3.0 Design loading

In this section

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.

In 2004 the design traffic loading, HN-HO-72, was modified by the introduction of a 1.35 load factor applied to normal live load in the serviceability limit state (SLS) load combinations.

In 2013 Waka Kotahi NZ Transport Agency commissioned research report RR 539 *A new vehicle loading standard for road bridges in New Zealand*[1] was published. Further research has been ongoing since then to develop new vehicle live load models, load combinations and load factors. This amendment 4 to the *Bridge manual* 3rd edition introduces updated vehicle live loads for the evaluation of bridges and culverts (see 7.4.4) and updated load combinations mostly using existing load factors (see 3.5). Work is continuing to develop a new vehicle live load model for design along with corresponding finalised load factors.

Until this work is completed and any further revisions published, the provisions of the *Bridge manual* shall be followed.

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.

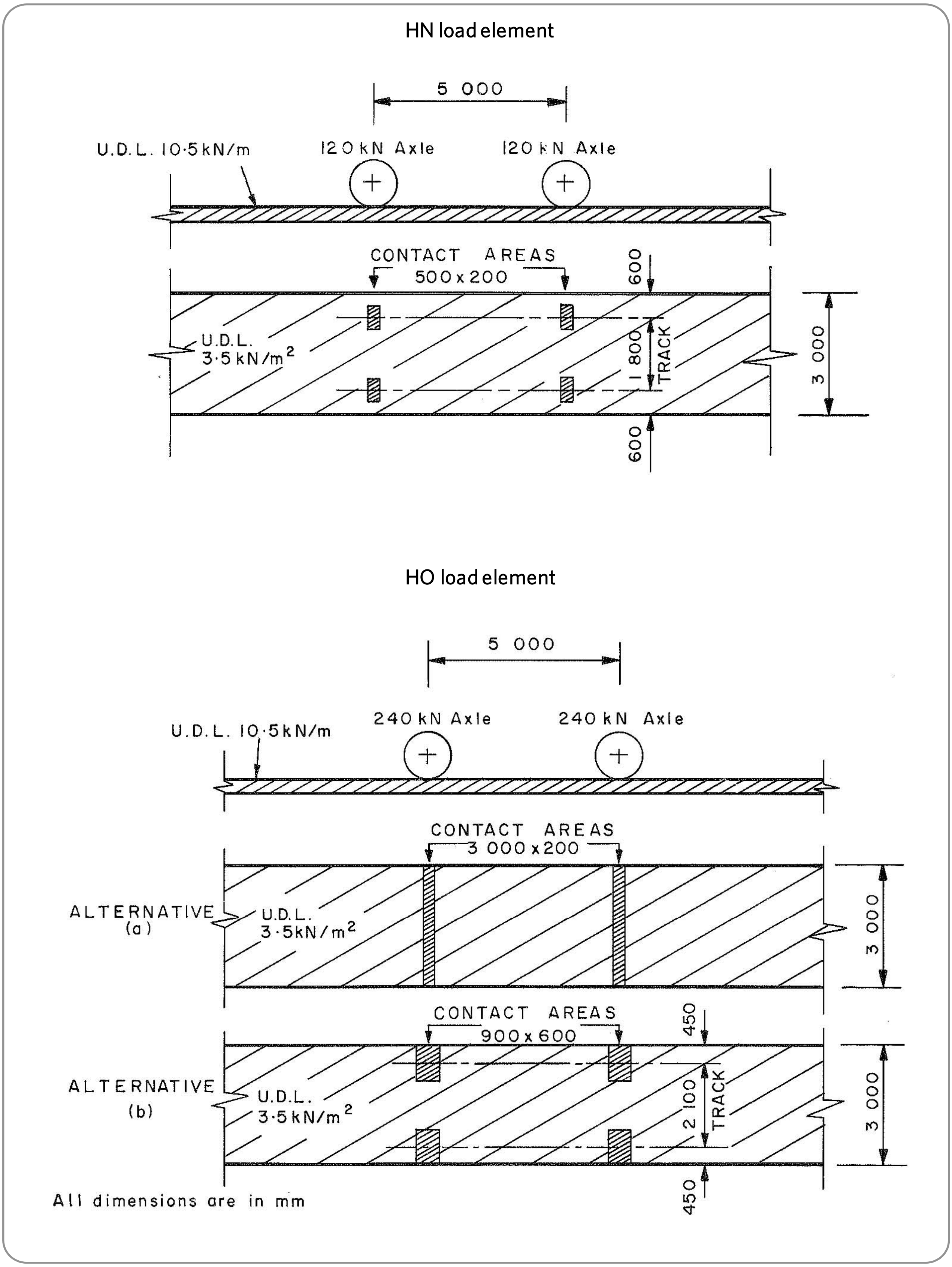
The first is a uniform load of 3.5kN/m², 3m wide, which may be continuous or discontinuous over the length of the bridge, as necessary to produce the worst effect on the member under consideration.

In addition to the uniform load, a pair of axle loads of 120kN 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 240kN 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 carriageway. The carriageway includes traffic lanes and shoulders. Raised or separated footpaths, cycle tracks or medians shall not be included in the carriageway unless the possibility of future reconfiguration of the carriageway is identified as a design requirement. On bridges the carriageway is bounded by either the face of a kerb or the face of a guardrail or other barrier.

b. The carriageway shall be divided into a number of load lanes of equal width as follows:

Width of carriageway	Number of load lanes
Less than 6m	1
6m 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.

For bridges with two carriageways separated by a median barrier, the number of load lanes considered in design shall be the sum of the load lanes for the two carriageways considered separately, unless future reconfiguration of the carriageway is identified as a design requirement.

c. For global effects, typically including the design of main members and elements transferring load between the 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 local effects, typically the design of deck slabs and their immediate supporting members (deck stringers and transoms etc), load elements are not restricted by the lanes as above but shall be placed anywhere within the carriageway, at such spacing as will give the worst effect, but not less than 3m centres transversely.

e. In order to represent a vehicle which has penetrated the guardrail, handrail or other barrier, or has mounted the kerb, if any, any slab and supporting members outside the carriageway shall be checked under HN wheel loads factored by the dynamic load factor. The wheels shall be positioned with their outer edge at the outer edge of the slab or kerb or anywhere inboard of that line. This may be treated as a load combination 4A (overload).

3.2.4 Combination of traffic loads

Two combinations of traffic loads shall be used for ultimate and serviceability limit state 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.

3.2.4 continued

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

Number of load elements (lanes loaded)	Reduction factor
1	1.0
2	0.9
3	0.8
4	0.7
5	0.6
6 or more	0.55

For overloads, the reduction factor for the HO load element shall be taken as 1.0. For additional load elements (lanes loaded), the reduction factors shall be as specified above (ie a reduction factor of 1.0 for the first additional load element reducing thereafter).

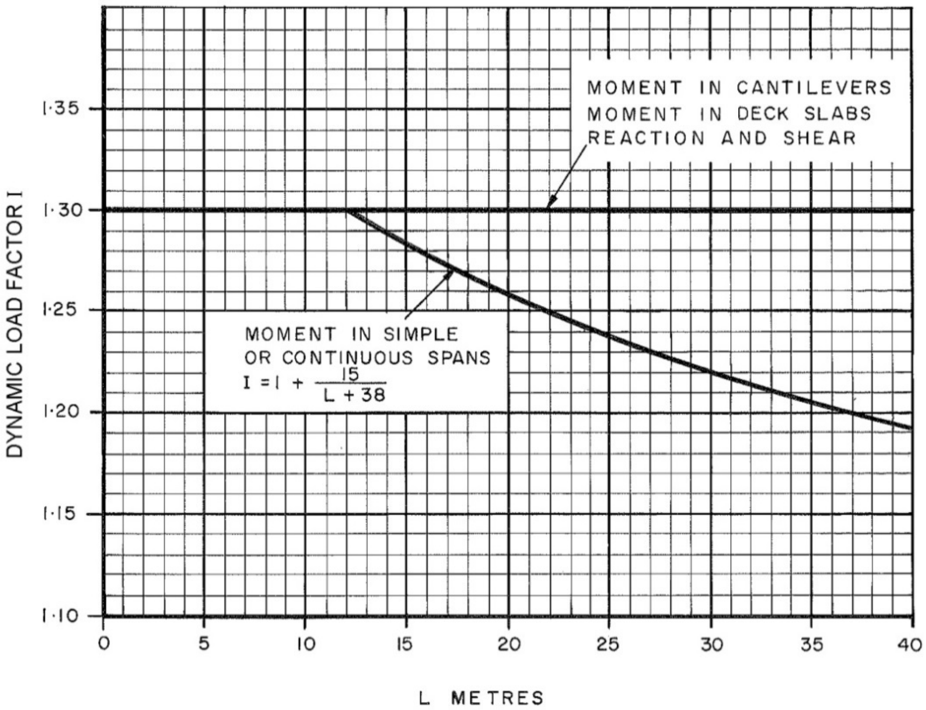
For the design of individual structural members, the number of load lanes that are loaded, applied in conjunction with the corresponding reduction factor, shall be selected and positioned to maximise the load effect on the structural member under consideration.

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.

Figure 3.2: Dynamic load factor for components above ground level and for bearings



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

3.2.5 continued

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.

3.2.6 Fatigue

The loading used in the fatigue assessment of steel bridges 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 has been developed, following on from the Waka Kotahi NZ Transport Agency commissioned research report RR 547 *Fatigue design criteria for road bridges in New Zealand*(2). The fatigue design of steel bridges shall use the TT530 fatigue loading model set in figure 3.3 unless otherwise specified by the road controlling authority. Further information is provided in the *Bridge manual commentary*.

This clause does not apply to fatigue design of roadway expansion joints. Fatigue loading for expansion joints is described in 4.7.3(b).

The fatigue loadings provided in this clause are not suitable for fatigue design of reinforced or prestressed concrete bridge components. Fatigue loading for concrete bridge components is described in 4.2.1(j).

a. Fatigue loading model (TT530)

   The TT530 fatigue loading model comprises three design load groups (figure 3.3):

   i. the TT530 vehicle representing a 530kN 8-axle truck-and-trailer
   ii. the 4-axle truck (representing the front of the TT530 vehicle)
   iii. the 2x75kN tandem axle set

   The 4-axle truck and the 2x75kN tandem axle set shall be used where they have a more severe effect than the TT530 vehicle (such as for components with continuous span lengths less than 15m). Use of automated methods applying the TT530 vehicle, but omitting axle sets with relieving effect, may be used as an alternative to the 4-axle truck and the 2x75kN tandem axle set.

   For assessment of main members for global effects the fatigue loading shall be located centrally within design lanes or centrally within marked traffic lanes where defined. The lane positions shall consider intended future carriageway reconfigurations in addition to the initial configuration.

   For assessment of local effects, the fatigue loading shall be located centrally within a 3m wide lane anywhere within the carriageway to maximise the fatigue effects on the component under consideration.

b. Fatigue load effects

   The fatigue load effects shall be determined from 100% of the effects of the TT530 fatigue loading, with a load factor of 1.0.

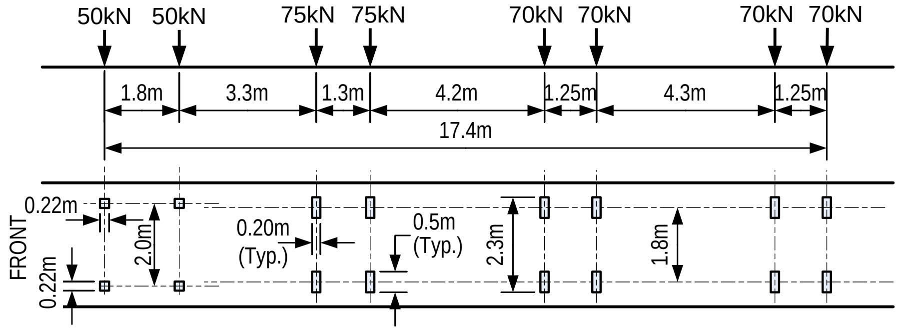
   A fatigue stress cycle shall be taken to be the maximum peak to peak stress range for a passage of the fatigue loading across the structure.

   Note: The dynamic load allowance (α) specified in AS 5100.2 *Bridge design* part 2 Design loads(3) clause 7.7.2, and the 70% factor on loading specified in AS 5100.2(3) clause 7.9, shall not be applied to the TT530 fatigue loading effects. Those factors were taken into consideration when determining the load factor of 1.0.

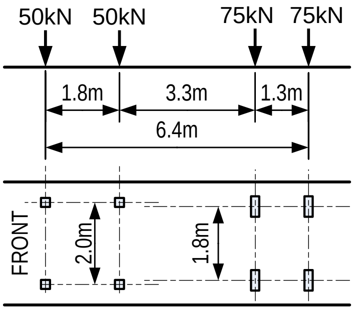
3.2.6 continued

Figure 3.3: TT530 fatigue loading model

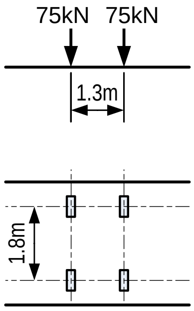
i. the TT530 vehicle (530kN 8-axle truck-and-trailer)



ii. the 4-axle truck (representing the front of the TT530 vehicle)



iii. the 2x75kN tandem axle set



c. Specific dynamic amplification factor for TT530 fatigue loading

A specific dynamic amplification factor (DAF) of 1.3 shall be applied to the TT530 fatigue loading within 6m of expansion joints or other discontinuities including the ends of integral bridges. For simplicity this DAF shall be applied to the actions on any cross section within 6m of the joint.

d. Loading from multiple lanes

Irrespective of the actual number of lanes on the bridge, the fatigue loading from only two lanes shall be considered to contribute to the fatigue damage of the component under consideration. The two lanes shall be referred to as Lane 1 and Lane 2, and shall be selected as follows:

- fatigue loading in Lane 1 causes the largest peak to peak stress range in the component under consideration
- fatigue loading in Lane 2 causes the second largest peak to peak stress range in the component under consideration

Lanes 1 and 2 may be in the same direction, or may be in opposing directions (whichever provides the two largest peak to peak stress ranges in the component under consideration), and will vary for different components under consideration.

The fatigue loadings in Lane 1 and Lane 2 shall be applied separately when calculating the peak to peak stress ranges (ie the fatigue loadings in Lane 1 and Lane 2 shall be assumed to occur at different points in time). The fatigue damage from Lane 1 and Lane 2 shall be combined using a Miner's summation or equivalent in accordance with the applicable materials standard (refer to 4.3.3(l) for application of AS/NZS 5100.6 *Bridge design* part 6 Steel and composite construction(4)).

3.2.6 continued

To allow for possible simultaneous fatigue loading from side-by-side running (or coincident vehicles from opposing directions) the total fatigue damage, obtained by combining the separate contributions from Lanes 1 and 2, shall be multiplied by factor $K_b Z$.

Where: $K_b = \dfrac{\text{Maximum peak to peak stress range at the component being considered caused by the fatigue loading in Lane 2}}{\text{Maximum peak to peak stress range at the component being considered caused by the fatigue loading in Lane 1}}$

Z = 1.0 for L ≤ 3.0m

= 1.5 for L ≥ 20m

= $0.71 + 0.61 \log_{10} L$ for 3.0m < L < 20m

$K_b Z$ shall not be less than 1.0

See 4.3.3(l) for the application of these rules to multiple lane loading of steel components.

e. Daily heavy vehicle counts per lane

Heavy vehicles shall include medium and heavy commercial vehicles plus buses, as defined in the *Monetised benefits and costs manual*(5), excluding light vehicles with gross mass up to 3500kg.

Table 3.1: Allocation of heavy-vehicles-per-day in one direction into heavy-vehicles-per-lane-per-day in that direction

Route description	No. of lanes in one direction	Proportion in Lane 1	Proportion in Lane 2
All routes	1	100%	100% in the opposing direction (unless a single lane bridge)
Motorways outside urban areas, expressways, and other rural routes	2 or more	100%	Nil (or 100% if in the opposing direction)
Urban motorways and arterial routes	2	65%	35% (or 65% if in the opposing direction)
		An additional check shall be carried out assuming 80% in the nominal "slow" lane, and 20% (or 80% if in the opposing direction) in the lane (other than the slow lane) that causes the largest peak to peak stress range in the component under consideration	
Urban motorways and arterial routes	3 or more	65%	45% (or 65% if in the opposing direction)
Other urban roads	2 or more	65%	35% (or 65% if in the opposing direction)

Note: For urban motorways and arterial routes with 3 or more lanes the total for Lane 1 and Lane 2 is 110% for lanes in the same direction

3.2.6 continued

The number of heavy-vehicles-per-day in one direction shall be based on the year the bridge is to be put into service. Guidance on use of heavy vehicle count information is included in C3 in the *Bridge manual commentary*. The number of heavy-vehicles-per-day assumed for low volume roads including ONRC Secondary Collector and Access road categories (refer table 2.1 and figures 2.1(a)-(c)) shall be at least 25 heavy-vehicles-per-day in each direction.

The number of heavy-vehicles-per-day in one direction shall be allocated into lanes in accordance with table 3.1.

f. Cycle counts for the TT530 fatigue loading

Unless otherwise specified by the road controlling authority, the number of fatigue stress cycles, from one traffic lane, to be used for the calculation of the fatigue load effects on the structural element under consideration shall be as follows:

For the TT530 fatigue loading:

$$\begin{array}{c}\text{Cycle}\\\text{count}\end{array} = \begin{array}{c}\text{Heavy-vehicles-per-lane-per-day on}\\\text{the year the bridge is put into service}\end{array} \times \begin{array}{c}\text{Equivalent cycles}\\\text{per heavy vehicle}\end{array} \times 1 \times 10^5 \times \begin{array}{c}\text{Route}\\\text{factor}\end{array}$$

Where: $\text{Equivalent cycles per heavy vehicle}$ = 2.0 for $L \leq 5$m

= $^{10}/_{L}$ for 5m<L<16.7m

= 0.6 for $L \geq 16.7$m

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

i. for positive bending moments and end shear, L is the span length in which the bending moment or shear force is being considered
ii. for negative moment over interior supports, L is the average of the adjacent span lengths
iii. for reactions, L is the sum of the adjacent span lengths
iv. for cross-girders, L is the sum of the two adjacent longitudinal deck span (eg twice the longitudinal spacing of equally spaced cross-girders).

Unless otherwise specified by the road controlling authority, route factors for New Zealand roads shall be as follows:

Route description	Route factor
Freight routes with a very high proportion of fully loaded long vehicles in one or both directions.	1.0
Typical long-haul freight routes, national and regional routes including expressways	0.8
Motorways, other rural freight routes	0.6
High-volume urban motorways and urban arterial routes	0.4
Urban roads	0.3

The above route factors allow for general access by HPMV conforming vehicles (see clause 7.1.2) on all route types. Route factor = 1.0 shall apply to exceptional cases such as logging, dairy factory, bulk aggregate supply, and port access routes where most of the freight vehicles in one direction are fully loaded. Further information is provided in the *Bridge manual commentary* C3.2.6(f).

3.2.6 continued

g. Service life adjustment

The cycle counts for the TT530 fatigue loading are intended for a service life of 100 years. For increases in design service life beyond 100 years the cycle counts for TT530 fatigue loading in (0) shall be increased by 2.2% per additional year.

3.2.7 Construction traffic loads

Where sections of a project are to be utilised as haul routes for the project construction and new structures are to be subjected to heavy construction vehicles exceeding the legal limits for road traffic, the structures shall be designed for the loading effects imposed by the nominated vehicles utilising the construction load combinations contained in section 3.5, with the loading effects considered as short term construction loads with an ultimate limit state load factor of 1.8 and a dynamic load factor I = 1.43, unless lower values can be justified by strict controls on load, speed and road roughness and are agreed with the road controlling authority.

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, eg 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 non-structural elements that are considered unlikely to vary during use of the structure, including traffic and pedestrian barriers, kerbs, footpaths, lighting and stormwater drainage. 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 the weight of non-structural elements added to the structure which may vary during the use of the structure, including services and their supports and road surfacing. Surfacing shall be allowed for at 1.5kN/m² whether the intention is to surface the bridge immediately or not. Where a levelling course is applied, the weight of the levelling course shall be in addition to the 1.5kN/m² superimposed dead load allowance for bridge deck surfacing.

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 0.25kN/m² for future services 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 due to the inertia of the structure, and the required structure ductility, shall be obtained from section 5. Earthquake effects on ground and soil structures (eg embankments, slopes and independent retaining walls) are specified in section 6. The earthquake increment of soil pressure acting on a structure shall be treated as an earthquake load when combining loads into load combinations as specified in 3.5.

In considering the stability and displacement of ground and soil structures (including earth retaining walls), unweighted peak ground accelerations, as specified in section 6, shall be used as the basis for deriving the earthquake loads acting.

In considering the strength design of structures (including locked-in structures and retaining walls), magnitude weighted peak ground accelerations, as specified in section 5, shall be applied in deriving the earthquake increment of soil pressure acting on the structure.

3.4.4 Shrinkage, creep and prestressing effects

The effects of shrinkage and creep of concrete, and shortening due to prestressing shall be taken into account. The resulting horizontal forces on substructure elements, whether transmitted by integral connection, shear keys, bearing friction or bearing stiffness (refer 3.4.19) shall be allowed for.

In the derivation of forces imposed on the structure due to these effects, consideration shall be given to the likelihood of cracking occurring in reinforced concrete piers and the influence this will have on their section rigidity. An appropriately conservative assessment of the forces to be adopted for the design of the structure shall therefore be made. The effects of creep in the pier in reducing the forces may be taken into account.

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

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

3.4.4 continued

Appropriate load factors for the effects of shrinkage and creep (SG) and prestressing (PS) are given in tables 3.2, 3.3 and 3.7.

Design shall consider the cases both before and after shrinkage and creep have occurred. The load factors for SG are considered to allow for the potential variation of ±30% in shrinkage and creep described in 4.2.1 (e) and NZS 3101 *Concrete structures standard*(6). Design for the effects of prestress shall consider the cases both before and after losses. Design shall also consider the prestress transfer case. At the ultimate limit state, load factors of $1.15DL + 1.15PS$ and $0.90DL + 1.15\ PS$ shall be used for the transfer case.

3.4.5 Wind

a. Wind load shall be applied to a bridge in accordance with the principles set out in BS 5400-2 *Steel concrete and composite bridges* part 2 Specification for loads(7) clause 5.3 contained within BD 37 *Loads for highway bridges*(8) appendix A, 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 30m for which aerodynamic effects may be critical, the principles forming the basis of BD 49 *Design rules for aerodynamic effects on bridges*(9) 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 1170.2 *Structural design actions* part 2 Wind actions(10) clauses 2.2 and 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:

$$V_r = \frac{V_d S_c T_c}{S_b T_g}$$

   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 5400-2(7) clause 5.3, contained within BD 37(8) appendix A.

   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 (including pedestrian loading) on the bridge, as defined in (a) above, the maximum site gust wind speed acting on adverse areas shall be the lesser of 37m/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 × 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 5400-2(7) clauses 5.3.3 to 5.3.6, contained within BD 37(8) appendix A.

3.4.6 Temperature effects

Temperature effects shall be allowed for in the design under the following load cases, which shall be treated as able to act separately or concurrently:

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:

3.4.6 continued

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

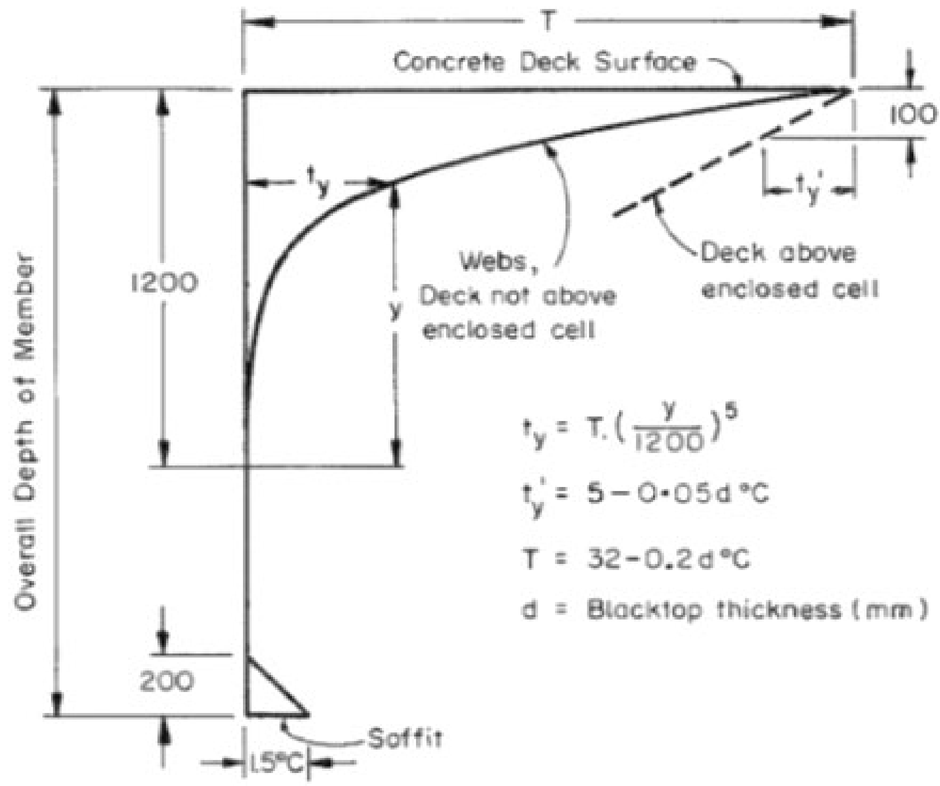
In the derivation of forces imposed on the structure due to these effects, consideration shall be given to the likelihood of cracking occurring in reinforced concrete piers and the influence this will have on their section rigidity. An appropriately conservative assessment of the forces to be adopted for the design of the structure shall therefore be made.

b. Differential temperature change

Allowance shall be made for stresses and movements, both longitudinal and transverse, resulting from temperature variation through the depth of the structure shown in figure 3.4. The effects of vertical differential temperature gradients shall be considered for both positive differential temperature conditions, where solar radiation has caused a gain in the top surface temperature, and negative temperature differential conditions, where re-radiation of heat from the section or snow fall results in a relatively low top surface temperature.

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

Figure 3.4: Temperature variation with depth



Notes:

(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 load combinations for structures under construction should be used (see **table 3.7**), with the value of T reduced to 27°C for the differential temperature load case.

(iii) The negative temperature variation to be considered shall be taken as that for bridge type 1 from figure 18.3 of AS 5100.2[3]. The value of T shall be set at 22°C (ie an assumed blacktop thickness of 50mm).

3.4.6 continued

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.

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

3.4.7 Construction and maintenance loads

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

All elements of structures that will be subjected to construction and maintenance loading (eg the bottom flange of box girders) shall be designed for a minimum access loading of 1.5kN/m², which need not act concurrently with traffic live loads.

3.4.8 Water flow and buoyancy

Loads due to water flow and buoyancy shall be applied to a bridge in accordance with AS 5100.2[(3)] clause 16 except as modified below:

a. Modification to AS 5100.2[(3)] clause 16.3.1

   In place of the 2000-year average recurrence interval (ARI) specified, the upper limit of the ultimate limit state ARI shall be taken as the inverse of annual probability of exceedance for the ultimate limit state given in table 2.1 of this manual.

b. Modification to AS 5100.2[(3)] clause 16.3.2

   In place of the serviceability limit state design floods specified by AS 5100.1 *Bridge design part 1 Scope and general principles*[(11)] clause 11.1, the ARI of the serviceability limit state design flood shall be taken as the inverse of the annual probability of exceedance for the relevant serviceability limit state (SLS 1 or SLS 2) given in table 2.1 of this manual.

c. Modification to AS 5100.2[(3)] clauses 16.5.2 and 16.5.3

   In place of equations 16.5.2(2) and 16.5.2(3) of AS 5100.2[(3)], the relative submergence (S_r) and proximity ratio (P_r) for use with figure 16.5.2(A) of AS 5100.2[(3)] and figure 3.6 (in place of figure 16.5.3 of AS 5100.2[(3)]) shall be taken as:

$$S_r = \frac{d_{wgs}}{d_s}$$

$$P_r = \frac{y_{gs}}{d_s}$$

Where: d_{wgs} = vertical distance from the girder soffit to the flood water surface upstream of the bridge

y_{gs} = average vertical distance from the girder soffit to the bed assuming no scour at the span under consideration

d_s = depth of the solid superstructure (excluding any railings but including solid parapets) projected on a plane normal to the water flow (see figure 3.5).

   Figure 3.5 shall be read in conjunction with figure 16.5.2(A) of AS 5100.2[(3)].

   In place of figure 16.5.3 of AS 5100.2[(3)], the superstructure lift coefficient shall be derived from figure 3.6.

3.4.8 continued

Figure 3.5: Superstructure drag and lift force dimensions

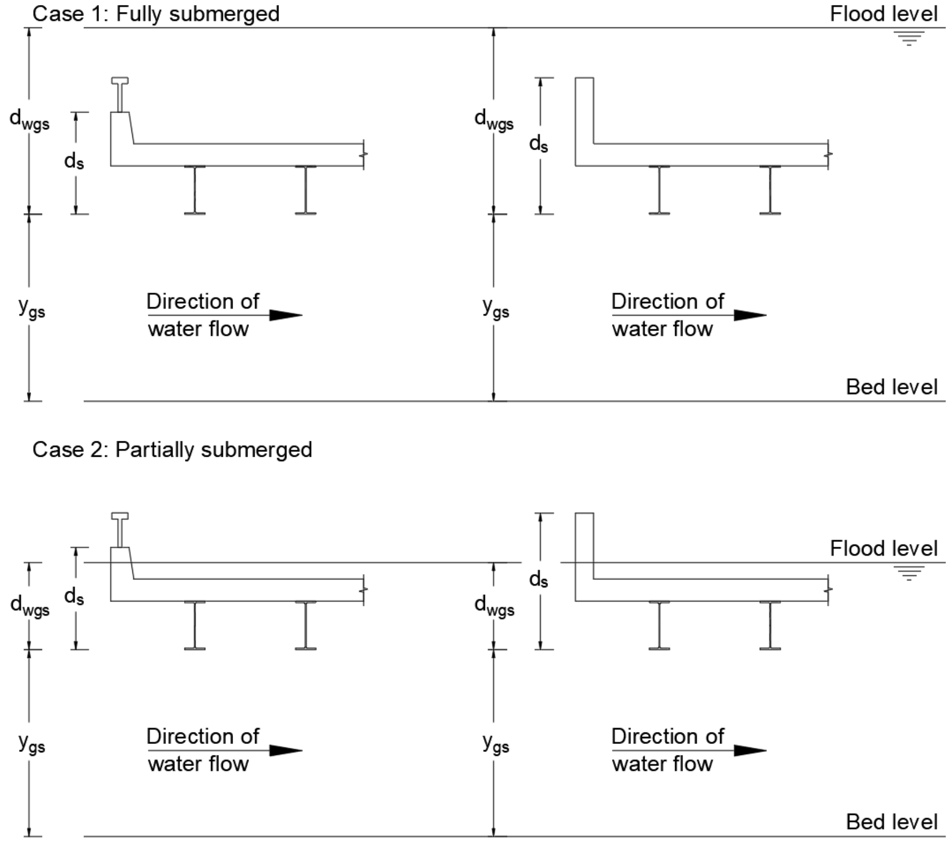
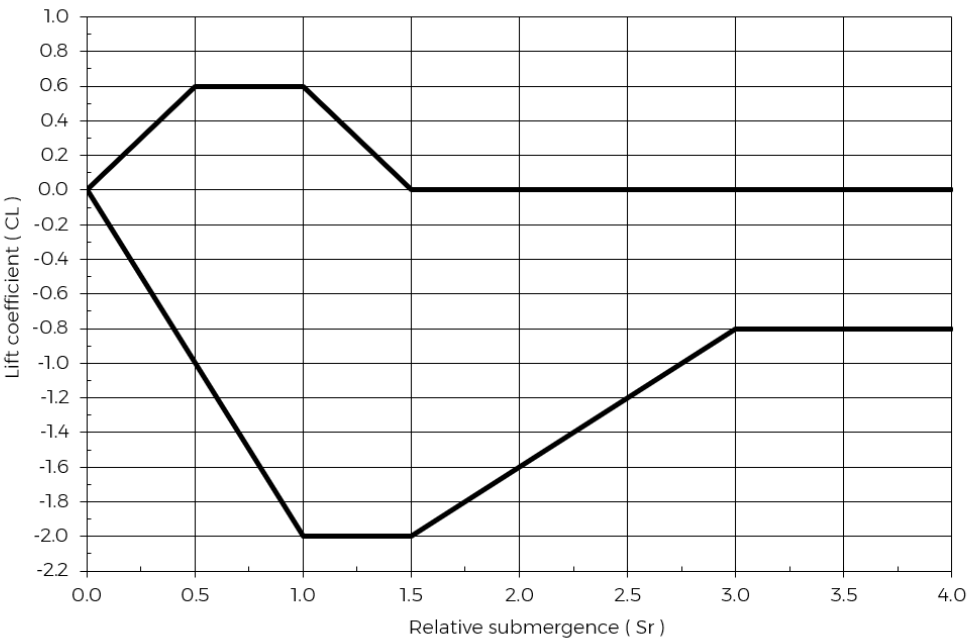


Figure 3.6: Superstructure lift coefficient



This method for determining drag and lift forces on bridges is not suitable for truss bridges and other through type structures where the flood level is above any solid face (ie S_r>1). Alternative approaches should be considered and if required detailed in the structure design statement. The possibility of debris impact on such structures would however have significant influence on the appropriateness of such a design. Some guidance may be found in *Hydrodynamic forces on inundated bridge decks*[12].

3.4.8 continued

d. Modification to AS 5100.2[3] clause 16.6.1

The depth of the debris mat varies depending on factors such as catchment vegetation, available water flow depth and superstructure span. In the absence of more accurate estimates, the minimum depth of the debris mat shall be half the water depth, but not less than 1.2m and not greater than 3m.

Both triangular shaped and rectangular shaped debris mats shall be considered (see figure 2.2 of this manual), noting that the size of mat determined from AS 5100.2 may differ from that determined in 2.3.5.

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.

Ordinary groundwater pressure shall correspond to the groundwater level with a 1 year annual probability of exceedance. Elevated groundwater pressure shall correspond to a groundwater levels with an annual probability of exceedance matching those given for serviceability and ultimate limit state floodwater actions in table 2.1. Conservatively the groundwater 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 1170.3 *Structural design actions* part 3 Snow and ice actions[13] for the annual probability of exceedance corresponding to the importance of the footbridge as defined in 2.1.3.

3.4.12 Earth loads

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

b. Earth retaining members shall be designed for static earth pressure plus either live load surcharge where appropriate or earthquake earth pressure in accordance with 6.2.4, whichever is more severe. Water pressure shall also be allowed for unless an adequate drainage system is provided.

For global analysis (of the whole structure), live load effects may be assumed equal to those of a surcharge pressure; in the case of HN (normal) traffic loading, 12kPa, and in the case of HO (overload) traffic loading, 24kPa.

For localised wheel load or other point load effects acting on retaining walls a method based on Boussinesq's equations or similar appropriate method shall be applied.

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.

3.4.12 continued

In some structures, for example concrete slab frame bridges, an increase in static earth pressure reduces the total load effect (eg moment) in some positions in the structure. When calculating the total load effect 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. In combining load effects, as specified in 3.5, the various loads transmitted by the soil shall be treated as follows:

- horizontal static earth pressure, vertical earth pressure, and negative skin friction shall be treated as earth pressures (EP)
- surcharge simulation of HN loading in some or all lanes shall be treated as a traffic live load (LL)
- surcharge simulation of HO loading in one lane with HN loading in some or all other lanes shall be treated as a traffic overload (OL)
- the earthquake increment of soil pressure (ΔP_E) shall be treated as an earthquake load (EQ)
- pressure due to water shall be treated as a groundwater loading (GW).

d. The effects of earthquake induced site instability, differential movements and liquefaction shall be considered as specified in section 6.

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. Footpaths and cycle tracks on highway bridges

i. A footpath or cycle track not considered as part of the carriageway in accordance with 3.2.3(a) shall be designed for a uniformly distributed load as follows:

- o when traffic loads are not considered in the same load case, between the limits of 2.6 and 5.0kPa as given by the expression $6.2 - S/25$, 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
- o when a single lane of traffic loading is considered in the same load case, the above loading multiplied by a factor of 0.8
- o when two or more lanes of traffic loading are considered in the same load case, the above loading multiplied by a factor of 0.4.

The structure shall also be checked for an overload case consisting of the HN wheel loads in accordance with 3.2.3(e).

ii. A footpath or cycle track considered as part of the carriageway, in accordance with 3.2.3(a), shall also be designed for the loads in (i) in conjunction with traffic loading on the remaining carriageway width.

iii. A footpath or cycle track on a highway bridge positioned out of reach of the traffic, eg underneath the carriageway, shall be designed as in (i) but without the overload.

b. Dedicated footbridges and cycle track bridges

A foot or cycle track bridge without traffic shall be designed for a uniformly distributed load between the limits of 2.6 and 5.0kPa, as given by the expression $6.2 - S/25$ where S is as defined in (a).

3.4.14 continued

c. In all cases where there is a likelihood of crowd loading, the maximum value of 5.0kPa 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. The factors for any concurrent traffic loading given in (a) shall still apply.

d. Where a footpath or cycle track is able to be accessed by horses or stock, the structure shall also be designed for a concentrated load of 8.0kN acting on a 100mm x 100mm square applied anywhere on the accessible areas.

e. For footpaths and cycle tracks of clear width 2.5m and greater, consideration shall be given to the need to design for specific maintenance, inspection and/or emergency vehicles. Proposed design provision for such vehicles shall be presented in the structure options report. Regardless of the provision made, the structure shall be designed for a concentrated load of 20kN on an area of 200mm x 200mm unless it is not possible for vehicles to access the path or track.

f. The cover concrete to embedded ducts and covers to ducts formed in footpaths and cycle tracks shall be designed to withstand the concentrated loads in (d) and (e) where relevant.

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 (ie 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 0.055m/s. The design load for this purpose shall be taken as the two 120kN axles of one HN load element. The following procedure is acceptable, but may be replaced by a rigorous analysis if desired:

a. Calculate the natural frequency of vibration. For a simply supported, rectangular in plan, span of prismatic section, the natural frequency is given by:

$$f = \frac{\pi}{2L^2}\sqrt{\frac{EIg}{w}}\ \mathrm{Hz}$$

Where: f = natural frequency (Hz)

E = modulus of elasticity (N/m^2)

I = second moment of area of the whole cross-section (m^4)

g = acceleration due to gravity (9.81 m/sec^2)

w = dead and superimposed dead load per unit length (N/m)

L = span (m)

For two and three span continuous bridges, standard solutions are available, eg *Bridge vibration study*(14). In other cases, including the cases of skewed bridges, curved bridges, and wide bridges where flexibility in the lateral direction is important, a computer analysis may be required.

3.4.15 continued

b. Calculate the amplitude of vibration:

$a = k\delta$

Where: a = Amplitude of vibration

k = 0.4 where f > 4Hz

= 0.75 where f < 4Hz

δ = deflection at midspan due to static design live load

In calculating δ, the two axles of the HN load may be assumed to be applied together at midspan.

c. Calculate velocity from:

$v = 2\pi f a$

Pedestrian and cycle bridges shall conform to the requirements of BS 5400-2[7] appendix B contained within BD 37[8] appendix A. Should the fundamental frequency of horizontal vibration of the bridge be found to be less than the 1.5Hz limit specified, a dynamic analysis to derive maximum horizontal acceleration may be undertaken in accordance with clause NA.2.44.7 of *National Annex (informative) to BS EN 1991-2:2003, Eurocode 1. Actions on structures* part 2 Traffic loads on bridges[15].

For pedestrian and cycle bridges with spans exceeding 30m, where aerodynamic effects may be critical, wind vibration effects as detailed in BD 49[9] shall be considered.

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 (total and differential), subsidence or ground deformation in the vicinity of the structure or approach embankment shall be taken into account.

Settlement, subsidence or ground deformation estimates used for this purpose shall be nominal values based on moderately conservative soil parameters, noting that the structural effects are factored up for ultimate limit state design.

Where there is potential for subsidence of the ground (such as due to groundwater changes, mining or liquefaction) 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

Locked-in forces in a structure that are caused by the erection sequence shall be allowed for. These may include both locked-in dead load forces and locked-in prestressing forces. They may also include locked-in effects due to temporary loads such as the weight of formwork, falsework, construction equipment and temporary prestressing acting on structural elements as they are built in. All locked-in effects, regardless of origin, shall be allowed for under dead loads (DL) and prestressing effects (PS) in load combinations.

3.4.18 Collision loads

a. General

Structures shall be designed to resist collision loads where:

- piers, abutments or superstructures of bridges over roads, railways or navigable rivers are located such that collisions are possible
- retaining walls are located such that collisions are possible and collision could result in the wall collapsing, partially or fully, onto the carriageway or endangering adjacent property
- bridge or other structure components at or above road level could be struck by vehicles.

3.4.18 continued

The intention behind these requirements is that the overall structural integrity of the structure should be maintained following a collision but that local damage to a part of the structure support or deck can be accepted.

In some circumstances, reduced collision loads may be considered if an appropriate protective barrier system is provided, collisions are considered highly improbable or the structure has sufficient redundancy to prevent collapse in the event of a collision.

Note that structure elements may be considered a hazard to road users under safe system principles as embodied in *Road to zero – NZ's road safety strategy*(16). Therefore there may be a requirement to install a traffic safety barrier system at piers, abutments or retaining walls regardless of whether they have been designed for collision loading or not.

Collision loads, applied as equivalent static loads, need only be considered at the ultimate limit state. Load factors to be considered at the moment of the collision shall be for load combination 5C given in table 3.3 unless specified otherwise. Load factors and combinations for any loads considered after collision are detailed in the following.

b. Collision load from road traffic

i. Collision with bridge substructure

Bridges over a highway shall be designed to resist a collision load of 2000kN applied to the piers or abutments supporting the bridge (including reinforced soil abutment walls), unless the piers or abutments concerned are located behind traffic barriers meeting performance level 5 or higher, as set out in appendix B, in which case they shall be designed to resist a collision load of 250kN. Each of these collision loads shall be applied horizontally 1.2m above ground level at an angle of 10 degrees from the direction of the centreline of the road passing under the bridge.

Where a pier or abutment consists of individual columns these shall each be designed to resist the collision load as detailed above. For a pier or abutment that consists of a wall, it shall be designed to resist the component of the collision load that is perpendicular to the face of the wall including the end face(s) of the wall facing oncoming traffic. At a corner both components of the load shall be applied simultaneously.

If there is any projection from a wall greater than 100mm that could snag a vehicle sliding along the wall face then the wall shall also be designed to resist the component of the collision load that is parallel to the wall, applied at the projection. In such instances the collision load applied to calculate the component shall vary linearly from 333kN at 100mm projection width to 2000kN at 600mm or greater projection.

The substructure, including 'redundant' piers or columns, may alternatively be designed for a reduced collision load of 250kN applied at any angle in the horizontal plane at 1.2m above ground level subject to the agreement of the road controlling authority, if:

- it can be demonstrated that the piers or abutments concerned are located such that collisions are highly improbable (eg where abutments are protected from collision by earth embankments or by considering the annual frequency for a bridge pier to be hit by a heavy vehicle (AFHBP) in accordance with clause 3.6.5.1 of AASHTO LRFD *Bridge design specifications*(17)); or

3.4.18 continued

- o a bridge has sufficient redundancy to prevent collapse under permanent loading plus live load using load factors for load combination 1A at the serviceability limit state given in table 3.2, with one pier or column removed (either one column to multi-column piers or the whole pier to single column piers). The effects of this load combination shall be assessed using ultimate limit state analysis; or
- o an abutment can be shown to have sufficient redundancy so that the bridge will not collapse in the event of a collision.

Where it is proposed that the full collision load with a bridge substructure is not to be designed for, where collisions are considered highly improbable or redundancy in the bridge structure is being relied on, the justification shall be included in the structure options report and structure design statement as details are developed.

ii. Collision with bridge superstructure

For bridges where the vertical clearance to the bridge superstructure is 6.0m or less from an underlying road carriageway, collision loads of 750kN acting normal to the bridge longitudinal direction and 375kN acting parallel to the bridge longitudinal direction (both loads acting in any direction between horizontal and vertically upward) shall be considered to act 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. The load normal to the carriageway shall be considered separately from the load parallel to the carriageway. Also where the vertical clearance to the bridge superstructure is 6.0m or less, all inner girders shall be designed for a soffit collision load of 75kN acting normal to the bridge longitudinal direction (and in any direction between horizontal and vertically upward).

For bridges where the vertical clearance to the bridge superstructure exceeds 6.0m (noting the requirements of figure A4 to make provision for settlement and road surfacing overlays in maintaining design vertical clearances) from an underlying road carriageway a collision load of 75kN acting normal to the bridge longitudinal direction 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.

Collision loads shall be treated as point loads, or may be distributed over a length of not more than 300mm of the impacted member. No other live load need be considered to coexist.

For concrete bridge superstructures, steel nosings shall be incorporated, above the approach traffic lanes, in the leading edge soffit of each beam where the vertical clearance to the bridge superstructure is 6.0m or less, and to the leading edge soffit of the leading beam only where the vertical clearance to the bridge superstructure is greater than 6.0m and less than 10m.

3.4.18 continued

The steel nosing for leading beams shall comprise composite 20mm thick plates extending vertically 200mm above the soffit and horizontally 200mm across the soffit. For other beams the steel nosing shall comprise composite 10mm thick plates extending vertically 150mm above the soffit and horizontally 150mm across the soffit. The nosings shall be galvanized (with galvanizing alone providing a time to first maintenance of 25 years) and coated over their full extent in accordance with NZTA S9 *Specification for coating steelwork on highway structures*(18) clause 8.6.3. If exposed to view, the nosings shall have a cover coat to blend with the adjacent surfaces. Consideration shall be given to the effects that any steel nosing has on beam flexural behaviour.

iii. Collision with retaining walls

Retaining walls shall be designed to resist collision loading where:

- o they are associated with bridges
- o they are not associated with bridges and vehicle collision could result in:
  - – part or all of the wall, including components such as precast concrete cladding panels, collapsing onto the traffic lanes of the carriageway
  - – failure of part or all of the wall, endangering adjacent property.

Collision loading shall consist of a load of 2000kN applied horizontally 1.2m above ground level at an angle of 10 degrees from the direction of the centreline of the road passing near the wall. Any face of the wall shall be designed to resist the component of the collision load that is perpendicular to the face. At a corner both components of the load shall be applied simultaneously. Collision loading on any projection from a wall shall be considered as for abutment walls in 3.4.18(b)(i).

A reduced load applied in a similar manner at a greater height up a retaining wall, varying in magnitude from 2000kN at 1.2m above ground level to 500kN at 5.0m, shall also be considered separately.

These collision loading requirements shall not apply to:

- o retaining walls associated with bridges that are located behind traffic barriers meeting performance level 5 or higher, as set out in appendix B
- o retaining walls not associated with bridges that are located behind traffic barriers meeting performance level 4 or higher, as set out in appendix B
- o retaining walls located such that collisions are highly improbable.

iv. Collision with the above deck level structure of through truss, tied arch and other similar bridge structures, protection beams and retaining wall props

Through truss, tied arch and other similar bridge structures with above deck level structure providing the primary structural support to spans shall be designed for collision from a vehicle traversing the bridge. The design collision loads specified herein shall also apply to the design of protection beams installed to protect the superstructure of low clearance bridges from collision from road vehicles and for collisions with props to retaining walls where the roadway is depressed below ground level in a trench.

Bridge structural elements projecting above deck level at either side of the bridge carriageway shall be protected from collision by rigid traffic barriers meeting performance level 5 or higher. Clearance between the barrier and structure shall be as required in 3.4.18(b)(vii).

3.4.18 continued

Bridge structural elements and other major elements projecting above the top of the side protection barriers or overhead of the road carriageway shall be designed for the collision loads given below. The load acting in the vertical plane normal to the bridge carriageway alignment shall be considered separately from the load acting in the vertical plane parallel to the bridge carriageway alignment. The loads shall be considered to act as point loads on the bridge elements in any direction between horizontal and vertically upwards. The load shall be applied to the element's leading corner nearest the carriageway considered in the direction of the vehicle travel.

The design collision loads shall be as follows, modified as specified below for the various structural elements:

- o Load acting in the vertical plane perpendicular to the bridge carriageway's longitudinal alignment: 375kN.
- o Load acting in the vertical plane parallel to the bridge carriageway's longitudinal alignment: 750kN.

Arch ribs, truss end posts and similar structural elements shall be designed for the full specified collision loading above, striking at all possible levels between the top of barrier level and 10m above road carriageway level.

The leading overhead structural member at each end of the bridge and within 10m of the carriageway shall be designed for the full collision loading specified.

Truss web members, arch rigid hanger members (as distinct from cable or single bar hangers) and overhead structural members within 10m of the carriageway, beyond 20m from the leading members, moving along the bridge in the direction of travel, shall be designed for one-third of the design collision load.

Truss web members, arch rigid hanger members (as distinct from cable or single bar hangers) and overhead structural members within 10m of the carriageway, within 20m from the leading members, moving along the bridge in the direction of travel, shall be designed for collision loading linearly interpolated with distance from the leading member to 20m from the leading member.

Collision loads shall be treated as point loads, or may be distributed over a length of not more than 300mm of the impacted member. No other live load other than the colliding vehicle, which shall be taken as the HN load element without lane load, need be considered to coexist at the moment of the collision. This vehicle load may be considered as an overload (OL) for the determination of load factors.

Single bar and cable hangers of tied and network arch structures shall satisfy the requirements of 4.9.

v. Non-concurrency of loading

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

vi. Exemptions

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 as a result of a collision. Such cases require justification in the structure options report and structure design statement as details are developed, and any variations to the requirements of this manual are subject to the agreement of the road controlling authority.

3.4.18 continued

vii. Collision protection

Where barriers are placed adjacent to a structure, or provide protection to a structure from vehicle collision, a minimum separation, to provide clearance to accommodate any barrier deflection and the colliding vehicle's tendency to roll over the barrier, shall be provided between the barrier front face and the face of the structure as follows:

- o Flexible or semi-rigid barriers: the working width of the barrier system, defined as the sum of the dynamic deflection of the barrier and the vehicle roll allowance (or the barrier system width if it is larger than the vehicle roll allowance). Refer to Austroads *Guide to road design* part 6 Roadside design, safety and barriers(19) clauses 5.3.15 to 5.3.17.
- o Performance levels 4 and 5 F type rigid barrier: vehicle roll allowance of 1.1m from the barrier front face. The dynamic deflection for a rigid barrier is zero.
- o Where rigid barriers are orientated normal to crossfall of the road sloping towards the structure, the separation shall be increased by 4.25m x the crossfall percentage/100.

c. Collision load from railway traffic

i. Collision with bridge substructure

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

Where bridge supports (ie abutment walls, piers or columns) are located within 20m of a rail track centreline the bridge shall be designed in accordance with one of the following:

- o Unless agreed otherwise by the road controlling authority and the railway authority, the bridge shall have sufficient redundancy to prevent collapse under permanent loading plus live load using load factors for load combination 1A at the serviceability limit state given in table 3.2, should part of an abutment wall or one or more pier or column be removed or rendered ineffective as a result of a collision. The number and location of supporting structures to be considered as removed by a train collision shall be determined by a risk analysis, and shall be subject to the agreement of the road controlling authority and the railway authority. The effects of this load combination shall be assessed using ultimate limit state analysis.

  For bridges over KiwiRail tracks, these provisions for design for redundancy to prevent collapse shall apply where bridge supports are situated within 5m of a rail track centreline (see KiwiRail *Railway bridge design brief*(20)).

  A 'redundant' bridge support shall be designed to resist a collision load of 250kN applied at any angle in the horizontal plane at 2m above rail level unless otherwise directed by the authorities noted above.
- o Alternatively, and with the agreement of the road controlling authority and the railway authority, the bridge supports shall be designed to resist collision loads.

  Where bridge supports are situated within 10m of a rail track centreline, they shall be designed to resist the following collision loads applied simultaneously:

  - 3000kN parallel to the rails
  - 1500kN normal to the rails

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

3.4.18 continued

Where bridge supports are situated between 10m and 20m from a rail track centreline they shall be designed to resist a collision load of 1500kN applied at any angle in the horizontal plane at 2m above rail level. This provision may be relaxed through a risk analysis subject to the agreement of the road controlling authority and the railway authority.

ii. Collision protection to bridge substructure

Bridge substructures shall be protected from collision in accordance with the requirements of the railway authority.

Bridges over KiwiRail tracks where bridge supports are situated within 5m of a rail track centreline shall be provided with collision protection consisting of an impact wall, designed in accordance with the KiwiRail *Railway bridge design brief*(20). The impact wall shall be standalone if the bridge is being designed for the redundancy requirements of 3.4.18(c)(i) or otherwise may be standalone or monolithic with the bridge supports being protected. The impact wall shall extend in length for not less than 2.0m to either side of the bridge support.

Bridges over KiwiRail tracks where bridge supports are situated greater than or equal to 5m and within 10m of a rail track centreline shall be protected by a robust kerb, the purpose of which is to reduce the momentum of a derailed train (see KiwiRail *Railway bridge design brief*(20)).

iii. Collision protection to bridge superstructure

Bridge superstructures where the vertical clearance is 5.5m or less from an underlying railway (noting any requirements of the railway authority to make provision for settlement or lifting of tracks in design vertical clearances) shall be designed for a 500kN collision load. The collision load shall be applied in any direction directed towards the bridge superstructure from the adjacent track centre-line, except downwards. Where the vertical clearance is more than 5.5m vertically above the railway track level, the bridge superstructure shall be designed for a 75kN collision load applied from the track centre-line in any direction except downwards.

The collision load shall not be applied in conjunction with the loads specified in 3.4.18(c)(i).

In addition and in all instances, any further requirements of the railway authority shall be satisfied.

The details of all provisions made for and agreements made with the railway authority shall be included in the structure design statement.

d. Collision load from shipping

Possible collision loads from shipping shall be considered. Bridge piers shall either be protected by auxiliary structures designed to absorb the collision energy, or they shall be designed to resist collision 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 structure design statement.

3.4.19 Bearing forces

Bridges shall be designed for the forces arising from the friction of sliding and rolling bearings, and the load displacement characteristics of elastomeric bearings.

The forces due to friction on bearings shall be calculated considering only the permanent loads acting on the bearings and shall be considered as acting in any of the sliding or rolling directions of the bearings.

Characteristic values of the coefficient of friction for stainless steel on PTFE sliding surfaces shall be as specified in 4.7.2.

Bearing friction forces shall have load factors applied as specified for bearing forces in 3.5.

Forces generated by elastomeric bearing movements shall be calculated for each load combination in 3.5 by multiplying the bearing stiffnesses specified in 4.7.2 (upper or lower bound as appropriate) by the serviceability or ultimate limit state movements generated by the relevant combination, utilising the load factors appropriate for each load or environmental effect.

3.4.20 Fire

Suitable loads, load factors and load combination shall be agreed with the road controlling authority for structures and environments where design for the effects of fire has been agreed with the road controlling authority to be necessary . Initial guidance on fire effects can be found in AS 5100.2[3] section 26 and specialist literature.

3.5 Combination of load effects

3.5.1 General

This amendment 4 to the *Bridge manual* 3rd edition introduces updated load combinations mostly using existing load factors for the design of completed structures and new load combinations and factors for structures under construction. Work is continuing to develop a new vehicle live load model and revised uniform temperature model for design, together with associated revised load factors.

3.5.1 continued

The effects of the loads described in 3.2 to 3.4 shall be combined by summating each load effect multiplied by the relevant load factors shown in tables 3.2 and 3.3 for the design of completed structures and in table **3.7** for the design of structures under construction, and as specified below:

a. In any combination, if a worse effect is obtained by omitting one or more of the non-permanent loads, this case shall be considered. Similarly the case of any 'permanent' load that is not always present (eg superimposed dead load, bearing friction, shrinkage and creep or settlement that are not initially present) shall be considered if a worse effect is obtained.
b. The load combinations specified cover general conditions. Provision shall also be made for other loads (eg snow and fire) where these might be critical. Combinations for such loads shall be agreed with the road controlling authority on a project specific basis, or shall be as detailed in the project principal's (or minimum) requirements for design and construct type contracts.

Locked-in effects due to erection sequence shall be included under dead load and prestressing as appropriate (see 3.4.17).

3.5.2 Specific load effects for design of completed structure	For the consideration of stability, maintenance of structural integrity and the design of bridge deck joints for seismic response, these aspects of design are not fully captured by tables 3.2 and 3.3 and reference shall also be made to 5.1.3 and 5.7.1(b). The design for load combinations that include earthquake and tsunami loading shall take into account the additional requirements specified in section 5 that are not applicable to design for non-seismic load combinations. The design for load combinations that include collision loading shall take into account the additional requirements specified in 3.4.18 that are not applicable to design for non-collision load combinations.
3.5.3 Specific load effects for structures under construction	a. Short-term construction loads (construction live loads) include the weights of workers, vehicles, hoists, cranes, other equipment and structural components subject to movement during the construction stage considered. b. Long-term construction loads are semi-permanent loads and can be thought of as construction dead loads. They include the weights of formwork, falsework, fixed appendages, stored material and lifting and launching devices not subject to movement during the construction stage considered. c. The required wind, flood, seismic, tsunami and collision 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 resistance shall all be taken into account. The loadings adopted shall give adequate protection in the circumstances being considered. d. Flood, water ponding, wind, seismic, tsunami or collision forces adopted for construction phase ultimate limit state design shall generally have a minimum return period of 100 years. Similarly, where relevant, forces adopted for construction phase serviceability limit state design, shall have a minimum return period of 1 year. Higher return periods may be appropriate for some structures. Lower return periods may be appropriate for short duration situations. e. The construction phase loads and combinations for a particular project shall be confirmed via the structure options reports. The construction phase combinations will need modification for balanced and anchor cantilever structures, launched structures and segmental construction – reference should be made to AS 5100.2(3) and AASHTO LRFD *Bridge design specifications*(17).

Table 3.2: Load combinations and load factors for the serviceability limit state for the design of completed structures

			Dead		Other permanent							Traffic				Environment						
			Self-weight	Superimposed	Bearing friction	Creep/shrinkage	Prestressing	Earth pressure	Settlement	Groundwater pressure (ordinary)	Ordinary water flow and buoyancy [d]	Normal vehicle	Special/normal vehicle	Horizontal effects	Pedestrian/cycle	Groundwater pressure (elevated) [a]	Floodwater flow and buoyancy, with scour [a]	Water ponding [a]	Wind [a]	Uniform temperature	Differential temperature
		Load symbol / **Combination**	DL	SD	BF	SG	PS	EP	ST	GW_O	OW	LLxl	OLxl	HE	FP	GW_E	FW	PW	WD	TP	DT
Permanent effects only		0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	-	0.50	0.50
Primary traffic (vehicle + pedestrian) with environmental	Traffic with temperature	1A	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	-	1.00	1.00	-	-	-	-	0.70	0.70
	Traffic with wind	1B	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	-	1.00	1.00	-	-	-	0.70	-	-
	Traffic with flood	1C	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	1.35	-	1.00	1.00	0.70[c]	0.70[b]	0.70	-	-	-
Environmental with traffic (vehicle + pedestrian)	Temperature with traffic	2A	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	0.70	0.70	-	-	-	-	1.00	1.00
	Wind with traffic	2B	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	0.70	0.70	-	-	-	1.00	-	-
	Flood with traffic	2C	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	1.00	-	0.70	0.70	1.00	1.00	1.00	-	-	-
Environmental combinations	Flood with wind	3A1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	1.00	1.00	1.00	0.70	-	-
	Wind with flood	3A2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	0.70[c]	0.70[b]	0.70	1.00	-	-
	Temperature with wind	3B1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	0.70	1.00	1.00
	Wind with temperature	3B2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	1.00	0.70	0.70
Special vehicles (permitted overload)	Special vehicle with temperature	4A1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	1.00/ 1.00	1.00	1.00	-	-	-	-	0.70	0.70
	Temperature with special vehicle	4A2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	0.70/ 0.70	0.70	0.70	-	-	-	-	1.00	1.00

Notes for table 3.2:

a. Load factor applied to serviceability limit state level flood, groundwater and wind actions.

b. If 0.70xFW results in a lower load than 1.00xOW, it shall be replaced by 1.00xOW.

c. If $0.70xGW_E$ results in a lower load than $1.00xGW_O$, it shall be replaced by $1.00xGW_O$.

d. Ordinary water flow and buoyancy to be taken as due to the flow with an AEP of 1 (ie 1 year event).

Table 3.3: Load combinations and load factors for the ultimate limit state for the design of completed structures [a]

			Dead [d]		Other permanent [d]							Traffic				Environment						Extreme		
		Combination	Self-weight	Superimposed	Bearing friction	Creep/shrinkage	Prestressing	Earth pressure	Settlement	Groundwater pressure (ordinary)	Ordinary water flow and buoyancy [c]	Normal vehicle	Special/normal vehicle	Horizontal effects	Pedestrian/cycle	Groundwater pressure (elevated)	Floodwater flow and buoyancy, with scour	Water ponding	Wind	Uniform temperature	Differential temperature	Earthquake	Tsunami	Collision loads
Load symbol			DL	SD	BF	SG	PS	EP	ST	GW_O	OW	LLxl	OLxl	HE	FP	GW_E	FW	PW	WD	TP	DT	EQ	TS	CO
Permanent effects only		0	1.35	1.35	1.35	1.35	1.00	1.35	1.35	1.00	1.35	-	-	-	-	-	-	-	-	0.50	0.50	-	-	-
Primary traffic with temperature	Normal vehicles and pedestrians	1A	1.35	1.35	1.30	1.35	1.00	1.50	1.35	1.00	1.35	2.25	-	2.25	1.50	-	-	-	-	1.00	1.00	-	-	-
	Pedestrians only (ie footbridges)	1AP	1.35	1.35	1.30	1.35	1.00	1.50	1.35	1.00	1.35	-	-	-	1.50	-	-	-	-	1.00	1.00	-	-	-
Traffic with wind and temperature		1B	1.35	1.35	1.30	1.35	1.00	1.50	1.35	1.00	1.35	1.80	-	1.80	1.20	-	-	-	1.00[b]	0.75	0.75	-	-	-
Environmental with traffic	Primary temperature with traffic	2A	1.35	1.35	1.30	1.35	1.00	1.50	1.35	1.00	1.35	1.35	-	-	1.00	-	-	-	-	1.70	1.25	-	-	-
	Primary flood with traffic	2C	1.35	1.35	1.30	1.35	1.00	1.50	1.35	-	-	1.35	-	-	1.00	1.00	1.30	1.00	-	-	-	-	-	-
Environmental combinations	Primary wind with flood and temperature	3A	1.35	1.35	1.30	1.35	1.00	1.50	1.35	-	-	-	-	-	-	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-
Special vehicles (permitted overload)	Special vehicles with pedestrians	4A	1.35	1.35	1.30	1.35	1.00	1.50	1.35	1.00	1.35	-	1.50/ 1.50	1.50	1.50	-	-	-	-	1.00	1.00	-	-	-
Extreme	Seismic	5A	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	-	0.50	0.50	1.00	-	-
	Tsunami	5B	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	-	-	0.50	0.50	-	1.00	-
	Collision with traffic	5C	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	-	-	1.00	-	-	-	-	0.50	0.50	-	-	1.00

Notes for table 3.3:

a. Also load combinations and load factors for damage control limit state (DCLS) and collapse avoidance limit state (CALS) in accordance with sections 5 and 6.
b. See 3.4.5(b) for wind speed to be considered in conjunction with traffic loading.
c. Ordinary water flow and buoyancy to be taken as due to the flow with an AEP of 1 (ie 1 year event) other than for extreme load combinations 5A (seismic) and 5C (collision) where it shall be taken as due to mean daily flow conditions.
d. Where the effect of a possible reduction in any permanent load is critical at ultimate limit state, use of a 1.00 factor shall be used for that load. See also 3.5.1(a).

Table 3.4: Ultimate limit state load factors for dead loads for the design of structures under construction

Type of material	Ultimate limit states where dead load	
	Reduces safety	Increases safety
Structural steel	1.10	0.95
Structural concrete and non-structural elements	1.20	0.90
Timber	1.25	0.80

(Note that the load factors in tables 3.4 to 3.6 are currently intended for use with structures under construction only.)

Table 3.5: Ultimate limit state load factors for superimposed dead loads for the design of structures under construction

Type of load	Ultimate limit states where dead load	
	Reduces safety	Increases safety
Permanent	1.40	0.80
Removable and future	1.40	0

Notes:

a. Permanent superimposed dead load (SDL) is SDL added at construction that is unlikely to be removed from the structure.
b. Removable SDL is SDL added at construction that is likely to be removed at some stage during the structure's life.
c. Future SDL is SDL that is allowed for in design but is not added at construction.

Table 3.6: Ultimate limit state load factors for earth pressure loads for the design of structures under construction

Type of soil	Ultimate limit states where soil	
	Reduces safety	Increases safety
All fills and in-situ soils	1.5	0.70

Notes for table 3.7 (following):

a. The load factor of 1.50 on long-term construction loads may be reduced to 1.35 if the loads are derived based on known dimensions and material weights. No reduction shall be used for stored materials.
b. The load factor of 1.50 on short-term construction loads assumes the loading and position is well controlled. A load factor of 1.80 should be used if in doubt.
c. Operating wind force (WD_O) due to nominated maximum operating wind speed in conjunction with short-term construction load.
d. 0.90 x design temperature considered to represent construction phase serviceability limit state temperature; 0.65 x design temperature considered to represent 70% x construction phase serviceability limit state temperature; 1.10 x design temperature considered to represent construction phase ultimate limit state temperature.
e. Ordinary water flow and buoyancy to be taken as due to the flow and level due to mean daily flow conditions.
f. Where the effect of a possible reduction in a permanent load is critical at ultimate limit state, use of the lower bracketed load factor, where shown, shall be used for that load.

Table 3.7: Load combinations and load factors for the design of structures under construction

		Load symbol	Serviceability limit state								Ultimate limit state								
Construction	Short-term	CS	-	1.00	0.70	1.00	-	-	-	-	-	1.50[b]	1.50[b]	-	-	-	-	-	-
	Long-term	CL	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	1.50[a]	1.50[a]	1.50[a]	1.50[a]	1.50[a]	1.00	1.00	1.00
	Wind (operating) [c]	WD_O	-	-	-	1.00		-	-	-	-	-	1.50	-	-	-	-	-	-
Extreme	Collision loads (construction)	CO_C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1.00
	Tsunami (construction)	TS_C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1.00	-
	Earthquake (construction)	EQ_C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1.00	-	-
Environment	Differential temperature	DT	0.50	0.65[d]	0.90[d]	0.50	-	-	0.90[d]	0.65[d]	0.50	0.90[d]	0.50	1.10[d]	-	0.50	0.50	0.50	0.50
	Uniform temperature	TP	0.50	0.65[d]	0.90[d]	0.50	-	-	0.90[d]	0.65[d]	0.50	0.90[d]	0.50	1.10[d]	-	0.50	0.50	0.50	0.50
	Wind (construction)	WD_C	-	-	-	-	0.70	1.00	0.70	1.00	-	-	-	-	-	1.00	-	-	-
	Water ponding (construction)	PW_C	-	-	-	-	1.00	0.70	-	-	-	-	-	-	1.00	1.00	-	-	-
	Floodwater flow and buoyancy, with scour (construction)	FW_C	-	-	-	-	1.00	0.70	-	-	-	-	-	-	1.30	1.00	-	-	-
	Groundwater pressure (elevated)	GW_E	-	-	-	-	1.00	0.70	-	-	-	-	-	-	1.00	1.00	-	-	-
Other permanent [f]	Ordinary water flow and buoyancy [e]	OW	1.00	1.00	1.00	1.00	-	-	1.00	1.00	1.35 (0.90)	1.30 (0.80)	1.30 (0.80)	1.30 (0.80)	-	-	1.00	-	1.00
	Groundwater pressure (ordinary)	GW_O	1.00	1.00	1.00	1.00	-	-	1.00	1.00	1.00	1.00	1.00	1.00	-	-	1.00	1.00	1.00
	Settlement	ST	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	1.50	1.50	1.50	1.50	1.50	1.00	1.00	1.00
	Earth pressure	EP	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35 (0.90)	See table 3.6					1.00	1.00	1.00
	Prestressing	PS	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	Creep/shrinkage	SG	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	1.30	1.30	1.30	1.30	1.30	1.00	1.00	1.00
	Bearing friction	BF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35	1.30	1.30	1.30	1.30	1.30	1.00	1.00	1.00
Dead [f]	Superimposed	SD	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35 (0.90)	See table 3.5					1.00	1.00	1.00
	Self-weight	DL	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.35 (0.90)	See table 3.4					1.00	1.00	1.00
	Combination		C0	C1A1	C1A2	C1B	C3A1	C3A2	C3B1	C3B2	C0	C1A	C1B	C2A	C2C	C3A	C5A	C5B	C5C
		Load symbol	Permanent effects and long-term construction only	Construction with temperature	Temperature with construction	Operational wind and temperature	Flood with wind	Wind with flood	Temperature with wind	Wind with temperature	Permanent effects and long-term construction only	Temperature	Operational wind and temperature	Primary temperature	Primary flood	Primary wind with flood and temperature	Seismic	Tsunami	Collision
				Long- and short-term construction/ environmental			Environmental/ long-term construction					Long- and short-term construction/ environmental		Environmental/ long-term construction			Extreme / long-term construction		

3.6 References

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