4.0 Analysis and design criteria

In this section

4.1 Analysis

Structural components shall be designed for the most adverse effects arising from eccentricity of loading or curvature of the bridge. The analysis method used shall take account of the relative stiffness of longitudinal and transverse members, and the stiffness used for reinforced concrete members shall take account of the effects of flexural cracking.

4.2 Reinforced concrete and prestressed concrete

4.2.1 General

Design shall be in accordance with NZS 3101.1&2 *Concrete structures standard*[1], with the following provisos:

a. Crack widths (clause 2.4.4.2)

Assessment of crack widths is required unless concrete tensile stresses do not exceed 0.0MPa at construction joints and $0.4\sqrt{f'_c}$ at other locations under all serviceability limit state load combinations. Crack widths shall not exceed the limits stated in table 4.1. Crack widths shall be assessed following the requirements of NZS 3101[1] clauses 2.4.4.6 and 19.3.3.5.3(c).

For prestressed concrete, concrete tension shall be avoided under permanent effects (to be taken as the serviceability limit state permanent effects load combination 0 in table 3.2 without a temperature contribution).

Table 4.1: Crack width limits

	Crack width limit for exposure classification	
	A2, B1, B2	C
Reinforced concrete – SLS permanent effects load combination 0	0.30mm	0.20mm
Reinforced concrete – all other SLS load combinations	0.40mm	0.30mm
Prestressed concrete – SLS load combinations excluding permanent effects combination 0	0.30mm	0.20mm

Care should be exercised when designing deep beams using the strut and tie method as cracks can become large when this method is used.

Deck reinforcement design shall be exempt from a check of crack widths when the empirical design method specified by NZS 3101[1] section 12.8 is used.

b. Design for durability (section 3)

i. General

For designs based on the use of concrete made with GP, GB or HE cement complying with NZS 3122 *Specification for Portland and blended cements (General and special purpose)*[2] with or without supplementary cementitious materials (SCM) complying with AS/NZS 3582 *Supplementary cementitious materials*[3], durability of the reinforced or prestressed concrete shall be designed for in accordance with the requirements of NZS 3101[1] except as modified herein.

4.2.1 continued

ii. Equivalent terminology

The term ‘design working life’ adopted in this manual shall be taken to equate to the term ‘specified intended life’ adopted by NZS 3101[1] clause 3.3.1.

iii. Site exposure classification

All parts of bridges, major culverts, stock underpasses, pedestrian/cycle subways and retaining walls shall be considered to be in an ‘exterior’ type of environment.

In ascertaining the site exposure classification, where specific evaluation of a site is proposed in accordance with NZS 3101[1] clause 3.4.2.4, Coastal frontage zone extent, or clause 3.4.2.5, Tidal/splash/spray zone, this shall be treated as a special study as described by AS/NZS 1170.0 *Structural design actions* part 0 General principles[4] and shall be fully documented in an appendix to the structure design statement.

In the case of the tidal/splash/spray zone, unless exposure classification C is adopted as the default, a special study is required to define the extent of the C zone in the vertical direction, which shall be taken to extend from the mean low water level at depth upwards to a height above sea level that is determined by prevailing wind and sea conditions.

The height of the upper boundary shall be defined by a decrease in the aggressiveness of the exposure environment such that the selected mix design for the B2 exposure classification can be demonstrated to achieve a 100-year life. To carry out this evaluation, a series of chloride profiles as a function of height above sea level will need to be determined on nearby concrete structures of similar exposure, to indicate the long-term surface chloride profile likely to be established in the concrete at the site under consideration. If nearby structures are not available, height profiles from closely comparable environments will need to be substituted.

The measured chloride concentrations are then to be employed as an input to a service life prediction model based on Fick’s laws to verify the height at which the B2 mix design becomes adequately durable, which shall then be taken as the upper boundary position for the C zone. The application of the service life prediction model shall comply with NZS 3101[1] part 2 clause C3.12.1 with adequate durability being taken as a minimum time to first rusting of 80 years.

Table 4.2: Requirements for concrete subjected to natural aggressive soil and groundwater attack

Chemical exposure classification	Maximum water cementitous ratio	Minimum cover (mm)	Minimum binder content (kg)	Additional requirement
XA1	0.50	65	340	---
XA2	0.45	65	370	SCM
XA3	0.40	75	400	SCM

Notes:

1. Binders containing combinations of cement and SCM (fly ash, slag or amorphous silica) provide significantly increased resistance to chemical attack mechanisms.
2. Where low pH and high exchangeable soil acid conditions prevail, an additional protection (eg protective coating or other form of physical protection) may be required. This may allow for reduction of originally specified concrete parameters.

4.2.1 continued

iv. Requirements for aggressive soil and groundwater exposure classification XA

Concrete in members subject to chemical attack shall be specified in accordance with table 4.2 which replaces table 3.4 in NZS 3101(1) for members with a design working life of 100 years. Such concrete shall be specified as 'special concrete' under NZS 3109 *Concrete construction*(5) clause 6.3.

v. Minimum concrete curing requirements

NZS 3101(1) table 3.5 is clarified as follows: Note 3 of the table shall be taken as applying to the curing of concrete associated with exposure classifications C, XA2 and XA3 only, with alternative curing methods being alternatives to water curing.

The use of heat accelerated curing (eg as specified by the NSW Roads and Maritime Services QA specification B80 *Concrete work for bridges*(6)) as an alternative method to water curing in the C exposure zone shall be subject to a special study to demonstrate equivalent performance.

Another potentially acceptable alternative method to water curing in the C exposure zone is sealed curing used in conjunction with concrete cover increased above that specified by NZS 3101(1) table 3.7. This approach shall also be subject to verification by a special study to establish the increase in concrete cover required to provide equivalent performance.

vi. Additional requirement for concrete exposure classification B2

Concrete for use in the exposure classification zone B2 shall have a minimum specified 28 day compression strength of not less than 40MPa.

vii. Life prediction models and durability enhancement measures

There are a number of alternative durability enhancing measures which can be taken to extend the life of concrete structures and provide the required durability other than those specified by NZS 3101(1) chapter 3. These include concrete coatings, corrosion inhibiting admixtures, galvanized or stainless steel reinforcement, controlled permeability formwork, glass fibre reinforced concrete (GRC) permanent formwork, and cathodic protection. Life prediction models offer an alternative approach to use of NZS 3101(1) table 3.7 for determination of covers for the C and B2 zones. Adoption of any of these or other alternative measures shall be the subject of a special study as described in AS/NZS 1170.0(4), and shall be fully documented in an appendix to the structure design statement. Where a life prediction model is proposed to be used, the structure design statement appendix shall include full details of the formulation and calibration of the model.

viii. Subways and culverts accessible by stock

A B2 exposure classification shall be adopted for the floor and bottom 1.2m of the height of walls of subways and box culverts accessible by stock due to possible exposure to stock effluent.

c. Friction losses (clause 19.3.4.2.3)

It should be noted that the apparent coefficient of friction for post-tensioned cables deflected at isolated points is likely to be significantly higher than that for equivalent cables curved over their whole length. This shall be taken into account in the design.

4.2.1 continued

d. Reinforced concrete slabs

i. Bridge deck slab thickness (table 2.3)

For a uniform concrete slab, monolithic with concrete webs, L_s shall be taken as the clear span.

For a haunched slab, monolithic with concrete webs, or tied down to steel girders, where thickness at root of haunch is at least 1.5 times thickness at centre of slab, L_s shall be taken as the distance between midpoints of opposite haunches.

For a uniform slab on steel girders, L_s shall be taken as the average of the distance between webs and the clear distance between flange edges.

For deck slabs designed by the empirical method of NZS 3101[(1)] clause 12.8, the minimum slab thickness requirements of that clause shall take precedence over the requirements of NZS 3101[(1)] table 2.3.

ii. Minimum area of shear reinforcement in slabs

In the absence of more detailed research, the definition of "highly repetitive loads" for the interpretation of NZS 3101[(1)] clause 9.3.9.4.13, relating to through thickness shear in slabs, shall be that due to ultimate limit state live loading corresponding to table 3.3 load combination 1A, but without pedestrian (FP) loading.

e. Shrinkage and creep effects in concrete

Assessment of shrinkage and creep effects shall be undertaken in accordance with NZS 3101[(1)] (including amendment 3). Design shrinkage strain shall be determined as the sum of chemical (autogenous) and drying shrinkage as given in NZS 3101[(1)] appendix E. It is noted that autogenous shrinkage is essentially complete at about 50 days after initial setting of the concrete.

In the application of the NZS 3101[(1)] procedure for shrinkage and creep to structural concrete mixes based on Type GP cement with restricted water demands and moderate workability (slump 50 – 100mm), the relative humidity factor (k_4) may be derived from table 4.3, based on the average relative humidity for the locality. Table 4.4 presents average basic drying shrinkage strains $(\varepsilon_{csd,b})$ for a range of New Zealand aggregates for use in NZS 3101[(1)] equation E-4 to determine ε_{csd}, the drying shrinkage strain. Alternatively, $\varepsilon_{csd,b}$ may be taken as equal to the 56 day drying shrinkage test result determined by using the method specified in AS 1012.13 *Methods of testing concrete* part 13 Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory[(7)], corrected for autogenous shrinkage over the drying period by subtracting $25(0.06f_c' - 1.0)$ microstrain.

The average relative humidity for a locality may be assessed from data available from the NIWA CliFlo[(8)] database through their website. Table 4.5 presents average and 9am relative humidities, derived as noted below the table, for various locations throughout New Zealand. Figure 4.1 presents 9am relative humidities which may be used to estimate the average relative humidity for locations for which data is not available in the CliFlo[(8)] database.

Note that for particularly dry parts of the country (eg Central Otago) the average relative humidity may vary quite significantly from the 9am relative humidity values. Conservative (low) assessments of relative humidities should be adopted as the basis for design. Guidance on using the CliFlo[(8)] database is provided in C4 in the *Bridge manual commentary*.

Consideration shall be given to the fact that Ɛε_{cs} has a range of ±30%. Note also that high slump (eg pump-type) concrete mixes may have significantly higher levels of shrinkage.

4.2.1 continued

For shrinkage sensitive structures, it is recommended that concrete suppliers who may potentially supply concrete for the structure be consulted about the shrinkage properties of their concrete, and that use of a super-plasticiser or shrinkage reducing admixture be considered. Such admixtures can significantly reduce shrinkage, although their use requires a higher degree of control in the production and placing of the concrete. Caution needs to be exercised in adopting very low shrinkage strains associated with the use of super-plasticisers and shrinkage reducing admixtures as there is a lack of published data, and thus uncertainty, over the long term shrinkage performance of concretes associated with their use. (Note that shrinkage reducing admixtures, which are most commonly used and do not need the concrete to be confined to be effective, should not be confused with shrinkage compensating admixtures, which generally induce initial expansion in the concrete before subsequent shrinkage takes place.)

In general, a higher water content results in greater shrinkage for concretes made from a particular combination of aggregates. This trend may be modified by the use of admixtures to reduce water content. Greater drying shrinkage may also occur with the following types of mix:

- specified strength over 50MPa
- cementitious binder content exceeding 380kg/m^3
- water to cement ratio less than 0.40.

Designers need to consider the potential for higher shrinkage for these types of concrete. This applies in particular to the design of deck slabs where restraint is provided by the supporting girders. It applies also to use of the empirical design method for deck slabs.

In addition to potentially greater drying shrinkage, these concretes, and concretes containing supplementary cementitious material, may have greater autogenous shrinkage. They also tend not to bleed, and consequently can exhibit greater plastic shrinkage. Plastic shrinkage may be severe in the case of low water/cement concrete containing supplementary cementitious material. Therefore the use of such concrete in deck slabs is not recommended.

Plastic shrinkage cracking occurs before the bond between concrete and reinforcing steel has developed, therefore the steel is ineffective in controlling this type of cracking. For such concretes, evaporation retarders (eg aliphatic alcohols) or misting should be used to reduce evaporation from the concrete surface and thereby to reduce plastic shrinkage. Use of micro synthetic fibres in the concrete mix can also be beneficial.

For concrete structures constructed in stages, the design shall take account of the shrinkage and creep effects of the concrete using an appropriate time dependent analysis. The final profile of the structure shall take account of the deflections that occur due to these effects over the life of the structure. In bridge superstructures, the post-construction deflection associated with these effects shall be less than span/1000.

Where precast concrete beams are made continuous by interconnection with reinforced in situ concrete at the intermediate supports, design for the effects of residual creep and differential shrinkage shall comply with the principles and general requirements of AS 5100.5 *Bridge design* part 5 Concrete(9) section 8.10.

Table 4.3: Relative humidity factor (k_4)

Relative humidity (%)	40	50	60	70	80	90
Relative humidity factor (k_4)	0.74	0.68	0.61	0.50	0.39	0.21

Figure 4.1: Map of New Zealand 9am relative humidities (RH%)

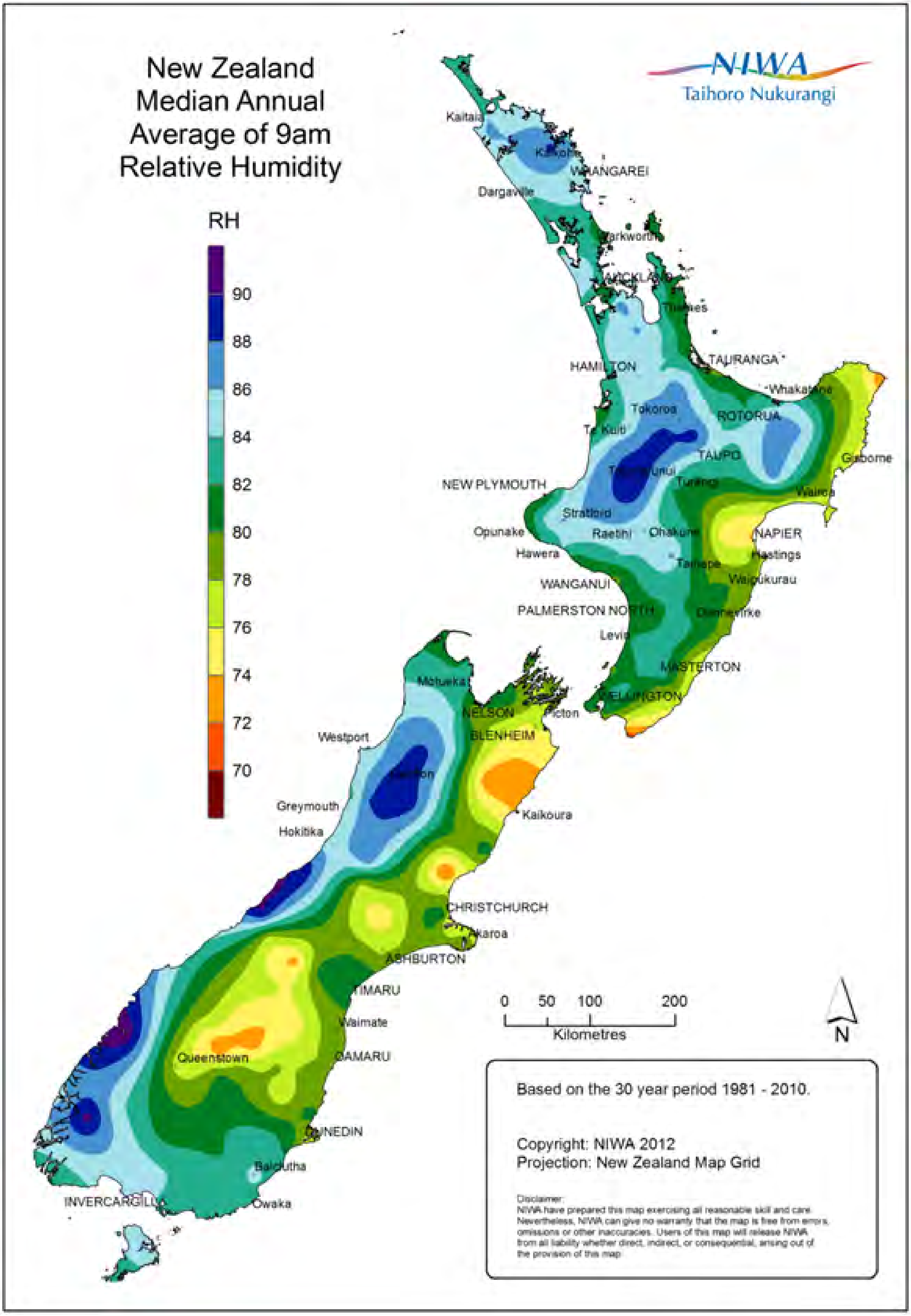


Figure 4.1 is reproduced with the permission of NIWA.

4.2.1 continued

Table 4.4: Basic drying shrinkage strain ($\varepsilon_{csd,b}$)

Location	Aggregate type	Basic drying shrinkage (microstrain)
Whangarei, Auckland Hunua, Hamilton	Northern greywacke	720
Hastings, Palmerston North, Masterton, Wellington, Blenheim, Kaikoura	Central greywacke	1100
Christchurch, Timaru, Oamaru	Southern greywacke	700
Auckland	Basalt*	660
Kaitaia, Tauranga	Other andesite / basalt gabro	960
New Plymouth, Taranaki	Taranaki andesite	730
Waiau	Limestone	390
Nelson	Greywacke – siltstone	1100
Westport Queenstown, Wanaka Invercargill	Granite – greywacke Schist – greywacke Igneous – greywacke	590
Dunedin	Phonolite	520

Notes:

* Use of Auckland basalt aggregate is declining. In general for shrinkage design for the Auckland locality it is recommended that the value for Auckland greywacke be adopted instead.

Table 4.5: Average and 9am relative humidities for various locations

Location	RH (%)	Location	RH (%)	Location	RH (%)	Location	RH (%)
Kaikohe	85 (86)	Taupo	79 (84)	Wellington	76-81 (80)	Ashburton	77 (76)
Whangarei	80 (84)	Taumarunui	82 (88)	Nelson	77 (80)	Timaru	80 (80)
Dargaville	80 (84)	Gisborne	78 (76)	Blenheim	75 (76)	Franz Josef	84 (88)
Auckland	77-82 (82)	New Plymouth	83* (80)	Westport	83 (84)	Queenstown	70* (78)
Hamilton	82 (84)	Waiouru	82 (84)	Kaikoura	75 (74)	Alexandra	66* (80)
Tauranga	77 (80)	Napier	76 (74)	Greymouth	79 (84)	Oamaru	80 (78)
Whakatane	80 (82)	Whanganui	77 (80)	Hokitika	85* (84)	Dunedin	78 (78)
Rotorua	83* (84)	Palmerston North	80 (82)	Hanmer	76 (80)	Invercargill	81 (82)
Te Kuiti	77 (84)	Masterton	78 (82)	Christchurch	78 (76)		

Notes:

RH values not within brackets are average RH assessed from NIWA CliFlo[8] data for the period 2008-12. RH values within brackets are 9am RH values assessed from figure 4.1.

* Conservative (low) estimates based on incomplete NIWA datasets. Data missing: Rotorua, New Plymouth, Hokitika – early hours of the morning; Queenstown – nighttime hours; Alexandra – nighttime hours and large and irregular gaps in daytime hours. In general, humidity is usually higher during the hours of darkness, peaking a little before dawn.

4.2.1 continued

f. Mechanical coupling and anchorage of reinforcing steel bars in concrete

i. General

(For the purposes of this clause 4.2.1(f) "road controlling authority" shall refer to Waka Kotahi NZ Transport Agency unless alternative provisions for approval of conformity have been established by the road controlling authority with jurisdiction over the structure in question.)

Mechanical couplers for the jointing of reinforcing steel, together with the associated reinforcing steel after any processing required for coupling, shall satisfy the requirements of NZS 3101[(1)] clauses 8.7.5 and 8.9.1.3, and either ISO 15835-1:2009 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 1 Requirements[(10)] for category S2 couplers (seismic 2 – violent) or ISO 15835:2018 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 1 Requirements[(11)] for category S couplers (seismic), except as modified herein. Mechanical anchors for the anchoring of reinforcing steel, together with the associated reinforcing steel after any processing required for anchoring, shall satisfy the requirements of NZS 3101[(1)] clauses 8.6.11 and 8.9.1.3, and ISO 15698-1 *Steel for the reinforcement of concrete - Headed bars* part 1 Requirements[(12)] for category S headed bars (seismic), except as modified herein. Where the requirements of these standards conflict, those contained in the NZS 3101[(1)] clauses referenced, including references therein to ISO 15825-1:2009[(10)] and ISO 15835-2:2009 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 2 Test methods[(10)], shall take precedence, unless specifically stated otherwise.

As per NZS 3101 clause 8.9.1.1(a) couplers shall not be located in or immediately adjacent to ductile and limited ductile plastic regions.

Couplers and anchors shall be legibly marked with identification as specified by NZS 3101[(1)] clause 8.7.5.5 and their method of installation shall be specified as required by NZS 3101[(1)] clause 8.7.5.6.

Where considered to be necessary, couplers and associated reinforcing steel after processing shall also meet the requirements of the requirements of NZS 3101[(1)] for high cycle fatigue and either those of ISO 15835-1:2009[(10)] for category F (fatigue) or those of ISO 15835-1:2018[(11)] for category F (fatigue); and anchors and associated reinforcing steel after processing shall also meet the requirements of NZS 3101[(1)] for high cycle fatigue and those of ISO 15698-1[(12)] for category F2 (fatigue). The required cycles and stress range for both couplers and anchors shall be as for category F couplers in ISO 15835-1:2009[(10)] or ISO 15835-1:2018[(11)].

Couplers and anchors manufactured from cast iron shall not be used.

Anchors, couplers and associated reinforcing steel shall comply with the brittle fracture resistance requirements set out in (iv).

Where, in the modification or strengthening of an existing structure, coupling to embedded reinforcement of unknown maximum ultimate tensile strength is proposed, the reinforcement shall either be tested to establish its ultimate tensile strength or a conservative over estimation made of its ultimate tensile strength as the basis for selection and design of the couplers in order to ensure that the performance requirements specified above are satisfied.

4.2.1 continued

ii. Demonstration of conformity of couplers and anchors

Demonstration of conformity for both couplers and anchors shall be by one of the following methods:

- product certification in accordance with ISO 15835-1:2009(10) or ISO 15835-3:2018 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 3 Conformity assessment scheme(11), together with factory production control testing as outlined below
- batch verification in accordance with ISO 15835-1:2009(10), together with satisfactory evidence of brittle fracture resistance
- approved qualification testing, together with factory production control testing, both as outlined below.

Note that ISO 15698(12) does not contain product certification requirements or batch verification requirements. The certification and batch verification requirements of ISO 15835(10) or (11) shall be applied to anchors to the extent possible.

Product certification shall be by a certification body accredited by JAS-ANZ (Joint Accreditation System of Australia and New Zealand) or accredited by other international accreditation schemes with the agreement of the road controlling authority. Product certification shall be location of manufacture specific, ie any certificate shall be applicable to couplers or anchors manufactured at a specific location only, and shall be for the requirements set out in (i) including brittle fracture resistance, and optionally including the high cycle fatigue requirements.

Approved qualification testing of coupler and anchor systems shall consist of qualification testing in accordance with ISO 15835-1:2009(10) annex A or ISO 15835-3:2018(11) for couplers, and ISO 15698-1(12) for anchors, demonstrating that the system meets the requirements set out in (i), including demonstration of brittle fracture resistance and optionally including high cycle fatigue resistance, all to the approval of the road controlling authority. Approvals will remain valid for an initial period of three years, following which they will require renewal. Requirements for renewal are to be confirmed by the road controlling authority.

Factory production control testing shall be in accordance with the factory production control requirements of ISO 15835-1:2009(10) annex A or ISO 15835-3:2018(11) for couplers, and ISO 15698-1(12) for anchors, and the requirements of (i), unless agreed otherwise with the road controlling authority. The rate of testing shall be one sample per batch of up to 2500 units for certified systems and one sample per batch of up to 1000 units for systems with approved qualification testing, again unless agreed otherwise with the road controlling authority.

Testing associated with product certification and qualification may be carried out in New Zealand rather than at location of manufacture. Testing associated with product certification, product qualification, factory production control and for batch verification shall be by an IANZ (International Accreditation New Zealand) accredited laboratory or equivalent agreed with the road controlling authority.

Test methods for couplers shall be either those specified in ISO 15835-2:2009 or those specified in ISO 15835-2:2018 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars* part 2 Test methods(11) for the categories of coupler detailed in (i). Test methods for anchors shall be those specified in ISO 15698-2 *Steel for the reinforcement of concrete - Headed bars* part 2 Test methods(12) for the categories of anchor detailed in (i).

4.2.1 continued

For all couplers and anchors and associated reinforcing steel bars delivered to site the following documentation shall be provided:

- product certificates together with factory production control testing certificates for the batches delivered; or
- evidence of qualification testing approved by the road controlling authority together with factory production control testing certificates for the batches delivered; or
- batch verification test certificates for the batches delivered.

In all cases the test certificates for the original manufacture of the reinforcing bar shall also be provided.

iii. Demonstration of reinforcement suitability and ongoing conformity for coupling and anchoring

It is noted that reinforcement conformity with AS/NZS 4671 Grade 300E or Grade 500E does not guarantee suitability for coupling and anchoring in accordance with the requirements set out in (i). The suitability of reinforcement from any manufacturer and produced by any manufacturing method for use in conjunction with a particular coupler and anchor system, together with ongoing conformity, shall be confirmed by one of the following methods:

- product certification as described in (ii) for couplers and anchors, where the product certificate includes the reinforcing bar as part of the system, together with ongoing factory production control testing as outlined below
- qualification testing as described below, to the approval of the road controlling authority, together with ongoing factory production control testing as outlined below
- batch verification as described in (ii), together with the associated couplers and anchors

Qualification testing of reinforcement for use with a coupler or anchor system shall be in accordance with the requirements given in (ii) for qualification of coupler and anchor systems, and may be carried out as part of the qualification testing for the system itself. Where reinforcement preparation for an anchor system is identical to that for a coupler system, qualification of reinforcement for the coupler system shall be deemed to also qualify the reinforcement for the anchor system.

Factory production control testing of reinforcement that is to be coupled or anchored shall be carried out, after processing as required, in accordance with the factory production control requirements for couplers and anchors set out in (ii), unless agreed otherwise with the road controlling authority. The rate of testing shall be one sample per batch of up to 2500 bars for certified products and one sample per batch of up to 1000 bars for reinforcement with approved qualification testing, again unless agreed otherwise with the road controlling authority.

iv. Brittle fracture resistance

In accordance with NZS 3101[(1)] clauses 8.6.11.4 and 8.7.5.2(a) respectively, mechanical anchors and couplers for the anchorage or jointing of reinforcing steel shall be proven by an appropriate test acceptable to the road controlling authority to possess resistance to brittle fracture down to a temperature of 0°C. Reinforcing steel bars shall meet the brittle fracture requirements of AS/NZS 4671 Steel for the reinforcement of concrete[(13)].

4.2.1 continued

Presently test methods to indicate resistance to brittle fracture are limited in availability or are in development. Therefore where couplers, anchors or bar stock equivalent to the finished material of machined couplers or anchors are of sufficient size to enable Charpy V notch test specimens to be cut from them, Charpy V notch testing may be used to demonstrate brittle fracture resistance. Where this test method is applied, a Charpy V-notch impact resistance equal to or greater than 27 joules shall be achieved when standard 10mm x 10mm test pieces are tested at 0°C in accordance with AS 1544.2 *Methods for impact tests on metals* part 2 Charpy V-notch(14) and assessed for acceptance as specified by AS/NZS 3678 *Structural steel – Hot-rolled plates, floorplates and slabs*(15) table 9. Test pieces of smaller cross section as listed in AS/NZS 3678(15) table 9 may be used when standard 10mm x 10mm cross-section test pieces are impractical. For these test pieces of smaller cross-section the acceptance criteria shall correspond to the L0 impact designation and test piece cross-section given in AS/NZS 3678(15) table 9.

g. Prestressed concrete

i. Design for shear

In the design of prestressed concrete members for shear, the concrete contribution to shear strength shall be computed in accordance with AASHTO *LRFD Bridge design specifications, customary U.S. units, 7th edition*(16), clause 5.8.3.4.3, noting that the AASHTO equations are based on imperial units. (In respect to NZS 3101(1) clause 19.3.11.2.2 equation 19-15, the NZS 3101:1995(17) Commentary noted that for beams subjected to point loads the equation should not be used, and also that the equation is not necessarily appropriate to continuous prestressed concrete members, such as a bridge superstructure, and that it may be non-conservative for thin webbed sections, common in bridge superstructures. This equation should therefore not be used.)

ii. Confining reinforcement and strand corrosion protection in pretensioned members

Adequate confining reinforcement shall be provided in the end zones of pretensioned prestressed concrete members to prevent splitting of the members.

The ends of prestressing strand in pretensioned members shall be protected from corrosion in such a manner that no maintenance of the corrosion protection is required within the design life of the element.

iii. External post-tensioning

External post-tensioning shall not be used in locations accessible to the public or where there is a significant risk of fire in proximity to the tendons.

iv. Detailing of prestressed members

Grout for and grouting of post-tensioning ducts shall comply with the Concrete Institute of Australia's publication CIA Z3 *Grouting of prestressing ducts*(18) for type A tendons. This shall include the provision of grout inlets, vents, caps and control valves or other approved methods of maintaining pressure and controlling flow.

In C and B2 exposure classifications as defined in NZS 3101(1), plastic post-tensioning duct with 2.0mm minimum wall thickness and coupled at all construction joints shall be used. The duct and accessories shall comply with *fib* Bulletin 7 *Corrugated plastic ducts for internal bonded post-tensioning*(19). In other exposure classifications corrugated galvanized steel ducts may be used. They shall be spirally wound from galvanized strip steel with a minimum wall thickness of 0.3mm and a coating weight in excess of 90 grams/m², and shall have welded or interlocking seams with sufficient rigidity to maintain the correct profile during concrete placement.

4.2.1 continued

All anchorages shall have grout caps and seals for grouting operations as required by CIA Z3[18]. For C and B2 exposure classifications, anchorages shall have permanent grout caps made from fibre reinforced polymer or HDPE, bolted to the anchorage and sealed with O rings or gaskets against the bearing plate.

Further guidance on design details for post-tensioning may be found in:

- FHWA *Post-tensioning tendon installation and grouting manual*[20]
- UK Concrete Society technical report 72 *Durable post-tensioned concrete structures*[21].

h. Precast prestressed hollow core unit decks

Precast prestressed hollow core deck units shall be provided with sufficient transverse stressing to provide shear transfer without relative movement, and without cracks opening (ie with zero tension) on the longitudinal joints between units at the deck top surface under all serviceability load combinations. Transverse tendons shall be provided with at least a double corrosion protection system.

A double corrosion protection system will generally comprise a continuous, full length, watertight, electrically non-conductive, corrosion resistant, durable duct with the void between the duct and the tendon fully infilled with a corrosion inhibiting grout (eg cement grout).

i. Reinforcing steel

All reinforcing steel shall comply with the requirements of NZS 3101[1]. References to AS/NZS 4671 in NZS 3101[1] may be taken to refer to AS/NZS 4671:2001[22] or AS/NZS 4671:2019[13] until 31 December 2022, and shall be taken to refer to AS/NZS 4671:2019[13] thereafter.

j. Design for fatigue

In the application of NZS 3101[1] clause 2.5.2.2, the stress range due to repetitive loading to be considered in flexural reinforcing bars shall be that due to normal vehicle live loading at the serviceability limit state (ie 1.35LLxI).

In the application of NZS 3101[1] clause 19.3.3.6.2, the stress range due to frequently repetitive live loading shall be that due to normal vehicle live loading at the serviceability limit state (ie 1.35LLxI). The stress range due to infrequent live loading shall be taken to be that due to the transient components of any of the serviceability limit state loads combinations in table 3.2 that include traffic loading.

k. Permissible stresses in reinforcement.

In reinforced concrete the tensile stress in reinforcement shall not exceed $0.8f_y$ under any serviceability limit state load combination. Deck reinforcement design shall be exempt from a tensile stress check when exempted from a crack width check as per (a). In prestressed concrete the limits on tensile stress in NZS 3101[1] clause 19.3.3.6.1 shall apply.

l. Bridge Piers

Regardless of shape or aspect ratio, bridge piers shall comply with the requirements of NZS 3101[1] section 10. Where bridge piers may be classified as walls, any additional requirements of NZS 3101[1] section 11 pertaining to walls shall also be satisfied. Where inconsistencies may exist between the requirements of section 10 and section 11, the requirements of section 10 shall take precedence unless otherwise justified and accepted by the road controlling authority.

Bridge piers shall also be designed for serviceability in accordance with NZS 3101[1] section 2.4 and, where applicable, section 19.

4.3 Structural steel and composite construction

4.3.1 General

Design for the steel componentry of bridge substructures, and any seismic load resisting componentry expected to behave inelastically, shall comply with NZS 3404:1997 *Steel structures standard*[23].

Design for the steel componentry of bridge superstructures, including seismic load resisting components expected to behave elastically, shall be in accordance with AS/NZS 5100.6 *Bridge design* part 6 Steel and composite construction[24].

Substructure composite steel and concrete members shall be designed for composite action in compliance with NZS 3101[1]. Alternatively, where the member is designed with a displacement ductility demand less than 1.0 in events up to CALS, the member may be designed for composite action in compliance with AS/NZS 5100.6[24]. Further requirements for the seismic design of composite steel and concrete substructure members are contained in section 5.

The above paragraphs apply also to the design of steel and composite componentry of major culverts, stock underpasses and pedestrian/cycle subways.

In addition to the brittle fracture requirements for plates and rolled sections of the withdrawn NZS 3404.1:2009 *Steel structures standard* part 1 Materials, fabrication, and construction[25] and AS/NZS 5100.6[24], consideration shall also be given to the brittle fracture of steel elements complying with standards other than those listed by NZS 3404.1:2009[25] section 2 and AS/NZS 5100.6[24] section 14 (eg fixings, high strength bars).

The design and construction of concrete deck slabs for composite bridges for all actions on the concrete deck shall be in accordance with NZS 3101[1], except that the design of shear connection between the concrete deck slab and steel girders and the design for longitudinal shear occurring within the deck slab and paps shall comply with AS/NZS 5100.6[24]. The requirements of AS/NZS 5100.6[24] clause 6.2.3, as they relate to the design of the concrete deck slab for the control of cracking due to thermal and shrinkage effects, where they require a greater quantity of reinforcement than required by NZS 3101[1], shall also be complied with.

Steelwork fabrication and erection shall comply with AS/NZS 5131 *Structural steelwork - Fabrication and erection*[26], and where not specified by AS/NZS 5131[26], with any additional requirements of the withdrawn NZS 3404.1:2009[25]. Steel material for the fabrication of structural components shall comply with the requirements of a standard listed in AS/NZS 5100.6[24] clause 2.2.1 or appendix H, or NZS 3404.1:2009[25] clause 2.2.1, and fasteners shall comply with AS/NZS 5100.6[24] clause 2.4 or NZS 3404.1:2009[25] section 2.3.

The Waka Kotahi NZ Transport Agency research report RR 525 *Steel-concrete composite bridge design guide*[27] provides guidance on the design of steel girder bridge superstructures to the 2004 edition of AS 5100.6[28].

4.3.2 Application of NZS 3404:1997[23]

a. Design loadings (clause 3.2.3)

   The design load combinations for the ultimate limit state (ULS) and serviceability limit state (SLS) shall be those specified in this manual.

b. Seismic design structural performance factor (clause 12.2.2.1)

   The structural performance factor (S_p) shall be as specified in this manual.

4.3.2 continued

c. Damping values and changes to basic design seismic load (clause 12.2.9)

Within this clause, the wording 'loadings standard' shall be replaced by 'the Waka Kotahi NZ Transport Agency *Bridge manual*'.

d. Methods of analysis of seismic-resisting systems (clause 12.3.2)

Within this clause, the wording 'loading standard' shall be replaced by 'the Waka Kotahi NZ Transport Agency *Bridge manual*'.

4.3.3 Application of AS/NZS 5100.6[24]

a. Application (clause 1.3)

Within clause 1.3 replace the second paragraph with: "In the design of steel concrete composite members, the general requirements of NZS 3101 pertaining to the design of concrete shall apply, where relevant, in addition to the requirements of AS/NZS 5100.6."

b. General – Aim (clause 3.1.1) and Design for the ultimate limit state (clause 3.1.2)

Within section 3.1, reference to AS 5100.1 *Bridge design* part 1 Scope and general principles[29] shall be replaced by New Zealand Transport Agency *Bridge manual*.

c. Design for the ultimate limit state (clauses 3.1.2 and 3.2)

The design of superstructure componentry remaining elastic when subjected to damage control limit state (DCLS) seismic loading shall be designed on the basis of the capacity design principles specified in section 5 of this manual (ie for the actions induced in them by yielding elements mobilising their overstrength capacity under seismic response), where these actions exceed those of other ultimate limit state load combinations.

d. Design for serviceability – Vibration of beams (clause 3.3.3) and Steel reinforcement (clause 3.3.5)

Within clause 3.3.3, reference to AS 5100.2 *Bridge design* part 2 Design loads[30] shall be replaced by New Zealand Transport Agency *Bridge manual*.

Within clause 3.3.5, reference to AS 5100.5 *Bridge design* part 5 Concrete[9] shall be replaced by New Zealand Transport Agency *Bridge manual*, clause 4.3.1.

e. Shear buckling capacity (clause 5.10.5.2)

At the end of clause 5.10.5.2, amend the reference to appendix A in the note to appendix B.

f. Composite beams and composite compression members – references to AS 5100.5[9] (sub-section 6.2, clause 6.8.5, and sub-sections 10.6, 10.7, 10.8)

Within clauses 6.2.1, 6.2.3, and 6.8.5, reference to AS 5100.5[9] shall be replaced by NZS 3101[1] and New Zealand Transport Agency *Bridge manual*.

Within clause 6.2.4, the design life of the bridge shall be taken to be as specified in 2.1.5 of this manual.

Within clauses 10.6.1.2, 10.6.1.4, and 10.6.1.5, concrete of normal weight may conform with NZS 3101[1] as an alternative to AS 5100.5[9], and reinforcement may be designed and detailed in accordance with NZS 3101[1] as an alternative to AS 5100.5[9]. Where longitudinal reinforcement is placed in contact with the steel section as shown in figure 10.6.1.4, the bond associated with reduction in the effective perimeter of the bar may be similarly determined based on NZS 3101[1].

4.3.3 continued

Within clause 10.6.2.3, if δ is less than 0.2 the column may be designed as a reinforced concrete column in accordance with NZS 3101[(1)], as an alternative to AS 5100.5[(9)].

Within clause 10.6.2.4, creep coefficients (φ_{cc}) appropriate to New Zealand concrete as determined from NZS 3101[(1)] shall be applied in place of values determined from AS 5100.5[(9)]. Note that the compression member's elastic flexural stiffness should only be modified for creep for the effects of sustained long term loading inducing flexure in the member.

Within clause 10.8.3.4, shrinkage strains (ε_{cse} or ε_{cs}) appropriate to New Zealand concrete as determined from NZS 3101[(1)] shall be applied in place of values determined from AS 5100.5[(9)].

g. Design for longitudinal shear (clause 6.8.5.2.1)

Within clause 6.8.5.2.1, no reliance shall be placed on cohesion contributing to the shear strength as it is considered to be unreliable. k_{co} shall be taken as zero for all surface conditions. The higher coefficients of friction (μ) specified by NZS 3101[(1)] clause 7.7.4.3 may be adopted in place of those specified by table 6.8.5.2.1.

h. Members subjected to axial tension (clause 9.2.1)

Within clause 9.2.1 add to the definition for A_n: "For threaded rods, the net area shall be taken as the tensile stress area of the threaded portion as defined in AS 1275."

i. Members subjected to axial compression - correction of terminology (section 10)

Correct terminology within this section as follows:

Within clauses 10.1, and 10.2.1 in the definitions of N_{us} and N_{uc} replace "nominal section capacity" with "design (dependable) section capacity at the ultimate limit state" and "nominal member capacity" with "design (dependable) member capacity at the ultimate limit state" respectively.

Amend the heading of clause 10.3.3 to "Design (dependable) capacity of a member of constant cross section" and within the first paragraph of the clause replace "ultimate member capacity (N_{uc})" with "design (dependable) member capacity (N_{uc}) at the ultimate limit state".

Amend the heading of clause 10.6.2 to "Design (dependable) section capacity at the ultimate limit state" and within the first paragraph of clauses 10.6.2.1 and 10.6.2.2 replace "ultimate section capacity (N_{us})" with "design (dependable) section capacity (N_{us}) at the ultimate limit state". Within the first paragraph of 10.6.2.3 insert "design (dependable)" before "plastic resistance".

Amend the heading of clause 10.6.3 to "Design (dependable) member capacity at the ultimate limit state".

Amend the heading of clause 10.6.3.3 to "Design (dependable) member capacity at the ultimate limit state of composite members of constant cross section" and within the first paragraph of clause 10.6.3.3 replace "nominal member capacity (N_{uc})" with "design (dependable) member capacity (N_{uc}) at the ultimate limit state", and in the definition of N_{us} replace "nominal section capacity" with "design (dependable) section capacity at the ultimate limit state".

Amend the heading of clause 10.6.3.4 to "Design (dependable) member capacity at the ultimate limit state of composite members of varying cross section". Within the first paragraph of clause 10.6.3.4 replace "nominal member capacity (N_{uc})" with "design (dependable) member capacity (N_{uc}) at the ultimate limit state", and in (a) replace "nominal section capacity (N_{us})" with "design (dependable) section capacity (N_{us})".

4.3.3 continued

j. Members subject to combined actions (section 11)

The additional constraints imposed by NZS 3404:1997[23] clause 8.1.5 in respect to clause 8.3.4.2 shall apply to the use of the alternative method of AS/NZS 5100.6[24] clause 11.3.4.

For requirements for double bolted or welded single angles eccentrically loaded in compression, omitted from section 11, refer to NZS 3404:1997[23] sub-section 8.4.6.

Delete from the heading of sub-section 11.5 the word "compression" as the sub-section also considers tension combined with bending.

k. Connections (section 12)

Within clause 12.2.7, amend the definitions of snug tight as follows: "The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a podger spanner to ensure that the load transmitting plies are brought into effective contact."

Within clause 12.3.1, replace the first paragraph with: "Connections carrying calculated design actions, except for lacing connections, shall be designed to transmit the greater of the design action in the member and the following minimum actions: "

Within clause 12.3.4, replace the first sentence of the second paragraph with: "Where a mixture of non-slip fasteners is used, sharing of the load may be assumed except in connections forming part of a seismic resisting system in which the sharing of actions on the same element between welds and bolts is not permitted."

To clause 12.3.6 add a second paragraph: "When using Sections 9 and 10 for the design of connection components, take α_b = 0.5 and determine the net area (A_n) by means of a rational design procedure which accounts for the distribution of axial force into the component."

Within clause 12.5.3.4 merge the last sentence of the clause to the definition of a_e, to read:

"a_e = minimum distance from the edge of a hole to the edge of a ply, measured in the direction of the component of the force, plus half the bolt diameter. The edge of a ply shall be deemed to include the edge of an adjacent bolt hole."

With reference to clause 12.5.4.1, tests in accordance with NZS 3404:1997[23] clause 9.3.3.2.1 and appendix K to determine the slip factor for friction type connections in shear are an acceptable alternative to those specified by AS 4100 *Steel structures*[31].

Within clause 12.6.8.1, replace the paragraph with: "Plug and slot welds may only be used to transmit shear in lap joints or to prevent buckling of lapped parts or to join component parts of built up members or to prevent out of plane buckling of doubler plates in joint panel zones."

Within clause 12.6.10.1, add as a second paragraph to (a): "The butt weld shall be made using welding consumables that will produce butt tensile test specimens in accordance with AS 2205.2.1 for which the minimum strength is not less than the corresponding values for the parent material."

4.3.3 continued

l. Fatigue loading (section 13)

Amendments to AS/NZS 5100.6:2017(24) clauses are included below. These may not reflect changes in subsequent amendments to AS/NZS 5100.6(24).

Clause 13.5:

Amend (a) as follows: “no appropriate fatigue load is available in AS 5100.2 or the Waka Kotahi NZ Transport Agency *Bridge manual*, or”

Clause 13.6.1:

Add: “The safe life method (see Clause 13.6.3) shall be used unless otherwise agreed with the road controlling authority.”

Clause 13.6.4:

The high consequence values of capacity reduction factor ϕ_{Mf} in AS/NZS 5100.6(24) table 13.6.4 shall apply where fatigue failure of the component being assessed would lead to overall collapse of the structure. Note that the designations CC1, CC2, CC3 in AS/NZS 5100.6(24) table 13.6.4 align with consequence class (as used in BS EN 1990 *Basis of structural design*(32) annex B) and not the construction category defined in AS/NZS 5100.6(24) appendix L and AS/NZS 5131(26).

Clause 13.9.2.1:

Replace clause with:

“The characteristic values of equivalent nominal stress range for 2 million cycles $\Delta\sigma_{E,2}$ and $\Delta\tau_{E,2}$ shall be determined as follows:

$$\Delta\sigma_{E,2} = \lambda \times \Delta\sigma_{max} \qquad \text{...13.9.2.1(1)}$$

$$\Delta\tau_{E,2} = \lambda \times \Delta\tau_{max} \qquad \text{...13.9.2.1(2)}$$

Where:

$\Delta\sigma_{max}$, $\Delta\tau_{max}$	=	maximum stress range caused by the fatigue loads specified in Waka Kotahi NZ Transport Agency *Bridge manual* Clause 3.2.6
λ	=	damage equivalent factor depending on the spectra, as specified in Clause 13.9.2.2”

Clause 13.9.2.2:

Damage equivalent factors (λ) for the TT530 fatigue loading model (replacing the factors specified in AS/NZS 5100.6(24) clause 13.9.2.2) shall be determined as follows:

$$\lambda = \lambda_C \times \lambda_S \times \lambda_L \times \lambda_R \times \lambda_M \times \lambda_Y$$

Where:

λ_C	=	factor for heavy-vehicles-per-lane-per-day (3.2.6.f)
λ_S	=	factor for stress type (not included in AS/NZS 5100.6(24))
λ_L	=	factor for effective span length (3.2.6.f)
λ_R	=	route factor adjustment (3.2.6.f)
λ_M	=	factor for loading from multiple lanes (3.2.6.d)
λ_Y	=	factor for service life (3.2.6.g)

These factors are defined below.

4.3.3 continued

i. λ_C factor for heavy-vehicles-per-lane-per-day

λ_C-values for the TT530 loading model shall be calculated with reference to table 4.6 or using

$\lambda_C = (0.05 \times \text{heavy-vehicles-per-lane-per-day})^{0.2}$

Table 4.6: λ_C factor for TT530 fatigue loading model

Heavy vehicles per lane per day	100	200	300	500	750	1000	1250	1500	2000	3000
λ_C	1.38	1.58	1.72	1.90	2.06	2.19	2.29	2.37	2.51	2.72

ii. λ_S factor for stress type (refer to the *Bridge manual commentary*)

λ_S = 1.24 for normal stress ranges $\Delta\sigma_{max}$

λ_S = 1.0 for shear stress ranges $\Delta\tau_{max}$

iii. λ_L factor for effective span length

For the TT530 fatigue loading model:

$\lambda_L = 2.0^{0.2} = 1.15$ for L≤5m

$\lambda_L = \left({}^{10}/_{L}\right)^{0.2}$ for 5m<L<16.7m, or

$\lambda_L = 0.6^{0.2} = 0.90$ for L≥16.7m

where L is as defined in 3.2.6(f).

iv. λ_R route factor adjustment

The λ_R-value is calculated using the route factor values in 3.2.6(f) as

$\lambda_R = (\text{route factor})^{0.2}$

Table 4.7: λ_R factor for TT530 fatigue loading model

Route description	Route factor	λ_R
Freight routes with a very high proportion of fully loaded long vehicles in one or both directions.	1.0	1.0
Typical long-haul freight routes, national and regional routes including expressways	0.8	0.96
Motorways, other rural freight routes	0.6	0.90
High-volume urban motorways and urban arterial routes	0.4	0.83
Urban roads	0.3	0.79

v. λ_M multiple lane adjustment factor

In accordance with 3.2.6(d) the fatigue loadings in Lane 1 and Lane 2 shall be applied separately when calculating the maximum peak to peak stress range, and to allow for possible simultaneous fatigue loading the total fatigue damage (after combining the separate contributions from Lanes 1 and 2) shall be multiplied by factor K_bZ.

The following equation combines the fatigue loadings from Lane 1 and Lane 2 and incorporates the factor K_bZ for possible simultaneous fatigue loading.

4.3.3 continued

$$\lambda_{M} = \left[\left(1+\frac{N_2}{N_1}K_b^5\right)K_bZ\right]^{0.2}$$

Where:

N_1	=	heavy-vehicles-per-lane-per-day in Lane 1
N_2	=	heavy-vehicles-per-lane-per-day in Lane 2
K_bZ	=	as defined in 3.2.6(d)

vi. λ_Y service life adjustment factor

The λ_Y-value for different service lives in years shall be as given in table 4.8.

Table 4.8: λ_Y factor for TT530 fatigue loading model

Life (years)	100	120	150
λ_Y	1.00	1.08	1.16

Clause 13.9.3:

Replace clause with:

"The characteristic value of modified nominal stress ranges for 2 million cycles $\Delta\sigma_{E,2}$ and $\Delta\tau_{E,2}$ shall be determined as follows:

$$\Delta\sigma_{E,2} = k_t\lambda \times \Delta\sigma_{max} \qquad \text{...13.9.3(1)}$$

$$\Delta\tau_{E,2} = k_t\lambda \times \Delta\tau_{max} \qquad \text{...13.9.3(2)}$$

Where:

$\Delta\sigma_{max}$, $\Delta\tau_{max}$	=	maximum stress range caused by the fatigue loads specified in the Waka Kotahi NZ Transport Agency *Bridge manual* Clause 3.2.6
λ	=	damage equivalent factor depending on the spectra, as specified in Clause 13.9.2.2
k_t	=	stress concentration factor to take account of the local stress magnification in relation to detail geometry not included in the reference $\Delta\sigma_R$-N curve

Note: k_t-values may be taken from PD 6695-1-9 Recommendations for the design of structures to BS EN 1993-1-9 or from appropriate finite element calculations."

Clause 13.9.4:

Replace clause with:

"Unless more accurate calculations are carried out, the design value of modified nominal stress range shall be determined as follows:

$$\gamma_{Ff}\Delta\sigma_{E,2} = \gamma_{Ff} \times k_1 \times \Delta\sigma^*_{E,2} \qquad \text{...13.9.4}$$

Where:

$\Delta\sigma^*_{E,2}$	=	modified nominal stress range calculated with a simplified truss model with pinned joints
γ_{Ff}	=	load factor for equivalent constant amplitude stress range = 1
k_1	=	magnification factor according to Table 13.7(A) and Table 13.7(B)"

4.3.3 continued

Clause 13.9.6:

The damage equivalent factor λ_V within clause 13.9.6 shall mean the damage equivalent factor (λ) recalculated using the slope m specified for the relevant fatigue strength curve. For example, clause 13.10.2 specifies m = 8 for headed studs, and so the damage equivalent factor λ_V can be calculated by replacing 5 and 0.2 with 8 and 0.125 in the equations in clause 13.9.2.2.

Clause 13.10.1:

Fatigue stress ranges for road bridge fatigue loading models shall be treated as variable amplitude stress spectra in Miner's summations.

AS/NZS 5100.6[24] equations 13.10.1(3) and 13.10.1(4) require corrections (pending) but are for constant amplitude stress ranges so shall not be used with road bridge fatigue loading models.

AS/NZS 5100.6[24] equations 13.10.1(5) and 13.10.1(6) shall be replaced with:

$\Delta\sigma_{\mathrm{R}}^{m} N_R = (0.585\Delta\sigma_{\mathrm{C}})^m \ 1{\times}10^7$ with m=5 for N > 10^7 ...13.10.1(5)

$\Delta\tau_{\mathrm{R}}^{m} N_R = (0.457\Delta\tau_{\mathrm{C}})^m \ 1{\times}10^8$ with m=9 for N > 10^8 ...13.10.1(6)

Clause 13.11.1:

Add:

"The verification formats in Appendix I Clause I1.6 may be used in conjunction with stress ranges and cycle counts for fatigue loads applied in multiple lanes as specified in the Waka Kotahi NZ Transport Agency *Bridge manual* Clause 3.2.6. The damage summation after combining the separate contributions from multiple lanes shall be multiplied by factor $K_b Z$ as defined in clause 3.2.6(d)."

m. Brittle fracture resistance (section 14)

Within clause 14.3, add to the end of the second paragraph: "The steel grade shall be selected to match the required steel type in accordance with 14.5.4."

Within table 14.5.1, replace the permissible service temperatures for steel types 2S and 5S with the following permissible service temperatures for these steel types:

Steel type	Permissible service temperature, °C					
	Thickness, mm					
	≤6	>6 ≤12	>12 ≤20	>20 ≤32	>32 ≤70	>70
2S & 5S	-35	-25	-15	-10	-10	-5

Within clause 14.5.3.4, replace paragraphs (b) and (e) with the following:

"(b) Three Charpy test specimens shall be taken from the area of maximum strain and tested at the design service temperature or lower."

"(e) Where a plate or component thickness prevents a 10 mm x 10 mm test piece from being used, the standard test specimen thickness given in Appendix K Table K4 closest to the plate or component thickness shall be used. Where the standard to which the steel complies does not specify minimum impact properties, the average absorbed energy for three test specimens and the minimum absorbed energy of each specimen shall be not less than those given in Table K4 for the relevant size of test specimen."

Within clause 14.5.4, following the first sentence add: "For steels conforming to BS EN 10025 and JIS G 3106 refer to Table H4.1 or NZS 3404:1997 Table 2.6.4.4."

4.3.3 continued

At the end of clause 14.5.4 add the following two paragraphs:

"Welding consumables selection shall be in accordance with AS/NZS 1554.1 or AS/NZS 1554.5.

The impact resistance of bolts must not be less than that for the grades of steel that they are joining."

n. Proof testing (sub-section 15.3)

Replace sub-section 15.3 with: "Proof load testing shall comply with Clause 7.6 of the New Zealand Transport Agency *Bridge manual*."

4.3.4 Seismic resistance

Where materials design codes other than NZS 3404:1997[23] are applied, if steel members are required to provide the ductility and energy dissipating capability of the structure, the principles set out in section 12 of NZS 3404:1997[23] shall be followed. The recommendations of the NZNSEE study group on *Seismic design of steel structures*[33] shall also be followed where applicable.

4.3.5 Fatigue design

Assessment of the fatigue resistance of steel structure components shall be based on the respective design standard adopted for the design of the component as per 4.3.1. For comment on the fatigue loading see 3.2.6.

Fatigue design parameters should be recorded in project hand-over documentation required by the *Highway structures design guide*[34], including:

- assumed heavy-vehicles-per-day in each direction and assumed allocation into heavy-vehicles-per-lane-per-day
- fatigue loading standard and fatigue loading vehicle(s)
- assumed lane arrangements for the fatigue vehicle placement
- assumed route factor
- fatigue assessment method and capacity reduction factors (AS/NZS 5100.6[24] clause 13.6) with corresponding requirements for future inspection and maintenance.

4.3.6 Durability and corrosion protection

a. Corrosion protection systems

Corrosion protection systems for structural steelwork shall comply with *Protective coatings for steel bridges: a guide for bridge and maintenance engineers*[35], NZTA S9 *Specification for coating steelwork on highway structures*[36], and SNZ TS 3404 *Durability requirements for steel structures and components*[37].

Primary structural members and elements not easily accessed or replaced (eg bearing plates, deck joint components) in steel shall be corrosion protected with a system capable of achieving a time to first maintenance of at least 40 years unless agreed otherwise with the road controlling authority.

Secondary steelwork elements (eg barriers, handrails) shall be corrosion protected with a system capable of achieving a time to first maintenance of at least 25 years.

The terminology "time to first maintenance" and "time to first major maintenance" shall be taken to have the same meaning and to be as defined by SNZ TS 3404[37] clause 1.7. Additional guidance on the selection of corrosion protection systems, in particular for those systems capable of achieving an expected life to first maintenance of in excess of 40 years, is given in *Protective coatings for steel bridges: a guide for bridge and maintenance engineers*[35] and SNZ TS 3404[37]. Where the corrosivity of the environment is such that achieving the above levels of performance is impractical or not economically viable, a lower level of performance may be proposed and justified within the structure design statement.

4.3.6 continued

Linkage bars and hold-down devices providing interconnection between the primary elements of the structure as per 2.1.7 shall satisfy the durability requirements of 2.1.6 and appendix C clause C1.6.

For steelwork coloured for aesthetic purposes (ie colours other than the generic grey from metallic coatings, such as thermal metal spray or galvanizing), the coloured top coat will typically require refurbishment every 10-15 years for polyurethanes, or up to 30 years for fluoropolymers, depending on the chosen colour and coating selected. As such, it is accepted that the time to first maintenance of at least 40 years is not achievable and a lower level of performance may be proposed. This will require approval from the road controlling authority for this departure from the requirements of this manual.

Thermal metal spray systems shall be seal coated as specified in NZTA S9 *Specification for coating steelwork on highway structures*(36) clause 11.6, for the given application, to give uniformity of appearance and increase durability.

b. Weathering steel

The design and detailing of weathering steel bridges shall be undertaken in accordance with HERA report R4-97 *New Zealand weathering steel guide for bridges*(38).

In addition, the following requirements shall be complied with:

- The weathering steel shall be resistant to brittle fracture and lamellar tearing in accordance with section 14 of AS/NZS 5100.6(24).
- Electrodes used for welding shall be of comparable weathering properties to that of the parent metal. For each type of electrode used, a test sample shall be made of a 10mm fillet weld on the parent weathering steel and tested to ensure that the weld metal's chemical composition is within the allowable range specified in the relevant standard for the parent metal.
- After fabrication and prior to erection, all weathering steel components shall be abrasive blast cleaned with non-metallic grit to SSPC SP 6/NACE No. 3 *Commercial blast cleaning*(39) to remove mill scale and other contaminants. This shall be immediately followed by a minimum of three cycles of wetting using potable water and drying, to assist in the formation of the protective patina and provide a uniform finish. In the event of fabrication being undertaken offshore from New Zealand, this shall be undertaken following delivery to New Zealand of the fabricated steelwork.

4.3.7 Certification of steel

All steel, bolts, nuts and washers shall comply with the requirements of AS/NZS 5131(26) and standards listed therein. Additional acceptable compliance standards to those listed in AS/NZS 5131(26), acceptable for compliance to, for specific materials, are:

- For nuts: AS 1112 *ISO metric hexagon nuts*(40)
- For washers: AS 1237.1 *Plain washers for metric bolts, screws and nuts for general purposes – General plan*(41)
- For high tensile bars: BS 4486 *Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete*(42)

Evidence of compliance with the specified standards shall be obtained and shall comprise test reports or test certificates prepared by a laboratory recognised by signatories to the International Laboratory Accreditation Cooperation (ILAC) Mutual Recognition Agreement (MRA) on behalf of the manufacturer. These documents are to be traceable to the specific batches of material used.

4.3.7 continued

Alternatively for fasteners, an IANZ (International Accreditation New Zealand) endorsed proof load and wedge test certificate showing they comply with the specified standard may be provided.

The requirements for the specification of structural steel, its supply and compliance are currently undergoing review and revision. In addition to the above current requirements note shall be taken of any Technical Advice Note published by Waka Kotahi NZ Transport Agency dealing with this subject (TAN#17-09 *Verification testing of steel materials* at the time of publication) and the mandatory requirements set out therein shall be complied with.

Construction categories as nominated in AS/NZS 5131 shall be in accordance with any Technical Advice Note published by Waka Kotahi NZ Transport Agency dealing with this subject; or any project principal's requirements; or as otherwise agreed with the road controlling authority and as detailed in the structure design statement.

4.4 Timber

4.4.1 General

Design shall be in accordance with the appropriate following standards, except as modified by 4.4.2:

- AS 5100.9 *Bridge design* part 9 Timber(43)
- NZS 3603 *Timber structures standard*(44) for the timber species and types of timber materials that it covers that are not covered by AS 5100.9(43)
- AS 1720.1 *Timber structures* part 1 Design methods(45) for the timber species and types of timber materials that it covers that are not covered by AS 5100.9(43) or NZS 3603(44)
- Characteristic stresses adopted for design to AS 1720.1(45) shall be in accordance with AS 1720.2 *Timber structures* part 2 Timber properties(46) and AS/NZS 2878 *Timber - Classification into strength groups*(47).

4.4.2 Strength reduction factors, characteristic stress/strength modification factors and live load factor

Strength reduction factors shall conform to those given in AS 5100.9(43) table 3.2 corresponding to the type of timber product and/or jointing being considered. Where not covered by AS 5100.9(43), strength reduction factors shall be derived from AS 1720.1(45) table 2.1, corresponding to the type of timber product (eg sawn timber, round timbers) and type of grading (eg visually graded, machine graded, proof graded) and/or jointing being considered. For secondary structural elements they shall be those for AS 1720.1(45) table 2.1 category 2 structural members and joints, and for primary structural elements they shall be those for AS 1720.1(45) table 2.1 category 3 structural members and joints.

For the grid system or parallel support system modification factor (k_4, k_5 or k_6 in NZS 3603(44) or k_9 in AS 1720.1(45)) to apply, in the event of the failure of a single supporting member, the overlying members or sheathing material shall be capable of transferring loads to the adjacent supporting members. Otherwise the grid system or parallel support system modification factor shall be taken as 1.0.

Where a bridge or other structure possesses smooth sealed approaches the live load dynamic load factor may be taken as follows:

Dynamic load factor = $1.0 + (I - 1.0) \times 0.7$

Where I is the dynamic load factor defined by 3.2.5.

4.4.3 Seismic resistance

Design shall comply with NZS 3603[44] clause 2.12 for seismic resistance except that the design loading shall be in accordance with this document.

4.4.4 Durability

In order to ensure long-term durability in timber bridge members, particular attention shall be given to the provisions of AS 5100.9[43] section 4 in general, and for stress laminated timber, of AS 5100.9[43] section 5, and to the following:

- in-service moisture content and the effects of its variation
- member deflections
- connection design.

4.5 Aluminium

Design shall be in accordance with AS/NZS 1664.1 *Aluminium structures* part 1 Limit state design[48] with the following provisos:

a. Loading (clause 2.3)

   The loads on the structure shall be in accordance with this document.

b. Loading combinations and load factors (clause 2.4)

   The required forces, moments and stresses for the applicable loads shall be determined by structural analysis for the load combinations as indicated in this document.

c. Earthquake (clause 2.5)

   All structures shall be designed for the loads and load combinations specified in this document. The limitations on structural ductility factor given in AS/NZ 1664.1[48] clause 2.4(b)(i) and (ii) shall apply. The structural performance factor (S_p) shall be as specified in this document.

4.6 Other materials

The criteria applying to the use of materials not mentioned in this document will be subject to the approval of the road controlling authority.

4.7 Bearings and deck joints

4.7.1 General

a. Design code

The design and performance of bearings and deck joints shall comply with AS 5100.4 *Bridge design* part 4 Bearings and deck joints[(49)] except as modified herein. Where there may be conflict between the requirements of AS 5100.4[(49)] and this document, this document shall take precedence.

b. Preferred bearing types

Bridge bearings shall preferably be elastomeric bearings. Where mechanical bearings are used they shall be pot or spherical approved sliding material (ASM) bearings (including sliding or sliding guided pot and spherical ASM bearings). Alternative bearing types may be considered provided their equivalence to that specified can be demonstrated to the satisfaction of the road controlling authority. Roller bearings shall not be used.

c. Elastomeric bearings

Reference to elastomeric bearings herein shall also include laminated elastomeric bearings fitted with a lead cylinder, commonly referred to as lead-rubber bearings, used for the dissipation of earthquake energy.

d. Deck joints

The number of deck joints in a structure shall be the practical minimum.

In principle, deck slabs should be continuous over intermediate supports, and bridges with overall lengths of less than the limits specified by 4.8.2(a) and skews of less than 30° should have integral or semi-integral abutments. It is accepted that deck joints may be necessary in longer bridges to cater for periodic changes in length. They may also be necessary where the structural system, adopted with the objective of minimising earthquake damage (eg base isolation with mechanical energy dissipation, or rocking piers), requires the structure to be free to displace.

The form of deck joints to be used shall be nominated in the structure options report or structure design statement and shall be subject to the approval of the road controlling authority. For bridges requiring deck joint gaps exceeding 25mm, the preferred form of deck joint for Waka Kotahi NZ Transport Agency is the single elastomeric seal retained by metal nosings. For very long bridges where appropriate, single seal deck joints at maximum spacing along the bridge length are preferred to multiple seal joints.

Deck joints with aluminium nosings shall generally not be used, although the road controlling authority may consider a departure to this requirement. The departure request shall address in particular the durability and robustness of aluminium nosings to withstand service conditions. Verification of in-service performance of the proposed joint on similar structures shall be provided.

e. Access and provision for inspection, maintenance and repair

Access and provisions for inspection, maintenance and repair of bearings and deck joints shall comply with 2.1.9.

4.7.2 Modifications and extensions to the AS 5100.4[49] criteria for bearings

a. Limit state requirements and robustness

Elastomeric, lead-rubber, pot and spherical ASM bearings shall be designed for the serviceability, ultimate and seismic damage control limit states.

The robustness and displacement capacity of bearings and their fixings shall be checked at the damage control limit state, and shall also be sufficient to ensure that sufficient overall integrity of the structure is maintained and collapse avoided under seismic response from a major earthquake (see table 5.1).

b. Bearings inspection and replacement

All bearings, other than thin elastomeric strip bearings less than 25mm in thickness, shall be able to be inspected and replaced without the removal of any structural concrete or cutting of steelwork.

Provision shall be made in the design for jacking from the substructure sills on which the beams are supported during bearing replacement. Replacement of bearings shall be possible with minimal disruption to traffic on the bridge, or to traffic beneath the bridge.

c. Design loads and movements and load factors

Reference in AS 5100.4[49] to design loads and load factors given in AS 5100.2[30] shall be replaced by reference to chapter 3 of this manual. For limits on movement and restraint of movement see AS 5100.4[49].

The potential variability in coefficients of friction of sliding surfaces and in the stiffnesses of elastomeric bearings shall be taken into account with the effect of both upper and lower bound values for these being considered in the design.

In the design of sliding contact surfaces incorporating PTFE, the coefficient of friction to be used for analysis shall be assessed on a conservative basis for the situation being considered. A coefficient of 0.00 shall be assumed as the coefficient of friction for situations where a minimum frictional force is appropriate. For situations where a maximum frictional force is appropriate the coefficients of friction stated in AS 5100.4[49] clause 11.2 may be adopted, except for seismic, tsunami and collision loading combinations where a coefficient of at least 0.15 shall be assumed. Further guidance may be obtained from section 14.7.2 of the AASHTO *LRFD Bridge design specifications*[50], and associated commentary and the AASHTO *Guide specifications for seismic isolation design*[51].

The upper and lower bound stiffnesses adopted in design for elastomeric bearings shall be consistent with the manufacturing tolerances specified for their shear and compression stiffnesses. For bearings manufactured to AS 5100.4[49] a tolerance on shear stiffness of ±20% is applicable. Tolerances on compression stiffness and rotational stiffness are not stated in the standard, but it is suggested that ±25% be adopted where these properties are important.

d. Anchorage of bearings

Bearings, other than thin elastomeric strip bearings less than 25mm in thickness, shall be positively anchored to the bridge structure above and below to prevent their dislodgement during response to the damage control limit state design intensity or greater earthquake unless the bridge superstructure is fully restrained by other means against horizontal displacement relative to the support. Reliance shall not be placed on friction alone to ensure safety against sliding. The bearing restraint system for horizontal load shall be designed to resist the full horizontal force to be transmitted by the bearing from the superstructure to the substructure.

4.7.2 continued

For laminated elastomeric bearings, horizontal restraint shall be provided by dowels or bolts engaging in thick outer shims within the bearing or by vulcanising the bearings to external plates that are fixed in position to the structure by bolts. External restraining cleats shall not be used. Dowels shall generally be located as close to the centre of the bearing (in plan) as practicable, to prevent them from disengaging due to deformation of the edges of the bearing under the high shear strain that may be developed during response to a strong earthquake.

Bearings shall be mounted on either cast concrete or proprietary shrinkage compensated mortar having a 28-day compressive strength of at least 50MPa. A smooth surface on the supporting concrete or mortar shall surround the bearing and have a dimension from the edges of the bearing of at least the height of the bearing.

Dowels, as a means of bearing lateral restraint, do not need to be removable to allow bearing replacement provided that the bridge superstructure can be jacked sufficiently to enable the bearings to be lifted, disengaged from the restraining dowels, and slid out of position.

e. Bearing set back from the edge of concrete bearing surfaces and confinement of bearing surfaces

Bearings shall be set sufficiently far back from the edge of concrete bearing surfaces to avoid spalling of the corner concrete, and where bearing pressures are high, confining reinforcement shall be provided to prevent tensile splitting of the concrete. Consideration shall be given to the redistribution of pressure on the concrete bearing surface due to horizontal loads such as from earthquake action.

f. Elastomeric bearings

Elastomeric bearings shall conform with the requirements of AS 5100.4[(49)], except that steel reinforcing plates may be a minimum of 3mm thick.

Wherever feasible, bearings shall be chosen from those commercially available, but this does not preclude the use of individual designs where circumstances justify it.

Under service conditions that exclude earthquake and tsunami effects and collision loading, the maximum shear strain in a bearing (measured as a percentage of the total rubber thickness being sheared) shall not exceed 50%. Under response to the damage control limit state design intensity earthquake and ultimate limit state tsunami and collision loading, plus other prevailing conditions such as permanent loads and shortening effects, corresponding to table 3.3 load combinations 5A, 5B and 5C, the maximum shear strain shall not exceed 100%.

In the design of elastomeric and lead-rubber bearings, the following considerations shall be given particular attention:

- In evaluating the stability against roll-over, consideration shall be given to the sensitivity of the stability to an extreme earthquake such as the collapse avoidance limit state event, as safety factors can be rapidly eroded.
- In bridges with prestressed concrete superstructures and the spans either continuous or tightly linked, consideration shall be given to the long-term effects of shrinkage and creep shortening of the superstructure due to the prestress on the bearings.

g. Sliding bearings

Bearings containing sliding surfaces shall have the sliding surfaces protected from dirt and debris ingress that may affect the performance of the bearing.

4.7.2 continued

h. Design life

In respect to bearings, AS 5100.4[49] clause 7.3 shall be replaced with the following:

Bearings shall satisfy the requirements of 2.1.6(b) which includes a minimum service life of 40 years. Cast in components shall have the same design life as the bridge.

i. Testing of laminated elastomeric bearings

Testing of laminated elastomeric bearing, with or without lead plugs, shall comply with AS 5100.4[49] appendix D extended and modified as follows:

Compression quality assurance tests shall be carried out on all bearings.

Bearings that are not subjected to relative horizontal displacements between superstructure and substructure need not have compression and shear stiffness tests carried out unless it is critical for the design of the structure or the restraint of the bearing that these parameters are confirmed.

For bearings that are subjected to relative horizontal displacements between superstructure and substructure, shear stiffness tests on a representative sample of each type and size of bearing shall be carried out. In order to determine the representative sample, all such bearings shall be subjected to a compression stiffness test. For each type and size, the two bearings with the highest compression stiffness and the two bearings with the lowest compression stiffness shall be chosen to make up pairs for shear stiffness testing. If there are more than 40 bearings of the same type and size, additional pairs of bearings shall be selected so that at least 10% of the bearings are tested for shear stiffness.

For bearings subjected to shear strains under the seismic DCLS load combination of less than 75%, the shear stiffness test shall be carried out as described in AS 5100.4[49] clause D4.2, modified as follows:

Replace (d) with:
While maintaining the compressive load, apply shear loads to the plate in a direction parallel to the short dimension of the bearings. Increase the shear load at a uniform rate of 25mm/minute from zero shear to 1.5 times the rated shear deflection (ie 75% shear strain) and maintain shear load for one minute. Carry out a visual examination at maximum shear displacement for misplaced steel plates, bond failure and surface defects(eg tears and splits), with test summary sheet and inspection personnel as for the compression quality assurance test.

For bearings subjected to shear strains under the seismic DCLS load combination of more than 75% (and less than 100%), the shear stiffness test shall be carried out as described in AS 5100.4 clause D4.2, modified as follows:

Replace (d) with:
While maintaining the compressive load, apply shear loads to the plate in a direction parallel to the short dimension of the bearings. Increase the shear load at a uniform rate of 25mm/minute from zero shear to 2.0 times the rated shear deflection (i.e. 100% shear strain) and maintain shear load for one minute. Carry out a visual examination at maximum shear displacement for misplaced steel plates, bond failure and surface defects (eg tears and splits), with test summary sheet and inspection personnel as for the compression quality assurance test.

4.7.2 continued

Replace (h) with:
While maintaining the compressive load, reapply the shear load at the uniform rate of 25mm/minute taking readings of the shear load at shear strains of 5%, 15%, 25% and 50%. At each reading level, maintain the shear deflection for 30 seconds prior to taking the shear load reading.

Replace (i) with:
From the recordings, calculate the mean shear stiffness and hence the shear modulus (G) between 5% and 25% strain and between 5% and 50% strain for one bearing. G shall be calculated in accordance with 12.7.2.

The shear stiffness of the bearings between 5% and 25% shear strain shall be within ±20% of the design value. Compression stiffness shall be within ±25% of the design value. The designer may choose to accept bearings outside these stiffness ranges if the bridge design can accommodate the additional variation.

For bridges where elastomeric bearings provide the entire seismic restraint to the superstructure, the designer shall consider augmenting the above shear stiffness testing requirements. Guidance may be obtained from the AASHTO *Guide specifications for seismic isolation design*(51).

4.7.3 Modifications to the AS 5100.4(49) criteria for deck joints

a. General requirements

The maximum opening of a deck joint will generally be determined by earthquake conditions at the damage control limit state. No limitation applies to the maximum design width of an open gap joint under these conditions.

The maximum width of open gap between expansion joint components at deck surface level, at the ultimate limit state, under non-seismic load combinations in table 3.3, shall not exceed 85mm.

Where pedestrians, cyclists or animals have direct access over deck joints, all open gap deck joints shall be sealed or covered, but deck joints otherwise do not need to be covered.

b. Design loads

Deck joints and their fixings shall be designed for the following loads in place of those specified by the AS 5100.4(49):

i. Vertical at ultimate limit state

The vehicle axle loads defined in 3.2.2 factored by the dynamic load factor defined in (iv). The ultimate limit state load factors to be applied shall be 2.25 to an HN axle load, and 1.50 to an HO axle load.

ii. Vertical at serviceability limit state

The HN axle load defined in 3.2.2 factored by the dynamic load factor defined in (iv) together with a serviceability limit state load factor of 1.35.

iii. Fatigue

The HN axle load defined in 3.2.2 factored by the dynamic load factor defined in (iv) together with a load factor of 0.80.

4.7.3 continued

iv. Dynamic load factor

A dynamic load factor shall be applied to the ultimate limit state and serviceability limits state vertical loads and the fatigue load. The dynamic load factor to be applied shall be taken as 1.60 except for modular bridge expansion joints for which it shall be derived as 1.0 plus the dynamic amplification factor (DAF) determined in accordance with AS 5100.4[(49)] clauses 20.3.9 and 20.3.10.

v. Longitudinal

The local vehicle braking and traction forces specified in 3.3.1, combined with any force due to the stiffness of, or friction in, the joint. The ultimate limit state load factor to be applied to the combined force shall be 1.35.

c. Movements

i. Deck joints shall be designed to accommodate the movements due to temperature, shortening and earthquake specified in 5.7.1 and to otherwise satisfy the requirements of 5.7.1.
ii. Deck joints shall be designed to accommodate the ultimate limit state movements from table 3.3 load combinations, except those for load combination 5A (seismic), and shall include the effect of beam end rotation under live load.

d. Anchorage

The second paragraph of AS 5100.4[(49)] clause 19.4 shall be replaced by the following:

Where the deck joint is attached by bolts, fully tensioned high tensile bolts of property class 8.8 or higher shall be used. The spacing of the bolts shall not be greater than 300mm and the bolts shall develop a dependable force clamping the joint to the concrete substrate, of not less than 500kN per metre length on each side of the joint.

Where appropriate, deck joint anchor bolts shall be sleeved through the deck and anchored on the underside with nuts & locknuts. Such hardware shall be replaceable.

e. Drainage

The AS 5100.4[(49)] clause 19.5 shall be replaced by the following:

Deck joints shall be watertight unless specific provision is made to collect and dispose of the water. Deck run-off shall be contained from spilling over the sides of the bridge. Sealed expansion joints, where the gap is sealed with a compression seal, elastomeric element or sealant, are preferred.

Open joints, where the gap is not sealed, shall be slightly wider at the bottom than at the top to prevent stones and debris lodging in the joint, and shall include a specific drainage system to collect and dispose of the water. Such drainage systems shall be accessible for cleaning.

The design of drainage systems shall accommodate a serviceability limit state movement across the deck joints of the bridge of not less than the greater of:

- one quarter of the calculated relative movement under the damage control limit state design earthquake conditions, plus long-term shortening effects where applicable, and one half of the temperature induced movement from the median temperature position; and
- long term shortening plus the full design temperature induced movement from the median temperature position

without sustaining damage. Under greater movements, the drainage system shall be detailed so that damage is confined to readily replaceable components only.

4.7.3 continued

f. Installation

Deck joints and the parts of the structure to which they are attached shall be designed so that the joint can be installed after completion of the deck slab in the adjacent span(s).

g. Design life and provision for access, resetting and replacement

In respect to deck joints, AS 5100.4[(49)] clause 7.3 shall be replaced with, and clauses 7.6, 7.7 and 7.9 modified by, the following:

Deck joints, with the exception of modular bridge expansion joints, shall satisfy the requirements of 2.1.6(b) which includes a minimum service life of 40 years. Cast in components shall have the same design life as the bridge. Notwithstanding the requirements of 2.1.6(b) for movement joints as a whole, replaceable elastomeric seal components of joints shall have a minimum service life of 15 years and shall be replaceable without the need for modification to the joint and adjacent structural elements. Modular bridge expansion joints shall satisfy the service life requirements of AS 5100.4[(49)] clause 20.3.2.

Asphaltic plug (elastomeric concrete) joints have a limited service life and are highly unlikely to meet the minimum life of 40 years for replaceable deck joint components specified by 2.1.6(b). The use of this form of deck joint in high traffic environments shall be avoided where possible and otherwise shall be subject to the approval of the road controlling authority.

In general most forms of large movement deck joints can be subject to deterioration and eventual failure within the design life of a bridge. Consideration shall be given to, and provision shall be made for, the eventual need to reset or replace deck joints in the design of the structure supporting the deck joint, including the provision of adequate access to facilitate the deck joint inspection, maintenance, and resetting or replacement. This applies especially to multiple seal modular deck joints and forms of deck joint involving mechanical componentry and below joint surface drainage systems.

4.7.4 Additional criteria and guidance for deck joints

a. Joint type and joint system selection

Movement joints shall be selected on the basis of low life-time costs and maintenance requirements, and user (vehicle, cyclist and pedestrian) safety.

Deck joints shall be designed to provide for the total design range and direction of movement expected for a specific installation. The guidance provided in appendix A of CD 357 *Bridge expansion joints*[(52)] shall be considered with respect to the movement capacity of common joint types.

Acceptance of a proprietary joint system shall be subject to that system satisfying the requirements of this manual and the additional project-specific performance requirements. All dimensional and performance requirements, including movement capacity, shall be specified in the design to enable manufacturers to offer joints that are best suited to meet the requirements.

The characteristics and performance history of a particular joint shall be reviewed to determine the suitability of the joint for a specific installation. The information provided in *Performance of deck expansion joints in New Zealand road bridges*[(53)] and *Bridge deck expansion joints*[(54)] shall be considered with respect to the performance history of deck joints.

4.7.4 continued

Proprietary deck joint suppliers shall provide a warranty on the serviceability (ie durability and performance) of their joint(s) for a period of ten years after installation. The warranty shall cover all costs associated with rectification of a joint, including traffic control costs.

b. Joint sealing elements

Joint sealing elements (eg compression seals, elastomeric membrane seals, sealants) shall be resistant to water, oil, salt, stone penetration, abrasion and environmental effects, and shall be readily replaceable. Compression seals shall not be used in situations where concrete creep shortening and/or rotation of the ends of beams under live loading will result in decompression of the seal.

Sealants shall be compatible with the materials with which they will be in contact. It is typically necessary to provide a separation barrier between sealant and bituminous deck surfacing. Irrespective of claimed properties, sealants shall not be subjected to more than 25% strain in tension or compression. The modulus of elasticity of the sealant shall be appropriate to ensure that, under the expected joint movement, the tensile capacity of the concrete forming the joint is not exceeded. The joint shall be sealed at or as near the mean of its range of movement as is practicable. Base support for joint sealants shall be provided by durable compressible joint fillers with adequate recovery and without excessive compressive stiffness.

Joint seals or sealant shall be set 5mm lower than the deck surface to limit damage by traffic.

c. Nosings

New bridges and deck replacements shall be designed with a concrete upstand the height of the carriageway surfacing thickness and at least 200mm wide between the deck joint and the adjacent carriageway surfacing. This is to act as a dam to retain the surfacing and to isolate the surfacing from any tensile forces imposed on the deck by the joint system.

d. Asphaltic plug (elastomeric concrete) joints

Asphaltic plug joints are in-situ joints comprising a band of specially formulated flexible material, commonly consisting of rubberised bitumen with aggregate fillers. The joint is supported over the gap by thin metal plates or other suitable components.

Except in retrofit applications where the existing structural configuration prevents these joint dimensional requirements being met, elastomeric concrete plug joints shall be designed and specified to have a minimum thickness of 75mm and a minimum width of bond with the structure on either side of the joint gap of 200mm. Such joints shall be designed by the supplier or the supplier's agent to take account of the predicted movements at the joint including rotation of the ends of the bridge decks to be joined due to traffic loads.

Where proposed for use in retrofit situations with dimensions less than those specified above, evidence shall be supplied to the road controlling authority of satisfactory performance of the joint system under similar or more demanding traffic conditions with a similar joint configuration over periods of not less than 5 years.

4.8 Integral and semi-integral abutments

4.8.1 Definitions

a. An integral abutment is defined as one that is built integrally with the end of the bridge superstructure and with the supporting piles or footing. The abutment therefore forms the end diaphragm of the superstructure and the retaining wall for the approach filling. The supporting piles are restrained against rotation relative to the superstructure, but are free to conform to superstructure length changes by pile flexure.

b. A semi-integral abutment is defined as an integral abutment that contains provision for relative rotation, but no more than limited translation, between the superstructure and the supporting piles or footing.

4.8.2 Design criteria

Integral and semi-integral abutments are acceptable for bridges that meet the following criteria:

a. Length between the rear faces of abutments not exceeding:

   – with concrete superstructure 70m
   – with steel superstructure main members 55m

   These values may be doubled for a length of superstructure that contains an intermediate temperature movement deck joint.

b. The abutment piles and surrounding soil shall possess adequate flexibility to enable superstructure length changes to occur without structural distress.

c. An approach settlement slab complying with 4.12.2 shall be attached to the back face of the abutment.

Integral and semi-integral abutments are acceptable for longer bridges provided rational analysis is applied to evaluate the effect of the superstructure length change on the supporting piles. Adequate measures shall also be taken to ensure the bridge approach remains serviceable.

4.8.2 continued

In addition to withstanding the normal design loading combinations, bridges with integral or semi-integral abutments shall be designed to avoid collapse of the bridge under the maximum considered earthquake event (MCE) as defined in sections 5 and 6. This may require that the bridge abutments and superstructure be designed to withstand the maximum passive pressure capacity able to be mobilised by the soil to act on the abutments.

The Waka Kotahi NZ Transport Agency research report 577 *Criteria and guidance for the design of integral bridges in New Zealand*(55) provides guidance further to this clause on the design of integral bridges in New Zealand.

4.8.3 Application of BA 42(56)

The design of integral abutments for resistance to longitudinal thermal movements and braking loads shall comply generally with the UK Design Manual for Roads and Bridges document BA 42 *The design of integral bridges*(56) as outlined below. For seismic loading and other loadings outside the scope of BA 42(56) reference should be made to alternative literature, such as:

- Waka Kotahi NZ Transport Agency research report 577(55)
- *Recommended LRFD guidelines for the seismic design of highway bridges*(57)
- *Backbone curves for passive lateral response of walls with homogenous backfills*(58).

4.8.3 continued

The general design requirements given in sections 1 and 2 of BA 42[56] should be adopted for bridges with integral abutments. Earth pressures on integral abutment walls arising from temperature movements shall be calculated using the provisions of section 3 of BA 42[56].

The following notes provide information on the design parameters used in BA 42[56] but not adequately defined and the changes required to BA 42[56] to make it consistent with the provisions of the *Bridge manual*. The applicability of the various documents referenced in BA 42[56] and cross-references to relevant provisions of the *Bridge manual* are also noted.

a. Sections 1.1, 2.4 and 2.15

   BD 57 *Design for durability*[59] provides design requirements, and BA 57 *Design for durability*[60] design advice on design for durability and information on various methods of achieving continuity between spans to eliminate deck joints, which may lead to a more durable design. Not all of the requirements or advice however is appropriate to New Zealand conditions. Design for durability shall comply with this manual and its supporting materials design standards which set out the requirements for durability design of materials and of various structural elements.

b. Section 1.4

   BD 30 *Backfilled retaining walls and bridge abutments*[61] provides design requirements on backfilled retaining walls and abutments and may be used in conjunction with BA 42[56].

c. Section 1.5

   BD 31 *The design of buried concrete box and portal frame structures*[62] provides design requirements on buried concrete box structures and may be used in conjunction with BA 42[56].

d. Section 2.5

   The limit of ±20mm is for thermally induced cyclic movements and is not intended to include creep, shrinkage or earthquake load induced movements. This limit may be exceeded subject to a rational analysis as outlined in 4.8.2 of this manual.

e. Section 2.6

   Temperature difference, shrinkage and creep effects are covered in 3.4.4 and 3.4.6 of this manual and should be used instead of the loads given in the referenced documents (ie BD 24 *The design of concrete highway bridges and structures. Use of BS 5400: Part 4: 1990*[63] and BD 37 *Loads for highway bridges*[64]).

f. Section 2.7

   The load factors specified in 3.5 of this manual shall be used instead of the factors specified in BD 37[64].

g. Section 2.8

   The load factors for passive pressure forces shall be as specified in 3.5 of this manual.

h. Section 2.9

   The soil material strength reduction factors given in 6.5.3 of this manual shall be used instead of the material partial safety factors specified in this section.

4.8.3 continued

i. Section 2.10

The characteristic thermal strains given in this section are not consistent with the provisions of this manual. They shall be calculated using the temperature differences given in 3.4.6 of this manual. For the purpose of estimating temperature induced pressures on the abutment walls the load factor applied to thermal strains shall be taken as 1.0.

j. Section 2.11

New Zealand mean temperatures can be found from NIWA's *New Zealand median annual average temperature (°C), 1981 – 2010*[65].

k. Section 2.16

In place of reference to BD 24[63], reference shall be made to NZS 3101[1] for serviceability requirements under design live loading.

l. Section 3.3

In BS 8002:1994 *Code of practice for earth retaining structures*[66] the soil design strength is defined as; "Soil strengths which are assumed will be mobilized at the occurrence of a limit state. The design value of soil strength is the lower of either the peak soil strength reduced by a mobilization factor or the critical state strength." Clause 3.1.8 of BS 8002:1994[66] states: "Single design values of soil strength should be obtained from consideration of the representative values for peak and ultimate strength. The value so selected will satisfy simultaneously the considerations of ultimate and serviceability limit states. The design value should be the lower of:

i. That value of soil strength, on the stress-strain relation leading to peak strength, which is mobilized at soil strains acceptable for serviceability. This can be expressed as the peak strength reduced by a mobilization factor M as given in 3.2.4 or 3.2.5; or
ii. That value which would be mobilized at collapse, after significant ground movements. This can generally be taken to be the critical state strength."

m. Section 3.4

For determining wall pressures arising from thermal expansion, values of K_p should be based on design ϕ' and δ = design ϕ'/2. K_p can be taken from the chart shown in figure 4.2 which has been adapted from BS EN 1997-1 *Eurocode 7. Geotechnical design* part 1 General rules[67] by adding a curve for δ/ϕ'= 0.5.

In combining load effects, as specified in 3.5, the pressure due to thermal expansion shall be treated as a temperature load (TP).

n. Section 3.5.1

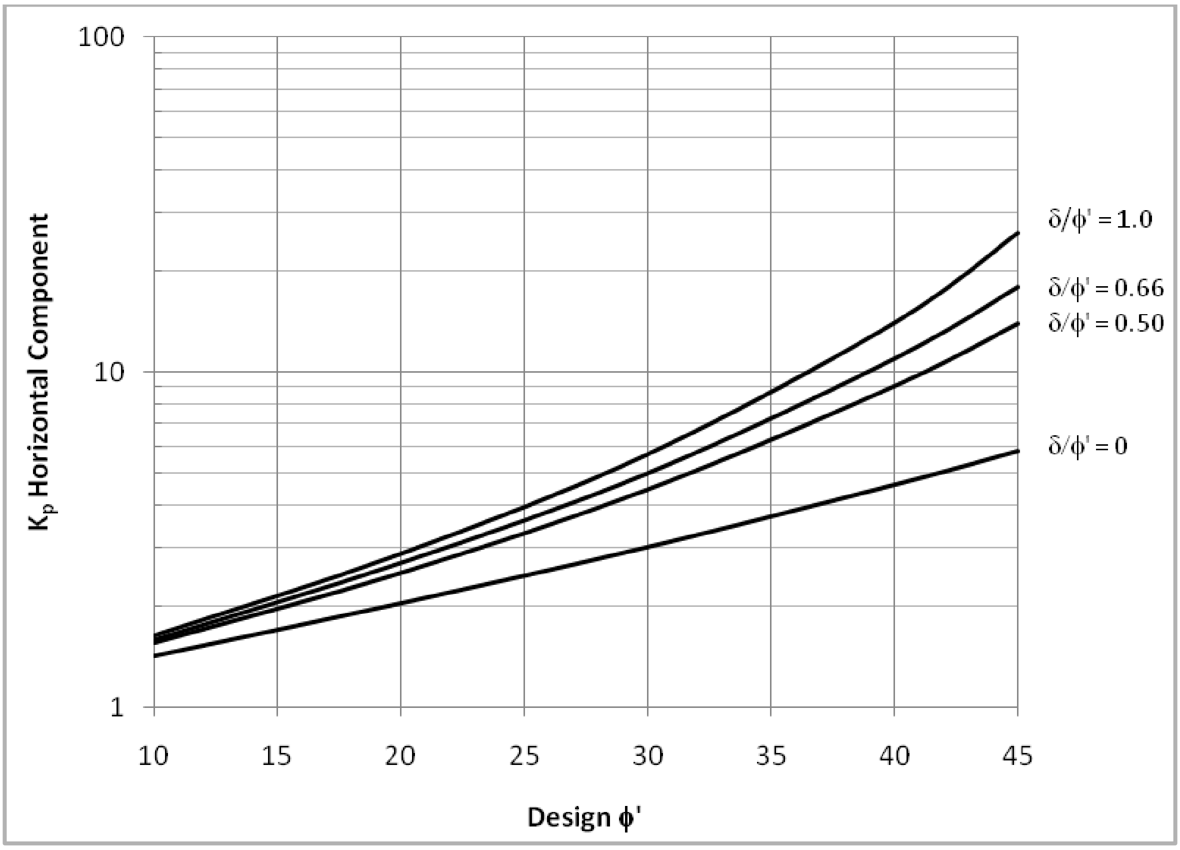
For wall heights up to 3m, K^* should be assumed to act uniformly over the height of the abutment wall.

4.8.3 continued

o. Sections 3.8, 3.9, and 3.15

Selected free draining granular backfill should be used within a distance behind the wall equivalent to twice the height of the abutment wall. The material should be carefully selected to allow displacement of the abutment wall under thermal expansion of the bridge. This may require any in situ rock or very stiff materials to be excavated and replaced with fill materials to accommodate such movements. The material should be compacted using hand compaction methods to avoid damage to the structure, minimise compaction pressures and displacement during the compaction process. Large compaction plant shall not be used. The compaction shall achieve a minimum of 95% of the maximum dry density, and comply with the requirements of specification TNZ F/01 *Earthworks construction*(68). In addition, a drainage layer and sub-soil drain (specifications NZTA F2 *Pipe subsoil drain construction*(69) or TNZ F/06 *Fabric wrapped aggregate subsoil drain construction*(70)) should be incorporated behind the wall to avoid groundwater pressures on the wall.

Figure 4.2: Coefficient of passive earth pressure (horizontal component) for vertical wall and horizontal backfill



4.9 Network and tied arch bridges – hanger supports

Hangers supporting the bridge deck from the main arch ribs shall be protected from vehicle collision by a barrier system of minimum performance level 5.

The design of each hanger and associated structure shall consider the following three scenarios and associated design requirements:

a. Scenario 1: Failure of a single hanger due to fatigue

   Design load case: $1.20DL + 1.20SDL + 1.20(LL{\times}I)$ + hanger loss dynamic forces

   Where:

   DL = dead load of structural components and non-structural attachments

   SDL = superimposed dead loads as specified in 3.4.2

   LL = HN (normal) traffic live load placed in their actual marked lanes, using lane factors in accordance with 3.2.4(a)

   I = dynamic load factor

   The hanger loss dynamic forces shall be taken as twice the static force in the hanger unless demonstrated by a suitable time history analysis that a lesser hanger loss dynamic force is appropriate. The hanger loss dynamic force shall not be taken to be less than 1.5 times the static force in the hanger.

b. Scenario 2: Continued bridge operation while a hanger is repaired

   Design load case: $1.35DL + 1.35SDL + 1.5(LL{\times}I) + FP$ + hanger exchange forces

   Where:

   DL = dead load of structural components and non-structural attachments

   SDL = superimposed dead loads as specified in 3.4.2

   LL = HN (normal) traffic live load placed in their actual marked lanes, using lane factors in accordance with 3.2.4(a)

   I = dynamic load factor

   FP = pedestrian and cycle track loading

   Hanger exchange forces include the redistribution of loads through the structure as a result of the missing hanger and/or replacement of the missing hanger when it is repaired.

   The above load combination is to be considered with the traffic live load placed in their marked lanes with each hanger in turn, one at a time, missing to evaluate the bridge under an 'unnoticed lost hanger' scenario.

c. Scenario 3: Hanger failure due to collision

   Within a single plane of hangers, all hangers within any 4.0m length taken at the level of the bottom chord, but not less than two hangers, adjacent to or crossing each other, shall be considered to break, one after the other, sequentially.

   Design Load Case: $1.20DL + 1.20SDL + 1.0(LL{\times}I)$

4.9 continued

Where:

DL = dead load of structural components and non-structural attachments

SDL = superimposed dead loads as specified in 3.4.2

LL = traffic live load taken as one 36 tonne vehicle positioned as described below

I = dynamic load factor

The traffic live load shall be distributed longitudinally along the bridge over a 12m length positioned symmetrically about the group of breaking hangers. (Eg in the case of there being only two hangers crossing each other that break, the live load shall be distributed symmetrically either side of the point where the hangers cross.) The vehicle shall be taken to be 2.5m wide positioned on the carriageway against the traffic barrier adjacent to the breaking hangers.

Loads and conditions shall be applied to the arch span as follows:

- One hanger shall be removed from the structural model (considered to have failed ahead of the second hanger).
- At the second hanger to fail position, the hanger shall be removed from the structural model and inward forces applied to the arch rib and bottom tension chord in the line of the hanger equal to $1.2F_{pu} - F$.

  Where:

  F_{pu} = characteristic failure load of the hanger

  F = load in the hanger before failure under $1.20DL + 1.20SDL + 1.0(LL \times I)$

  Inward forces means: When applied to the arch rib, the force is acting in the direction towards the bottom tension chord, and when applied to the bottom tension chord, the force is acting in the direction towards the arch rib.

- The arch shall remain stable. Yielding is to be avoided in the arch rib and in all elements that could lead to instability or that are not easily repaired.
- The effect of the hangers potentially being deflected out of the plane of the arch at the time when they break, imposing lateral loading on the arch, shall be considered. The level at which the cable is struck shall be taken as anywhere between the top of the barrier and 4.3m above deck level.
- Also the effects of the hanger being deflected sideways in a collision on end connections of the hanger to the arch rib and bottom tension chord are to be considered. Hanger connections are to be detailed to allow for their easy repair and hanger replacement throughout the design life of the bridge.

4.10 Buried structures

4.10.1 General

The design and construction of corrugated metal structures shall comply with AS/NZS 2041 *Buried corrugated metal structures*(71) (the most relevant and up-to-date version or part thereof) except as modified or superseded herein.

The design of concrete box culvert structures shall comply with NZS 3101(1).

The design of precast concrete pipes shall comply with AS/NZS 3725 *Design for installation of buried concrete pipes*(72) and AS/NZS 4058 *Precast concrete pipes (pressure and non-pressure)*(73) except as modified or superseded herein.

The design requirements set out below shall supersede those included in the AS or NZS standards for defining loads and the load application to the buried structure. The requirements of the respective AS and NZS standards shall be used for determining the internal forces and actions for the buried structure and for determining the appropriate acceptance criteria, except as otherwise amended.

The design and detailing of buried structures shall be such that the design working life is achieved without reconstruction or major rehabilitation within that period (except as outlined below). Sufficient investigation shall be undertaken to ensure the aggressiveness of the site (corrosion, abrasion and chemical attack) is appropriately evaluated and the structure designed for durability accordingly.

For sites where the buried structure is under large fill heights or in a location where future replacement or rehabilitation may be very expensive, longer service life options or options including specific provision for future rehabilitation (eg installing an oversize pipe to allow for future sleeving) shall be considered.

Design of corrugated steel structures shall be on the basis of one of the following approaches:

a. For a design working life of 100 years with, in addition to the initial galvanizing, sacrificial wall thickness provided to compensate for the loss of section due to corrosion, or a supplementary corrosion protection system provided, capable of enabling the structure to achieve the specified durability.

b. For a design working life of 50 years, but oversized sufficiently to enable sleeving at the end of its life with a smaller sized barrel satisfying the waterway requirements of this manual. The adoption of this option shall be based on a comparison of the 'whole of life' costs over a 100-year period of this option, including the cost of sleeving, with the cost of option (a). Use of this option shall be subject to the approval of the road controlling authority.

Where a steel invert is provided and significant abrasion over the life of the structure is anticipated, a concrete invert shall be installed at initial construction, unless agreed otherwise with the road controlling authority. The concrete invert shall be suitably detailed, reinforced and tied to the culvert floor and the base of the walls up to 300mm above the normal water flow surface. The upstands against the culvert walls shall have their top surfaces sloped inwards so that water does not pond against the galvanizing. Placement of the concrete lining should ideally be delayed for six months or more following construction of the culvert to allow initial flexing and settlement of the structure to cease.

4.10.1 continued

Clause 2.8.2.1 of AS/NZS 2041.2[71] shall be modified by adding the following to the end of the first paragraph:

- Each uncoated area for renovation shall not exceed 1000mm². If uncoated areas are larger, the article containing such areas shall be regalvanized unless agreed otherwise between the road controlling authority and the galvanizer.

Pipe and corrugated metal culverts shall be provided with not less than 600mm of cover and where possible all other culverts, pedestrian/cycle subways and stock underpasses shall be provided with the same cover. Where the cover is to be less than 600mm, an alternative pavement design should be provided and measures taken to reduce reflected pavement cracking and differential settlement at the road surface to acceptable levels, including the provision of settlement slabs (see 4.12.2) unless they are agreed with the road controlling authority to not be necessary. Additional cover shall be provided where necessary to accommodate existing utility services, future services where there is a policy or agreement to do so and existing or planned longitudinal stormwater drainage.

In determining the size and shape of the buried structures appropriate consideration shall be given to fish passage, climate change, and inspection and maintenance requirements. Design shall consider the effects of vibration, settlement, batter stability, piping/erosion and possible earthquake induced ground deformation or liquefaction on the structure.

The Austroads *Guidelines for design, construction, monitoring and rehabilitation of buried corrugated metal structures*[74] provides useful guidance.

4.10.2 Rigid buried structures

Rigid buried structures include concrete box, concrete arch and precast concrete pipe. Precast concrete pipes used on state highways shall be steel reinforced.

Design live loadings and their application shall be as follows:

- The full range of load case and combinations as specified in section 3 shall be evaluated and met at both the serviceability and ultimate limit states.
- The HN and HO load footprints applied to the pavement shall be as specified in section 3. The load spread through the fill above the buried structure shall use the AS 5100.2[30] section 7.12 'double trapezoidal prism' consisting of 0.5:1 load spread in the top 0.2m and 1.2:1 load spread through the remaining cover depth when the cover depth equals or exceeds 0.4m. The 3.5kPa traffic load UDL shall be applied with no load spread. The dynamic load factor shall be applied as set out in 3.2.5.
- When the cover depth is less than 0.4m the HN and HO footprints shall be applied directly to the top surface of the buried structure, in conjunction with the 3.5kPa UDL, to generate the worst internal action effect. The dynamic load factor appropriate to the least design cover depth shall be applied as set out in 3.2.5.
- Rigid buried structure design shall include the 1.35HN serviceability limit state combination.
- For precast concrete pipes, for the situation under consideration, the class of pipe required shall be derived from AS/NZS 4058[73] based on the required proof load capacity determined in accordance with AS/NZS 3725[72].

4.10.3 Semi-rigid and flexible buried structures

Semi-rigid and flexible buried structures include corrugated metal structures. Design live loadings and their application shall be as follows:

- The full range of load case and combinations as specified in section 3 shall be evaluated and met.

4.10.3 continued

- Unless a soil - structure interaction analysis (which takes structure stiffness, foundation stiffness and the type, compaction and drainage of the backfill into account) is undertaken then the HN-HO-72 live load pressure to be applied to the crown of the buried structure shall be determined as follows:

  $$P_v = 1.35(32H^{-1.852} + 3.5)$$

  Where:

  P_v = vertical pressure in kPa on the plan projected area of the structure due to HN-HO-72 live loads including dynamic load effects and the serviceability load factor of 1.35 on live load.

  H = minimum depth of cover in m measured from the trafficked surface level to the crown of the pipe

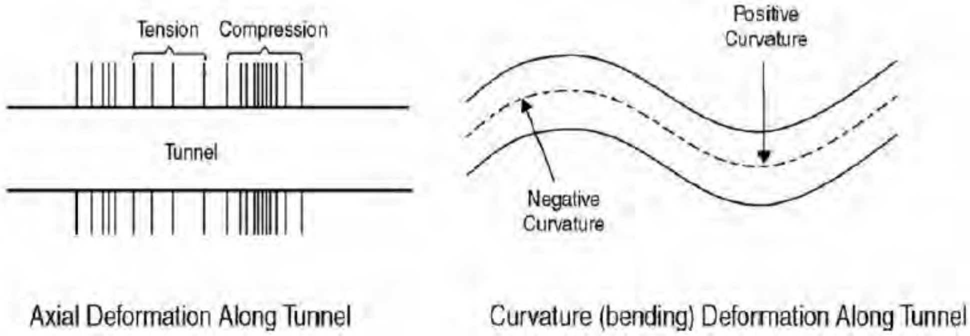
  This equation is appropriate for cover depths greater than or equal to 0.6m and includes HN, HO and dynamic load factor effects. For the purpose of serviceability limit state design it is appropriate to assume that the 1.35HN traffic pressure governs for cover depths less than 0.9m using load combination 1A from table 3.2 or 3.3 and HO traffic pressure governs for cover depths equal to or greater than 0.9m using load combination(s) 4A from table 3.2 or 3.3. For the ultimate limit state load combination 1A (table 3.3) the traffic pressure determined from the above formula shall be divided by 1.35 to determine the basic HN traffic load pressure before applying the specified ultimate limit state load factor of 2.25 to the HN load.

- Unless the depth of cover to the structure equals or exceeds the diameter or span of the structure no load reduction shall be made for soil arching effects.

4.10.4 Earthquake loading on buried structures

The earthquake response of underground structures shall be considered with reference to the three principal types of deformations: axial, curvature and racking (rectangular cross-sections), or ovaling (circular cross-sections). Axial and curvature deformations develop when seismic waves propagate either parallel or obliquely to the longitudinal axis of the structure (figure 4.3). The general behaviour of a long structure subjected to a component of parallel wave deformation is similar to that of an elastic beam embedded in the soil. In simplified analyses, the structure is assumed to be flexible relative to the surrounding soil or rock, and to respond with the same deformation pattern as in the free-field elastic seismic waves. These simplified analyses are often employed for pipelines that have relatively small cross-sectional areas and for the preliminary analyses of tunnels and culverts. When the structure is stiff in the longitudinal direction relative to the surrounding soil, it will not be compliant with the soil or rock deformations. For this case, interaction effects need to be considered by employing either numerical methods or approximate solutions developed from wave propagation theory for beams on an elastic foundation (see *Seismic design of tunnels*(75)).

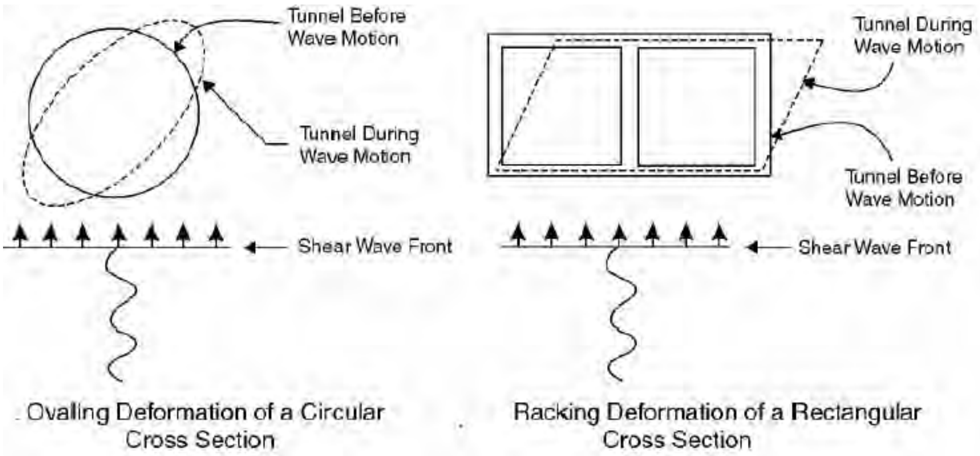
Figure 4.3: Axial and curvature deformations (from FHWA-NHI-11-032(76))



4.10.4 continued

Ovaling or racking deformations develop in an underground structure when the seismic waves propagate in a direction perpendicular to, or with a significant component perpendicular to, the longitudinal axis, resulting in distortion of the cross-section (see figure 4.4). For this case, and if the structure is relatively long, a plane-strain two-dimensional analysis is usually employed.

Figure 4.4: Axial and curvature deformations (from FHWA-NHI-11-032[76])



The racking performance of the cross-section of an underground structure subjected to earthquake ground motions can be undertaken using the following steps:

a. evaluation of the free-field peak shear strain in the soil at the average depth of the structure

b. evaluation of the elastic and post-elastic stiffness of the structure

c. evaluation of the racking deformation of the structure from the free-field strain, structure stiffness and soil-structure interaction curves

d. evaluation of the member forces in the structure from the racking deformation.

The shear distortion of the ground from vertically propagating shear waves is usually considered to be the most critical and predominant effect producing racking type deformations in underground structures. Numerical methods are often applied to estimate the free-field shear distortions, particularly in sites with variable stratigraphy. Computer codes such as SHAKE based on one-dimensional wave propagation theory for equivalent linear systems can be used to carry out these types of analyses.

An alternative to undertaking numerical analyses to predict free-field shear deformations is to use theory of elasticity analytical solutions for vertically propagating shear waves in a layer of uniform thickness. These solutions provide sufficiently good approximations for the design of smaller underground structures, particularly where the site soil properties are not known in any detail. They are useful for preliminary analysis work and provide a verification method for more sophisticated numerical analysis. Analytical solutions are presented in *Earthquake design of rectangular underground structures*[77].

Because of both soil-structure interaction and dynamic inertial effects, the soil shear strains near the structure are generally significantly different to the free-field shear strain at the corresponding depth in the soil layer. Often the mass change at the cavity created by the structure is small in relation to the total mass in the layer acting in unison with the structure and dynamic inertial effects are sufficiently small to be neglected.

4.10.4 continued

In contrast to the influence of soil inertial effects, soil-structure interaction effects may produce significant changes in the shear strains near the structure. If a cavity in the soil is unlined or lined with a very flexible structure, the shear strains may be greater than in the free-field. If a stiff structure is inserted in the soil cavity, then the shear strains may be less than the free-field.

In assessing soil-structure interaction effects on underground structures it is usual to define shear strain deformation and flexibility ratios. The shear strain deformation ratio (R) is defined by:

$$R = \frac{Shear\ deformation\ of\ structure\ embedded\ in\ soil\ (including\ Interaction)}{Free - field\ shear\ deformation\ over\ height\ of\ structure}$$

The flexibility ratio (F_r) is defined by:

$$F_r = \frac{Shear\ flexibility\ of\ free\ standing\ structure\ without\ soil\ interaction}{Shear\ flexibility\ of\ soil\ block\ of\ same\ overall\ dimensions\ of\ the\ structure}$$

The flexibility ratio (F_r) can be readily computed from the soil shear modulus (G) and the structure flexibility. Methods for calculating the shear deformation ratio (R) from the flexibility ratio have been presented for both circular and rectangular sections in *Seismic design of tunnels*(75), *Seismically induced racking of tunnel linings*(78), *A simplified evaluation method for the seismic performance of underground common utility boxes*(79), *Earthquake design procedures for rectangular underground structures*(80) and *Earthquake design of rectangular underground structures*(77). The structure shear deformation including soil-structure interaction can be calculated from the shear deformation ratio (R) and estimates of the peak free-field shear deformation during the design earthquake event. The structure shear deformation is then used to calculate the earthquake-induced stresses in the members of the structure using analytical or numerical methods (Wang, 1993 and Wood, 2005).

Alternatively, finite element analyses based on the estimated free-field shear deformation can be used to calculate the earthquake induced stresses in the structure. If these more complex analyses are undertaken the four-step procedure using the F_r ratio can be used to check the results.

4.10.5 Backfilling around buried structures

Guidance on the selection of fill materials, method of placement and compaction of backfill for culverts can be found in AS/NZS 2041.2(71), sections 2.4 and 2.12. The following provides further clarifications to those requirements.

The tests below have been prescribed in NZS 4402 *Methods of testing soils for civil engineering purposes – Soil tests*(81) for the purposes of evaluating soil compaction:

- NZS 4402.4.1.1 *New Zealand standard compaction test*(82)
- NZS 4402.4.1.2 *New Zealand heavy compaction test*(83)
- NZS 4402.4.1.3 *New Zealand vibrating hammer compaction test*(84)
- Section 4.2 *Determination of the minimum and maximum dry densities and relative density of a cohesionless soil*, which includes the following laboratory tests:
  - NZS 4402.4.2.1 *Minimum dry density*(85)
  - NZS 4402.4.2.2 *Maximum dry density*(86)

4.10.5 continued

Clause 2.12.5 of AS/NZS 2041.2[71] allows compaction level acceptance criteria to be based on test results from any of the above tests. The choice on which of these tests should be used shall be based on how closely the test procedures prescribed in the standards simulate the workings of the compaction equipment used in the field. If the use of heavy and vibratory compaction equipment is not required, the maximum dry density (MDD) from the NZ standard compaction test (NZS 4402.4.1.1:1986) will be appropriate and the dry density in each layer of fill (in accordance with clause 2.12.4.2 of AS/NZS 2041.2[71]) shall be compacted to at least 95% of the MDD.

Relative density from NZS 4402.4.2.1:1988 and NZS 4402.4.2.2:1988 should only be used as a compaction acceptance criteria if the select fill contains less than 12% by mass of non-plastic fines passing a 0.075mm sieve and there is a stringent need to minimise compression in the backfill, in which case an indication of the highest possible value of dry density for the backfill would be required. For these cases each layer of select fill shall be compacted to not less than 70% of the relative density. As noted in the standards, laboratory results from NZS 4402.4.2.2:1988 are highly sensitive to the capability of the vibratory table used in the test, and strict adherence to the mechanical specifications given in NZS 4402.4.2.2:1986 for the vibratory table is necessary for the compaction test results to be repeatable.

4.11 Bridges subject to inundation by flooding

Where it is proposed to place a bridge over a waterway, care should be taken to so locate it that immersion will not be likely to occur. In cases where immersion is unavoidable or cannot readily be designed for, then it may be appropriate to consider other forms of construction in preference to prestressed concrete, which is particularly vulnerable to the effects of immersion.

When a bridge is covered by floodwaters the upthrust on the structure exerted by the water cancels out some of the dead load acting downwards. In prestressed concrete bridges the upthrust of the water combines with the upthrust due to draped or eccentric prestressing tendons and this may lead to unfavourable stress distribution in the beams; especially so if air is entrapped between the girders, so increasing the volume of water displaced.

In all cases where waterway crossings are to be constructed, careful consideration shall be given to stresses induced under submerged conditions. Ducts should be formed through the girder webs as close to the underside of the deck slab as possible, preferably by means of a short length of pipe which can be left in place so as to offset the loss of section. These ducts should be placed in positions that will be most effective to releasing entrapped air, giving considerations to the grades, vertical curves and crossfalls to which the bridge may be constructed.

In the case of composite construction, where a cast in-situ deck is poured onto prestressed concrete or steel beams, sufficient steel must be incorporated across the interface between the beams and the deck slab to resist the tendency of the deck to separate from the beams under the uplift forces acting under submergence; and to place the air escape ducts as high up as possible to reduce this tendency.

4.12 Miscellaneous design requirements

4.12.1 Proprietary items

Wherever proprietary items are required as part of the structure, allowance shall be made as far as possible for any brand to be used. Brand names shall not be quoted in the documents unless it is essential to the design that a particular brand is used.

4.12.2 Settlement slabs

A settlement slab shall be provided at every bridge abutment unless they are agreed with the road controlling authority to not be necessary. Examples where a settlement slab may not be necessary are abutments with no earth filling or low volume or low speed environments. Settlement slabs shall be provided for buried structures (ie culverts, stock underpasses and pedestrian/cycle subways) as detailed in 4.10.1. Settlement slabs shall extend over the full width of expressway carriageways (ie the width of traffic lanes plus shoulders) but may be omitted over the width of medians, unless the possibility of future reconfiguration of the carriageway is identified as a design requirement.

The slab shall be simply supported along one edge by the abutment, and shall be designed for dead and live load, assuming that it spans at least three-quarters of its actual length, in the longitudinal direction of the bridge or road. Slabs shall be at least 2m in length and sloped to divert surface water from flowing down the abutment/soil interface. Sub-surface drainage of this water shall be considered. For bridges longer than 30m the slab shall be at least 3m in length. The slab shall be a minimum of 600mm below the road surface. The effects set out in 2.5 shall be considered. In addition to being supported at one edge by the structure, the settlement slabs shall also be tied longitudinally to the structure to prevent them being pulled off their seating through the actions of settlement beneath them or by earthquake actions. The design yield strength of the connection of the slab to the structure, factored by the strength reduction factors given in NZS 3101[1], shall be at least 1.3 times the nominal sliding resistance of the settlement slab.

Settlements slabs, in addition to catering for settlements behind abutments arising from normal service conditions, shall accommodate settlements that may arise from earthquake actions and flood events, the magnitude of which may be large and unexpected. Settlement slabs shall be detailed to be able to accommodate at least 500mm of settlement behind the abutments over their length arising from an extreme scour or CALS event. The bearing strip supporting the settlement slab does not need to be designed for this settlement, but loss of seating must be avoided.

4.12.3 Deck drainage

In general, stormwater shall be collected and specific provision made for its disposal. On bridges that are waterway crossings stormwater may be discharged over the edge of the deck unless prohibited by the resource consent.

Deck drainage shall be designed to the standards adopted for the highway drainage system. In particular, the outlet pipes and pipe system shall be designed for a rainfall event with a return period of not less than 20 years including the effects of predicted climate change. Guidance on the design for surface drainage may be obtained from *Highway surface drainage: Design guide for highways with a positive collection system*[87], except that more up-to-date sources of information to that referenced should be drawn on for the estimation of design storm rainfall. These sources include:

- local rainfall databases, as may be held by the regional council responsible for the locality under consideration
- *High intensity rainfall design system (HIRDS)* version 3[88], a web based program for estimating rainfall frequency

4.12.3 continued

- *Climate change effects and impacts assessment: A guidance manual for local government in New Zealand – 2nd edition*(89)
- *The frequency of high intensity rainfalls in New Zealand, part 1*(90).

The deck drainage of bridges, and of other structures if directly carrying traffic, shall be designed to ensure that any ponding in any part of any traffic lane is limited to a maximum depth of 4mm of sheet flow above any surface texture during a two year return period rainfall.

To avoid pipes silting up from deposits left by slow flowing water pipes shall be laid on as steep a grade as practicable and not less than the figures below for the listed diameters:

Diameter (mm)	150	225	300	375
Grade (%)	1.0	0.5	0.35	0.25

The visual impact of drainage pipework shall be minimised. Ducting or pipework shall not run along external faces of structures or vertically down pier or abutment faces that are visible to the public. Longitudinal deck drainage collection pipes on bridges shall not be located on the outside of or below the outside beams. Feeder pipes from catch pits may be visible, but shall be concealed as far as possible. Deck drainage shall not be carried in steel pipework. Pipework carried in hollow members shall also comply with the relevant requirements of 4.12.5.

Pipework shall be supported by a support system. The support system and its spacing shall ensure that no appreciable sag occurs to the pipe under the design load for the pipe full case.

Pipework shall incorporate movement joints or other mechanisms to allow for the serviceability limit state design movements of the bridge calculated for the serviceability limit state load combinations given in table 3.2. The relative movement between parts of the structure under one quarter of the earthquake relative movement plus the relative movement due to long term shortening plus one half of the temperature induced relative movement from the median temperature position shall also be provided for.

Drainage pipe material shall comply with NZTA F3 *Specification for pipe culvert construction*(91).

All components of the drainage system shall have a life to first maintenance of not less than 30 years. The drainage system shall be replaceable without modification or removal of any structural concrete or steelwork. This does not however preclude casting pipework into concrete piers.

All components of the bridge deck drainage systems shall be designed to be self-cleansing, and shall be detailed to allow adequate access for future inspection, maintenance and cleaning. A minimum collector pipe internal diameter of 150mm shall be provided. All drains shall be capable of being cleared of blockages under routine maintenance activities without the need for closure of carriageways beneath the structure. Manholes, if required, shall not be located within the road carriageways.

The deck drainage system shall be detailed to ensure water does not leak onto visible surfaces, causing staining or corrosion, or onto bearings or energy dissipating devices. Positive fall drainage shall be provided on all bearing shelves, under expansion joints and behind all earth retaining abutments and walls. Drip grooves shall be provided at the edge of all slab soffits. Deck movement joints shall be made watertight.

Sumps in the bridge deck shall be positioned and detailed in a manner that will ensure traffic ride is not affected and that will provide for future resurfacing of the bridge deck.

4.12.4 Drainage of hollow structural elements

If hollow structural components are adopted, then positive fall drain holes of 40mm minimum diameter shall be provided at all low points within the voids regardless of their susceptibility to ponding. In the case of bridge superstructure slab or beam elements, drain holes shall be provided in each void and shall discharge directly to the outside through the soffit of the element. All such drain holes shall be accessible for maintenance.

4.12.5 Services

Agreement shall be reached with network utility operators of services, over support conditions required for services and minimum spacing requirements between different services. Network utility operators shall be made aware of the extent and direction of movement at expansion joints, due both to length changes and seismic acceleration. Where a lesser standard of design for earthquake resistance has been accepted by the road controlling authority than that specified by this manual (eg due to the high cost of mitigating liquefaction effects), network utility operators whose services are proposed to be carried on the structure shall be advised of the design standard accepted to be adopted for the structure and shall be responsible for ensuring that appropriate measures are adopted to achieve continuity of operation of their services following a major earthquake event.

The implications of possible bridge overloading due to leakage or rupture of pipes carrying water or other fluids inside a box girder or other hollow member shall be considered, and adequate drainage shall be provided.

Special approvals and conditions apply to the installation of pipelines carrying flammable fluids (including gas). Such pipelines shall not be carried inside box girders.

Regulations requiring minimum spacing between different types of services (eg as may apply between water supply and sanitary sewer; and between gas and electricity) shall be complied with. The final location of ducts shall be discussed and agreed with utility operators.

Services carried on a bridge deck or trafficked deck of a buried structure shall be adequately protected against possible loading by vehicle wheels, horses and stock.

Unless otherwise directed by the road controlling authority, on new bridges with either hollow core unit superstructures or raised footpaths, in addition to the known services and where practical to do so, a nominal provision shall be made for future services to be carried by the installation or casting into these elements of 2 × 150mm diameter uPVC ducts with durable draw wires installed. These ducts shall be located within the width of the footpath or the outermost available voids for hollow core decks unless alternative locations are directed or accepted All unused ducts shall be provided with water-tight terminations located outside of the sealed carriageway.

4.12.6 Date and loading panels

All structures subjected to direct traffic loading shall have displayed details of the date of construction and design live loading.

Each structure designed to HN-HO-72 loading shall have this information displayed on two panels, as shown in figure 4.5. The panels shall be of bronze or other approved material of equivalent durability.

The panels shall be located one at each end of the structure on the left hand side of approaching traffic and in a conspicuous location, eg on the top surface of footpaths or safety kerbs, on the carriageway face of concrete barriers, or on the deck behind the line of the guardrail clear of any subsequent sealing work.

Structures designed to other loadings shall have similar panels.

4.12.7 Load limiting devices and shock load force transfer devices

a. Abutment knock-off elements and deck slab knock-up elements

Abutment 'knock-off' elements and deck slab 'knock-up' elements, at deck joints, designed to be displaced under response of the bridge to strong earthquakes, thereby allowing freedom of movement of the bridge superstructure without significant interaction with adjacent structure, shall be:

- stable under traffic loads at the ultimate limit state
- able to resist the forces imposed on the knock-off or knock-up element by an attached deck joint at the ultimate limit state displacements under service conditions that exclude earthquake effects
- able to be dislodged without significant damage to adjacent structural elements. Abutment knock-off elements are not to be dowelled to the abutment back wall.

b. Earthquake energy dissipating devices

Devices for dissipating earthquake energy, that also act to limit the earthquake forces mobilised within the structure, shall comply with 5.6.14 and shall also ensure that 5.1.2 (c) will be satisfied.

c. Shock load force transfer devices

Devices designed to accommodate slow rates of movement between adjacent structural elements interconnected by the device without significant transfer of force due to the movement, but designed to lock-up and provide force transfer under shock loading from an earthquake, shall be designed with sufficient ideal strength to resist the forces imposed on them. The forces imposed on the devices shall be assessed from a rational analysis of the structure assuming overstrength to have developed in plastically yielding elements of the structure.

4.12.8 Confinement of embedded fixings

Embedded fixings forming part of the primary load path for transferring forces into the structure (eg side protection barrier fixings) or transferring forces from the superstructure to the substructure (eg bearing and base isolation system fixings, holding down bolts) subjected to lateral loading, or that may become subjected to lateral loading through such events as the seizure of bearings or damage to shear keys, shall be adequately confined to prevent splitting of the surrounding concrete.

4.12.9 Anti-graffiti finish

The selection and application of an anti-graffiti finish shall be undertaken in accordance with NZTA S10 *Specification for anti-graffiti coatings*(92).

Environmentally and structurally friendly anti-graffiti coatings with a design life of at least 5 years for sacrificial coatings and 10 years for permanent coatings shall be applied to all new structures if required by the road controlling authority. The coatings will require approval by the road controlling authority prior to use on the works. The extent of application for each element of the structures shall be:

- 1.2m from an accessible top edge
- 2.7m above adjacent ground level or base level, and
- 1.5m horizontally from an accessible substructure element
- both faces of a rigid traffic barrier.

The extent of the application shall be increased where required for urban design.

4.12.10 Accommodation of signage and lighting columns

Any lighting columns and the posts of signs provided on structures shall be located on the outside (non-traffic side) of the side protection barriers.

On bridges with rigid barriers the horizontal clearance to posts and columns from the base of the traffic face of the side protection barrier shall be not less than 1.1m regardless of whether the barrier/deck connections are frangible or not. Clearances to posts and columns on bridges with semi-rigid barriers should be considered on a case-by-case basis and detailed in the structure design statement.

Signs and lighting overhanging the carriageway shall maintain the vertical clearances from the carriageway specified in figure A4.

4.12.11 Deck surfacing

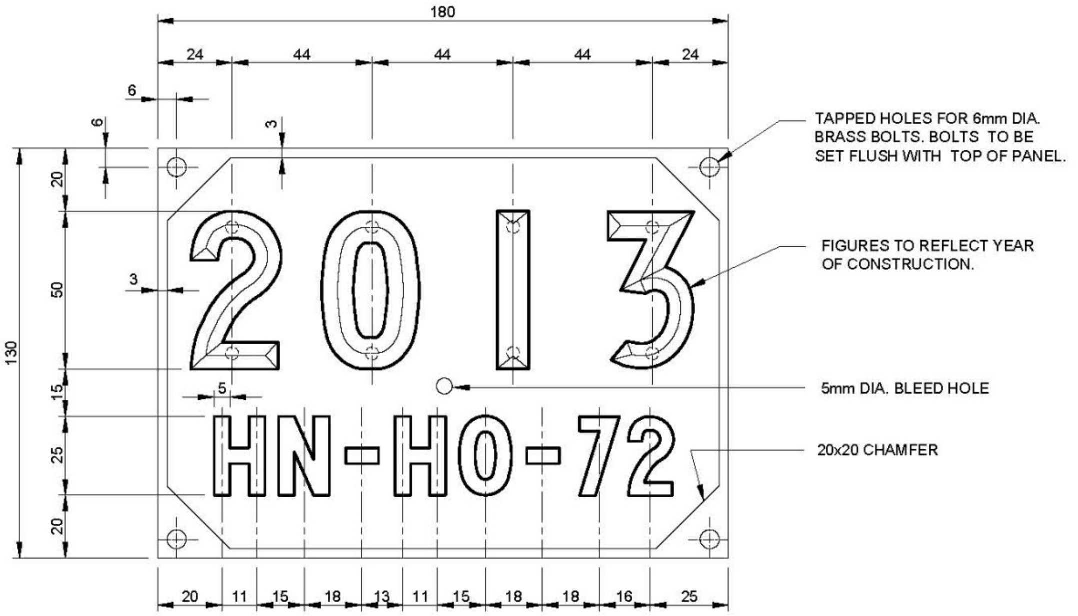
Bridge decks comprised of precast components will generally have an irregular concrete surface due to the variations in thickness and hog between adjacent units. An asphaltic levelling course shall be applied to provide smooth surfaces conforming to the design longitudinal profile and transverse crossfalls of the bridge with allowance made for the thickness of the final surfacing to be applied subsequently. The thickness of the levelling course should approach the practical minimum required to enable its laying. Also a tack coat, generally an emulsion, is required to ensure adhesion between the bridge deck and the asphaltic levelling course.

Bridge decks cast in situ should generally be constructed to the design longitudinal profile and transverse crossfalls of the bridge with allowance made for the thickness of the final surfacing and should not require application of an asphaltic levelling course. Where supported on precast concrete or steel beams, paps constructed between the top surface of the beams and soffit of the deck should be used to adjust the level of the deck to the desired profile where necessary.

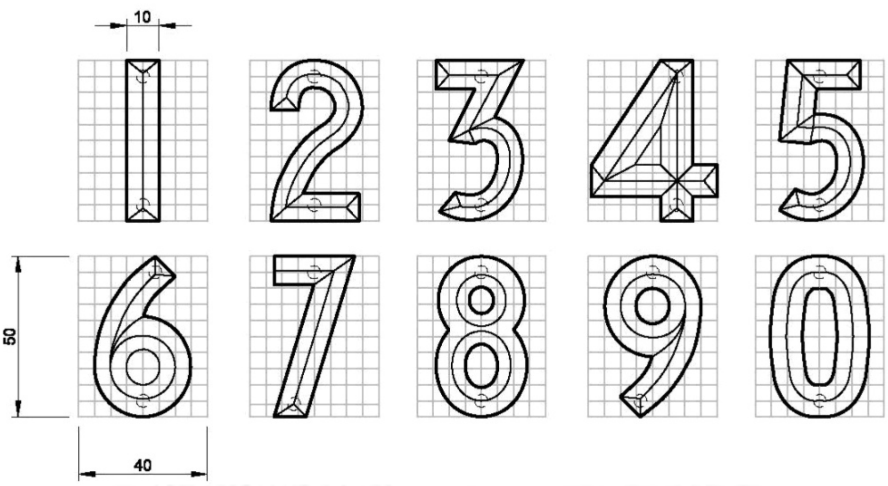
The final deck surfacing shall match the surfacing applied on the bridge approaches to maintain consistency in the visual appearance and driving and braking characteristics of the surfacing.

Figure 4.5: Date and loading panel HN-HO-72 loading

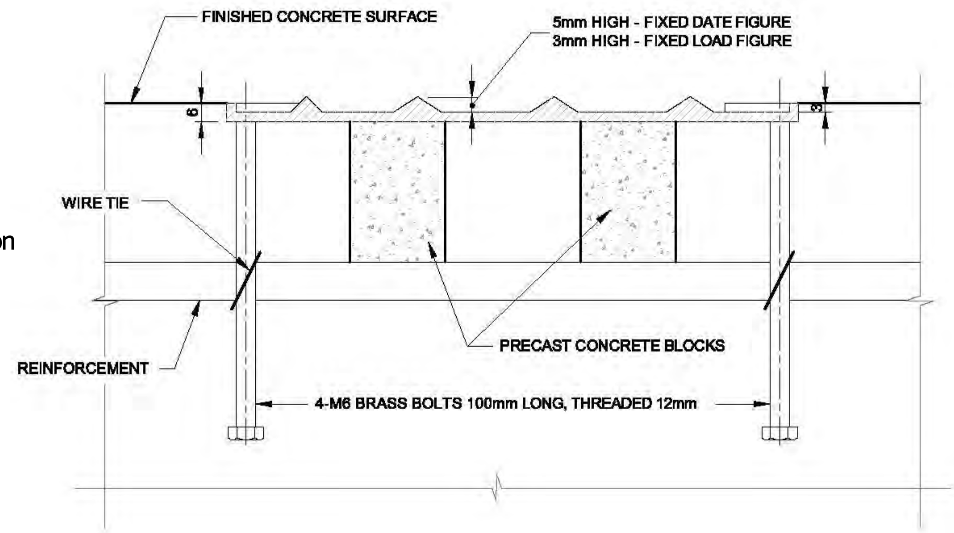
Plan



Date figure proportions



Typical panel section and installation



Notes

1. The panel is to be made of bronze (or other approved material of similar durability), supplied with bolts.
2. The panel shall be located on the left side of approaching traffic, on top of abutment wing walls, kerb or deck surface.

4.13 References

(1) Standards New Zealand NZS 3101.1&2:2006 *Concrete structures standard*. (Incorporating Amendment No. 3: 2017)

(2) Standards New Zealand NZS 3122:2009 *Specification for Portland and blended cements (General and special purpose)*.

(3) Standards Australia and Standards New Zealand jointly AS/NZS 3582. ________*Supplementary cementitious materials*
Part 1:2016 Fly ash
Part 2:2016 Slag – Ground granulated iron blast furnace (AS 3582.2)
Part 3:2016 Amorphous silica.

(4) Standards Australia and Standards New Zealand jointly AS/NZS 1170.0:2002 *Structural design actions.* Part 0 General principles. (Incorporating Amendment No. 5: 2011)

(5) Standards New Zealand NZS 3109:1997 *Concrete construction*.

(6) Roads and Maritime Services (2021) *Concrete work for bridges*. QA specification B80, Sydney, NSW, Australia.

(7) Standards Australia AS 1012.13:2015 *Methods of testing concrete*. Part 13 Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory.

(8) National Institute of Water and Atmospheric Research *CliFlo – The national climate database*. Last accessed 28 March 2022. <http://cliflo.niwa.co.nz/>.

(9) Standards Australia AS 5100.5:2017 *Bridge design.* Part 5 Concrete.

(10) International Organization for Standardization ISO 15835:2009 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars.*
Part 1 Requirements. Superseded.
Part 2 Test methods. Superseded.

(11) International Organization for Standardization ISO 15835:2018 *Steels for the reinforcement of concrete - Reinforcement couplers for mechanical splices of bars.*
Part 1 Requirements
Part 2 Test methods.
Part 3 Conformity assessment scheme.

(12) International Organization for Standardization ISO 15698:2012(R2018) *Steel for the reinforcement of concrete - Headed bars*.
Part 1 Requirements.
Part 2 Test methods.

(13) Standards Australia and Standards New Zealand jointly AS/NZS 4671:2019 *Steel for the reinforcement of concrete*.

(14) Standards Australia AS 1544.2-2003 (R2017) *Methods for impact tests on metals.* Part 2 Charpy V-notch.

(15) Standards Australia and Standards New Zealand jointly AS/NZS 3678:2016 *Structural steel – Hot-rolled plates, floorplates and slabs*. (Incorporating Amendment No. 1: 2017)

(16) American Association of State Highway and Transportation Officials (2014) *LRFD Bridge design specifications, customary U.S. units, 7th edition.* Washington DC, USA. Superseded.

(17) Standards New Zealand NZS 3101 Parts 1 and 2:1995 *Concrete structures standard – The design of concrete structures*. Replaced by NZS 3101.1&2:2006.

(18) Concrete Institute of Australia (2007) CIA Z3 *Grouting of prestressing ducts*. Sydney, NSW, Australia.

(19) Federation International du Béton (2000) *Corrugated plastic ducts for internal bonded post-tensioning*. *fib* Bulletin 7, Lausanne, Switzerland.

(20) Federal Highway Administration (2013) *Post-tensioning tendon installation and grouting manual.* US Department of Transportation, Washington DC, USA.

(21) The Concrete Society (2010) *Durable post-tensioned concrete structures*. Technical report 72, The Concrete Society, United Kingdom.

(22) Standards Australia and Standards New Zealand jointly AS/NZS 4671:2001 *Steel reinforcing materials*.

(23) Standards New Zealand NZS 3404 Parts 1 and 2:1997 *Steel structures standard*.

(24) Standards Australia and Standards New Zealand jointly AS/NZS 5100.6:2017 *Bridge design*. Part 6 Steel and composite construction.

(25) Standards New Zealand NZS 3404.1:2009 *Steel structures standard*. Part 1 Materials, fabrication, and construction. Withdrawn.

(26) Standards Australia and Standards New Zealand jointly AS/NZS 5131:2016 *Structural steelwork - Fabrication and erection*. (Incorporating Amendment No. 1: 2020)

(27) Waka Kotahi NZ Transport Agency (2013) *Steel-concrete composite bridge design guide.* Research report 525, Wellington.

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

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

(30) Standards Australia AS 5100.2:2017 *Bridge design.* Part 2 Design loads.

(31) Standards Australia AS 4100:2020 *Steel structures*.

(32) British Standards Institution *BS EN 1990:2002 Basis of structural design*.

(33) New Zealand National Society for Earthquake Engineering (1985) *Seismic design of steel structures*. Deliberations of the society's study group, Bulletin of New Zealand National Society for Earthquake Engineering, December 1985.

(34) Waka Kotahi NZ Transport Agency (2016) *Highway structures design guide*. Wellington.

(35) Waka Kotahi NZ Transport Agency (2017) *Protective coatings for steel bridges: a guide for bridge and maintenance engineers*. Wellington.

(36) Waka Kotahi NZ Transport Agency (2020) NZTA S9 *Specification for coating steelwork on highway structures*. Wellington.

(37) Standards New Zealand SNZ TS 3404:2018 *Durability requirements for steel structural and components*.

(38) New Zealand Heavy Engineering Research Association (2020) *New Zealand weathering steel guide for bridges*. Report R4-97, Manukau City.

(39) The Society for Protective Coatings and NACE International jointly (2007) SSPC SP 6/NACE No. 3 *Commercial blast cleaning*. Pittsburgh, PA, USA.

(40) Standards Australia AS 1112: _______ *ISO metric hexagon nuts*
Part 1:2015 Style 1 – Product grades A and B
Part 2:2015 Style 2 – Product grades A and B
Part 3:2015 Product grade C
Part 4:2015 Chamfered thin nuts – Product grades A and B.

(41) Standards Australia AS 1237.1-2002 (R2015) *Plain washers for metric bolts, screws and nuts for general purposes.* Part 1 General plan.

(42) British Standards Institution BS 4486:1980 *Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete*.

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

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

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

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

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

(48) Standards Australia and Standards New Zealand jointly AS/NZS 1664.1:1997 (R2020) *Aluminium structures*. Part 1 Limit State Design.

(49) Standards Australia AS 5100.4:2017 *Bridge design.* Part 4 Bearings and deck joints.

(50) American Association of State Highway and Transportation Officials (2021) *LRFD Bridge design specifications, 9th edition.* Washington DC, USA.

(51) American Association of State Highway and Transportation Officials (2014) *Guide specifications for seismic isolation design*, 4th edition. Washington DC, USA.

(52) Highways England (2020) CD 357 *Bridge expansion joints*. TSO, London, United Kingdom.

(53) Bruce SM and Kirkcaldie DK (2000) *Performance of deck expansion joints in New Zealand road bridges.* Research report 186, NZ Transport Agency, Wellington.

(54) Burke MP (1989) *Bridge deck expansion joints*. National Cooperative Highway Research Program Synthesis of Highway Practice 141, Transportation Research Board, Washington DC, USA.

(55) Waka Kotahi NZ Transport Agency (2015) *Criteria and guidance for the design of integral bridges in New Zealand.* Research report 577, Wellington.

(56) Highways England (2003) BA 42/96 Amendment No. 1 *The design of integral bridges*. TSO, London, United Kingdom. Withdrawn.

(57) Applied Technology Council and Multidisciplinary Center for Earthquake Engineering Research (2003) *Recommended LRFD guidelines for the seismic design of highway bridges*, MCEER/ATC 49. Buffalo NY, USA.

(58) Khalili-Tehrani P, Taciroglu E and Shamsabadi A (2010) *Backbone curves for passive lateral response of walls with homogenous backfills*. Published in *Soil-foundation-structure interaction* – Orense, Chouw & Pender (eds), Taylor & Francis Group, London, United Kingdom.

(59) Highways England (2001) BD 57/01 *Design for durability*. TSO, London, United Kingdom. Withdrawn.

(60) Highways England (2001) BA 57/01 *Design for durability*. TSO, London, United Kingdom. Withdrawn.

(61) Highways England (1987) BD 30/87 *Backfilled retaining walls and bridge abutments*. TSO, London, United Kingdom. Withdrawn.

(62) Highways England (2001) BD 31/01 *The design of buried concrete box and portal frame structures*. TSO, London, United Kingdom. Withdrawn.

(63) Highways England (1992) BD 24/92 *The design of concrete highway bridges and structures. Use of BS 5400: Part 4: 1990*. TSO, London, United Kingdom. Not in use. (See Highways England (2015) BD 44/15 *The assessment of concrete highway bridges and structures*. TSO, London, United Kingdom. Withdrawn.)

(64) Highways England (2001) BD 37/01 *Loads for highway bridges*. TSO, London, United Kingdom. Withdrawn.

(65) National Institute of Water and Atmospheric Research *New Zealand median annual average temperature (°C), 1981 – 2010*. Last accessed 28 March 2022. <www.niwa.co.nz/education-and-training/schools/resources/climate/overview>.

(66) British Standards Institution BS 8002:1994 *Code of practice for earth retaining structures.* Superseded, withdrawn.

(67) British Standards Institution BS EN 1997-1:2004+A1:2013 *Eurocode 7. Geotechnical design.* Part 1 General rules.

(68) Waka Kotahi NZ Transport Agency (1997) TNZ F/01 *Earthworks construction*. Wellington.

(69) Waka Kotahi NZ Transport Agency (2013) NZTA F2 *Pipe subsoil drain construction*. Wellington.

(70) Waka Kotahi NZ Transport Agency (2003) TNZ F/06 *Fabric wrapped aggregate subsoil drain construction*. Wellington.

(71) Standards Australia and Standards New Zealand jointly
AS/NZS 2041: _______ *Buried corrugated metal structures*
Part 1:2011 Design methods (Incorporating Amendment No. 1: 2018)
Part 2:2011 Installation
Part 3 Assessment of existing structures (In prep.)
Part 4:2010 Helically formed sinusoidal pipes
Part 5 Helically formed ribbed pipes (In prep.)
Part 6:2010 Bolted plate structures
Part 7 Bolted plate structures with transverse stiffeners (In prep.)
Part 8 Metal box structures (In prep.).

(72) Standards Australia and Standards New Zealand jointly
AS/NZS 3725:2007 *Design for installation of buried concrete pipes.*

(73) Standards Australia and Standards New Zealand jointly
AS/NZS 4058:2007 *Precast concrete pipes (pressure and non-pressure).*

(74) Austroads (2011) *Guidelines for design, construction, monitoring and rehabilitation of buried corrugated metal structures*, AP-T196-11. Sydney, NSW, Australia.

(75) Wang J (1993) *Seismic design of tunnels*. Monograph 7, Parsons Brinckerhoff Quade and Douglas Inc, New York, NY, USA.

(76) Federal Highway Administration (2011) *LRFD Seismic analysis and design of transportation geotechnical features and structural foundations*. NHI course no. 130094 reference manual, Geotechnical Engineering Circular 3, publication FHWA-NHI-11-032, US Department of Transportation, Washington DC, USA.

(77) Wood JH (2005) *Earthquake design of rectangular underground structures*. Proceedings of the 2005 NZ Society for Earthquake Engineering Annual Conference, Wairakei Resort, Taupo.

(78) Penzien J (2000) *Seismically induced racking of tunnel linings*. Earthquake Engineering and Structural Dynamics, volume 29.

(79) Nishioka T and Unjoh S (2003) *A simplified evaluation method for the seismic performance of underground common utility boxes*. Proceedings of the 2003 Pacific Conference on Earthquake Engineering, Auckland.

(80) Wood JH (2004) *Earthquake design procedures for rectangular underground structures*. Project report to Earthquake Commission, EQC project no. 01/470, rev B.

(81) Standards New Zealand NZS 4402:1986 *Methods of testing soils for civil engineering purposes – Soil tests.*

(82) Standards New Zealand NZS 4402.4.1.1:1986 *Soil compaction tests - Determination of the dry density/water content relationship* - Test 4.1.1 New Zealand standard compaction test.

(83) Standards New Zealand NZS 4402.4.1.2:1986 *Soil compaction tests - Determination of the dry density/water content relationship* - Test 4.1.2 New Zealand heavy compaction test.

(84) Standards New Zealand NZS 4402.4.1.3:1986 *Soil compaction tests - Determination of the dry density/water content relationship* - Test 4.1.3 New Zealand vibrating hammer compaction test.

(85) Standards New Zealand NZS 4402.4.2.1:1988 *Soil compaction tests - Determination of the minimum and maximum dry densities and relative density of a cohesionless soil* - Test 4.2.1 Minimum dry density.

(86) Standards New Zealand NZS 4402.4.2.2:1988 *Soil compaction tests - Determination of the minimum and maximum dry densities and relative density of a cohesionless soil* - Test 4.2.2 Maximum dry density.

(87) Oakden GJ (1977) *Highway surface drainage: Design guide for highways with a positive collection system*. Roading Directorate, Ministry of Works and Development, Wellington.

(88) National Institute of Water and Atmospheric Research *High intensity rainfall design system (HIRDS)* version 3. Last accessed 28 March 2022. <http://hirds.niwa.co.nz/>.

(89) Ministry for the Environment (2008) *Climate change effects and impacts assessment: A guidance manual for local government in New Zealand – 2nd edition*. Wellington.

(90) Tomlinson AI (1980) *The frequency of high intensity rainfalls in New Zealand, part 1.* Water and Soil Technical Publication19, Ministry of Works and Development, Wellington.

(91) Waka Kotahi NZ Transport Agency (2010) NZTA F3 *Pipe culvert construction*. Wellington.

(92) Waka Kotahi NZ Transport Agency (2020) NZTA S10 *Specification for anti-graffiti coatings*. Wellington.

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