7.0 Evaluation of bridges and culverts

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</table>
7.1 Introduction

7.1.1 General

a. Objective

The objective of evaluation of an existing bridge, culvert, stock underpass or subway is to obtain parameters which define its load carrying capacity. Two parameters are required – one for main members and one for the deck.

The overall procedure is summarised in 7.1.5. The process shall take account of the actual condition of the structure and the characteristics of the traffic and other loads. If at some future date any of the conditions change significantly, the structure shall be re-evaluated accordingly.

b. Rating and posting

Evaluation may be carried out at four load levels (see definitions in 7.1.2):

- Rating evaluation
  
  Rating parameters define the structure’s capacity using overload load factors or stress levels that are appropriate for overweight vehicles.

- Posting evaluation
  
  Posting parameters define the structure’s capacity using live load factors or stress levels that are appropriate for Class 1 conforming vehicles.

- HPMV evaluation
  
  HPMV evaluation defines the structure’s capacity under the effects of high-productivity motor vehicle (HPMV) conforming vehicles using the same live load factors or stress levels as posting.

- 50MAX evaluation
  
  50MAX evaluation defines the structure’s capacity under the effects of 50MAX conforming vehicles using the same live load factors or stress levels as posting.

Because much of the procedure is identical for these types of evaluation, the criteria are presented together and where appropriate, the different procedures are set out side by side on the page.

c. Culverts, stock underpasses and subways

Culverts, stock underpasses and subways shall be treated on the same basis as bridges (with generally no distinction being made in this section 7), except that further evaluation of a culvert stock underpass or subway is not required, provided the following apply:

- it has a span less than 2m, and
- it has more than 1m of fill over it, and
- it is undamaged, and
- there are no unusual circumstances.

For most culverts, stock underpasses and subways, evaluation of the top slab as a deck will be sufficient.

7.1.2 Definitions

Class 1 conforming vehicle: A vehicle that is loaded to the general mass limits set out for heavy motor vehicles in part A of schedule 2 in the Land Transport Rule: Vehicle Dimensions and Mass 2002\(^1\), and is thus able to travel on Class 1 roads as defined in section 3: Classification of roads of the Heavy Motor Vehicle Regulations 1974\(^2\) without restriction.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>50MAX conforming vehicle</td>
<td>A proforma vehicle that is loaded to the general mass limits set out for heavy motor vehicles in part A of schedule 2 in the Land Transport Rule: Vehicle Dimensions and Mass 2002, but with table 6 thereof amended to allow a vehicle mass varying linearly between 44,000kg at 16.5m wheelbase to a maximum of 50,000kg at 20.0m wheelbase. This is a variant high-productivity motor vehicle.</td>
</tr>
</tbody>
</table>
| 50MAX evaluation load | A load consisting of 50MAX conforming vehicles in some or all load lanes on the bridge, taken to be:  
- for loaded lengths up to and including 25m: 0.85HN, including dynamic load factors, and  
- for loaded lengths greater than 25m: 0.90HN, including dynamic load factors.  
See 7.4.4 for further details. |
| HPMV conforming vehicle | A vehicle carrying a divisible load that is loaded to the mass limits set out for high-productivity motor vehicles (HPMVs) in part B of schedule 2 in the Land Transport Rule: Vehicle Dimensions and Mass 2002. |
| HPMV evaluation load | A load consisting of HPMV conforming vehicles in some or all load lanes on the bridge, taken to be:  
- for loaded lengths up to and including 25m: 0.90HN, including dynamic load factors, and  
- for loaded lengths greater than 25m: 0.95HN, including dynamic load factors.  
See 7.4.4 for further details. |
| Live load capacity | The section capacity, in terms of the net unfactored service load, of a critical member or group of members at load factors, or stress limits appropriate to conforming vehicles. See 7.4.2. |
| Load lane | Lanes used for the positioning of elements of live loading on the bridge. The number of load lanes shall generally equal the number of marked lanes on the bridge. See 7.4.4 for further details. |
| Loaded length | The length over which loads may be applied. See 7.4.4 for further details. |
| Overload capacity | The section capacity, in terms of the net unfactored service load, of a critical member or group of members at load factors, or stress limits appropriate to overweight vehicles. See 7.4.2. |
| Overweight vehicle | A vehicle carrying an indivisible load that exceeds the load limits set out in the Land Transport Rule: Vehicle Dimensions and Mass 2002 and therefore requires an overweight permit. |
| Posting | The proportion of the Class 1 posting load which the bridge can withstand under live load criteria. It is expressed as a percentage of Class 1 for main members and as a specific axle load for decks. |
| Posting load | A load consisting of Class 1 conforming vehicles in some or all load lanes on the bridge, taken to be 0.85HN, including dynamic load factors. See 7.4.4 for further details. |
7.1.2 continued

Rating: The proportion of the rating load which the bridge can withstand under overload criteria. It is expressed as a percentage, defined as the class for main members, and an alphabetic symbol defined as the grade for decks.

Rating load: A load consisting of one lane containing an overweight vehicle loaded to the maximum which would be allowed to cross a Class 100 Grade A bridge unsupervised, as set out in the Overweight permit manual\(^3\) (taken as 0.85HO), plus, where critical, some or all other load lanes on the bridge loaded with HPMV evaluation load including dynamic load factors. See 7.4.4 for further details.

7.1.3 Rating requirements

a. These requirements apply to all bridges, major culverts (greater than 3.4m\(^2\) waterway), stock underpasses and subways on roads controlled by authorities participating in the NZ Transport Agency’s (NZTA) policy for overweight permits as set out in the Overweight permit manual\(^3\). This requires an inventory of structural capacity for overload to be maintained for each of these structures. This is expressed as the rating, defined in 7.1.2. By comparing a specific overweight vehicle with the rating load, and use of the structure rating, an estimate of the effect of the vehicle on the structure can be made, as described in the Overweight permit manual\(^3\).

In the case of state highways and some of the major alternative routes, the inventory is in the form of basic moment and shear, or other capacities of bridge members stored in the overweight permit system (OPermit)\(^4\). This enables the effects of a specific overweight vehicle on any bridge to be determined more accurately than by use of the rating alone.

b. The procedures set out in section 7 are intended to be used for existing bridges which require evaluation. New bridges designed to HN-HO-72, and fully complying with the design requirements of this document, also require rating and the methods could be used for this. However, unless rating information is readily available, or there are unusual circumstances, all new bridges shall be evaluated on their design capacities. Since the rating load is 0.85 times the design load, the class is 100/0.85 = (say) 120%, and the grade is A. Capacities entered into OPermit should be the design values of HO or HO + HN moment, shear or other parameters as appropriate, with dynamic load factors and eccentricity.

7.1.4 Posting requirements

If a bridge has insufficient capacity to sustain loads up to the maximum allowed for heavy motor vehicles by the general mass limits specified in part A of schedule 2 of the Land Transport Rule: Vehicle Dimensions and Mass 2002\(^1\) at normal live load factors or stress levels, or at higher stress levels as permitted by 7.4.3, it is required to be posted with a notice showing its allowable load, or posting, as defined in 7.1.2.

Posting of a bridge shall comply with section 11: Protection of bridges of the Heavy Motor Vehicle Regulations 1974\(^2\).

7.1.5 Evaluation procedure

The steps necessary for a full evaluation, either for rating or posting, are shown in table 7.1. Details of each step will be found in the clauses referenced.

The evaluation of bridges for their capacity for HPMVs and 50MAX vehicles shall adopt the same procedure as for a posting evaluation.
### Table 7.1: Evaluation procedure

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Carry out site inspection (7.2.1).</td>
</tr>
<tr>
<td>2</td>
<td>Determine appropriate material strengths (7.3).</td>
</tr>
<tr>
<td>3</td>
<td>Identify critical section(s) of the main supporting members and the critical effect(s) on them (7.4.1).</td>
</tr>
<tr>
<td>4</td>
<td>Determine the overload capacity and/or the live load capacity at each critical main member section (7.4.2).</td>
</tr>
</tbody>
</table>
| 5    | If rating is being done manually:  
  - Analyse the structure for effects of rating or posting load at each critical section (7.4.4).  
If data is to be entered into OPermit:  
  - Follow the requirements for main member element data in the OPermit bridge structural data guide (7.4.7). |
| 6    | Determine rating or posting (7.4.6). |
| 7    | Concrete deck:  
  - Determine if the criteria for empirical design based on assumed membrane action are satisfied (7.5.2).  
  - Determine if the simplified evaluation method is applicable (7.5.3(a)). |
| 8    | Timber deck  
  - If simplified method is applicable:  
    - determine ultimate wheel load (7.5.3(b)).  
  - If simplified method is not applicable:  
    - determine section capacity per unit width at critical locations in slab (7.5.4(a)).  
  - Determine section capacity of the nominal width of deck considered to carry one axle (7.5.5(a)). |
| 9    | Analyse the deck for rating or posting loads (7.5.4(b)).  
  - Determine moments due to rating or posting axle loads (7.5.5(b)). |
| 10   | Determine deck capacity factor (DCF) and/or allowable axle load. (7.5.3(c)) (7.5.4(c)) (7.5.5(c)) |
| 11   | If data is to be entered into OPermit, follow the requirements for deck element data in the OPermit bridge structural data guide (7.5.7). |

### 7.2 Inspection and dynamic load factors

#### 7.2.1 Inspection

Appropriate inspection shall be carried out as a part of the evaluation of the load carrying capacity of any bridge. This is required to determine member condition and to verify dimensions. Where necessary, the extent of corrosion or decay shall be determined by physical measurement.

The following significant characteristics of the carriageway and traffic shall be assessed:

- position of lane markings
- roughness of deck and approaches
- mean speed of heavy traffic
- heavy traffic type and proportion of the total vehicle count.

#### 7.2.2 Dynamic load factors

Appropriate dynamic load factors shall be determined for the various bridge members. Each value shall be:

i. either the design value from 3.2.5 or in the case of timber elements from 4.4.2, or

ii. a value derived from site measurements.

A measured value shall be used if the design value is considered to be unrealistic.
7.2.2 continued

Dynamic measurements shall be made under heavy loads which are representative of actual traffic, in terms of both mass and speed, at either rating load level or posting load level or both. A sufficient number of vehicles shall be included to give confidence in the statistical values chosen.

The dynamic load values derived shall be those which are exceeded by less than 5% of vehicles in either category.

For posting, HPMV and 50MAX evaluation, a reduced dynamic load factor may be used in the following instances:

- NZ Transport Agency state highways - as per posted speed limit
- other roads - as per posted speed limit, or as specified within the 50MAX or HPMV permit where the vehicle speed is restricted.

The dynamic load factor may be reduced as follows:

<table>
<thead>
<tr>
<th>Speed</th>
<th>Dynamic load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>30km/h</td>
<td>(I - 1) x 0.67 + 1</td>
</tr>
<tr>
<td>10km/h</td>
<td>(I - 1) x 0.33 + 1</td>
</tr>
</tbody>
</table>

Where I is the dynamic load factor appropriate for unrestricted heavy traffic.

7.3 Material strengths

Material strengths for calculation of section capacity shall be determined as described below. The strengths used shall be characteristic values as defined in the relevant material code, or as determined in 7.3.6. Where testing is undertaken a laboratory with IANZ accreditation for the test being undertaken or other appropriate agency shall be used. The basis of the material strengths used for determining section capacity shall be clearly stated in the evaluation calculations or any accompanying report.

7.3.1 Concrete

Concrete compressive strength shall be determined by one of the following methods:

a. From drawings, specification or other construction records.

b. From the following nominal historical values:

<table>
<thead>
<tr>
<th>Construction date</th>
<th>Concrete type</th>
<th>Specified strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 1932</td>
<td>Reinforced</td>
<td>14</td>
</tr>
<tr>
<td>1933 to 1940</td>
<td>Reinforced</td>
<td>17</td>
</tr>
<tr>
<td>1941 to 1970</td>
<td>Reinforced</td>
<td>21</td>
</tr>
<tr>
<td>1971 and later</td>
<td>Reinforced</td>
<td>25</td>
</tr>
<tr>
<td>1953 and later</td>
<td>Prestressed</td>
<td>34</td>
</tr>
</tbody>
</table>

c. From cores cut from the bridge.

Cores shall be taken from areas of low stress, in the members being analysed, and so as to avoid reinforcing and prestressing steel. Cutting and testing shall be in accordance with NZS 3112.2 Methods of test for concrete part 2 Tests relating to the determination of strength of concrete(5).
7.3.1 continued

Where core tests are carried out, the statistical analysis described in 7.3.6 shall be applied to determine the compressive strength value to be used in calculations.

7.3.2 Steel reinforcement

The characteristic yield strength of reinforcement shall be determined by one of the following methods. It should be noted that if the steel is of unusually high strength, sections may in fact be over-reinforced and the restriction referred to in 7.4.5(a) shall apply:

a. From drawings, specification or other construction records.

b. From the following nominal historical values:

<table>
<thead>
<tr>
<th>Construction date</th>
<th>Characteristic yield strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 1932</td>
<td>210</td>
</tr>
<tr>
<td>1933 to 1966</td>
<td>250</td>
</tr>
<tr>
<td>1967 and later</td>
<td>275</td>
</tr>
</tbody>
</table>

c. From tensile tests of bar samples of appropriate diameter removed from the bridge members being analysed. Testing shall be in accordance with BS EN ISO 6892-1

Metallic materials Tensile testing part 1 Method of test at ambient temperature\(^6\).

d. From non-destructive tests of bars of appropriate diameter in situ, after removal of cover concrete. The method used shall have been authenticated by correlation with tests in accordance with BS EN ISO 6892-1\(^6\).

Test locations shall be on the members being analysed, chosen so as to be unaffected by bends or welded splices in bars.

Where testing is performed as in (c) or (d), the statistical analysis described in 7.3.6 shall be applied to determine the characteristic value to be used in calculations. A separate analysis shall be performed for each bar diameter.

7.3.3 Prestressing steel

The characteristic yield strength or the 0.2\% proof stress of prestressing steel shall be determined by one of the following methods:

a. From drawings, specification or other construction records.

b. From the lowest alternative value specified in BS 5896 Specification for high tensile steel wire and strand for the prestressing of concrete\(^7\) for the wire or strand diameter.

7.3.4 Structural steel

The characteristic yield strength of structural steel shall be determined by one of the following methods:

a. From drawings, specification or other construction records.

b. From the following nominal historical values:

<table>
<thead>
<tr>
<th>Construction date</th>
<th>Characteristic yield strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 1940</td>
<td>210</td>
</tr>
<tr>
<td>1941 and later</td>
<td>230</td>
</tr>
</tbody>
</table>

c. From tensile tests of coupons removed from the members being analysed, in areas of low stress. Testing shall be in accordance with BS EN ISO 6892-1\(^6\).

d. From non-destructive tests of the steel in situ.
7.3.4 continued
Where testing is performed as in (c) or (d), the statistical analysis described in 7.3.6 shall be applied to determine the characteristic value to be used in calculations.

7.3.5 Timber
Characteristic stresses shall be in accordance with NZS 3603 *Timber structures standard* (8), or where applicable, AS 1720.2 *Timber structures part 2 Timber properties* (9) and AS/NZS 2878 *Timber – Classification into strength groups* (10). Where the species of timber is unknown, it may be determined by removing 10mm diameter core samples from the bridge and submitting them for expert analysis.

Characteristic stresses shall be based either on the lowest grading of any member in the bridge, or on the actual grading of each timber member, according to the visual grading rules of NZS 3631 *New Zealand timber grading rules* (11) or where applicable, AS 3818.6 *Timber – Heavy structural products – Visually graded part 6 Decking for wharves and bridges* (12), AS 3818.7 *Timber – Heavy structural products – Visually graded part 7 Large cross-section sawn hardwood engineering timbers* (13) or AS 2858 *Timber – Softwood – Visually stress-graded for structural purposes* (14). The moisture content shall be determined from core samples cut from the bridge.

Characteristic stress/strength modification factors shall comply with the applicable standard NZS 3603 (8) or AS 1720.1 *Timber structures part 1 Design methods* (15), except as modified by 4.4.2.

Determination of design stresses for timber is discussed in *Strength and durability of timber bridges* (16).

7.3.6 Analysis of test results
In order to obtain characteristic strength values for calculation purposes, results of steel and concrete tests shall be analysed statistically. Each test result shall be the mean of tests on at least two samples taken from one location in the structure or the mean of two (or more as required by specific test procedures) non-destructive tests from one location on a bar or member. For analysis, a group of test results shall originate from similar members or from identical bar diameters as appropriate. Tests shall be taken at sufficient locations to ensure that results are representative of the whole structure, or the entire group of similar members, as appropriate.

When assessing how representative the test results are, consideration should be given to the spread and amount of sampling across the structural members being considered, and should take into account the possibility that materials in different spans may have been produced in different batches. Where possible, non-destructive testing should be carried out on the most critical members.

a. Estimating characteristic strength of materials functioning individually

An acceptable method of analysis to determine the characteristic strength of materials acting individually, such as concrete compressive strength, or the yield strength of individual reinforcing bars, is:

\[ f_{\text{individual}} = \overline{x} - ks \]

Where:

- \( f_{\text{individual}} \) = the characteristic strength of the individual material
- \( \overline{x} \) = the mean of the group of test results
- \( k \) = a one-sided tolerance limit factor
- \( s \) = the standard deviation of the test results

\( k \) shall be determined on the basis that at least a proportion \( (P) \) of the population will be greater than the value calculated, with a confidence \( (\alpha) \).
Values of \( k \) for various values of \((P)\), \((\alpha)\) and \((n)\) the number of test results, are given in table 7.2.

It is recommended that for structural and reinforcing steel, \((P)\) and \((\alpha)\) should both be 0.95 and that for concrete, \((P)\) and \((\alpha)\) should both be 0.90.

b. Estimating characteristic strength of a group of reinforcing bars

This methodology is based on the principle that the average strength of a group of bars has a lower standard deviation than the strength of an individual bar. It may be suitable for reinforcing bars functioning as a group, such as tensile reinforcement located within a reinforced concrete beam. It is reliant upon a small amount of ductility within the reinforcement, as individual bars may reach yield strength prior to the characteristic strength of the group of bars being reached.

\[
f_{\text{group}} = \bar{X} - \frac{ks}{\sqrt{N}}
\]

Where:

- \( f_{\text{group}} \) = the characteristic yield strength (stress) of the group (MPa)
- \( \bar{X} \) = the mean yield strength (stress) of a series of tests (MPa)
- \( k \) = a one-sided tolerance limit factor
- \( s \) = the sample standard deviation of yield strength from the series of tests
- \( N \) = the number of bars functioning as a group (ie in tension) at the location of the member being assessed

Values of \( k \) for various values of \((P)\), \((\alpha)\) and \((n)\) the number of test results, are given in table 7.2. The values of \((P)\) and \((\alpha)\) shall be in accordance with method (a).

This approach may not be suitable for shear reinforcement where the number of individual bars contributing to shear resistance at a section is likely to be small, and the assumption of independence of the reinforcing bars may not be appropriate.

The application of this approach to specific strength evaluations requires the professional judgement of a suitably experienced structural engineer, and must be considered on a case-by-case basis. In applying this approach the engineer shall be satisfied that tests have been taken at sufficient locations to represent the member being evaluated, or the entire group of similar members, as appropriate, including making due allowance for any anomalies in the test results and any significant variations between different members. Where these conditions cannot be satisfied, method (a) shall be used.

The background to this approach is provided in addendum 7A.
Table 7.2: One-sided tolerance limit factors for a normal distribution

<table>
<thead>
<tr>
<th>P</th>
<th>Values of k for α = 0.90</th>
<th>Values of k for α = 0.95</th>
</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>0.900</td>
<td>0.950</td>
</tr>
<tr>
<td>7</td>
<td>2.333</td>
<td>2.894</td>
</tr>
<tr>
<td>8</td>
<td>2.219</td>
<td>2.755</td>
</tr>
<tr>
<td>9</td>
<td>2.133</td>
<td>2.649</td>
</tr>
<tr>
<td>10</td>
<td>2.065</td>
<td>2.568</td>
</tr>
<tr>
<td>11</td>
<td>2.012</td>
<td>2.503</td>
</tr>
<tr>
<td>13</td>
<td>1.928</td>
<td>2.403</td>
</tr>
<tr>
<td>15</td>
<td>1.866</td>
<td>2.329</td>
</tr>
<tr>
<td>16</td>
<td>1.842</td>
<td>2.299</td>
</tr>
<tr>
<td>17</td>
<td>1.820</td>
<td>2.272</td>
</tr>
<tr>
<td>19</td>
<td>1.781</td>
<td>2.228</td>
</tr>
<tr>
<td>20</td>
<td>1.765</td>
<td>2.208</td>
</tr>
<tr>
<td>21</td>
<td>1.750</td>
<td>2.190</td>
</tr>
<tr>
<td>22</td>
<td>1.736</td>
<td>2.174</td>
</tr>
<tr>
<td>23</td>
<td>1.724</td>
<td>2.159</td>
</tr>
<tr>
<td>24</td>
<td>1.712</td>
<td>2.145</td>
</tr>
<tr>
<td>30</td>
<td>1.657</td>
<td>2.080</td>
</tr>
<tr>
<td>35</td>
<td>1.623</td>
<td>2.041</td>
</tr>
<tr>
<td>40</td>
<td>1.598</td>
<td>2.010</td>
</tr>
<tr>
<td>45</td>
<td>1.577</td>
<td>1.986</td>
</tr>
<tr>
<td>50</td>
<td>1.560</td>
<td>1.965</td>
</tr>
</tbody>
</table>

Adapted from Tables for one-sided statistical tolerance limits\(^1\).
7.4 Main member capacity and evaluation

7.4.1 General

The bridge overload and/or live load capacity shall be determined in terms of the net unfactored service load at the critical section of any member or group of identical members which could be critical under any live loading. The capacity of a member may be in any terms, ie moment, shear, torsion, direct force, bearing or an interaction relationship between any of these.

Assumptions which may be made about the behaviour of specific structures in defined circumstances are set out in 7.4.5.

7.4.2 Section capacity

The gross section capacity shall be calculated using the criteria specified in 4.2 to 4.6 for design.

Where conventional analysis fails to demonstrate adequate shear capacity the use of an alternative less conservative method permitted by clause 7.5.9 of NZS 3101.1&2 Concrete structures standard(18) for the evaluation of shear capacity for concrete elements (eg utilising modified compression field theory or strut and tie analysis) may be considered.

For details of the modified compression field theory approach, refer to CAN/CSA-S6 Canadian highway bridge design code(19). For details of the strut and tie approach, refer to clause 7.5.9 and appendix A of NZS 3101(18).

The measured effects of corrosion or other deterioration shall be taken into account if appropriate.

From the gross section capacity shall be subtracted the dead load effect, and any other effect considered to be significant, all factored as necessary to give the overload capacity or the live load capacity as required.

Load factors for rating, posting, HPMV and 50MAX evaluations at the ultimate limit state (see 7.4.2(a)) shall be taken from tables 7.3 and 7.4.

Other effects to be considered shall be those included in the following load combinations of tables 3.1 and 3.2:

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting, HPMV and 50MAX evaluations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combination 4</td>
<td>Combination 1A or 2A</td>
</tr>
</tbody>
</table>

a. For members for which evaluation at the ultimate limit state (ULS) is appropriate:

\[
R_o = \frac{\phi R_i - \gamma_o(DL) - \Sigma(\gamma(Other\ Effects))}{\gamma_o} \\
R_L = \frac{\phi R_i - \gamma_o(DL) - \Sigma(\gamma(Other\ Effects))}{\gamma_L}
\]

Where:
- \( R_o \) = overload capacity
- \( R_L \) = live load capacity
- \( R_i \) = section strength, using material strength determined from 7.3
- \( \phi \) = strength reduction factor from table 7.5
- \( DL \) = dead load effect
- \( \gamma_o \) = overload load factor from table 7.3
- \( \gamma_L \) = live load factor from table 7.3
- \( \gamma_D \) = dead load factor from table 7.4
- \( \gamma \) = load factor(s) on other effects, taken from table 3.2
b. For prestressed concrete members for which evaluation at the serviceability limit state (SLS) is appropriate:

For rating

\[
R_0 = \left( \frac{\text{Gross capacity at stress } f_o}{Z_o} \right) - (DL) - \left( \frac{\text{Other Effects}}{Z_o} \right)
\]

For posting, HPMV and 50MAX evaluations

\[
R_L = \left( \frac{\text{Gross capacity at stress } f_L}{Z_o} \right) - (DL) - \left( \frac{\text{Other Effects}}{Z_o} \right)
\]

or for members constructed in stages, where section properties vary between stages

\[
R_0 = \left[ f_o - \sum \left( \frac{DL_n}{Z_n} \right) - \sum \left( \frac{\text{Other Effects}}{Z_o} \right) \right] Z_F
\]

\[
R_L = \left[ f_L - \sum \left( \frac{DL_n}{Z_n} \right) - \sum \left( \frac{\text{Other Effects}}{Z_o} \right) \right] Z_F
\]

Where:

- \( f_o \) = allowable stress appropriate to overweight vehicles
- \( f_L \) = allowable stress appropriate to conforming vehicles
- \( DL_n \) = dead load effect for construction stage \( n \)
- \( Z_n \) = section modulus applicable to stage \( n \)
- \( Z_o \) = section modulus applicable to other effects
- \( Z_F \) = section modulus in final condition

If a prestressed concrete member is found to have inadequate capacity under SLS evaluation, the bridge element should be investigated further to determine the likely implications. The requirement for any posting should then be discussed with the road controlling authority (with reference made to the ULS capacity of the bridge).

For the rating evaluation of prestressed concrete members at the serviceability limit state, the permissible stresses and stress range applicable to load combinations including traffic overload on bridges specified in NZS 3101(18) shall not be exceeded. In section 19 of NZS 3101(18) the terminology “frequently repetitive live loading” shall be read to be normal live loading (load type LL) and “infrequent live loading” shall be read to be overload (load type OL).

For the posting, HPMV and 50MAX evaluation of prestressed concrete members at the serviceability limit state, the following criteria shall apply:

- The vehicle load effect shall be taken as that due to 1.35 x load x I (see 7.4.6).
- The permissible stress in compression in concrete due to service loads or normal live load for bridges, specified by NZS 3101(18) shall not be exceeded. This permissible stress may however be increased by 20% for load combinations excluding differential temperature, where a higher permissible stress is already permitted.
- The permissible extreme fibre tensile stresses under service loads specified in NZS 3101(18) shall not be exceeded. Where treated as Class U or T members and the tensile stress is the limiting criterion, the member may be assessed as a cracked (Class C) member.
- The permissible stress range in prestressed and non-prestressed reinforcement due to frequently repetitive live loading specified by NZS 3101(18) may be increased by 20%.
- The maximum allowable crack width specified by 4.2.1(a) assessed in accordance with NZS 3101(18) shall not be exceeded.
For the posting, HPMV and 50MAX evaluation of prestressed concrete bridges satisfying the criteria for adoption of higher stress levels in 7.4.3, with members assessed at the serviceability limit state in accordance with 7.4.2(b), the following criteria apply:

- The vehicle load effect shall be taken as that due to 1.35 x load x I (see 7.4.6).
- Where compression in the concrete is the limiting criterion, $f_L$, the allowable stress in the member, may be taken as 30% greater than the permissible stress in compression of concrete under normal live load for bridges specified by NZS 3101(18) for load combinations excluding differential temperature, and 10% greater for load combinations including differential temperature.
- The permissible stress range in prestressed and non-prestressed reinforcement due to frequently repetitive live loading specified by NZS 3101(18) may be increased by 30%.

### Table 7.3: Rating, posting and HPMV evaluation live load ULS load factors*

| Rating loads | $\gamma_0$ | 1.49 |
| Posting loads | $\gamma_L$ | 1.90 or 1.75** |
| HPMV and 50MAX evaluation loads | $\gamma_L$ | 1.90 or 1.75** |

* In no case shall the load factor on the total of all gravity load effects be less than 1.25.

** 1.75 may be adopted only when the conditions for adopting higher stress levels, as set out in 7.4.3, are satisfied.

### Table 7.4: Dead load ULS load factors ($\gamma_D$)*

| Wearing surface, nominal thickness | 1.40 |
| In situ concrete, nominal sizes | 1.20 |
| Wearing surface, measured thickness | 1.10 |
| In situ concrete, measured dimensions and verified density | 1.10 |
| Factory precast concrete, verified density | 1.10 |
| Structural steel | 1.10 |

* In no case shall the load factor on the total of all gravity load effects be less than 1.25.

### Table 7.5: Strength reduction factors ($\phi$)

<table>
<thead>
<tr>
<th>Superstructure condition</th>
<th>Critical section properties based on:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>construction drawings and assessed sound material</td>
</tr>
<tr>
<td>Elastic analysis method</td>
<td></td>
</tr>
<tr>
<td>Good or fair</td>
<td>1.00$\phi_D$</td>
</tr>
<tr>
<td>Deteriorated</td>
<td>0.80$\phi_D$</td>
</tr>
<tr>
<td>Seriously deteriorated</td>
<td>0.70$\phi_D$</td>
</tr>
</tbody>
</table>

Where $\phi_D$ is the applicable strength reduction factor given by the materials design standard, or for timber given by 4.4.2.
7.4.3 Higher allowable stress levels for Class 1 posting and HPMV and 50MAX evaluations

In the evaluation of bridges for posting when subjected to Class 1 conforming vehicle loading, or for their capacity to sustain HPMV and 50MAX conforming vehicle loading, higher stress levels (ie lower load factors) may be justified where only a small number of bridges are restrictive on an important route. For this approach to be adopted, all of the following criteria shall be met:

i. The bridge must be one of a small number of bridges restricting vehicles on an important route.
ii. The deterioration factors for the bridge shall be accurately assessed. This shall be confirmed by undertaking an initial inspection to assess the condition of the bridge.
iii. The engineer shall be satisfied that the structure has a ductile failure mode.
iv. The accuracy of the bridge structural data shall be confirmed (ie shear and moment capacities and eccentricity values must be confirmed).
v. The bridge shall be inspected at no more than six-monthly intervals to observe any structural deterioration.
vi. The engineer shall be satisfied that early replacement or strengthening is feasible.

The decision to implement a specific inspection programme for a critical bridge to justify higher working stresses shall be discussed with the road controlling authority to ensure that the heavy motor vehicle, HPMV or 50MAX demand for a particular route justifies the cost of regular inspections. This decision is only expected to be made for bridges with a high heavy motor vehicle, HPMV or 50MAX demand, that are one of only a few critical bridges on a route, that are in good condition, and where regular inspections would be relatively easy to undertake.

7.4.4 Live loading and analysis

The bridge shall be considered to be loaded with elements of live loading at their most adverse eccentricity in load lanes defined as follows:

The number of load lanes shall generally equal the number of marked lanes on the bridge. Load lanes shall generally be demarcated by the lane markings, except that shoulders shall be combined with the adjacent marked lanes to form load lanes. Where the combined width of the shoulder and marked lane exceeds 4.5m, a loading arrangement with the edge load lane width reduced to 4.5m, with a commensurate increase in the width of the adjacent load lane, shall also be considered.

For single lane or un-marked bridges, the number of load lanes shall not be less than that determined in accordance with 3.2.3(b), and they shall be of equal width.

Dynamic load factors shall be included, as described in 7.2.2. Reduction factors as specified in 3.2.4 shall be applied to each combination of vehicle loads.

a. A bridge with one load lane shall be loaded as follows:

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
<th>For HPMV evaluation</th>
<th>For 50MAX evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85HO</td>
<td>0.85HN</td>
<td>Up to 25m loaded length 0.90HN</td>
<td>Up to 25m loaded length 0.85HN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Greater than 25m loaded length 0.95HN</td>
<td>Greater than 25m loaded length 0.90HN</td>
</tr>
</tbody>
</table>
7.4.4 continued

b. A bridge with two or more load lanes shall normally be loaded as follows:

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
<th>For HPMV evaluation</th>
<th>For 50MAX evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 25m loaded length</td>
<td>0.85HO in the most adverse lane, together with 0.90HN in some or all other marked lanes, where critical</td>
<td>0.85HN in some or all marked lanes</td>
<td>Up to 25m loaded length</td>
</tr>
<tr>
<td>Greater than 25m loaded length</td>
<td>0.85HO in the most adverse lane, together with 0.95HN in some or all other marked lanes, where critical</td>
<td>Greater than 25m loaded length</td>
<td>0.95HN in some or all marked lanes</td>
</tr>
</tbody>
</table>

For all evaluations, if the case of one lane loaded is more critical, this configuration shall be used.

For loadings in (a) and (b) the loaded length shall be defined as follows:

i. For positive bending moments and end shear, the loaded length is the span length in which the bending moment or shear force is being considered.

ii. For negative moment over interior supports, the loaded length is the average of the adjacent spans.

iii. For reactions, the loaded length is the sum of the adjacent spans.

iv. For transoms, the loaded length is twice the longitudinal spacing of the transoms.

The above definitions shall not apply to 3.2.5 or figure 3.2.

The bridge shall be analysed assuming elastic behaviour to determine the effects of the above loads at the critical locations for which capacities have been determined. Analysis shall take into consideration the relative stiffnesses of the various members, and their end conditions. Stiffness values for reinforced concrete members shall allow for the effects of cracking.

7.4.5 Assumptions for specific structural situations

a. Over-reinforced concrete sections

The intent of clause 9.3.8.1 of NZS 3101(18) shall be complied with. The capacity of a reinforced concrete section shall not be taken as more than that derived using the area of tension steel which would correspond to a distance from the extreme compression fibre to the neutral axis of 0.75\(C_b\).

\(C_b\) is the distance from extreme compression fibre to neutral axis at balanced strain conditions, as defined in clause 7.4.2.8 of NZS 3101(18).

b. Concrete kerbs cast onto a composite deck

Where a kerb has been cast directly onto the deck over its full length and has at least a nominal amount of reinforcing steel connecting it to the deck, and is within the effective flange width of the beam, the moment capacity of the outer beam may be calculated assuming that the kerb is an integral part of it, with the following provisos:

i. The area of concrete in the kerb shall be assumed to be 50% of its actual area, to allow for shear lag effects, unless tests indicate otherwise.

ii. The neutral axis shall not be taken to be above the level of the deck surface.
7.4.5 continued

c. Concrete handrails

No reliance shall be placed on the contribution to longitudinal bending capacity of beams by concrete handrails.

d. Steel beams with non-composite concrete deck

No account shall be taken of such a non-composite deck in determining the bending capacity of the beams, except insofar as it may stiffen the beam top flanges, and thus increase their buckling load. Friction shall not be considered to contribute to composite action, nor to the stiffening of top flanges.

e. Steel beams with timber deck

Effective lateral support of the beam flanges by the deck shall only be assumed if the timber deck fastenings are adequate in number and condition.

f. Continuous or framed-in beams

For beams with full moment continuity between spans, of normal proportions and showing no signs of distress, the following simplified procedure may be followed:

The overall moment capacity of each span may be converted to that of an equivalent simple span by subtracting (algebraically) the midspan positive moment capacity from the mean of the two negative moment capacities at its supports. This will give the overall ordinate of the moment of resistance diagram, and both dead and live load moments may then be calculated as though it were a simple span. This procedure shall not be followed for a short span whose length is less than 60% of an adjacent long span, nor for live load effect on a span adjacent to a free cantilever span. The possibility of uplift at an adjacent support shall be considered.

g. Spans built into abutments

Reinforced concrete T-beam spans built monolithically with their abutments may be considered for treatment as in (f), with the following provisos:

i. If negative moment yield at abutments can be shown to occur at a load greater than 85% of that at which midspan positive moment yield occurs, the working load capacity may be based on the full yield capacity of the section at all locations.

ii. If negative moment yield at abutments occurs at a lesser load than 85% of that at which midspan positive moment yield occurs:

   o either the net unfactored service load capacity may be based on the full yield capacity at the abutments, with a reduced yield capacity at midspan, corresponding to the actual moment when abutment yield occurs, or

   o the net unfactored service load capacity may be calculated assuming zero abutment moment capacity.

In any case, where negative moment capacity is to be relied on, the ability of the abutments to resist the overall negative moments without excessive displacement, either by foundation reaction or by earth pressure, or both shall be assured.

h. Horizontal support restraint

Where the bearings and supports of a beam possess sufficient strength and stiffness horizontally, the horizontal support reaction to live loading may be taken into account where appropriate.
7.4.6 Rating, posting, HPMV and 50MAX evaluations

For each critical location in the bridge the rating, posting, HPMV and 50MAX evaluations shall be calculated as described below. In each of the calculations the denominator shall include the effects of eccentricity of load and of dynamic load factors. \( R_o \) and \( R_L \) are the section capacities calculated as 7.4.2.

If data is to be entered into OPermit, the CLASS calculation is not necessary (see 7.4.7).

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ CLASS = \left( \frac{R_o \times 100}{\text{Rating load effect}} \right)_{\text{min}} ] %</td>
<td>[ GROSS = \left( \frac{R_L \times 100}{\text{Posting load effect}} \right)_{\text{min}} ] %</td>
</tr>
</tbody>
</table>

The minimum value for any member in the bridge, except the deck, shall be recorded in a structural inventory as the CLASS for manual calculations during processing of overweight permits in accordance with the Overweight permit manual\(^{(23)}\). For this purpose, any value of CLASS more than 120% shall be recorded as 120%.

For HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting with HPMV or 50MAX evaluation load effects as applicable replacing posting load effect in the GROSS equation. If the value of GROSS is less than 100% then the bridge is unable to carry full HPMV loading or 50MAX loading. A reduced dynamic load factor may be used in accordance with 7.2.2.

7.4.7 Highway permits data

For all state highway bridges, major culverts, stock underpasses and subways and some local authority structures including bypass routes, the basic rating data described above is stored in OPermit. A description of the form in which the data is required and the calculations which the program performs is contained in OPermit bridge structural data guide\(^{(4)}\).

7.5 Deck capacity and evaluation

7.5.1 General

Evaluation procedures for the following are given in this clause:

- Reinforced concrete decks by empirical design, based on assumed membrane action.
- Reinforced concrete decks by the simplified evaluation method.
- Reinforced concrete decks by elastic plate bending analysis.
- Timber decks.

A reinforced concrete deck panel may be evaluated against the criteria for the empirical design of concrete decks based on membrane action as per 7.5.2.
7.5.1 continued

Otherwise generally, a reinforced concrete deck panel which is supported on four sides should be evaluated by the simplified evaluation method if it meets the criteria listed in 7.5.3(a). All remaining reinforced concrete deck panels should be evaluated by the elastic plate bending analysis method. In addition, reinforced concrete deck slabs shall be evaluated for their punching shear capacity for wheel loads, taking into account deterioration of the bridge deck using the factors in table 7.5.

It shall be assumed that vehicle wheels can be transversely positioned anywhere between the kerbs or guardrails, but generally no closer than the restriction imposed by the 3m wide load lane of HN loading (figure 3.1). Ordinarily, any vehicle wheel loads positioned outside the restriction imposed by the 3m wide load lane of HN loading, such as a wheel located at the outer edge of a carriageway against a kerb, shall be treated as a load combination 4 (overload), using loads in tables 7.7 and 7.8. For narrow bridges where wheel loads will frequently be positioned closer to the kerb or guardrail than represented in figure 3.1, evaluation of load combination 1 (normal traffic) shall be carried out based on the expected range of wheel positions of normal traffic for the specific structure geometry.

7.5.2 Reinforced concrete decks: empirical design based on assumed membrane action

Where the requirements for empirical design based on assumed membrane action in accordance with NZS 3101\(^{18}\) clause 12.8.2 are satisfied, the deck slab shall be considered to have adequate resistance to HN-HO-72 loading.

7.5.3 Reinforced concrete decks: simplified evaluation method

a. Criteria for determining applicability of the simplified evaluation method

The simplified evaluation method takes account of membrane action in the slab, and is based on test results. Evaluation of both composite* and non-composite reinforced concrete deck slab panels may be determined by this method provided the following conditions are satisfied:

- The slab panel shall be supported on all sides by steel or concrete beams, girders or diaphragms.
- Cross-frames or diaphragms shall be continuous between external beams or girders, and the maximum spacing of such cross-frames or diaphragms shall be as follows:
  o Steel I beams and box girders of steel or concrete: 8.0m.
  o Reinforced and prestressed concrete beams: at supports.
- The ratio of span length \((L_s)\) to minimum slab thickness shall not exceed 20. In skew slabs where the reinforcing has been placed parallel with the skew, the skew span, \(L_s / \cos \gamma\) shall be used, where \(\gamma\) = angle of skew.
- The span length \((L_s)\) or \(L_s / \cos \gamma\) shall not exceed 4.5m.
- The concrete compressive strength shall not be less than 20MPa.
- The slab thickness, or for slabs of variable thickness the minimum slab thickness, shall be not less than 150mm.

\(^{18}\) For the purposes of this clause, any steel beam and concrete deck bridge designed compositely (but not necessarily meeting current composite design requirements), or any concrete beams cast monolithically and interconnected with reinforcement with a concrete deck, shall be considered to be composite.

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7.5.3 continued

- There shall be an overhang beyond the centre line of the outside beam of at least 0.80m measured perpendicular to the beam. The overhang shall be of the minimum slab thickness used to determine the span to thickness ratio above. This condition may be considered satisfied if there is an integral continuous concrete kerb or barrier which provides a combined cross-sectional area of slab and kerb or barrier not less than the cross-sectional area of 0.80m of deck slab.

b. Deck strength in terms of wheel load

For rating (HO wheel contact area alternative (b) of figure 3.1 assumed), the unfactored ultimate resistance \( R_i \) of a composite or non-composite deck slab shall be calculated as follows:

\[
R_i = R_d F_q F_c
\]

Where \( R_d \) is taken from figure 7.1 or 7.2, as applicable, for the deck thickness \( d \) and the deck span being considered; \( F_q \) is a correction factor based on the value of reinforcement percentage \( q \) where \( q \) is the average of the lower layer reinforcement percentages at the midspan of the slab, in the two directions in which the reinforcement is placed; and \( F_c \) is a correction factor based on the concrete strength \( f'_c \).

The values of \( F_q \) and \( F_c \) shall be taken from figure 7.1 or 7.2, as applicable, or obtained from those figures by linear interpolation.

For deck thicknesses other than those shown in figures 7.1 and 7.2, the value of \( R_i \) shall be obtained by linear interpolation.

For posting, HPMV and 50MAX evaluations (HN wheel contact area assumed) the value of \( R_i \) obtained shall be multiplied by 0.6.

The dead load and other load effects are ignored in this method.

The “design” strength reduction factor \( \phi_D \) for the simplified evaluation method is 0.5. The strength reduction factor \( \phi \) used for evaluation shall be taken from table 7.6, by multiplying \( \phi_D \) by the appropriate factor. In this table, deck deterioration is quantified by the crack-to-reinforcing ratio (CRR) defined as follows:

\[
CRR = \frac{\text{Total length of visible cracks}}{\text{Total length of bottom reinforcement in both directions}} \times 100
\]

The above lengths shall be measured in a 1.2m square area on the bottom of the slab, central between supports.

c. Rating and posting evaluations

For each type of slab panel in the bridge, the parameters shall be calculated as follows. Rating and posting wheel loads shall be taken from tables 7.7 and 7.8. Dynamic load factor \( I \) shall be as described in 7.2.2. \( \gamma_0 \) and \( \gamma_L \) shall be taken from table 7.3.

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deck capacity factor (DCF)</strong></td>
<td><strong>Allowable axle load (kg)</strong></td>
</tr>
</tbody>
</table>
| \[
\text{DCF} = \frac{\text{Overload wheel load capacity}}{\text{Rating load effect}}_{\text{min}}
\] | \[
\text{Allowable axle load} = \frac{\text{Liveload wheel load capacity}}{\text{Posting load effect}}_{\text{min}} \times 8200
\] |
| \[
= \frac{\phi R_i}{\gamma_0 \times 95 \times I}_{\text{min}}
\] | \[
= \frac{\phi \times (0.6 R_i)}{\gamma_L \times 40 \times I \times 8200}_{\text{min}}
\] |
d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting. If the allowable axle load determined is less than 8800kg for HPMV or 8200kg for 50MAX then the bridge is unable to carry full HPMV or 50MAX loading as applicable.

Further analysis may show that the bridge is able to carry specific full HPMV or 50MAX loads, or limited HPMV or 50MAX loading.

**Table 7.6:** Strength reduction factors ($\phi$) for slabs evaluated by the simplified evaluation method

<table>
<thead>
<tr>
<th>Superstructure condition</th>
<th>Slab section properties based on:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>construction drawings and assessed sound material</td>
</tr>
<tr>
<td>Good or fair (CRR ≤40%)</td>
<td>$0.90\phi_D$</td>
</tr>
<tr>
<td>Deteriorated (CRR = 70%)</td>
<td>$0.60\phi_D$</td>
</tr>
<tr>
<td>Seriously deteriorated (CRR = 100%)</td>
<td>$0.30\phi_D$</td>
</tr>
</tbody>
</table>

**Table 7.7:** Deck rating loads

<table>
<thead>
<tr>
<th>Axle type</th>
<th>Axle load (kN)</th>
<th>Wheel track and contact area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Twin-tyred</td>
<td>105</td>
<td>As for HN axle</td>
</tr>
<tr>
<td>Single-tyred, large tyres</td>
<td>190*</td>
<td>As for HO axle, alternative (b)</td>
</tr>
<tr>
<td>2/8-tyred oscillating axles, spaced 1.0m</td>
<td>133</td>
<td>As for HO axle, alternative (a)</td>
</tr>
</tbody>
</table>

**Table 7.8:** Deck posting, HPMV and 50MAX evaluation loads

<table>
<thead>
<tr>
<th>Axle type</th>
<th>Axle load (kN)</th>
<th>Wheel track and contact area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1 and 50MAX</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Twin-tyred</td>
<td>80*</td>
<td>As for HN axle</td>
</tr>
<tr>
<td>Four-tyred oscillating</td>
<td>93</td>
<td>4/250 x 150mm areas equally spaced within 2500mm overall width</td>
</tr>
<tr>
<td>2/Twin-tyred axles, spaced 1.0m</td>
<td>71</td>
<td>As for HN axle</td>
</tr>
<tr>
<td>HPMV</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Twin-tyred</td>
<td>86*</td>
<td>As for HN axle</td>
</tr>
<tr>
<td>Four-tyred oscillating</td>
<td>93</td>
<td>4/250 x 150mm areas equally spaced within 2500mm overall width</td>
</tr>
<tr>
<td>2/Twin-tyred axles, spaced 1.0m</td>
<td>74</td>
<td>As for HN axle</td>
</tr>
</tbody>
</table>

* Wheel loads from these axles are used for evaluation by the simplified evaluation method in 7.5.3(c).
Figure 7.1: $R_d$ (kN) for composite concrete deck slabs

Figure 7.2: $R_d$ (kN) for non-composite concrete deck slabs

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Third edition, Amendment 1
Effective from September 2014
7.5.4 Reinforced concrete decks: plate bending analysis

a. Section capacity at critical locations

The deck slab live load or overload flexural capacity shall be determined using the methodology described in 7.4.2(a), in moment per unit width at critical locations in the slab. A simplification may be made in the case of a slab which is considered to act as a one-way slab, that is, if it has an aspect ratio of at least 4. Provided it has a positive moment capacity in the long-span direction at least 50% of that in the short-span direction, all moment capacities in the long-span direction may be ignored.

b. Live loading and analysis

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting, HPMV and 50MAX evaluations</th>
</tr>
</thead>
<tbody>
<tr>
<td>The deck shall be considered to be loaded with the most adverse of the axles or axle groups listed in the Overweight permit manual(1), at a vehicle axle index (VAI) of 1.3. The number of loaded axles shall be limited to produce a vehicle gross index (VGI) of up to 1.75. For deck spans up to 3m, these may be reduced to the three alternatives described in table 7.7.</td>
<td>The deck shall be considered to be loaded with the most adverse of the axles or axle sets described in the Land Transport Rule: Vehicle Dimensions and Mass 2002(20).</td>
</tr>
<tr>
<td>For Class 1 and 50MAX vehicles: schedule 2, part A, General mass limits, tables 1 to 6.</td>
<td>For Class 1 and 50MAX vehicles: schedule 2, part B Mass limits for high-productivity motor vehicles, tables 1 to 6.</td>
</tr>
<tr>
<td>For deck spans up to 3m, these may be reduced to the alternatives described in table 7.8.</td>
<td>For deck spans up to 3m, these may be reduced to the alternatives described in table 7.8.</td>
</tr>
</tbody>
</table>

The slab shall be analysed for the loads given in tables 7.7 and/or 7.8 assuming elastic behaviour, and shall be assumed to act as a thin plate in which membrane action is not taken into account. The moment effects of the various loads on the critical locations shall be calculated.

c. Rating and posting evaluations

For each critical location in the slab, the evaluation shall be calculated as described below. In both calculations, the denominator shall include dynamic load factors as in 7.2.2, and the numerator shall be as described in (a). The value of DCF or axle load adopted shall be the minimum for the bridge.

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck capacity factor (DCF)</td>
<td>Allowable axle load (kg)</td>
</tr>
</tbody>
</table>
| \[
| DCF = \left( \frac{Overload \ capacity \ at \ critical \ location}{Rating \ load \ effect} \right)_{\text{min}} \]
| \[
| = \left( \frac{Liveload \ capacity \ at \ critical \ location}{Posting \ load \ effect} \right)_{\text{min}} \times 8200 \]
| The minimum value for the bridge shall be recorded as the DCF for the bridge. | The minimum value for the bridge shall be rounded to the nearest 500kg and if less than 8200kg, shall be recorded after the word AXLES, in panel 1 of the heavy motor vehicle bridge limit sign, shown in diagram R5-9, schedule 1 of the Land Transport Rule: Traffic Control Devices 2004(20). |

d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting. If the allowable axle load determined is less than 8800kg for HPMV or 8200kg for 50MAX, then the bridge is unable to carry full HPMV or 50MAX loading as applicable.
7.5.5 Timber decks

a. Section capacity of nominal width

It is assumed that timber decks generally consist of a plank system spanning transversely between longitudinal main beams. Other systems shall be evaluated using the principles described, varying the details to suit.

Unless data is to be entered into OPPermit (see 7.4.7), the live load or overload moment capacity for timber decks consisting of planks spanning transversely between main beams shall be determined for the nominal width of section considered to carry one axle. The nominal widths given in (i) to (vi) below may be assumed unless investigations indicate other criteria. If the timber deck planks are continuous over two or more spans, the section capacity may be assumed increased by 25%, provided live load moments are calculated on a simple span basis.

Terms are defined as follows:

- **Plank width** is the larger cross-sectional dimension of a deck plank, regardless of its orientation, in metres. It is the actual dimension, not the call dimension.

- **Deck span** is the span of the planks between the centres of areas of bearing, in metres.

- **Contact length** is the dimension, perpendicular to the plank span, of a wheel contact area, and is assumed to be 0.25m.

- **Nominal width**:
  i. For planks laid flat, without running planks at least 50mm thick, the nominal width is equal to the width of a whole number of planks, and is greater than the contact length by not more than one plank width.
  ii. For planks laid flat, with running planks at least 50mm thick, the nominal width is equal to the width of a whole number of planks, and is greater than the contact length by not more than two plank widths.
  iii. For nail laminated deck, with planks on edge, fabricated into baulks with no shear connection between them, the nominal width is: 0.250m + 0.4 x (Plank width) x (Deck span).
  iv. For nail-laminated deck, with planks on edge, end laminations well supported and:
    o fabricated in baulks with shear connection between them by steel dowels or other means, or
    o fabricated in baulks and having running planks over them more than 50mm thick, or
    o fabricated in situ, continuously across the beam span, with no unconnected joints between laminations, the nominal width is: 0.250m + 0.8 x (Plank width) x (Deck span).
  v. For glue-laminated deck, with planks on edge, fabricated in baulks with no shear connection between them, the nominal width is: 0.250m + 1.5 x (Plank width) x (Deck span).
  vi. For glue-laminated deck, with planks on edge, otherwise as for (iv), the nominal width is: 0.250m + 3.0 x (Plank width) x (Deck span).

Dead load may be neglected in the above calculation.

b. Live loading and analysis

The transverse moments due to the various axles described in tables 7.7 and/or 7.8 on the span between beams shall be calculated assuming the deck planks are simply supported.
7.5.5 continued  
c. Rating and posting evaluations

For the nominal width at the midspan section of a timber deck span, the evaluation shall be calculated as described below. In both calculations, the numerator shall be as described in (a).

The value of DCF or axial load adopted shall be the minimum for the bridge.

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck capacity factor (DCF)</td>
<td>Allowable axle load (kg)</td>
</tr>
<tr>
<td>$\frac{\text{Overload capacity of nominal width}}{\text{Rating load effect}}_{\min}$</td>
<td>$\frac{\text{Live-load capacity of nominal width}}{\text{Posting load effect}}_{\min} \times 8200$</td>
</tr>
<tr>
<td>The minimum value for the bridge shall be recorded as the DCF for the bridge.</td>
<td>The minimum value for the bridge shall be rounded to the nearest 500 kg, and if less than 8200 kg, shall be recorded after the word AXLES, in panel 1 of the heavy motor vehicle bridge limit sign, shown in diagram R5-9, schedule 1 of the Land Transport Rule: Traffic Control Devices 2004(20).</td>
</tr>
</tbody>
</table>

d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting. If the allowable axle load determined is less than 8800 kg for HPMV or 8200 kg for 50MAX then the bridge is unable to carry full HPMV or 50MAX loading as applicable.

7.5.6 Deck grade

In 7.5.3(c), 7.5.4(c) and 7.5.5(c) the rating calculation has produced a DCF. For issue of permits by the manual method, the DCF shall be converted to a grade, using the relationship given in table 7.9.

<table>
<thead>
<tr>
<th>Table 7.9: Relationship between DCF and grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCF</td>
</tr>
<tr>
<td>≥ 1.00</td>
</tr>
<tr>
<td>1.00 &gt; DCF ≥ 0.89</td>
</tr>
<tr>
<td>0.89 &gt; DCF ≥ 0.78</td>
</tr>
<tr>
<td>0.78 &gt; DCF ≥ 0.67</td>
</tr>
<tr>
<td>0.67 &gt; DCF</td>
</tr>
</tbody>
</table>

7.5.7 Highway permits data

The statement in 7.4.7 applies but, for decks, only the DCF is required.

7.6 Proof loading

Proof loading may be undertaken in addition to the procedure described in 7.1 to 7.5, either to verify the theoretical findings and assumptions made, or to extend the load limits where the results of the procedure are considered to be not representative of the structure's actual behaviour.

Proof loading shall not be relied on to determine load limits for bridges with features such as those described in 7.6.2(a)(iv) and (v), without either modifying the structure, or multiplying the load factors of 7.4.2 by 1.5.
7.6.1 Preliminary

a. Objective

The objective of proof loading shall be to determine experimentally the safe load limit for either overweight loads or normal loads or both, expressed as defined in 7.4.6, 7.5.3(c), 7.5.4(c) and 7.5.5(c).

b. Scope

These requirements apply to main member spans of all materials up to 30m, and to decks. Proof loading of spans larger than 30m may require additional criteria.

c. Analysis

Before testing of any bridge, adequate analysis shall be performed to determine its likely behaviour, including its failure mode.

d. Personnel

Personnel engaged in proof loading shall be experienced and competent, in order to minimise the risk associated with loading beyond the linear range.

e. Risk

The risk of failure or damage being induced by testing shall be clearly stated to the controlling authority.

7.6.2 Analysis

a. Objectives

The objectives of the analysis shall be:

i. To model the structural behaviour up to yield level.

ii. To assess the amount of redundancy in the structural system and its implications for behaviour.

iii. To determine if the bridge failure mode is likely to be ductile or not.

iv. To identify and evaluate features which would give an apparent enhancement of strength up to proof-load level but which could be followed by sudden failure. Such features may include a non-composite deck as described in 7.4.5(d).

v. To identify and evaluate features which are likely to affect the distribution of loads differently at proof load level and at yield load level, such as a stiff concrete handrail, as described in 7.4.5(c).

b. Evaluation of main members

The bridge shall be analysed for the rating and/or posting load as described in 7.4.4, to determine the load effects at the critical location. It shall also be analysed for the actual test loading configuration proposed to be used. This shall be chosen so that it will produce approximately the same relative effects on critical members as the evaluation loading described in 7.4.4. If there is more than one critical effect to be monitored, the load may need to be applied in more than one place, eg to induce both maximum moment and shear in a beam.

c. Evaluation of decks

Sufficient analysis shall be carried out to determine which of the axle configurations in tables 7.7 or 7.8 is most critical, and the critical load position(s). The likely failure mode(s) shall be determined.
7.6.3 Load application, instrumentation and procedure

a. The nature and magnitude of the proof load, and/or any prior modification of the structure, shall be consistent with the objectives of 7.6.2(a).

b. For evaluation of main members lanes shall be loaded to represent the effects of the evaluation loads described in 7.4.4, including dynamic load factors as in 7.2.2.

For evaluation of decks, contact areas corresponding to the most critical of the axle loads of tables 7.7 or 7.8 shall be loaded, to represent the evaluation load including dynamic load factors.

c. If the failure mode is likely to be non-ductile or there is little redundancy in the structure, a jacking system shall be used to apply the load in preference to gravity because of the added control it gives against inadvertent failure.

d. Appropriate strains, deflections and crack widths shall be recorded and correlated with the applied load. Care shall be taken to eliminate errors due to thermal movement. A plot of critical effect(s) against load shall be monitored to ensure that the limits set in 7.6.4 are not exceeded. The test load shall be applied in approximately equal increments, at least four of which shall lie on the anticipated linear part of the response curve. Critical effects shall be recorded in a consistent manner, immediately after the application of each load increment.

e. During incremental loading, the next increment of load shall not be applied until displacement under the previous increment of load has stabilised. Following application of the final increment of load the total proof load shall be applied for not less than fifteen minutes after the displacement has stabilised.

7.6.4 Load limit criteria

a. Main members

Loading shall not exceed either:

i. the load which, together with dead load effects, produces 80% of the yield load on the critical member, as determined by the analysis of 7.6.2, or

ii. that at which the response of the critical member deflection exceeds the value which would be predicted by linear extrapolation of the initial part of the load/response curve by the following percentage.

<table>
<thead>
<tr>
<th>Member material</th>
<th>Percentage offset</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel</td>
<td>10</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td>15</td>
</tr>
<tr>
<td>Reinforced concrete, composite steel/concrete</td>
<td>20</td>
</tr>
<tr>
<td>Timber</td>
<td>25</td>
</tr>
</tbody>
</table>

b. Decks

Loading shall not exceed either:

i. 80% of the load (on the same contact area) calculated to produce yield in the deck, or

ii. that at which the deck local deflection exceeds a value determined as in (a)(ii) above.
7.6.4 continued

c. Concrete cracking criteria

Under proof loading to establish the safe load limit for normal loads, at the maximum load, critical crack widths of reinforced concrete and prestressed concrete shall be recorded. Also under proof loading to establish the safe load limit for overloads, the crack widths under a level of loading equivalent to normal live load shall be recorded. If such cracks are wider than allowed under 4.2.1(a), then regular inspection shall be instituted, specifically to detect any ongoing deterioration of the cracking and possible corrosion.

7.6.5 Rating, posting, HMPV and 50MAX evaluations

a. Correlation of analysis and test results

The results of testing shall be compared with predicted results from the analysis of 7.6.2. The reasons for major differences between predicted and actual behaviour shall be resolved before adoption of rating or posting parameters based on tests.

b. Main members

Rating and posting parameters shall be calculated as in 7.4.6. In the calculations $R_i$ shall be the calculated effect at the critical location of the maximum applied test load divided by $(0.8 \times \gamma_L)$. $R_o$ shall be the same value divided by $(0.8 \times \gamma_o)$.

Rating, posting, HPMV and 50MAX load effects shall be taken from the analysis of 7.6.2 and shall include dynamic load factors.

c. Decks

Parameters shall be calculated as follows:

<table>
<thead>
<tr>
<th>For rating</th>
<th>For posting</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCF [= \frac{T_o}{0.8 \times \gamma_o \times (Rating \ load) \times I} ]</td>
<td>Allowable axle load (kg) [= \frac{T_L \times 8200}{0.8 \times \gamma_L \times (Posting \ load) \times I} ]</td>
</tr>
</tbody>
</table>

Where $T_o$ and $T_L$ are the maximum applied wheel or axle loads on the contact areas specified in tables 7.7 and 7.8 respectively. Rating and posting loads are the appropriate wheel or axle loads from tables 7.7 and 7.8.

d. HPMV and 50 MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting. If the allowable axle load determined is less than 8800kg for HPMV or 8200kg for 50MAX then the bridge is unable to carry full HPMV or 50MAX loading as applicable.
7.7 References


(7) British Standards Institution BS 5896:2012 High tensile steel wire and strand for the prestressing of concrete. Specification


(9) Standards Australia AS 1720.2-2006 Timber structures. Part 2 Timber properties.

(10) Standards Australia and Standards New Zealand jointly AS/NZS 2878:2000 Timber – Classification into strength groups.


(13) Standards Australia AS 3818.7-2010 Timber – Heavy structural products – Visually graded. Part 7 Large cross-section sawn hardwood engineering timbers.


(17) Lieberman GJ (1957) Tables for one-sided statistical tolerance limits. Technical report no. 34, Applied Mathematics and Statistics Laboratory, Stanford University, California for Office of Naval Research, USA.

(18) Standards New Zealand NZS 3101.1&2:2006 Concrete structures standard.

(19) Canadian Standards Association (2014) S6-14 Canadian highway bridge design code.

Addendum 7A  Guidance note – Estimating the characteristic strength of a group of reinforcing bars

7A.1 Introduction

In 7.3.6(a) it is recommended that the characteristic strength of reinforcing steel, where determined by testing, be the 5% percentile strength and determined with 95% confidence. This approach determines the characteristic yield strength which represents the performance of an individual bar.

In practice, reinforcing bars are rarely loaded individually, and the tensile demands are shared. The probability of two independent events with probability \( P_1 \) and \( P_2 \) occurring simultaneously is \( P_1 \times P_2 \). For two independent bars, the probability of both bars having strength less than the 5th percentile is 0.25% (ie 5% x 5%). This shows that the total strength of two bars is less variable than the strength of individual bars; the combined characteristic strength of the two bars will therefore be greater than twice the characteristic strength of an individual bar.

As a result, in many cases use of a higher characteristic strength representing the performance of a group of bars may be justified.

7A.2 Background

The distribution of yield strength for a particular grade of reinforcement within a structure is generally accepted to be approximated by a normal distribution. The value of strength which less than 5% of test results would fall below is called the characteristic strength. Figure 7A.1 illustrates a standard normal distribution with a mean of zero and a standard deviation of 1; the hatched area represents the 5% probability of falling below the characteristic value.

**Figure 7A.1 Standard normal distribution**

An estimate of the characteristic strength based on a sample of \( n \) test results can be represented by:

\[
F_k = \bar{X} - ks
\]

Where:

\[
\bar{X} = \text{the mean of the sample} = \frac{1}{n} \sum_{i=1}^{n} X_i
\]

\( k = \) the “tolerance limit factor”, which is the number of standard deviations below the mean of the point where the characteristic value lies. This value is sourced from statistical tables. For a normally distributed large population, the value of \( k \) is 1.65. When dealing with a sample, the value of \( k \) increases as the sample size decreases or the desired level of confidence increases.
7A.2 continued

\[ s = \text{ the sample standard deviation which is given by:} \]

\[ s = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (X_i - \bar{X})^2} \]

7A.3 Derivation of "characteristic strength" for a group of bars

If \( X \) and \( Y \) are two independent random variables that are normally distributed the sum of \( X \) and \( Y \) is also normally distributed with mean and variance as follows:

Mean:
\[ \mu_{X+Y} = \mu_X + \mu_Y \]

Variance:
\[ \sigma^2_{X+Y} = \sigma^2_X + \sigma^2_Y \]

Here the symbols \( \mu \) and \( \sigma \) represent the mean and standard deviation of the entire population. Variance is the square of the standard deviation.

If \( N \) bars are resisting tension at a given location in a beam, the probability distribution of the total strength (ie the total yield force) of the group of bars can be defined by:

Mean:
\[ \mu_{Total} = \mu_1 + \mu_2 + \cdots + \mu_N \]

Variance:
\[ \sigma^2_{Total} = \sigma^2_1 + \sigma^2_2 + \cdots + \sigma^2_N \]

If all the bars are taken from the same population \( \mu_1 = \mu_2 = \cdots = \mu \) and \( \sigma_1 = \sigma_2 = \cdots = \sigma \), therefore:

\[ \mu_{Total} = N \mu \]

\[ \sigma_{Total}^2 = N \sigma^2 \text{ and } \sigma_{Total} = \sqrt{N} \sigma \]

If the population parameters are estimated from tests, the applicable mean is \( \bar{X} \) and the standard deviation is \( s \). We therefore have:

\[ F_{Total} = N \bar{X} - ks_{Total} = N \bar{X} - ks\sqrt{N} \]

Where:

\[ F_{Total} = \text{the estimated total characteristic strength (yield force (kN)) of a group of } N \text{ bars of a particular size resisting tension at a given location in a beam} \]

\[ \bar{X} = \text{the mean value of a series of } n \text{ representative tests (kN)} \]

\[ s = \text{the sample standard deviation of that series of } n \text{ tests} \]

\[ k = \text{the "tolerance limit factor" for the specific combination of tolerance limit (eg 5th percentile), sample size (n), confidence limit (eg 95%). This value is sourced from tables such as table 7.2} \]

This equation can be expressed in terms of the characteristic yield strength (stress) of the group (MPa) as follows:

\[ f_{Total} = \frac{F_{Total}}{NA_{bar}} = \frac{\bar{X}_{stress} - ks\sqrt{N}}{NA_{bar}} \]

\[ = \bar{X}_{stress} - \frac{k s_{stress}}{\sqrt{N}} \]

Where:

\[ f_{Total} = \text{the characteristic yield strength (stress) of the group (MPa)} \]

\[ A_{bar} = \text{the cross-sectional area of the reinforcing bar (mm²)} \]

\[ \bar{X}_{stress} = \text{the mean yield strength (stress) of a series of } n \text{ tests (MPa)} \]

\[ s_{stress} = \text{the sample standard deviation of yield strength from that series of } n \text{ tests} \]
7A.4 Application

The application of this approach to specific strength assessments requires the professional judgement of a suitably experienced structural engineer, and must be considered on a case-by-case basis. It is reliant upon a small amount of ductility within the reinforcement, as individual bars will reach yield strength prior to the characteristic strength of the group of bars being reached.

This approach may not be suitable for shear reinforcement where the number of individual bars contributing to shear resistance at a section is likely to be small, and the assumption of independence of the reinforcing bars may not be appropriate.

As described in 7.3.6, in applying this approach the engineer shall also be satisfied that tests have been taken at sufficient locations to represent the whole structure, or the entire group of similar members, as appropriate to the assessment, including making due allowance for any anomalies in the test results.

Where possible, non-destructive sampling (ie hardness testing) should be carried out on the most critical members.