3.0 Design loading

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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 the NZTA commissioned research report RR 539 A new vehicle loading standard for road bridges in New Zealand was published. Subsequent to this research a review of vehicle live load models, load combinations and load factors to be used has commenced. Until this work is completed and any revisions published, the current 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

HN load element

U.D.L. 10.5 kN/m
120 kN Axle
120 kN Axle

CONTACT AREAS 500 x 200

U.D.L. 3.5 kN/m²

HO load element

U.D.L. 10.5 kN/m
240 kN Axle
240 kN Axle

CONTACT AREAS 3000 x 200

ALTERNATIVE (a) U.D.L. 3.5 kN/m²

CONTACT AREAS 900 x 600

ALTERNATIVE (b) U.D.L. 3.5 kN/m²

All dimensions are in mm
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:

<table>
<thead>
<tr>
<th>Width of carriageway</th>
<th>Number of load lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 6m</td>
<td>1</td>
</tr>
<tr>
<td>6m but less than 9.7m</td>
<td>2</td>
</tr>
<tr>
<td>9.7m but less than 13.4m</td>
<td>3</td>
</tr>
<tr>
<td>13.4m but less than 17.1m</td>
<td>4</td>
</tr>
<tr>
<td>17.1m but less than 20.8m</td>
<td>5</td>
</tr>
</tbody>
</table>

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

c. For global effects, typically the 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 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 4 (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:

<table>
<thead>
<tr>
<th>Number of load elements (lanes loaded)</th>
<th>Reduction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>0.7</td>
</tr>
<tr>
<td>5</td>
<td>0.6</td>
</tr>
<tr>
<td>6 or more</td>
<td>0.55</td>
</tr>
</tbody>
</table>

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.

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 is not available, although one is in development following on from the NZTA commissioned research report RR 547 Fatigue design criteria for road bridges in New Zealand\(^2\).

In the interim, steelwork may be designed for the effects of fatigue in accordance with AS 5100.6-2004 Bridge design part 6 Steel and composite construction\(^3\) as modified by the New Zealand Heavy Engineering Research Association document *Recommended draft fatigue design criteria for bridges version 3*\(^4\). The draft fatigue design criteria shall be amended so that the fatigue design traffic load is applied in each marked traffic lane rather than the design lanes.

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.
3.2.6 continued

**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.

3.2.7 Live loading due to construction vehicles and plant

Where sections of a project are to be utilised as haul routes for the project construction and new structures are to be subjected to loading effects imposed on them by construction vehicles and plant that exceed those imposed by the Bridge manual specified design HN (normal) live loading applied in as many lanes as produces the worst design loading effect on elements of the structures, the structures shall be designed for the loading effects imposed by the construction vehicles and plant treated as normal live loading.

Where construction vehicles and plant are to traverse over a new structure or alongside a new earth retaining structure and will impose on that structure loading effects exceeding those imposed by the design normal live loading applied to the design of the structure, the passage of such vehicles shall be subject to NZTA’s permit procedures for overweight vehicles.

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. Consequently, 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, barriers, lamp standards, kerbs, services 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 (e.g., 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. Transmission of horizontal forces from superstructure to substructure by bearing restraint 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.

Appropriate load factors for the effects of shrinkage and creep (SG) and prestressing (PS) are given in tables 3.1 and 3.2.

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 clause 5.3 contained within BD 37 Loads for highway bridges 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 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 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 clause 5.3, contained within BD 37 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 \( 37 \times \frac{S_c}{S_b} \) m/s and \( V_r \) m/s, as specified above.

c. Wind forces shall be calculated using the method of BS 5400-2 clauses 5.3.3 to 5.3.6, contained within BD 37 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:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>For steel structures</td>
<td>±25°C</td>
</tr>
<tr>
<td>For concrete structures</td>
<td>±20°C</td>
</tr>
</tbody>
</table>

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.

Figure 3.3: 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 combination 5B should be used, 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(9). The value of T shall be set at 22°C (ie an assumed blacktop thickness of 50mm).
3.4.6 continued

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 3.3. 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.

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 (e.g., 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 pressure

Loads due to water pressure shall be applied to a bridge in accordance with AS 5100.2 Bridge design part 2 Design loads clause 16 except as modified below:

a. Modification to AS 5100.2 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.

Where the critical design condition occurs at an ARI of less than the upper limit of the ultimate limit state $\text{ARI}_1$, the ultimate limit state load factor ($\gamma_{FL}$) shall be taken as:

$$\gamma_{FL} = 2 - \left( \frac{1}{\log\left(\frac{\text{ARI}_1}{20}\right)} \right) \times \left( \log\left(\frac{\text{ARI}_2}{20}\right) \right)$$

b. Modification to AS 5100.2 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 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 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 2.3.5 and figure 2.2 of this manual).
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 groundwater level with a 1/50 annual probability of exceedance. 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 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.

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).
3.4.12 continued

- The earthquake increment of soil pressure (ΔP₁) shall be treated as an earthquake load (EQ).
- Pressure due to water shall be treated as a ground water 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. 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:

- When traffic loads are not considered in the same load case, 5.0kPa.
- When traffic loads are considered in the same load case, between the limits of 1.5 and 4.0kPa 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 in accordance with 3.2.3(e).

b. 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 (a) in conjunction with traffic loading on the remaining carriageway width.

c. 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 (a) 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 5.0kPa, as given by the expression 6.2 - S/25 where S is as defined in (a).

e. 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.

f. Where a footpath or cycle track is able to be accessed by horses or stock, the structure shall also be designed for a point loading of 8.0kN acting on a 100mm x 100mm square applied anywhere on the accessible areas. The cover concrete to embedded ducts and covers to ducts formed in footpaths and cycle tracks shall be designed to withstand this loading.

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:
3.4.15 continued

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

\[ f = \frac{\pi}{2L^2} \sqrt{\frac{EI}{w}} \text{ Hz} \]

Where:
- \( f \) = natural frequency (Hz)
- \( E \) = modulus of elasticity (N/m²)
- \( I \) = second moment of area of the whole cross-section (m⁴)
- \( g \) = acceleration due to gravity (9.81 m/sec²)
- \( w \) = dead and superimposed dead load per unit length (N/m)
- \( L \) = span (m)

For two and three span continuous bridges, standard solutions are available, e.g. Bridge vibration study. 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.

b. Calculate the amplitude of vibration:

\[ a = k\delta \]

Where:
- \( a \) = Amplitude of vibration
- \( k \) = 0.4 where \( f > 4\)Hz
- \( k \) = 0.75 where \( f < 4\)Hz
- \( \delta \) = deflection at midspan due to static design live load

In calculating \( \delta \), 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 appendix B contained within BD 37 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 NA to BS EN 1991-2 UK National Annex to Eurocode 1. Actions on structures part 2 Traffic loads on bridges.

For pedestrian and cycle bridges with spans exceeding 30m, where aerodynamic effects may be critical, wind vibration effects as detailed in BD 49 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, 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 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 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 considered as specified in 3.4.4.

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.

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 The safe system approach. 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 3C given in table 3.2 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.
3.4.18 continued

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 (AFHPB) in accordance with clause 3.6.5.1 of AASHTO LRFD Bridge design specifications(15)); or
- 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 3.1, 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
- 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 plates shall be galvanized, and if exposed to view, 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:

- they are associated with bridges
- 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:

- retaining walls associated with bridges that are located behind traffic barriers meeting performance level 5 or higher, as set out in appendix B
- retaining walls not associated with bridges that are located behind traffic barriers meeting performance level 4 or higher, as set out in appendix B
- 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:

- Load acting in the vertical plane perpendicular to the bridge carriageway’s longitudinal alignment: 375kN.
- 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 vehicle 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:

- **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\(^{(16)}\) clauses 6.3.15 to 6.3.17.

- **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.

- **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.

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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 (i.e., 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:

- **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 3.1, 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\(^{(17)}\)).

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.

- **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.
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\textsuperscript{(17)}. 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\textsuperscript{(17)}).

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.
3.5 Combination of load effects

The effects of the loads described in 3.2 to 3.4 shall be combined by summatting each load effect multiplied by the relevant load factors shown in tables 3.1 and 3.2, and as specified below:

a. In any combination, if a worse effect is obtained by omitting one or more of the transient items, this case shall be considered. Similarly the case of any ‘permanent’ load that is not always present (eg superimposed dead load, shrinkage and creep or settlement that are not initially present) shall be considered if a worse effect is obtained in addition to the reduced load effect noted in table 3.2.

b. The required wind and 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 wind and seismic resistance shall all be taken into account. The load components of combinations 5A and 5C shall give adequate protection in the circumstances being considered.

c. The load combinations specified cover general conditions. Provision shall also be made for other loads where these might be critical.

d. 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 captured by tables 3.1 and 3.2 and reference shall be made to 5.1.3 and 5.7.1(b). The design of elastomeric bearings for load combination 3A of table 3.1 shall comply with 4.7.2 for the level of earthquake loading and displacements specified therein. The design for load combinations that include earthquake loading shall take into account the additional requirements specified in section 5 for seismic design that are not applicable to design for non-seismic load combinations.
Table 3.1: Load combinations and load factors for the serviceability limit state

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Notes: 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.
† Ordinary water pressure and buoyancy to be taken as due to the flow with an ARI of 1 year other than for load combination 3A (seismic) where it shall be taken as due to mean daily flow conditions.
Table 3.2: Load combinations and load factors for the ultimate limit state

<table>
<thead>
<tr>
<th>Combination</th>
<th>Load symbol</th>
<th>Primary normal traffic cases</th>
<th>Secondary normal traffic cases</th>
<th>Primary lateral load cases</th>
<th>Traffic overload case</th>
<th>Construction cases</th>
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<tbody>
<tr>
<td>1A</td>
<td>BL</td>
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<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
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<td>EL</td>
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<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
<td>1.35 (1.00)</td>
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<td>1.35 (1.00)</td>
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<td>1.35 (1.00)</td>
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<td>1.35 (1.00)</td>
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</tbody>
</table>

Notes: Where the effect of a possible reduction in a permanent load is critical, use of the lower bracketed load factors shall be considered.
- 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.
- γp shall be as defined in 3.4.8(a).
- Ordinary water pressure and buoyancy to be taken as due to the flow with an ARI of 1 year other than for load combination 3A (seismic) where it shall be taken as due to mean daily flow conditions.
- Combination 3D applies only to the design of footbridges.
3.6 References


(3) Standards Australia AS 5100.6-2004 Bridge design. Part 6 Steel and composite construction. Superseded.


(10) Standards Australia AS 5100.1:2017 Bridge design. Part 1 Scope and general principles.


