3 Design Loading

Table of Contents

3.1 Introduction

All structures shall be designed for the following loads, which shall be considered to act in various combinations, as set out in 3.5, except for lightly trafficked rural bridges - refer to Appendix D.

3.2 Traffic Loads - Gravity Effects

3.2.1 General

Traffic loading shall be HN-HO-72. A detailed description of this loading and its application is given below. The loads described shall be used for design of all members from deck slabs to main members and foundations.

3.2.2 Loads

(a) HN (Normal) Loading

An element of normal loading represents a single stream of legal traffic, and is the load applied to a 3m wide strip of deck, running the entire length of the structure. It is shown diagrammatically in Figure 3.1. The element consists of two parts.

The first is a uniform load of 3.5 kN/m^2, 3m wide, which may be continuous or discontinuous over the length of the bridge, as necessary to produce the worst effect on the member under consideration.

In addition to the uniform load, a pair of axle loads of 120 kN each, spaced at 5m, shall be placed to give the worst effect on the member being designed. Only one pair of axle loads shall exist in each load element, regardless of the length of bridge or number of spans. For design of deck slabs, the wheel contact areas shown shall be used, but for design of other members, such detail is unnecessary and point or line loads may be assumed.

(b) HO (Overload) Loading

An element of overweight loading is also shown diagrammatically in Figure 3.1. It consists of, firstly, the same uniform load as described above. In addition, there is a pair of axle loads of 240 kN each, spaced at 5m. In this case, there are two alternative wheel contact areas, and the one that has the most adverse effect on the member being considered shall be used.

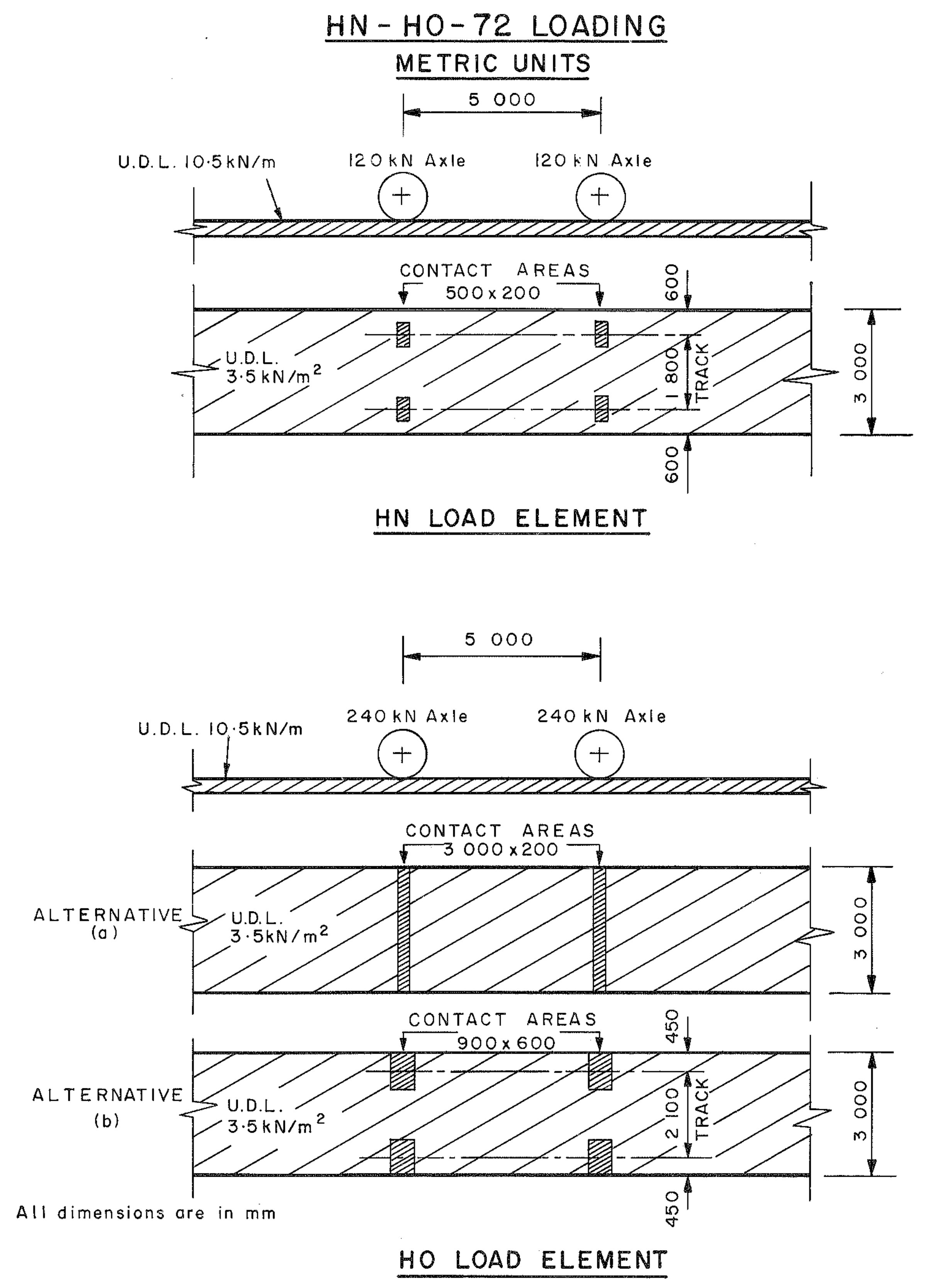


Figure 3.1 HN-HO-72 Traffic Loading

3.2.3 Transverse Load Position

(a) The above load elements shall be applied to an area defined as the roadway. The roadway includes carriageway and shoulders. If the bridge carries a cycle track adjacent to and on the same level as the carriageway and shoulders, the cycle track shall also be included in the roadway for design purposes, whether or not it is in the first instance separated by a guardrail. A raised median shall not be included in the roadway. The roadway is bounded by either the face of a kerb or the face of a guardrail or other barrier.

(b) The roadway shall be divided into a number of load lanes of equal width as follows:-

Width of Roadway	Number of Load Lanes
Less than 6.0m	1
6.0m but less than 9.7m	2
9.7m but less than 13.4m	3
13.4m but less than 17.1m	4
17.1m but less than 20.8m	5

Note: Load lanes as defined above are not to be confused with traffic lanes as physically marked on the road surface.

(c) For design of main members, the load elements shall be applied within each load lane as defined above, but may have as much eccentricity within the lane as their width of 3m allows. Even if the number of traffic lanes as finally marked on the bridge will be different from that obtained from the table above, the number tabulated shall be used for design purposes.

(d) For design of deck slabs and median slabs and their immediate supporting members, load elements are not restricted by the lanes as above, but shall be placed anywhere within the roadway and on the median, at such spacing as will give the worst effect, but not less than 3m centres transversely.

In order to represent a vehicle which has penetrated the guardrail or handrail and mounted the kerb, if any, the slab shall also be checked under an HN wheel load factored by the dynamic load factor. The wheel shall be positioned with its outer edge at the outer edge of the slab or kerb. This may be treated as a Group 4 (overload) combination.

3.2.4 Combination of Traffic Loads

Two combinations of traffic loads shall be used for design purposes.

(a) Normal Live Load

In this combination, as many elements of HN loading shall be placed on the bridge as will give the worst effect on the member being considered, complying with the rules for positioning set out in 3.2.3.

(b) Overload

In this combination, any one element of HN loading in the live load combination shall be replaced by an element of HO loading, chosen so as to give the most adverse effect on the member being considered.

To allow for the improbability of concurrent loading, where appropriate, the total loading may be multiplied by a factor varying according to the number of elements in the load case, thus:

Number of Elements	Reduction Factor
1	1.0
2	0.9
3	0.8
4	0.7
5	0.6
6 or more	0.55

This reduction factor shall be applied to the overload as well as the normal live load.

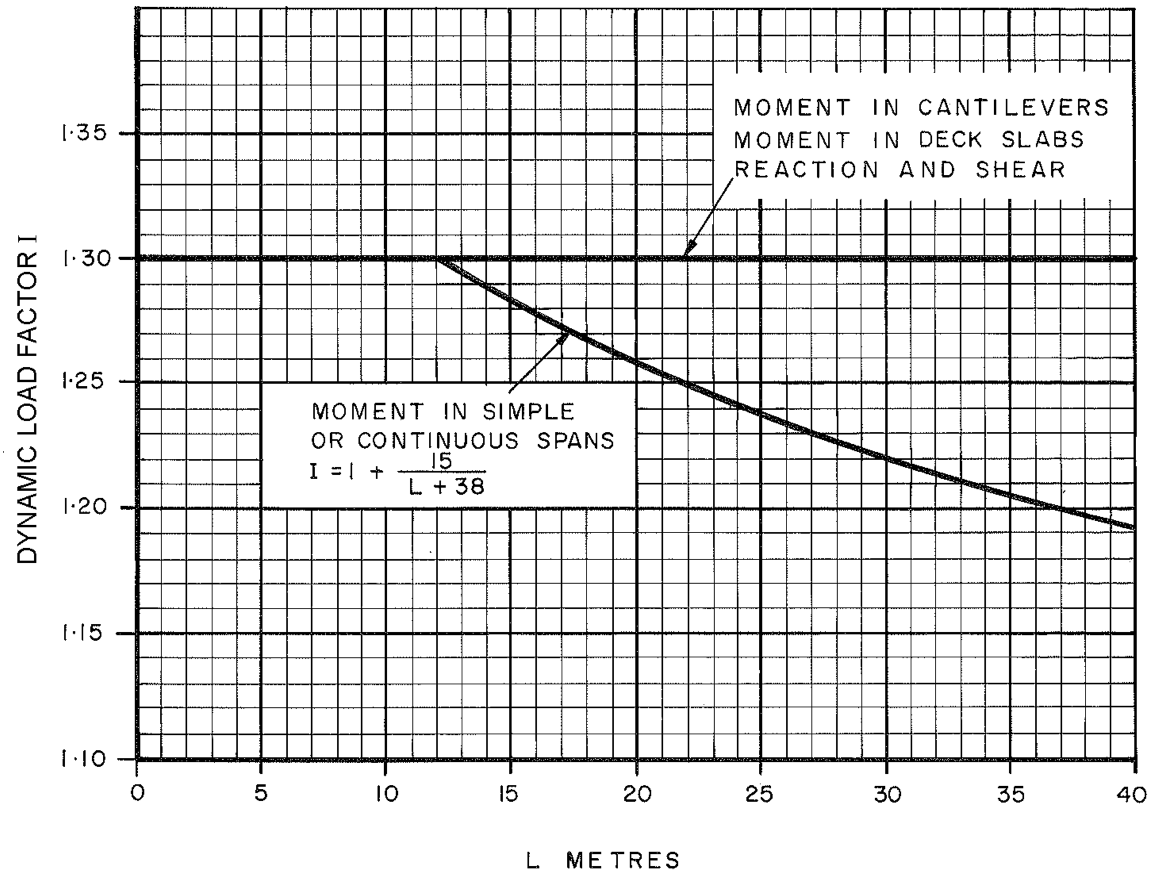
The number of design lanes that are loaded shall be selected to maximise the load affect on the structural member under consideration.

3.2.5 Dynamic Load Factor

Normal live load and overload shall be multiplied by the dynamic load factor applicable to the material and location in the structure of the member being designed.

The dynamic load factor for use in the design of all components which are above ground level shall be taken from Figure 3.2.

The dynamic load factor for use in the design of components which are below ground level shall be 1.0, to allow for the fact that vibration is damped out by the soil, except that for top slabs of culvert type structures, the dynamic load factor shall be reduced linearly with depth of fill, from 1.30 for zero fill to 1.00 for 1m of fill.



L is the span length for positive moment, and the
average of adjacent span lengths for negative moment.

Figure 3.2 : Dynamic Load Factor for Components Above Ground Level and for Bearings

3.2.6 Fatigue

The loading used in the fatigue assessment shall at least represent the expected service loading over the design life of the structure, including dynamic effects. This should be simulated by a set of nominal loading events described by the distribution of the loads, their magnitudes, and the number of applications of each nominal loading event.

A standard fatigue load spectrum for New Zealand traffic conditions is not available. The loading in BS 5400: Part 10: 1980[(1)] clause 7.2.2 may be used, but is likely to predict fatigue lives shorter than those which would be achieved in practice.

In a case where fatigue details significantly influence the design, an appropriate loading spectrum shall be developed, taking account of current and likely future traffic.

3.3 Traffic Loads - Horizontal Effects

3.3.1 Braking and Traction

For local effects, a horizontal longitudinal force, equal to 70% of an HN axle load, shall be applied across the width of any load lane, at any position on the deck surface, to represent a skidding axle.

For effects on the bridge as a whole, a horizontal longitudinal force shall be applied at deck surface level in each section of superstructure between expansion joints. The magnitude of the force shall be the greater of two skidding axle loads as above, or 10% of the live load which is applied to the section of superstructure, in each lane containing traffic headed in the same direction. In some cases, e.g., on the approach to an intersection, or for a bridge on a grade, it may be appropriate to allow for a greater force.

Consequent displacement of the structure shall be allowed for.

3.3.2 Centrifugal Force

A structure on a curve shall be designed for a horizontal radial force equal to the following proportion of the live load. The reduction factors of 3.2.4 shall be applied, but the dynamic load factor of 3.2.5 shall not be applied.

$$C = 0.008S^2 / R$$

Where : C = centrifugal force as a proportion of live load

S = design speed, km/h

R = radius, m.

The force shall be applied 2m above the road surface level, but the consequent variation in wheel loads need not be considered in deck design. Consequent displacement of the structure shall be allowed for.

3.4 Loads Other Than Traffic

3.4.1 Dead Load

This shall consist of the weight of the structural members, and any other permanent load added or removed before the structural system becomes complete. When calculating the weight of concrete members, care shall be taken to use a density appropriate to the aggregates available in the area, plus an allowance for embedded steel.

3.4.2 Superimposed Dead Load

This shall consist of all permanent loads added after the structural system becomes complete. It shall include handrails, guardrails, lamp standards, kerbs, services and road surfacing. Surfacing shall be allowed for at 1.5 kN/m^2, whether the intention is to surface the bridge immediately or not.

An allowance shall be made for future services in addition to the weight of actual services installed at the time of construction. A minimum allowance of 0.25 kN/m^2 shall be applied as a uniformly distributed load over the full width and length of the bridge deck.

3.4.3 Earthquake

The design shall allow for the effects of earthquakes, by considering:

- The possibility of earthquake motions in any horizontal direction
- The potential effects of vertical earthquake motions
- The available structure ductility.

The magnitude of the force and the required structure ductility shall be obtained from Section 5.

3.4.4 Shortening

The effects of shrinkage and creep of concrete, and shortening due to prestressing shall be taken into account. Transmission of horizontal forces from superstructure to substructure by bearing restraint shall be allowed for.

The section rigidity assumed for a reinforced concrete pier which resists the resulting forces shall be that of the cracked section. The effects of creep in the pier in reducing the forces may be taken into account.

In composite structures, differential shrinkage between elements shall be allowed for.

The secondary effects of shrinkage, creep and prestressing shall be allowed for in continuous and statically indeterminate structures.

3.4.5 Wind

(a) Wind load shall be applied to a bridge in accordance with the principles set out in BS 5400, Part 2, Specification for Loads[(2)], Clause 5.3, contained within BD 37/01 Appendix A[(3)], giving consideration to wind acting on adverse and relieving areas as defined in Clause 3.2.5 of that standard. For footbridges with spans exceeding 30 m, for which aerodynamic effects may be critical, the principles forming the basis of BD 49/01, *Design Rules for Aerodynamic Effects on Bridges*[(4)] shall be applied.

(b) The design gust wind speeds acting on adverse areas of a bridge without live load being present, for the ultimate and serviceability limit states, shall be calculated in accordance with AS/NZS 1170, Part 2[(5)], Clause 2.2 to 2.3 for the annual probability of exceedance corresponding to the importance of the bridge as defined in 2.1.3.

The design gust wind speeds acting on relieving areas of a bridge without live load being present shall be derived from the following equation:

$V_r = V_d.S_c.T_c/(S_b.T_g)$

Where: V_r = design gust wind speed acting on relieving areas

V_d = design gust wind speed acting on adverse areas

S_c, T_c, S_b, and T_g are factors defined in, and derived from, BS 5400, Part 2[2], Clause 5.3.

The height of a bridge shall be measured from ground level or minimum water level to the deck level.

For the case where wind load is applied to a bridge structure and live load on the bridge, as defined in (a) above, the maximum site gust wind speed acting on adverse areas shall be the lesser of 37 m/s and V_d m/s as specified above, and the effective coexistent value of wind gust speed acting on parts affording relief shall be taken as the lesser of $37 \times S_c/S_b$. m/s and V_r m/s, as specified above.

(c) Wind forces shall be calculated using the method of BS 5400, Part 2, Specification for Loads[2], Sections 5.3.3 to 5.3.6, contained within BD 37/01 Appendix A[3].

3.4.6 Temperature Effects

(a) Overall Temperature Changes

Allowance shall be made for both forces and movements resulting from variations in the mean temperature of the structure, as below:

For steel structures	± 25°C
For concrete structures	± 20°C

The section rigidity assumed for a reinforced concrete pier that resists the forces shall be that of the cracked section.

(b) Differential Temperature Change

Allowance shall be made for stresses, both longitudinal and transverse, resulting from the temperature variation through the depth of the structure shown in Figure 3.3.

The criteria shall be used for all structural types and all materials except timber.

In the case of a truss bridge, the temperature variation shall be assumed to occur only through the deck and stringers, and any chord members attached to the deck, and not through web members or chord members remote from the deck.

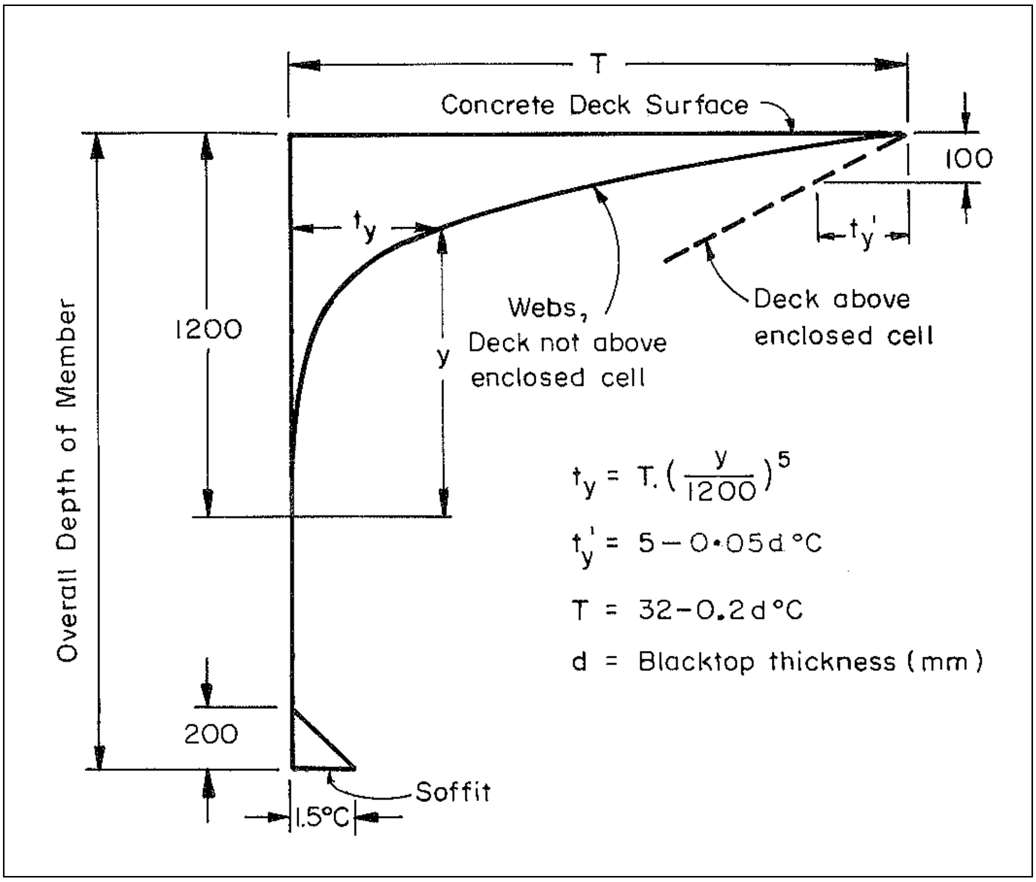


Figure 3.3: Temperature Variation With Depth

Note that:

(i) For structures shallower than 1400mm, the two parts of the solid curve are to be superimposed.

(ii) On a bridge that is to be surfaced, the temporary unsurfaced condition shall also be checked. For this condition, the value of T may be reduced to 27°C. Load Group 5B should be used, with the above reduced temperature load replacing the term "0.33TP".

For analysis of reinforced concrete members under differential temperature, the properties of the cracked section shall be used.

3.4.7 Construction Loads

Allowance shall be made for the weight of any falsework or plant that must be carried by the structure, because of the anticipated method of construction. This does not obviate the necessity of checking, during construction, the capacity of the structure for the contractor's actual equipment.

3.4.8 Water Pressure

(a) All piers subject to the force of flowing water shall be designed to resist a pressure acting on their face area normal to the flow, varying with depth below surface level, as below:

$P = KV^2$

Where: P = pressure in kPa on projected area

V = water velocity in m/s at the level being considered

K = pier shape factor x angle of attack factor, as below

For pressures acting on faces normal to the direction of flow, the pier shape factor shall be taken as:

0.7	For a square ended pier
0.35	For a circular pier
0.35	For a pier with semi-circular ends
0.4	For a pier with angled ends (i.e. wedge-nosed, sharper than 90°)
1.1	For a superstructure

Slab or wall type piers angled to the direction of the flow and partially or fully submerged superstructures inclined by superelevation shall also be designed for pressure perpendicular to their plane due to the water flow acting on their area parallel to the water flow.

For pressures acting normal to the direction of flow, the angle of attack factor is dependent on the angle of attack of the pier or superstructure to the direction of the water flow and shall be taken as follows, with coefficients for intermediate angles being interpolated:

Angle of attack	Factor
0°	0
5°	0.25
10°	0.45
20°	0.45
30° or greater	0.5

The flood flows for which water pressure shall be considered shall be as specified in 2.3.2. The flow producing ordinary water pressure shall be taken as the flow with an average recurrence interval (ARI) of 1 year.

(b) Buoyancy shall be allowed for in assessing vertical reactions with which water pressure must be combined. Calculations of water pressure under flood conditions shall make allowance for scour, with a suitable margin on the scour depth.

(c) Where a significant amount of driftwood is carried, water pressure shall also be allowed for on a driftwood raft lodged against the pier. The size of the raft is a matter of judgement, but as a guide, dimension A in Figure 3.4 should be half the water depth, but not greater than 3m. Dimension B should be half the sum of adjacent span lengths but not greater than 15m. Pressure shall be calculated using the formula in (a) above, with $K = 0.5$.

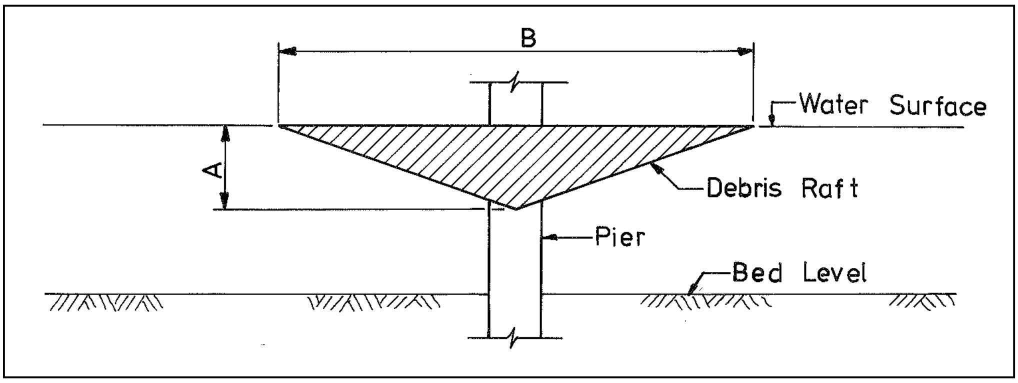


Figure 3.4 : Debris Raft for Pier Design

3.4.9 Groundwater on Buried Surfaces

Groundwater pressures shall be based on the groundwater levels and pressures measured from an appropriate programme of site investigations, with allowance for seasonal, long term and weather dependent fluctuations, and considering the reliability and robustness of any drainage measures incorporated in the design. Consideration shall also be given to flood situations and also incidents such as possible break in any water pipes or other drainage services.

The groundwater pressure shall correspond to not less than the ground water level with a 1/50 probability of exceedance. Conservatively the ground water level may be taken as being at the ground surface provided that artesian or sub-artesian pressures are not present.

3.4.10 Water Ponding

The load resulting from water ponding shall be calculated from the expected quantity of water that can collect when primary drainage does not function.

3.4.11 Snow

Snow loading need only be considered at the ultimate limit state for footbridges.

The design snow load shall be determined from AS/NZS 1170 Part 3 for the annual probability of exceedance corresponding to the importance of the bridge as defined in 2.1.3.

3.4.12 Earth Loads

(a) Earth loads shall include horizontal static earth pressure (active, at-rest and passive), horizontal earthquake earth pressure, vertical earth pressure, surcharge pressure and hydrostatic pressure from groundwater. It also includes negative skin friction (downdrag) loads on piles.

(b) Earth retaining members shall be designed for either static earth pressure plus live load surcharge where appropriate, or earthquake earth pressure in accordance with 5.6, whichever is more severe. Water pressure shall also be allowed for unless an adequate drainage system is provided. Live load effects may be assumed equal to those of a surcharge pressure caused by 0.6m of fill.

In calculating static earth pressures, consideration shall be given to the influence of wall stiffness, foundation and tie-back stiffness (where appropriate) and the type, compaction and drainage provisions of the backfill. Active, at-rest or passive earth pressure shall be used as appropriate.

In some structures, for example concrete slab frame bridges, an increase in static earth pressure reduces total moment in some positions in the structure. When calculating the total design moment at those positions, a maximum of half the benefit due to static earth pressure shall be used in the load combination. Loads on foundations due to downdrag (or negative friction) and to plastic soil deformation, shall be included.

(c) The effects of earthquake induced site instability, differential movements and liquefaction shall be considered.

3.4.13 Loads on Kerbs, Guardrails, Barriers and Handrails

Kerbs, guardrails, barriers and handrails shall be designed in accordance with Appendix B.

3.4.14 Loads on Footpaths and Cycle Tracks.

(a) A footpath or cycle track on the same level as the roadway (whether or not separated by a guardrail) shall be included as part of the roadway, and designed for the loads in 3.2.

(b) A footpath or cycle track raised above the roadway behind a kerb (whether or not separated by a guardrail) shall be designed for a uniformly distributed load as follows:

- when traffic loads are not considered in the same load case, 5.0 kPa;

- when traffic loads are considered in the same load case, between the limits of 1.5 and 4.0 kPa as given by the expression 5.0 - S/30, where S, the loaded length in metres, is that length of footpath or cycle track which results in the worst effect on the member being analysed.

The structure shall also be checked for an overload case consisting of the HN wheel loads positioned with wheel outer edges at the outer edge of the slab.

(c) A footpath or cycle track on a highway bridge, positioned out of reach of the traffic, e.g., underneath the roadway, shall be designed as in (b), but without the overload.

(d) A foot or cycle track bridge without traffic shall be designed for a uniformly distributed load between the limits of 2.0 and 5.0 kPa, as given by the expression 6.2 - S/25, where S is as defined in (b).

(e) In all cases where there is a likelihood of crowd loading, the maximum value of 5.0 kPa should be considered, regardless of the loaded length. Examples are access to a sports stadium, or where the bridge could become a vantage point to view a public event.

3.4.15 Vibration

All highway bridges shall be checked for the effects of vibration due to traffic loads. The criteria below shall be complied with for bridges carrying significant pedestrian or cycle traffic, and those where vehicles are likely to be stationary for a significant portion of the time (i.e. near intersections, with or without traffic signals). Other bridges should comply with the criteria where economically justifiable.

The maximum vertical velocity during a cycle of vibration due to the design load shall be limited to 0.055 m/s. The design load for this purpose shall be taken as the two 120 kN axles of one HN load element.

Pedestrian and cycle bridges shall conform to the requirements of BS 5400, Part 2[(2)], Appendix B, contained within BD 37/01 Appendix A[(3)].

3.4.16 Settlement, Subsidence and Ground Deformation

Horizontal and vertical forces and displacements induced on or within the structure as a result of settlement, subsidence or ground deformation in the vicinity of the structure or approach embankment shall be taken into account.

Where there is potential for subsidence of the ground, such as due to groundwater changes, mining, liquefaction etc, the effects of this on the structures, and the performance requirements for the road link shall be taken into consideration in the development and design of appropriate mitigation measures.

3.4.17 Forces Locked-In by the Erection Sequence

Forces that are locked-in to a structure due to the erection sequence shall be allowed for. These may arise due to the weight of formwork, falsework and construction equipment acting on structural elements as they are built-in.

The secondary effects of prestressing shall be included with the effects of shortening in 3.4.4.

3.4.18 Collision Loads

(a) General

Piers and abutments supporting road bridges over other roads, railways or navigable rivers shall be designed to resist accidental collision loads. Alternatively, a protective barrier system shall be provided.

Collision loads need only be considered at the ultimate limit state, except that their effect on elastomeric bearings shall be considered at the serviceability limit state.

(b) Collision Load from Road Traffic

Where the piers or abutments supporting an overbridge are not located behind rigid or flexible traffic barriers meeting performance level 4 or higher, as set out in Appendix B, and they are located within 5.0 m of the edge of the underlying road carriageway, they shall be designed to resist a nominal equivalent static load of 1000 kN applied at an angle of 10 degrees from the direction of the centreline of the road passing under the bridge. The load shall be applied 1.2 m above ground level. Flexible barriers shall be positioned to allow for the dynamic deflection of the barrier.

The requirements of the Transit New Zealand Geometric Design Manual for clear zones shall also be met. Where piers and abutments are located within clear zones, suitable protective barriers shall be provided.

A nominal collision load of 50kN (equivalent static load) shall be considered to act as a single point load on the bridge superstructure at any location along the bridge and in any direction between the horizontal and vertically upwards. The load shall be applied at the level of the soffit of the outside girders, or at the level of the outer soffit corners of a box girder or slab superstructure.

Vehicle collision load on the supports and on the superstructure shall be considered to act non-concurrently.

Vehicle collision loads on bridge abutments need not be considered when abutments are protected from collision by earth embankments.

An exception to the above requirements will be considered where providing such protection would be impractical or the costs would be excessive, providing that the structure has sufficient redundancy to prevent collapse. Such cases require justification, and any variations to the requirements of this Manual are subject to the agreement of Transit New Zealand.

(c) Bridge Piers Adjacent to Railways

Where possible, rail crossings should be a clear span between abutments.

Where piers are necessary, and they are situated within 5.5 m of a rail track centreline, they shall be designed to resist the following minimum impact loads applied simultaneously (equivalent static loads):

2000 kN parallel to the rails

1000 kN normal to the rails

Both loads shall be applied horizontally, at 2 m above rail level.

In addition, any requirements of the appropriate railway authority shall be satisfied.

(d) Ship Impact on Bridge Piers

Possible impact loads from shipping shall be considered. Bridge piers shall either be protected by auxiliary structures designed to absorb the impact energy, or they shall be designed to resist impact from vessels operating under both normal conditions and extreme events that could occur during the life of the bridge. Design loads shall be assessed and included in the Design Statement.

3.5 Combination of Load Effects

The effects of the loads described in 3.2 to 3.4 shall be combined in groups as shown in Tables 3.1 and 3.2, and as specified below.

(a) In any group, if a worse effect is obtained by omitting one or more of the transient items, this case shall be considered.

(b) The required seismic resistance of structures during construction is difficult to specify in a general manner. Variables such as duration of construction stage, vulnerability of the structure and surroundings at each stage, and cost to temporarily improve the seismic resistance shall all be taken into account. The designer shall be satisfied that the load components of Group 5C give adequate protection in the circumstances being considered.

(c) The load groups specified cover general conditions. Provision shall also be made for other loads where these might be critical, e.g., vehicle or ship impact on piers.

(d) In the tables, the following abbreviations are used:

CF = Centrifugal effects of traffic loads
CN = Construction loads, including loads on an incomplete structure
CO = Collision loads
DL = Dead load, including superimposed dead load
EL = Locked-in forces due to the erection sequence

EP	=	Earth pressure
EQ	=	Earthquake effects
FP	=	Pedestrian and cycle track live load
FW	=	Flood water pressure and buoyancy, with scour
GW	=	Ground water
HE	=	Horizontal effects of traffic loads
I	=	Dynamic load factor
LL	=	Normal live load (gravity effects)
OL	=	Overload combination of traffic loads (gravity effects)
OW	=	Ordinary water pressure and buoyancy
PW	=	Water ponding
SG	=	Shortening effects
SN	=	Snow load
ST	=	Settlement
TP	=	Temperature effects, overall and differential
U	=	Design load for consideration of member strength
WD	=	Wind load

Table 3.1: Load Combinations for the Serviceability Limit State

Group	Loads
1A	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP
1B	DL + EL + GW + EP + OW + SG + ST + TP
2A	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP + HE + TP
2B	DL + EL + GW + EP + OW + SG + ST + CF + 1.35LLxI + FP + HE + WD
2C	DL + EL + GW + EP + FW + PW + SG + ST + CF + 1.35LLxI + FP + HE
3A	DL + EL + GW + EP + OW + SG + ST + EQ + 0.33TP
3B	DL + EL + GW + EP + FW + PW + SG + ST + WD
3C	DL + EL + GW + EP + OW + SG + ST + CO + 0.33 TP
4	DL + EL + GW + EP + OW + SG + ST + OLxI + 0.5FP + 0.33TP
5A	DL + EL + GW + EP + OW + SG + 0.33WD + CN
5B	DL + EL + GW + EP + OW + SG + 0.33TP + CN
5C	DL + EL + GW + EP + OW + SG + 0.33EQ + CN

Note: Where the effect of a possible reduction in permanent load is critical, replacement of the "permanent load" by "0.9 x permanent load" shall be considered.

For combinations 1A, 2A, 2B and 2C, the 1.35 factor applied to normal live load (LL) is to allow for the effects of closed-up stationary vehicles.

Combination 3C only applies to the design of elastomeric bearings.

Table 3.2: Load Combinations and Load Factors for the Ultimate Limit State

Group	Loads and Load Factors
1A	U = 1.35 (DL + EL + 1.35EP + OW + SG + ST + 1.67(CF + LLxI) + 1.30FP) + GW
1B	U = 1.35 (DL + EL + 1.35EP + OW + SG + ST + 1.25TP) + GW
2A	U = 1.20 (DL + EL + EP + OW + SG + ST + CF + LLxI + FP + HE + TP) + GW
2B	U = 1.35 (DL + EL + EP + OW + SG + ST + CF + LLxI + FP + HE) + GW + WD
2C	U = 1.35 (DL + EL + EP + SG + ST + CF + LLxI + FP + HE) + GW + FW + PW
3A*	U = 1.00 (kDL + EL + 1.35 (EP + OW) + SG + ST + EQ + 0.33TP) + GW
3B	U = 1.10 (DL + EL + 1.25 EP + SG + ST) + GW + FW + PW + WD
3C	U = 1.00 (DL + EL + 1.35 (EP + OW) + SG + ST + 2.00CO + 0.33TP) + GW
3D	U = 1.20 (DL + EL + EP + OW + SG + ST + TP) + GW + PW + SN + 0.33WD
4	U = 1.35 (DL + EL + EP + OW + SG + ST + 1.10(CF + OLxI) + 0.70FP + 0.33TP) + GW
5A	U = 1.35 (DL + EL + EP + OW + SG + 1.10CN) + GW + 0.33WD
5B	U = 1.35 (DL + EL + EP + OW + SG + 0.33TP + 1.10CN) + GW
5C	U = 1.35 (DL + EL + EP + OW + SG + 0.33EQ + 1.10CN) + GW

* k = 1.3 or 0.8 whichever is more severe, to allow for vertical acceleration. k=1.0 when considering the vertical earthquake response specified by clause 5.2.6

Note : Where the effect of a possible reduction in a permanent load is critical, replacement of "permanent load" by "permanent load/j" shall be considered (where j is the load factor outside the bracket).

Combination 3D applies only to the design of footbridges.

3.6 References

(1) BS 5400: *Steel, Concrete and Composite Bridges*, Part 10:1980, “Code of Practice for Fatigue”, British Standards Institute.
(2) BS 5400, *Steel Concrete and Composite Bridges*, Part 2:1978, “Specification for Loads”, British Standards Institute.
(3) BD 37/01, *Loads for Highway Bridges,* 2001, The Highways Agency, London.
(4) BD 49/01, *Design Rules for Aerodynamic Effects on Bridges*, 2001, The Highways Agency, London
(5) AS/NZS 1170.2:2002 *Structural Design Actions*, Part 2: Wind Actions, Standards Australia and Standards New Zealand jointly,