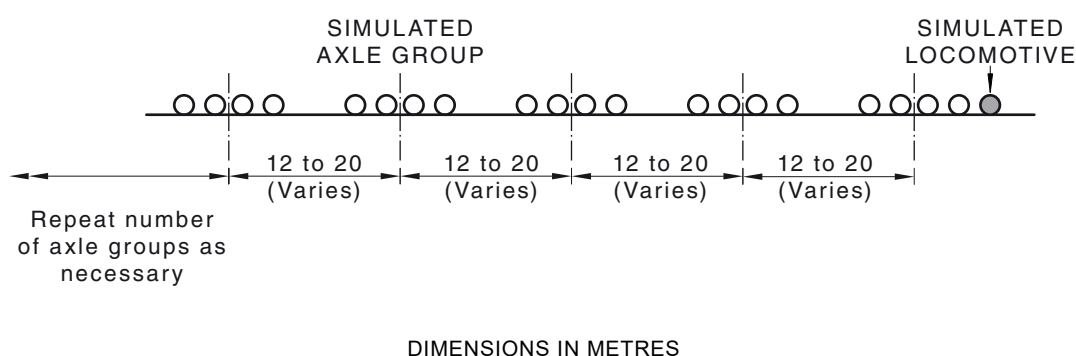


FIGURE 9.2(B) 300LA RAIL TRAFFIC LOADS—AXLE GROUP SPACINGS

9.3 Light rail traffic design load

The light rail traffic design load shall be based on 150LA, derived by multiplying the 300LA rail traffic loads specified in Clause 9.2 by a factor of 0.5, with nine (9) axles.

The nine axles shall comprise the simulated locomotive axle plus two groups of four trailing axles.

For bridges where queuing of light rail vehicles is possible, additional trailing axle groups may be included, as specified by the relevant authority

9.4 Multiple track factor for rail bridges

When loading a number of tracks simultaneously, the multiple track factors given in Table 9.4 shall be used, as appropriate.

The number of tracks loaded and the load patterning shall be selected to produce the most adverse effects in the member under consideration.

TABLE 9.4
MULTIPLE TRACK FACTORS

Number of tracks loaded	Multiple track factor
1	1.00
2	1.00 for both tracks
3	1.00 for two tracks and 0.85 for the third track
4	1.00 for two tracks, 0.85 for the third track and 0.70 for the fourth track
5 or more	1.00 for two tracks, 0.85 for the third track, 0.70 for the fourth track and 0.60 for the remaining tracks

9.5 Dynamic load allowance

9.5.1 General

The dynamic load allowance (α) for rail traffic load effects shall be a proportion of the static rail traffic load, and shall be calculated by the methods specified in this Clause. It shall have the same value for all structural material types. The value of α shall depend on the characteristic length (L_α).

The dynamic load allowance shall apply to both the ULS and SLS.

The design action is equal to $(1 + \alpha) \times$ the load factor $\times$ the action under consideration.

In cases where a member acts in two different modes, e.g. as a deck support and also as part of the main girder, the dynamic load allowance shall be calculated separately for the structural actions in each mode, and the actions summed.

9.5.2 Characteristic length (L_α)

For main girders and components of rail bridge superstructures, the characteristic length (L_α) for each component shall be dependent on the structural geometry. The values of L_α for superstructure elements shall be as given in Table 9.5.2.

For abutments, L_α shall be the length of the supported span.

For piers, L_α shall be the sum of the lengths of the loaded adjacent spans.

For bearings, L_α shall be per Table 9.5.2 for the member support.

TABLE 9.5.2
CHARACTERISTIC LENGTH (L_{α})

Case No.	Bridge members, types of bridge	Characteristic length (L_{α}) m
Floor members		
1	Stringers	Cross-girder spacing +3.0
2	End stringers	Cross-girder spacing
3	Cantilevered stringers	0.5
4	Cross-girders, including cantilevered cross-girders, loaded by simply supported stringers and continuous deck elements	Twice the cross-girder spacing +3.0
5	End cross-girders, including cantilevered end cross-girders	4.0
6	Deck slabs between supports	Span of the main girders or twice the span of the deck slab, whichever is less
7	Cantilevered deck slabs	Span of the main girders or twice the distance between each support, whichever is less
8	Suspension bars or supports loaded by cross-girders only	The values to be used shall correspond to those applying to the cross-girder, as given in Cases 4 and 5
Main girders		
9	Simply supported main girders	Span of main girders
10	Continuous main girders over n spans where— $L_m = 1/n (L_1 + L_2 + \dots + L_n)$	for— $n = 2 \quad 3 \quad 4 \quad \geq 5$ $x = 1.2 \quad 1.3 \quad 1.4 \quad 1.5$ $L_{\alpha} = xL_m, \text{ but } \leq L_{\max.}$
11	Cantilever portions of cantilever bridges	Length of the cantilevered portion plus the span of any suspended girder supported by the cantilever
12	Suspended girders of suspended span bridges	Span of the suspended girder
13	Arches	Half span
14	Buried structures (e.g. pipes, culverts, buried arches)	Half span
15	Plate web girders at bottom of welded stiffeners	0.5
16	Truss members: (a) Top and bottom chords (b) Verticals (c) Diagonals not intersected by members complying with this Standard (d) Diagonals intersected by members complying with this Standard	Three times the length from adjacent panel points Three times the length between chords Three times the horizontal or vertical projection, whichever is the shorter Six times the horizontal or vertical projection of the overall length, whichever is the shorter
17	Lattice girder members: (a) Top and bottom flanges and webs (b) Lattice members	As for main girders Six times the horizontal or vertical projection of the overall length from web to web, whichever is the shorter

(continued)

TABLE 9.5.2 (continued)

Case No.	Bridge members, types of bridge	Characteristic length (L_α) m
18	Bracing members:	
	(a) Horizontal or vertical members parallel to or perpendicular to the track	Three times the member length
	(b) Diagonal members with respect to Item (a), if not intersected by members complying with this Standard	Three times the projected length horizontally or vertically, parallel to or perpendicular to the track, whichever is the shorter
	(c) Diagonal members, with respect to Item (a), if intersected by members complying with this Standard	Six times the projected overall length horizontally or vertically, parallel to or perpendicular to the track, whichever is the shorter

LEGEND:

- n = number of continuous main girder spans
 L_1, L_2, L_n = span lengths of a continuous structure, in metres
 x = coefficient (Case 10)
 L_m = average span length (Case 10)
 $L_{max.}$ = largest of the spans L_1, L_2, L_n , in metres
 L_α = characteristic length

9.5.3 Dynamic load allowance for bending effects

The value of the dynamic load allowance (α) for bending moment for ballasted deck spans, open deck spans or spans with direct rail fixation spans shall be calculated as follows:

$$\alpha = \left(0.2 \leq \left\{ \frac{2.16}{L_\alpha^{0.5} - 0.20} - 0.27 \right\} \leq 0.67 \right) \quad \dots 9.5.3$$

where

L_α = characteristic length in metres

The value of α for steam locomotives shall be increased by 20%.

Transoms for open deck bridges and local effects for direct fixed tracks shall be designed with a dynamic load allowance of 1.0.

Where a transition approach to a bridge abutment is not provided, α shall be increased by not less than 50% of the calculated dynamic load allowance unless otherwise approved by the relevant authority.

9.5.4 Application

For all parts of the structure extending down to the ground level, the dynamic load allowance (α) shall be as specified in Clause 9.5.3.

For all parts of the structure below ground, culverts and soil steel structures below the ground level, α shall be linearly transitioned from the ground level value to zero (0) at a cover depth of 2 m. For structures in embankments, the ground level shall be taken as the underside of the ballast.

The dynamic load allowance established for the appropriate cover depth shall apply to the entire structure. The depth of the cover shall be measured from the underside of the ballast.

9.5.5 Dynamic load allowance for other load effects

The dynamic load allowance (α) for shear, torsion and reactions shall be taken as 0.67 of the value for bending moment.

Where the application of the dynamic load allowance leads to greater safety or stability, e.g. against overturning, α shall be taken as zero (0).

Where deflections are to be calculated for serviceability loads, including dynamic load allowance, 0.67 of the dynamic load allowance (α) shall be used.

Where a transition approach to a bridge abutment is not provided, α shall be increased by not less than 50% of the calculated dynamic load allowance unless otherwise approved by the relevant authority.

9.5.6 *Dynamic load reversal*

Consideration shall be given to the reversal of the dynamic response to live load. (Vibrations may continue and slowly decay after passing of traffic.) In particular, the minimum reaction on bearings shall allow for the reduction, which may occur from the results of the dynamic effects.

9.6 Distribution of rail traffic load

9.6.1 *General*

The distribution of rail traffic load to the supporting members shall be calculated using a rigorous analysis in accordance with the appropriate clauses of the relevant material Part of the AS(AS/NZS) 5100 series.

In the absence of a rigorous analysis, rail traffic loads shall be deemed to be distributed as set out in Clauses 9.6.2 to 9.6.5, as appropriate.

9.6.2 *Open deck steel rail bridges*

Timber bridge transoms shall be designed on the assumption that the maximum wheel load on each rail shall be distributed equally to all transoms or fractions thereof within a length of 1.2 m, but shall be not greater than three transoms.

For the design of beams, the rail traffic load shall be distributed and shall be applied via the transoms as above. In such cases, additional longitudinal distribution of such loads shall not be assumed.

9.6.3 *Ballasted deck steel rail bridges*

Provided that sleepers are spaced at no more than 700 mm centres, and not less than 150 mm of ballast is provided under them, the load from each axle may be uniformly distributed longitudinally over a length of 1.1 m, and uniformly distributed laterally over a width equal to the length of the sleeper plus the minimum distance from the bottom of sleeper to the top of the beams. This width shall be not greater than 4.0 m, the distance between track centres of multiple track bridges, or the width of the deck between ballast retainers.

9.6.4 *Ballasted deck concrete rail bridges*

Rail traffic loads on ballasted deck concrete bridges shall be uniformly distributed longitudinally over a length of 1.1 m, plus the depth of ballast under the sleeper, plus twice the effective depth of slab. The total length shall be not greater than the axle spacing.

The loads shall be uniformly distributed laterally over a width equal to the length of the sleepers plus the depth of ballast below the bottom of the sleepers, plus twice the effective depth of the concrete slab, unless limited by the extent of the structure. This width shall not be greater than the distance between centres of adjacent tracks on multiple track bridges.

9.6.5 *Direct fixation*

The distribution of rail wheel loads through directly fixed track shall be determined on the basis of the relative stiffness of the rail, the rail fixing supports and the superstructure.

9.7 Horizontal forces

9.7.1 Centrifugal forces

For rail bridges on horizontal curves, allowance shall be made for the centrifugal effects of rail traffic load by applying a centrifugal force (F_c) corresponding to each axle load horizontally through a point 2 m above the top of the rail.

The horizontal centrifugal force resulting from rail traffic loads shall be proportional to the design rail traffic load, and for each axle, F_c (in kilonewtons) shall be calculated as follows:

$$F_c = \frac{V^2 A}{rg} \quad \dots 9.7.1$$

where

V = design speed, in metres per second

A = axle load, in kilonewtons

r = radius of curve, in metres

g = acceleration due to gravity (9.81 m/s²)

The specified centrifugal force shall not be increased by the dynamic load allowance.

Centrifugal and nosing forces due to rail traffic load shall not be applied simultaneously.

9.7.2 Braking and traction forces

9.7.2.1 General

Rail bridges of all deck types shall be designed for loads arising from longitudinal braking and traction forces in combination with their coexisting rail traffic loads. These forces shall be applied to the top of rail and be determined using one of the following methods:

- (a) The empirical method.
- (b) The rational method.

The empirical method requires that occupied tracks be loaded for the full length of the bridge. The rational method permits partial loading of tracks.

Where account is to be taken of tracks that are subject to variable and/or partial rail vehicle loadings, the rational method shall be used as approved by the relevant rail authorities.

Braking and traction forces for a 300LA rail vehicle shall be taken to act at heights of 2.0 m and 0.9 m respectively above top of rail. For specific rail vehicles, these heights shall be derived from data supplied by their manufacturers if required.

Traction forces shall be assumed to be applied uniformly through the axles of driver cars or locomotives, as applicable, whilst braking forces shall be assumed to be applied uniformly through all axles, unless otherwise directed by the relevant rail authorities.

Dynamic load allowances shall not be applied to braking and tractions forces derived from either of the above methods.

For bridges that carry both rail traffic and road traffic, the relevant rail and road authorities shall be consulted.

9.7.2.2 Empirical method

For a 300LA rail vehicle, the longitudinal forces per track applied to a bridge shall be calculated as follows:

- (a) Braking forces:

$$BF = 200 + 15L_{LF} \quad \dots 9.7.2.2(1)$$

where

BF = longitudinal braking force, in kilonewtons

L_{LF} = total length of the bridge, in metres

(b) Traction forces:

TF = $200 + 25L_{LF}$ for $L_{LF} \leq 25$ m . . . 9.7.2.2(2)

$825 + 15(L_{LF} - 25)$ for $25 \text{ m} < L_{LF} \leq 50$ m . . . 9.7.2.2(3)

$1200 + 7.5(L_{LF} - 50)$ for $50 \text{ m} < L_{LF} \leq 250$ m . . . 9.7.2.2(4)

$2700 + 5.0(L_{LF} - 250)$ for $250 \text{ m} < L_{LF}$. . . 9.7.2.2(5)

where

TF = longitudinal traction force, in kilonewtons

L_{LF} = total length of the bridge, in metres

For specific rail traffic loads, the above braking and traction forces shall be multiplied by a factor equal to the equivalent line load intensity (in kilonewtons per metre) of the specific rail traffic load divided by the 300LA line load intensity of 100 kN/m. For a 150LA light rail vehicle, this factor shall be taken as 0.5.

For multiple track bridges, the above longitudinal rail forces shall be multiplied by the appropriate multiple coexistence factors from Table 9.7.2.2 when used in combination to produce the most adverse effect.

TABLE 9.7.2.2

BRAKING AND TRACTION FORCES MULTIPLE COEXISTENCE FACTORS

Number of rail vehicles	Multiple coexistence factor
1	1.00
2	1.00 for both rail vehicles
3	1.00 for two rail vehicles and 0.5 for the third rail vehicle
4 or more	1.00 for two rail vehicles, 0.5 for the third rail vehicle and 0.25 for the remaining rail vehicles

9.7.2.3 Rational method

The relevant rail authorities shall stipulate the bridge-specific design parameters to be used in applying the rational method, which shall include, but not be limited to, the following data:

- (a) Rail vehicle specifications, including the following where applicable:
 - (i) Critical rail vehicle dimensions.
 - (ii) Rail vehicle tare and passenger loads.
 - (iii) Rail vehicle traction characteristics.
 - (iv) Rail vehicle braking characteristics.
- (b) Minimum clear longitudinal distances between consecutive rail vehicles on the same track.
- (c) Guidelines for multiple rail vehicle combinations.

For the purposes of this Clause, a rail vehicle is defined as a single train comprising—

- (i) driver cars and trailer cars for an electric passenger train; or

- (ii) locomotives and carriages for a diesel passenger train; or
- (iii) locomotives and wagons for a diesel freight train; or
- (iv) special rail vehicle consist, such as a heritage train.

A bridge shall be analysed using the above parameters so as to produce the most adverse effect. Where this requires a bridge to be loaded simultaneously with a number of rail vehicles, the forces produced by each of these rail vehicles shall be multiplied by their respective multiple coexistence factors derived from Table 9.7.2 when combined with the effects from the other rail vehicles that are also present on the bridge.

9.7.2.4 *Distribution of forces*

The distribution of the longitudinal forces derived in Clauses 9.7.2.2 and 9.7.2.3 through a bridge structure shall be computed using a valid rail-structure interaction model approved by the relevant rail authorities. This model shall take account of all relevant factors, such as the following, where applicable:

- (a) Bridge-specific rail vehicle design parameters.
- (b) Track and bridge alignment.
- (c) Continuity/jointing of rails, including rail load transfer devices.
- (d) Stiffnesses of rails, turnouts, crossovers and the like.
- (e) Stiffnesses of ballast and transoms.
- (f) Longitudinal deformation characteristics of rail fixings.
- (g) Longitudinal shear stiffnesses of bridge bearings.
- (h) Longitudinal articulation of bridge superstructures.
- (i) Longitudinal stiffnesses of bridge piers and abutments, including their foundations.
- (j) Differential temperatures between rail and bridge,
- (k) Shrinkage and creep of concrete superstructures.
- (l) For the empirical method, modelling of the trackform for the extent of the bridge deck only. For the rational method, modelling of the trackform and the appropriate rail vehicle loadings for 100 m minimum beyond each bridge abutment.

9.7.3 *Nosing loads*

Rail bridges that are intended to carry 300LA rail traffic loads shall be designed to resist a lateral nosing load of 100 kN applied at top of rail level in either direction and at any point along the structure.

For light rail, the nosing load shall be the value for 300LA loading multiplied by a factor of 0.5. For rail loads other than 300LA, this load shall be adjusted by the ratio of the maximum axle load in the specified design traffic load to the 300 kN load. Nosing loads—

- (a) shall not be increased by the dynamic load allowance; and
- (b) shall not be reduced at low speeds.

NOTE: Nosing loads are independent from the speed.

Centrifugal and nosing forces due to rail traffic load shall not be applied simultaneously.

9.7.4 *Loads on ballast kerbs*

For ballasted rail bridges with kerbs required to retain the ballast and the effects of transverse horizontal loads from rail traffic, the loads shall include ballast pressure and the effects of centrifugal force and nosing load.

The horizontal load from an individual axle shall be distributed longitudinally along the face of the kerb over a length of 1 m plus twice the distance from facing end of sleeper to the inner face of the kerb. The load shall be applied at the top of sleeper level.

9.8 Fatigue load

9.8.1 Fatigue design traffic load

The fatigue design traffic load for rail bridges shall be the design rail traffic load and half of the design dynamic load allowance. The distance between the centres of the axle groups (L_v) shall be varied between 12 m and 20 m to produce the maximum fatigue design stress range (f^*) (see Clause 9.8.2).

9.8.2 Fatigue design stress range (f^*)

The fatigue design stress range (f^*) in any element of a bridge structure, shall be derived from the passage of the fatigue design traffic load over the bridge. It shall be the algebraic difference between the maximum and minimum stresses caused by that load.

NOTE: Stresses and stress ranges caused by other load effects need not be included.

9.8.3 Effective number of stress cycles (n)

The effective number of stress cycles (n) of the fatigue design stress range (f^*) to be considered in the design of the structure shall be calculated as follows:

$$n = C_T n_T \quad \dots 9.8.3$$

where

C_T = base number of load cycles for the track category, as given in Table 9.8.4

n_T = number of equivalent stress cycles of amplitude f^* per train, which depends on L_f and L_v , as given in Table 9.8.3

L_f = span of main girders, trusses or stringers; *or*
cross-girder spacing for cross-girders

L_v = distance between the centres of axle groups (i.e. the length of the vehicle)

TABLE 9.8.3
VALUES OF n_T

L_f	n_T
< 2.5	240
$2.5 < L_f < 9.0$	60.0
$9.0 < L_f < 25.0$	$60 \left(\frac{(2L_v - L_f)}{L_f} \right)^3 + 2$ Max. 60 Min. 2
> 25.0	2.0

9.8.4 Track category for fatigue load

The base number of load cycles (C_T) for fatigue load depends on the track category. It shall be not less than the values given in Table 9.8.4 unless otherwise approved by the relevant authority.