7.0 Evaluation of bridges and culverts

In this section

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.

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 general access 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 if the road controlling authority has granted an exemption, or 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 I vehicle: A vehicle that is able to travel on Class I roads as defined in section 3: *Classification of roads* of the Heavy Motor Vehicle Regulations 1974[1] without restriction. (Note that this is an archaic term but it is still in use for bridge posting limits on older signage.)

7.1.2 continued

General access vehicle:	A vehicle that is loaded to the axle mass and total mass limits set out in parts 1 and 2 respectively of schedule 3 in the Land Transport Rule: Vehicle Dimensions and Mass 2016(2) and is thus able to operate without a permit, subject to any specific route or bridge restrictions.
50MAX conforming vehicle:	A proforma vehicle that is loaded to the axle mass and total mass limits set out for general access vehicles in parts 1 and 2 of schedule 3 in the Land Transport Rule: Vehicle Dimensions and Mass 2016(2), but with table 2.1 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.
50MAX evaluation load:	A load consisting of 50MAX conforming vehicles in some or all load lanes on the bridge, as represented by a set of nominated axle groups and reference vehicles including dynamic load factors. See 7.4.4 for application of load and further details.
HPMV conforming vehicle:	A vehicle carrying a divisible load that is loaded to the axle mass and total mass limits set out for high-productivity motor vehicles (HPMVs) in parts 3 and 4 respectively of schedule 3 in the Land Transport Rule: Vehicle Dimensions and Mass 2016(2).
HPMV evaluation load:	A load consisting of HPMV conforming vehicles in some or all load lanes on the bridge, as represented by a set of nominated axle groups and reference vehicles including dynamic load factors. See 7.4.4 for application of load and 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.
Main member:	A structural member, spanning longitudinally or transversely, other than a bridge deck member (ie slab or plank).
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 2016(2) and therefore requires an overweight permit.
Posting:	The proportion of the general access vehicles posting load which the bridge can withstand under live load criteria. It is expressed as a percentage of general access vehicle mass limits for main members and as specific axle set loads for decks. Also, the process of undertaking a posting evaluation.
Posting load:	A load consisting of general access vehicles in some or all load lanes on the bridge, as represented by a set of nominated axle groups and reference vehicles, including dynamic load factors. See 7.4.4 for application of load and further details.

7.1.2 continued	Rating:	The process of undertaking a rating evaluation.
	Rating load:	A load consisting of one lane containing an overweight vehicle (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. Under the *Overweight permit manual*(3) (withdrawn), the performance of a bridge under this load was reported using the terms Class and Grade. The rating load represented an overweight vehicle loaded to the maximum which would be allowed to cross a Class 100 Grade A bridge unsupervised.
	Specialist vehicle:	A vehicle of certain specialised types carrying a divisible load, as defined in clause 5.11(2) in section 1 of part 1 in the Land Transport Rule: Vehicle Dimensions and Mass 2016(2). A variant high-productivity motor vehicle and evaluated as such.

7.1.3 Rating requirements

a. These requirements apply to all bridges, major culverts (greater than 3.4m² waterway), stock underpasses and subways on roads controlled by authorities participating in the Waka Kotahi NZ Transport Agency policy for overweight permits as set out in the *Vehicle dimensions and mass permitting manual* (volume 1)(4). The rating process requires an inventory of structural capacity for overload to be maintained for each of these structures. This capacity information is used by the Waka Kotahi NZ Transport Agency overweight permit system (OPermit)(5) to compare specific overweight vehicle load effects with the stored structural capacity for overload. A description of the form in which the data is required and the calculations which the program performs is contained in the *OPermit bridge structural data guide*(5).

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 may be evaluated on their design capacities. Capacities entered into OPermit may be derived from the member capacity values resulting from the overload design loading, including dynamic load factors and eccentricity.

7.1.4 Posting requirements

If a bridge has insufficient capacity to sustain axle masses and total load masses up to the maximum allowed for general access vehicles specified in parts 1 and 2 respectively of schedule 3 of the Land Transport Rule: Vehicle Dimensions and Mass 2016(2) 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.

The implementation of posting of a bridge shall comply with 7.6.

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>
<tr><td>Step 1</td><td colspan="3">Carry out site inspection (7.2.1).</td></tr>
<tr><td>Step 2</td><td colspan="3">Determine appropriate material strengths (7.3).</td></tr>
<tr><td>Step 3</td><td colspan="3">Identify critical section(s) of the main supporting members and the critical effect(s) on them (7.4.1).</td></tr>
<tr><td>Step 4</td><td colspan="3">Determine the overload capacity and/or the live load capacity at each critical main member section (7.4.2).</td></tr>
<tr><td>Step 5</td><td colspan="3">Analyse the structure for effects of rating or posting load at each critical section (7.4.4).</td></tr>
<tr><td>Step 6</td><td colspan="3">Determine posting (7.4.6), or for rating follow the requirements for main member element data in the OPermit bridge structural data guide[5].</td></tr>
<tr><td>Step 7</td><td colspan="2">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)).</td><td>Timber deck</td></tr>
<tr><td>Step 8</td><td rowspan="2">If simplified method is applicable:
• determine ultimate wheel load (7.5.3(b)).</td><td>If simplified method is not applicable:
• determine section capacity per unit width at critical locations in slab (7.5.4(a)).</td><td>Determine section capacity of the nominal width of deck considered to carry one axle (7.5.5(a)).</td></tr>
<tr><td>Step 9</td><td>Analyse the deck for rating or posting loads (7.5.4(b)).</td><td>Determine moments due to rating or posting axle loads (7.5.5(b)).</td></tr>
<tr><td rowspan="2">Step 10</td><td colspan="3">Determine deck capacity factor (DCF) and/or allowable axle load.</td></tr>
<tr><td>(7.5.3(c))</td><td>(7.5.4(c))</td><td>(7.5.5(c))</td></tr>
<tr><td>Step 11</td><td colspan="3">If data is to be entered into OPermit, follow the requirements for deck element data in the OPermit bridge structural data guide[5].</td></tr>
</table>

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

A dynamic load factor of $I = 1.30$ shall be used except as follows:

- for timber elements the design value from 4.4.2 shall be used
- a measured value, derived from site measurements, shall be used if the above value is considered to be unrealistic.

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.

7.2.2 continued

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:

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

The dynamic load factor may be reduced as follows:

Speed	Dynamic load factor
30km/h	(I - 1) x 0.67 + 1
10km/h	(I - 1) x 0.33 + 1

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:

Construction date	Concrete type	Specified strength (MPa)
Up to 1932	Reinforced	14
1933 to 1940	Reinforced	17
1941 to 1970	Reinforced	21
1971 and later	Reinforced	25
1953 and later	Prestressed	34

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[(6)].

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:

Construction date	Characteristic yield strength (MPa)
Up to 1932	210
1933 to 1966	250
1967 and later	275

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 room temperature(7).

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(7).

   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 ultimate tensile 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 by the most probably applicable standard specifications for the wire or strand diameter in force at the time of construction of the structure. Strengths specified by historical standard specifications and MWD design manuals are presented in C7.3.3 in the *Bridge manual commentary*.

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:

Construction date	Characteristic yield strength (MPa)
Up to 1940	210
1941 and later	230

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(7).

d. From non-destructive tests of the steel in situ.

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.

Strength reduction factors and characteristic stress/strength modification factors shall comply with the applicable standard NZS 3603[8], AS 5100.9 *Bridge design* part 9 Timber[15] or AS 1720.1 *Timber structures* part 1 Design methods[16], as specified by 4.4.2.

Determination of design stresses for timber is discussed in *Strength and durability of timber bridges*[17].

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:

   $$f_{individual} = \overline{X} - ks$$

   Where:

$f_{individual}$	=	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 (α).

   Values of k for various values of (P), (α) and (n) the number of test results, are given in table 7.2.

7.3.6 continued

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

(Note that further research is being undertaken on the appropriateness of this statistical analysis for all materials. For example, the typically asymmetric nature of concrete strength is not well represented by a normal distribution where the standard deviation and spread of test results is large.)

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.

$$f_{group} = \overline{X} - \frac{ks}{\sqrt{N}}$$

Where:

f_{group}	=	the characteristic yield strength (stress) of the group (MPa)
$\overline{X}$	=	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), (α) and (n) the number of test results, are given in table 7.2. The values of (P) and (α) 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 C7.3.6 in the *Bridge manual commentary*.

Table 7.2: One-sided tolerance limit factors for a normal distribution

Values of k for α = 0.90					Values of k for α = 0.95				
n \ P	0.900	0.950	0.990	0.999	n \ P	0.900	0.950	0.990	0.999
2	10.253	13.090	18.500	24.582	2	20.581	26.260	37.094	49.276
3	4.258	5.310	7.340	9.651	3	6.156	7.655	10.552	13.857
4	3.187	3.957	5.437	7.128	4	4.163	5.145	7.042	9.215
5	2.742	3.400	4.666	6.112	5	3.407	4.202	5.741	7.501
6	2.494	3.091	4.242	5.556	6	3.006	3.707	5.062	6.612
7	2.333	2.894	3.972	5.201	7	2.755	3.399	4.641	6.061
8	2.219	2.755	3.783	4.955	8	2.582	3.188	4.353	5.686
9	2.133	2.649	3.641	4.772	9	2.454	3.031	4.143	5.414
10	2.065	2.568	3.532	4.629	10	2.355	2.911	3.981	5.203
11	2.012	2.503	3.444	4.515	11	2.275	2.815	3.852	5.036
12	1.966	2.448	3.371	4.420	12	2.210	2.736	3.747	4.900
13	1.928	2.403	3.310	4.341	13	2.155	2.670	3.659	4.787
14	1.895	2.363	3.257	4.274	14	2.108	2.614	3.585	4.690
15	1.866	2.329	3.212	4.215	15	2.068	2.566	3.520	4.607
16	1.842	2.299	3.172	4.164	16	2.032	2.523	3.463	4.534
17	1.820	2.272	3.136	4.118	17	2.001	2.486	3.415	4.471
18	1.800	2.249	3.106	4.078	18	1.974	2.453	3.370	4.415
19	1.781	2.228	3.078	4.041	19	1.949	2.423	3.331	4.364
20	1.765	2.208	3.052	4.009	20	1.926	2.396	3.295	4.319
21	1.750	2.190	3.028	3.979	21	1.905	2.371	3.262	4.276
22	1.736	2.174	3.007	3.952	22	1.887	2.350	3.233	4.238
23	1.724	2.159	2.987	3.927	23	1.869	2.329	3.206	4.204
24	1.712	2.145	2.969	3.904	24	1.853	2.309	3.181	4.171
25	1.702	2.132	2.952	3.882	25	1.838	2.292	3.158	4.143
30	1.657	2.080	2.884	3.794	30	1.778	2.220	3.064	4.022
35	1.623	2.041	2.833	3.730	35	1.732	2.166	2.994	3.934
40	1.598	2.010	2.793	3.679	40	1.697	2.126	2.941	3.866
45	1.577	1.986	2.762	3.638	45	1.669	2.092	2.897	3.811
50	1.560	1.965	2.735	3.604	50	1.646	2.065	2.863	3.766

Adapted from *Tables for one-sided statistical tolerance limits*(18).

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*(19) 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*(20). For details of the strut and tie approach, refer to clause 7.5.9 and appendix A of NZS 3101(19).

The Waka Kotahi NZ Transport Agency research report 602 *Evaluation of shear connectors in composite bridges*(21) provides guidance on the evaluation of shear connectors in steel-concrete composite bridge superstructures.

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 3.2 and 3.3:

For rating	For posting, HPMV and 50MAX evaluations
Combination 4A	Combination 1A

Notes: For bridges for which combinations 4A and 1A may not be critical, consideration should be given to utilising the other effects from other combinations together with appropriate traffic live load factors. Combination 1A now includes temperature effects, combination 4A contains temperature effects with higher load factors than previously.

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

For rating	For posting, HPMV and 50MAX evaluations
$R_o = \dfrac{\phi R_i - \gamma_D(DL) - \Sigma(\gamma(Other\ Effects))}{\gamma_o}$	$R_L = \dfrac{\phi R_i - \gamma_D(DL) - \Sigma(\gamma(Other\ Effects))}{\gamma_L}$

Where: R_o = overload capacity

R_L = live load capacity

R_i = section strength, using material strength determined from 7.3

7.4.2 continued

ϕ = strength reduction factor from table 7.5

DL = dead load effect

γ_o = overload load factor from table 7.3

γ_L = live load factor from table 7.3 (differs for reference vehicle and axle group loadings)

γ_D = dead load factor from table 7.4

γ = load factor(s) on other effects, taken from table 3.3

b. For prestressed concrete members for which evaluation at the serviceability limit state (SLS) is appropriate:

For rating	For posting, HPMV and 50MAX evaluations
$R_o = \begin{pmatrix} Gross\ capacity \\ at\ stress\ f_o \end{pmatrix} - (DL) - \begin{pmatrix} Other \\ Effects \end{pmatrix}$	$R_L = \begin{pmatrix} Gross\ capacity \\ at\ stress\ f_L \end{pmatrix} - (DL) - \begin{pmatrix} Other \\ Effects \end{pmatrix}$

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

$R_o = \left[f_o - \Sigma\left(\frac{DL_n}{Z_n}\right) - \Sigma\left(\frac{Other\ Effects}{Z_o}\right)\right] Z_F$	$R_L = \left[f_L - \Sigma\left(\frac{DL_n}{Z_n}\right) - \Sigma\left(\frac{Other\ Effects}{Z_o}\right)\right] Z_F$

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(19) shall not be exceeded. In section 19 of NZS 3101(19) 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 1.30 x load x I, for reference vehicle loading, and 1.35 x load x I, for axle group loading (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(19) 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.

7.4.2 continued

- The permissible extreme fibre tensile stresses under service loads specified in NZS 3101(19) 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(19) may be increased by 20%.
- The maximum allowable crack width specified by 4.2.1(a) assessed in accordance with NZS 3101(19) 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 1.30 x load x I, for reference vehicle loading, and 1.35 x load x I, for axle group loading (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(19) 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(19) may be increased by 30%.

Table 7.3: Rating, posting, HPMV and 50MAX evaluation live load ULS load factors for main members*

Rating loads		γ_o	1.50
Posting loads HPMV and 50MAX evaluation loads	Reference vehicle loading	γ_L	1.80 or 1.65**
	Axle group loading	γ_L	1.90 or 1.75**

* In no case shall the load factor on the total of all gravity load effects (live and dead) be less than 1.25.

** 1.65 or 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 (γ_D)*

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

* In no case shall the load factor on the total of all gravity load effects (live and dead) be less than 1.25.

7.4.2 continued

Table 7.5: Strength reduction factors (ϕ)

Superstructure condition	Critical section properties based on:	
	construction drawings and assessed sound material	measured dimensions or verified as-built drawings, and measured sound material
Good or fair	1.00ϕ_D	1.00ϕ_D
Deteriorated	0.80ϕ_D	0.90ϕ_D
Seriously deteriorated	0.70ϕ_D	0.80ϕ_D

Where ϕ_D is the applicable strength reduction factor given by the materials design standard, or for timber given by 4.4.2.

The values in table 7.5 may be used for an initial assessment, but if a more reliable assessment is required, then the appropriate strength reduction factor shall be assessed based on specific investigation of the particular structural actions and the condition of the relevant materials.

7.4.3 Higher allowable stress levels for general access posting and HPMV and 50MAX evaluations

In the evaluation of bridges for posting when subjected to general access 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, whilst being cognisant of the extent of any disruption created if early replacement was required and the availability of any detour routes.

The decision to implement a specific inspection programme for a critical bridge to justify higher working stresses (or lower load factors from table 7.3) 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 analysed assuming elastic behaviour to determine the effects of the loads defined below 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.

7.4.4 continued

The bridge shall be loaded with elements of rating, general access, 50MAX and HPMV evaluation 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.

For single lane or un-marked bridges greater than 40m in length, or where approaching drivers may not see each other before the bridge, the possibility of passing vehicles shall be considered. Where passing vehicles are considered, both vehicles shall be centred within their available travelling width. Where a low-speed crossing is likely in this instance, a lower dynamic load factor may be appropriate.

Dynamic load factors shall be included, as described in 7.2.2.

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

For rating	For posting	For HPMV evaluation	For 50MAX evaluation
0.85HO	General access evaluation loadings defined in (d)	HPMV evaluation loadings defined in (c)	50MAX and general access evaluation loadings defined in (d)

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

For rating	For posting	For HPMV evaluation	For 50MAX evaluation
0.85HO in the most adverse lane, together with HPMV evaluation loadings in some or all other marked lanes, where critical	General access evaluation loadings defined in (d) in some or all marked lanes	HPMV evaluation loadings defined in (c) in some or all marked lanes	50MAX and general access evaluation loadings defined in (d) in some or all marked lanes

Accompanying lane factors or multiple vehicle factors as specified in (f) shall be applied to each combination of vehicle loads.

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

c. HPMV evaluation loading

The most onerous effects produced by the following reference vehicles shall be taken to represent the effects of HPMV conforming vehicles:

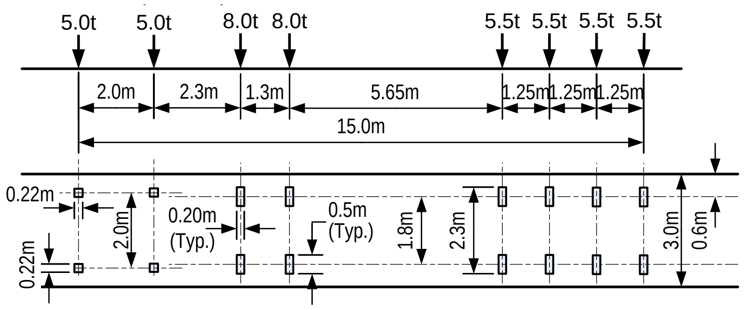
i. 8-axle 48.0 tonne semi-trailer (figure 7.1(i))
ii. 10-axle 55.0 tonne B-train (figure 7.1(ii))
iii. 9-axle 61.0 tonne truck-and-trailer (figure 7.1(iii))

Additionally, for short spans the effects of the HPMV axle groups listed in table 7.9 shall be considered.

7.4.4 continued

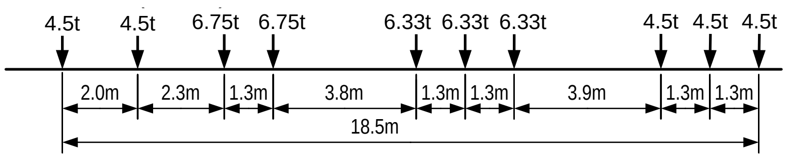
Figure 7.1: HPMV reference vehicles

i. 8-axle semi-trailer 48 tonne, 15m wheelbase (A224-48t)

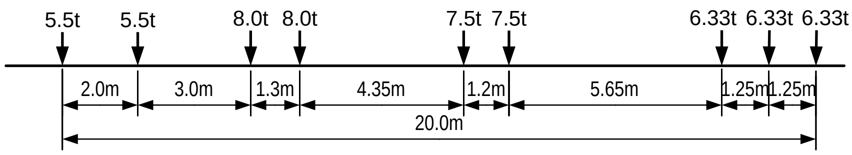


(Note all subsequent reference vehicles are similar in plan for tyre size and position within the load lane, with leading, generally lighter, axle(s) being considered single tyred and remainder double tyred)

ii. 10-axle B-train 55 tonne, 18.5m wheelbase (B2233-55t)



iii. 9-axle truck-and-trailer 61 tonne, 20m wheelbase (R22T23-61t)



d. 50MAX and general access evaluation loading

The most onerous effects produced by the following reference vehicles shall be taken to represent the effects of general access vehicles:

i. 5-axle 32.0 tonne articulated vehicle (figure 7.2(i))
ii. 8-axle 45.0 tonne semi-trailer (figure 7.2(ii))
iii. 8-axle 46.0 tonne B-train (figure 7.2(iii))
iv. 8-axle 46.0 tonne truck-and-trailer (figure 7.2(iv))

Additionally, for short spans the effects of the general access axle groups listed in table 7.9 shall be considered.

For 50MAX evaluations, the following additional reference vehicles are also required:

v. 9-axle 50.0 tonne B-train (figure 7.2(v))
vi. 9-axle 50.0 tonne truck-and-trailer (figure 7.2(vi))

e. HPMV, 50MAX and general access loading positioning

Reference vehicles or axle groups shall be positioned in the most onerous position within the load lane for the section and loading effect under consideration. Axles with relieving effect shall be omitted.

Loadings for simply supported span lengths greater than 36m or for continuous span lengths greater than 20m or for piers supporting combined span length greater than 55m shall consider multiple vehicles from the above lists in the same lane (platoons).

7.4.4 continued

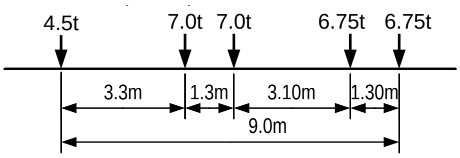
The minimum headway between vehicles shall be 17.0m for general access, 50MAX and HPMV vehicles (i) and (ii), and 19.0m for HPMV vehicle (iii), measured between the rear axle of the front vehicle and the front axle of the rear vehicle. The combination of vehicles and headways shall be chosen so as to produce the worst effect, noting that the above headways are minimum values and longer gaps may be more onerous for continuous spans.

Reduction factors for accompanying vehicles are as specified in (f). Dynamic load factors shall be applied to all vehicles in the platoon.

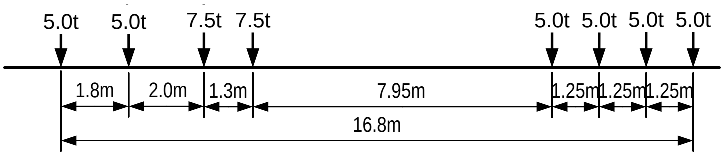
Further guidance on vehicle selections, positioning, and accompanying vehicle factors is provided in the *Bridge manual commentary* (C7.4.4).

Figure 7.2: General access and 50MAX reference vehicles

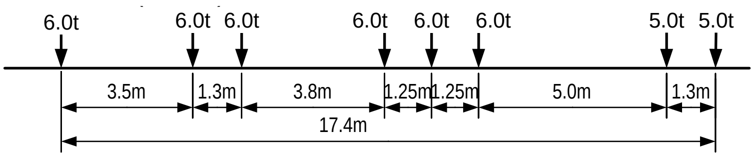
i. 5-axle articulated vehicle 32 tonne, 9m wheelbase (A122-32t)



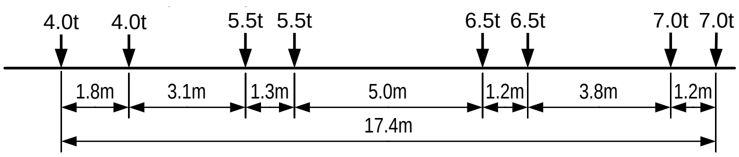
ii. 8-axle semi-trailer 45 tonne, 16.8m wheelbase (A224-45t)



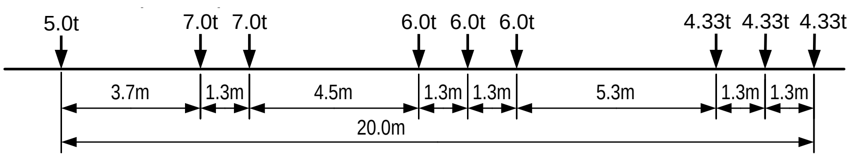
iii. 8-axle B-train 46 tonne, 17.4m wheelbase (B1232-46t)



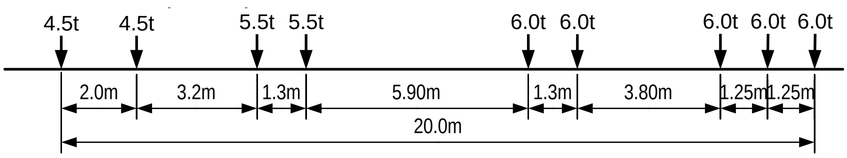
iv. 8-axle truck-and-trailer 46 tonne, 17.4m wheelbase (R22T22-46t)



v. 9-axle B-train 50 tonne, 20m wheelbase (50MAX) (B1233-50t)



vi. 9-axle truck-and-trailer 50 tonne, 20m wheelbase (50MAX) (R22T23-50t)



7.4.4 continued

f. Accompanying lane and multiple vehicle factors

If more than one lane is loaded with HPMV, GA or 50MAX loading, accompanying lane factors shall be applied to the loads or load effects for each lane as follows:

Number of loaded lanes	Accompanying lane factor (ALF_i)
1 lane loaded	1.0
2 lanes loaded	1.0 for first lane; and 0.8 for second lane
3 or more lanes loaded	1.0 for first lane; and 0.8 for second lane; and 0.4 for third and subsequent lanes

In the above, note that lanes are designated as follows for effects on the element under consideration:

First lane: the loaded lane giving the largest effect

Second lane: the loaded lane giving the second largest effect

Third lane: the loaded lane giving the third largest effect, etc.

Where there are multiple vehicles present in the same lane (platoons), the individual vehicles in the lane shall be considered as separate lanes for the purposes of applying appropriate accompanying lane (vehicle) factors. Thus, the accompanying lane factor (ALF) shall be 1.0 for the first vehicle, 0.8 for the second vehicle and 0.4 for the third and subsequent vehicles (where first means the vehicle with largest effect rather than leading vehicle in a platoon, etc).

For more than one loaded lane, the accompanying lane and vehicle factors are both applied (ie the second vehicle in the second lane has a factor of 0.8x0.8=0.64). See table C7.10 in the *Bridge manual commentary*.

Alternatively, where multiple vehicles are present in the same lane and in adjacent lanes, the above accompanying lane factors may be applied to loadings on a per vehicle basis rather than to the load effects determined separately for each lane. Further guidance is provided in the *Bridge manual commentary* (C7.4.4(e)).

For rating, the reduction factor for the 0.85HO load 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).

7.4.5 Assumptions for specific structural situations

a. Over-reinforced concrete sections

The intent of clause 9.3.8.1 of NZS 3101[19] 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 $0.75C_b$.

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[19].

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:

7.4.5 continued

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.

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. Note however that the Waka Kotahi NZ Transport Agency research report 602(21) provides evaluation procedures for partially composite bridges.

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:
   - 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
   - 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.

7.4.5 continued

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.

i. Longitudinal shear capacity at construction joints in reinforced concrete T-beam bridges

The longitudinal shear capacity at deck/beam construction joints on reinforced concrete T-beam bridges should be reviewed, where there is evidence of joint cracking or movement. Poor construction joint quality affecting composite action has been found on various bridges due to contamination with construction debris, a lack of concrete surface preparation, or minimal shear reinforcement across the construction joint.

For all concrete T-beam bridges subject to capacity assessment, a thorough inspection should be undertaken to identify possible non-composite action at the construction joint.

The assessment of longitudinal shear capacity shall be in accordance with Waka Kotahi NZ Transport Agency guidelines if available, or otherwise clause 7.7 of NZS 3101[(19)]. Allowance should be made for the effect of any construction defects, in particular on the friction coefficient. The effect of reduced composite action on other capacities (eg vertical shear and flexure) shall be evaluated.

7.4.6 Posting, HPMV and 50MAX evaluations

For each critical location in the bridge the posting, HPMV and 50MAX evaluations shall be calculated as described below. In each of the calculations the load effect in the denominator shall include the effects of eccentricity of load and of dynamic load factors. R_L is the section capacity calculated as 7.4.2.

As the member overload capacity is entered into OPermit, a similar calculation for rating (known historically as CLASS) is not necessary (see 7.4.7).

For posting	For HPMV and 50MAX evaluations
$GROSS = \left[\dfrac{R_L \times 100}{Posting\ load\ effect}\right]_{\min} \%$ The minimum value for any member in the bridge, except the deck, shall be rounded to the nearest 10%. This value shall be used to define the gross weight limits on the posting weight limit sign. Refer to 7.6.2. If the speed is restricted by inserting a value in panel 3 of the sign, the dynamic load factor may be reduced in accordance with 7.2.2.	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 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, rating data 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*[(5)].

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.

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 4A (overload), using loads in tables 7.8 and 7.9. 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 1A (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[(19)] 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:
     - Steel I beams and box girders of steel or concrete: 8.0m.
     - Reinforced and prestressed concrete beams: at supports.

* 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.

7.5.3 continued

- 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, $L_s/Cos\ Y$ shall be used, where Y = angle of skew.
- The span length (L_s) or $L_s/Cos\ Y$ 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.
- 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 7.3 or 7.4, 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 7.3 or 7.4, as applicable, or obtained from those figures by linear interpolation.

For deck thicknesses other than those shown in figures 7.3 and 7.4, 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 (ϕ_D) for the simplified evaluation method is 0.5. The strength reduction factor (ϕ) used for evaluation shall be taken from table 7.7, by multiplying ϕ_D by the appropriate factor. In this table, deck deterioration is quantified by the crack-to-reinforcing ratio (CRR) defined as follows:

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

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 7.8 and 7.9. Dynamic load factor (I) shall be as described in 7.2.2. γ_o and γ_L shall be taken from table 7.6.

Figure 7.3: R_d (kN) for composite concrete deck slabs

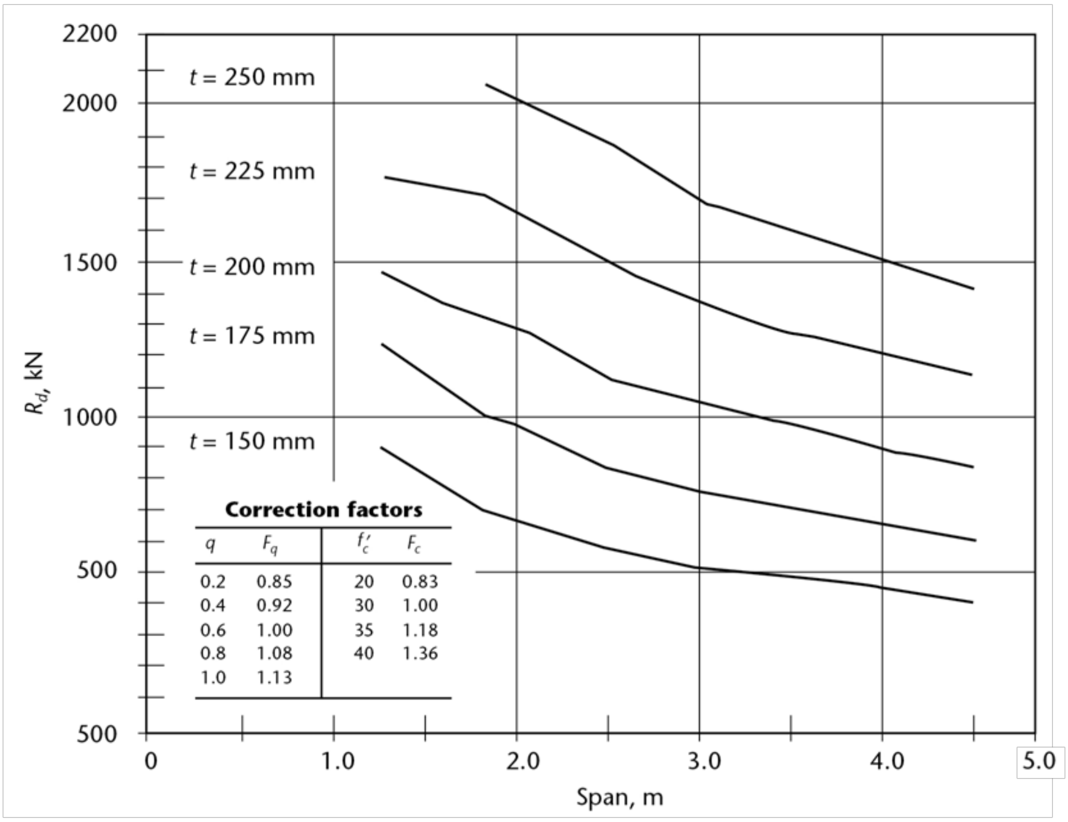
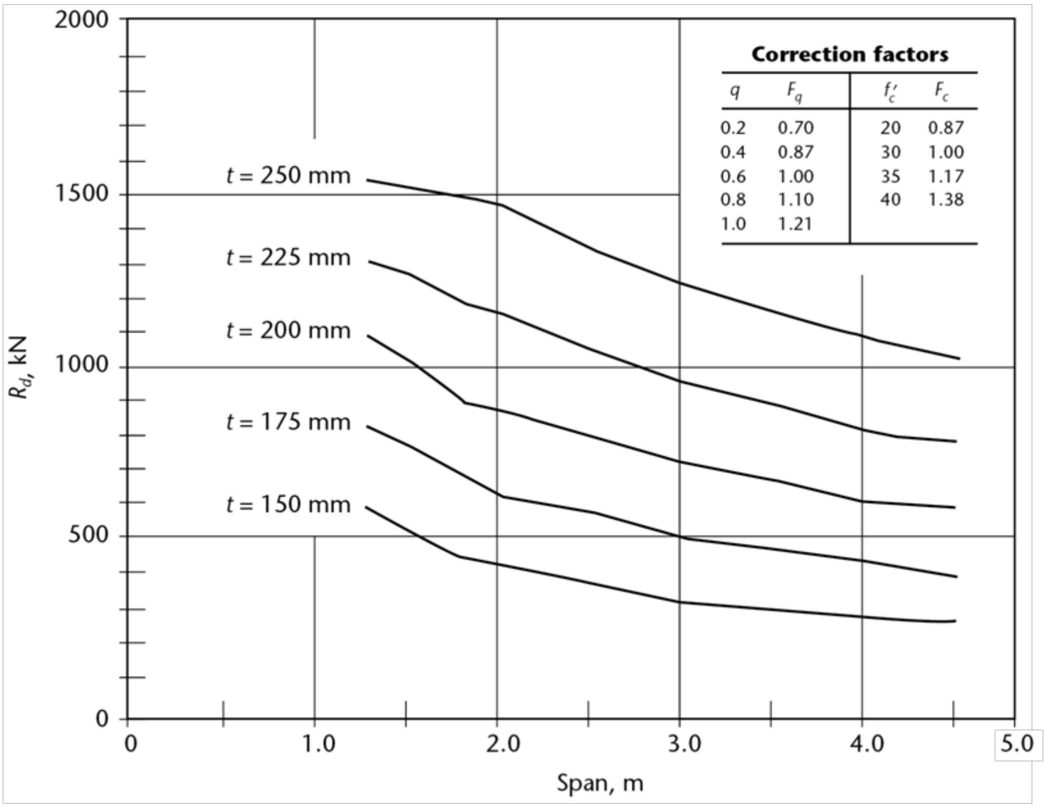


Figure 7.4: R_d (kN) for non-composite concrete deck slabs



7.5.3 continued

For rating	For posting
Deck capacity factor (DCF) $= \left[\frac{Overload\ wheel\ load\ capacity}{Rating\ load\ effect}\right]_{\min}$ $= \left[\frac{\phi R_i}{\gamma_O \times 95 \times I}\right]_{\min}$	**Single axle limit (kg)** $= \left[\frac{Liveload\ wheel\ load\ capacity}{Posting\ load\ effect} \times 8200\right]_{\min}$ $= \left[\frac{\phi \times (0.6R_i)}{\gamma_L \times 40 \times I} \times 8200\right]_{\min}$ **Tandem axle set limit (kg)**† $= \left[\frac{Liveload\ wheel\ load\ capacity}{Posting\ load\ effect} \times 14500\right]_{\min}$ $= \left[\frac{\phi \times (0.6R_i)}{\gamma_L \times 40 \times I} \times 14500\right]_{\min}$ These minimum values shall be used to define axle set limits on the posting weight limit sign as per 7.6.2.

d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall follow the same procedure as for posting (with 8800 utilised in the single axle limit equation for HPMV). If the allowable single axle load determined is less than 8800kg for HPMV or 8200kg for 50MAX then the bridge is unable to carry HPMV or 50MAX loading as applicable.

Table 7.6: Rating, posting, HPMV and 50MAX evaluation live load ULS load factors for decks

Rating loads	γ_o	1.50
Posting loads HPMV and 50MAX evaluation loads	γ_L	1.90*

* A lower load factor of 1.75 may be considered with the express agreement of the road controlling authority, typically through a departure request. Generally, the conditions for adopting higher stress levels, as set out in 7.4.3, will need to be satisfied.

Table 7.7: Strength reduction factors (ϕ) for slabs evaluated by the simplified evaluation method

Superstructure condition	Slab section properties based on:	
	construction drawings and assessed sound material	measured dimensions or verified as-built drawings, and measured sound material
Good or fair (CRR ≤40%)	$0.90\phi_D$	$1.00\phi_D$
Deteriorated (CRR = 70%)	$0.60\phi_D$	$0.70\phi_D$
Seriously deteriorated (CRR = 100%)	$0.30\phi_D$	$0.40\phi_D$

† Work is ongoing to determine how tandem axle limits can be appropriately obtained using the simplified evaluation method. Practitioners should therefore undertake such evaluations as indicated but with caution, noting that the simplified evaluation method as currently documented is only intended for use with single axles/wheels.

7.5.3 continued

Table 7.8: Deck rating loads

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

Table 7.9: Deck posting, HPMV and 50MAX evaluation loads

Axle type	Axle load (kN)	Wheel track and contact area
Posting (general access) and 50MAX		
Twin-tyred	80*	As for HN axle
Four-tyred oscillating	93	4/250 x 150mm areas equally spaced within 2500mm overall width
2/Twin-tyred axles, spaced 1.0m	71	As for HN axle
3/Single tyred axles, spaced 1.25m	59	6/300 x 200mm areas at 2300mm overall width
HPMV		
Twin-tyred	86*	As for HN axle
Four-tyred oscillating	93	4/250 x 150mm areas equally spaced within 2500mm overall width
2/Twin-tyred axles, spaced 1.0m	74	As for HN axle
3/Single tyred axles, spaced 1.25m	62	6/300 x 200mm areas at 2300mm overall width

* Wheel loads from these axles are used for evaluation by the simplified evaluation method in 7.5.3(c).

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, but with γ_o and γ_L taken from table 7.6. 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

For rating	For posting, HPMV and 50MAX evaluations
The deck shall be considered to be loaded with the most adverse of the axles or axle groups listed in the *Vehicle dimensions and mass permitting manual* (volume 1)(4), 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 7.8.	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 2016(2): • For general access and 50MAX vehicles: schedule 3, part 1, *Axle mass limits - General access*, tables 1.1 to 1.5. • For HPMV vehicles: schedule 3, part 3 *Maximum axle mass for heavy motor vehicles operating on a HPMV or specialist vehicle permit*, tables 3.1 to 3.6. For deck spans up to 3m, these may be reduced to the single and tandem axle sets described in table 7.9.

7.5.4 continued

The slab shall be analysed for the loads given in tables 7.8 and/or 7.9 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 load effect in the denominator shall include dynamic load factors as in 7.2.2, and the capacity in the numerator shall be as described in (a). The value of DCF or axle load adopted shall be the minimum for the bridge.

For rating	For posting
Deck capacity factor (DCF) $= \left[\dfrac{Overload\ capacity\ at\ critical\ location}{Rating\ load\ effect}\right]_{min}$ The minimum value for the bridge shall be recorded as the DCF for the bridge.	**Single axle limit (kg)** $= \left[\dfrac{Liveload\ capacity\ at\ critical\ location}{Posting\ load\ effect} \times 8200\right]_{min}$ **Tandem axle set limit (kg)** $= \left[\dfrac{Liveload\ capacity\ at\ critical\ location}{Posting\ load\ effect\ of\ tandem\ axle} \times 14500\right]_{min}$ Also, for slabs with transverse spans greater than 3m: **Tri axle set limit (kg)** $= \left[\dfrac{Liveload\ capacity\ at\ critical\ location}{Posting\ load\ effect\ of\ tandem\ axle} \times 18000\right]_{min}$ **Quad axle set limit (kg)** $= \left[\dfrac{Liveload\ capacity\ at\ critical\ location}{Posting\ load\ effect\ of\ tandem\ axle} \times 20000\right]_{min}$ These minimum values shall be used to define axle set limits of the posting weight limit sign as per 7.6.2.

d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall consider loading comprising the most adverse axles or axle sets described in (b). If this loading is greater than the deck's live load capacity at the critical location, then the bridge is unable to carry 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 OPermit (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, using the methodology described in 7.4.2 and specifically 7.4.2(a), but with γ_O and γ_L taken from table 7.6. 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.

Dead load may be neglected in these calculations.

7.5.5 continued

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:
     - fabricated in baulks with shear connection between them by steel dowels or other means, or
     - fabricated in baulks and having running planks over them more than 50mm thick, or
     - 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).

b. Live loading and analysis

The transverse moments due to the various axles described in tables 7.8 and/or 7.9 on the span between beams shall be calculated assuming the deck planks are simply supported.

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 capacity in the numerator shall be as described in (a).

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

7.5.5 continued

For rating	For posting
Deck capacity factor (DCF) $= \left[\frac{Overload\ capacity\ of\ nominal\ width}{Rating\ load\ effect}\right]_{\min}$	**Single axle limit (kg)** $= \left[\frac{Liveload\ capacity\ of\ nominal\ width}{Posting\ load\ effect} \times 8200\right]_{\min}$
The minimum value for the bridge shall be recorded as the DCF for the bridge.	The minimum value shall be used to define single axle limits for the posting weight limit sign as per 7.6.2.

Note – Tandem axle limits are not expected to be required for the vast majority of timber decks. However, a tandem axle set limit can be applied in accordance with the method in 7.5.4(c) where restrictive decks are sufficiently stiff (ie thick glulam) to warrant one.

d. HPMV and 50MAX evaluations

Evaluations for HPMV and 50MAX loading shall consider live loading described in (b). If this loading is greater than the deck's live load capacity for the nominal width of deck under consideration, then the bridge is unable to carry HPMV or 50MAX loading as applicable.

7.5.6 Deck grade

Not used.

7.5.7 Highway permits data

See 7.4.7.

7.6 Posting implementation

7.6.1 Statutory requirements

The statutory requirements relating to the posting of bridges are given in section 11 of the Heavy Motor Vehicle Regulations 1974[1].

7.6.2 Heavy vehicle bridge limit sign

a. Sign format

Posted weight and/or speed limits shall be displayed in accordance with Land Transport Rule: Traffic Control Devices 2004[22] using sign types R5-9.1 to R5-9.4. Advance warning of restrictions can be given on sign type A45-6. Limits on axle sets and vehicles with small numbers of axles may be removed for longer span structures where they have been shown to be non-critical.

b. Speed limit

A bridge speed limit restriction may be included on the sign, standalone or in conjunction with a weight restriction. This will usually be imposed to reduce vehicle dynamic loading effects on the bridge. A speed restriction is often the first limit imposed, being easiest to implement and less restrictive to traffic than a weight restriction. The speed limit shall be approved by the road controlling authority. Speed limit restrictions shall be applied in multiples of 10km/h.

c. Axle set limits

The axle set limits shall be the lesser of:

- the limits derived from an assessment of the deck capacity in 7.5.3(c), 7.5.4(c), 7.5.5(c), or of main member capacity in 7.4.2 where axle loads are critical. For restrictive deck elements evaluated using the simplified evaluation method and/or with spans up to 3m, the ratio of (tandem axle set limit/14,500) shall be multiplied by 18,000 and 20,000 to obtain the limits for tri and quad axles respectively

7.6.2 continued

- the limits derived by multiplying the general access axle set limits in table 7.10 by the '% GROSS' limit described in 7.4.6. The purpose of this is to ensure that the mass limits of vehicle axle groups comply with the assessed gross weight restriction.

The minimum values for axle set limits shall be rounded to the nearest 200kg and used to define the limits on the posting weight limit sign.

Table 7.10: Axle set limits

Axle set type	Mass limit (tonnes)
Single axle	8.2
Tandem axles	14.5
Tri-axles	18
Quad-axles	20

d. Gross weight limit

The gross weight limit shall be the lesser of:

- the limits derived by multiplying the gross weight limits in table 7.11 by the '% GROSS' described in 7.4.6, rounded to the nearest whole tonne, or
- 44 tonnes, or up to 46 tonnes for vehicles fully compliant with the requirements in table 2.2 of the Land Transport Rule: Vehicle Dimensions and Mass 2016[(2)].

Table 7.11: Gross weight limits

No. of axles	Gross weight (tonnes)
2	15
3	21
4	25
5	31
6	36
7	40
8	44
9 or more	49

7.7 Proof loading

7.7.1 Preliminary

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 7.7.2(a)(iv) and (v), without either modifying the structure, or multiplying the load factors of 7.4.2 by 1.5.

7.7.1 continued

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.

7.7.2 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 7.8 or 7.9 is most critical, and the critical load position(s). The likely failure mode(s) shall be determined.

7.7.3 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 7.7.2(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 7.8 or 7.9 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 7.7.4 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.

7.7.4 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 7.7.2, 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.

Member material	Percentage offset
Structural steel	10
Prestressed concrete	15
Reinforced concrete, composite steel/concrete	20
Timber	25

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.

7.7.4 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.

7.7.5 Rating, posting, HPMV and 50MAX evaluations

a. Correlation of analysis and test results

The results of testing shall be compared with predicted results from the analysis of 7.7.2. 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 x γ_L). R_o shall be the same value divided by (0.8 x γ_o). γ_o and γ_L shall be taken from table 7.3.

Rating, posting, HPMV and 50MAX load effects shall be taken from the analysis of 7.7.2 and shall include dynamic load factors.

c. Decks

Parameters shall be calculated as follows:

For rating	For posting and 50MAX
DCF	**Allowable axle load (kg)**
$= \left[\frac{T_o}{0.8 \times \gamma_o \times (Rating\ load) \times I}\right]$	$= \left[\frac{T_L \times 8200}{0.8 \times \gamma_L \times (Posting\ load) \times I}\right]$

For HPMV
Allowable axle load (kg)
$= \left[\frac{T_L \times 8800}{0.8 \times \gamma_L \times (HPMV\ load) \times I}\right]$

Where T_o and T_L are the maximum applied wheel or axle loads obtained from the proof loading, applied on the contact areas specified in tables 7.8 and 7.9 respectively. γ_o and γ_L shall be taken from table 7.6. Rating, posting, HPMV and 50MAX loads are the appropriate wheel or axle loads from tables 7.8 and 7.9.

For HPMV and 50MAX, if the allowable axle load determined is less than 8800kg or 8200kg respectively then the bridge is unable to carry HPMV or 50MAX loading as applicable.

7.8 References

(1) Ministry of Transport (1974) Heavy Motor Vehicle Regulations 1974. Wellington.

(2) Ministry of Transport (2016) Land Transport Rule: Vehicle Dimensions and Mass 2016. Wellington.

(3) Waka Kotahi NZ Transport Agency (1995) *Overweight permit manual* (SM070). Wellington. Withdrawn.

(4) Waka Kotahi NZ Transport Agency (2021) *Vehicle dimensions and mass permitting manual*. Volume 1 Applying for and operating under an overweight, overdimension, HPMV or specialist vehicle permit. Wellington.

(5) Waka Kotahi NZ Transport Agency (2016) O*Permit bridge structural data guide*. Wellington.

(6) Standards New Zealand NZS 3112.2:1986 *Methods of test for concrete.* Part 2 Tests relating to the determination of strength of concrete. (Incorporating Amendment No. 2: 2000)

(7) British Standards Institution BS EN ISO 6892-1:2019 *Metallic materials. Tensile testing.* Part 1 Method of test at room temperature.

(8) Standards New Zealand NZS 3603:1993 *Timber structures standard*. Under revision. (Incorporating Amendment No. 4: 2005). Under revision.

(9) Standards Australia AS 1720.2-2006 *Timber structures.* Part 2 Timber properties. (Incorporating Amendment No. 1: 2006).

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

(11) Standards New Zealand NZS 3631:1988 *New Zealand timber grading rules*.

(12) Standards Australia AS 3818.6-2010 *Timber – Heavy structural products – Visually graded.* Part 6 Decking for wharves and bridges.

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

(14) Standards Australia AS 2858-2008 *Timber – Softwood – Visually stress-graded for structural purposes*.

(15) Standards Australia AS 5100.9:2017 *Bridge design.* Part 9 Timber.

(16) Standards Australia AS 1720.1-2010 *Timber structures.* Part 1 Design methods. (Incorporating Amendment No. 2: 2015)

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(20) Canadian Standards Association (2019) S6-19 *Canadian highway bridge design code*.

(21) Waka Kotahi NZ Transport Agency (2016) *Evaluation of shear connectors in composite bridges*. Research report 602, Wellington.

(22) Ministry of Transport (2004) Land Transport Rule: Traffic Control Devices 2004 (plus subsequent amendments). Wellington.