

4 Analysis and Design Criteria

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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, *Code of Practice for the Design of Concrete Structures*⁽¹⁾, with the following provisos:

(a) Crack Widths (Table 3.4), and Stresses for Prestressed Concrete (Table 16.1)

Load Category II shall be applied to Group 1A loads, and Load Category IV shall be applied to all other load groups as defined in Tables 3.1 and 3.2 of this document.

(b) Design for Durability (Section 5)

All parts of bridges shall be considered to be in an "external" type of environment.

Minimum concrete covers for a design life of 100 years shall be not less than those set out in Table 4.1.*

* Table 4.1 is an interim requirement pending further study.

Table 4.1 : Minimum Required Concrete Covers for a Design Life of 100 years

The following tables are replacements for NZS 3101: Part 1:1995 Table 5.5. The following covers apply to concrete construction incorporating type GP Portland cement complying with NZS 3122⁽²⁾. Concrete construction incorporating alternative cement types are not covered by the following tables and shall be evaluated to establish their performance. The following tables do not apply to concrete construction where protective coatings are applied. Exposure classifications are as defined in NZS 3101⁽¹⁾ Section 5, and in all other respects the durability requirements of NZS 3101⁽¹⁾ Section 5 shall apply.

Part A : Standard Formwork and Standard Compaction*

Exposure Classification	Specified compressive strength f_c'			
	25	30	40	50
	Minimum required cover (mm)			
A1	30	25	20	20
A2	35	30	25	25
B1	50	40	35	30
B2	65	60	50	40
C***				70
Cast in contact with non-aggressive ground	75	75	75	75
Cast with damp proof course in contact with non-aggressive ground	50	50	50	50

Part B : Precast Construction with Rigid Formwork and Intense Compaction**

Exposure Classification	Specified compressive strength f_c'			
	25	30	40	50
	Minimum required cover (mm)			
A1	25	20	20	20
A2	35	30	25	25
B1	45	35	30	30
B2	55	50	40	35
C***				70

Notes

- * Part A shall apply where concrete is cast in formwork complying with AS 3610⁽³⁾ and is transported, placed and compacted so as to:
- limit segregation or loss of materials;
 - limit premature stiffening;
 - produce a monolithic mass between joints, the extremities of members, or both;
 - completely fill the formwork to the intended level, expel entrapped air, and closely surround all reinforcement, tendons, ducts, anchorages and embedments; and provide the specified finish to the formed areas of the concrete.
- ** Part B shall apply where the concrete is cast in rigid forms and subjected to intense compaction, such as obtained with vibrating tables or form vibrators.
- *** In addition to the specified cover, a type GP Portland cement content of 350 kg/m³ and water cement ratio not exceeding 0.4 is required.

Footnote: Standards of formwork and compaction are as adopted by the AS 5100: *Bridge Design*⁽⁴⁾ (refer clause 5.4.10.3)

(c) Friction Losses (Clause 16.3.2)

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.

(d) Reinforced Concrete Deck Slab Thickness (Table 3.3)

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 4.2.2 of this document, the minimum slab thickness requirements of 4.2.2 of this document shall take precedence over the requirements of NZS 3101⁽¹⁾.

4.2.2 Reinforced Concrete Deck Slab Design

Two methods of deck slab panel design are available for reinforced concrete deck slabs on beams or girders.

The methods are an empirical design based on assumed membrane action, and design based on an elastic plate bending analysis, as outlined below. Where the dimensional and structural limitations of the empirical design method are not met, or for deck cantilevers, the elastic plate bending analysis design method shall be used.

(a) Empirical Design Based on Assumed Membrane Action

Slabs satisfying the requirements below and designed in accordance with this method need not be analysed, and the requirements of NZS 3101⁽¹⁾ clauses 3.3.2.5 and 3.3.3 and Sections 8 and 9 shall be waived. The requirements of NZS 3101⁽¹⁾ clauses 3.7.2.1 and 3.7.2.2 shall not be applied to the reinforcement aligned perpendicular to the direction of traffic flow unless that reinforcement contributes to the strength of a member supporting the deck slab. The durability requirements of NZS 3101⁽¹⁾ shall otherwise be satisfied.

Empirical design may be used only if the following conditions are satisfied:

- the supporting components are made of steel and/or concrete,
- there are at least three longitudinal girder webs in the system,
- the deck is fully cast-in-place,
- the deck is of uniform depth, except for haunches at beam flanges and other local thickening,
- the deck is made composite with the supporting structural components,

- all cross frames or diaphragms extend throughout the cross section of the bridge between external girders, and the maximum spacing of such cross frames or diaphragms shall be as follows:
 - Steel I and Box Girders: 8.0m
 - Reinforced and prestressed concrete girders: at support lines
- the ratio of span length, L_s , to slab thickness, (excluding a sacrificial wearing surface where applicable), shall not exceed 15,
- the maximum span length, L_s , does not exceed 4.0m,
- the minimum slab thickness is not less than 165mm excluding a sacrificial wearing surface where applicable,
- the core depth is not less than 90mm. The core depth = slab thickness - (wearing surface + top and bottom cover thicknesses),
- there is an overhang beyond the centreline of the outside beam of at least 5 times the slab thickness. This condition may be considered satisfied if the overhang is at least 3 times the slab thickness and a structurally continuous concrete kerb or barrier is made composite with the overhang,
- the specified 28 day compressive strength of the deck concrete is not less than 30 MPa.

For slabs meeting the above conditions, the deck reinforcement shall comprise:

- two layers of isotropic reinforcement
- reinforcing steel shall be Grade 430 or better
- the outer layer of reinforcement in each face of the slab shall be placed normal to the beams
- the minimum amount of reinforcing shall be 0.3% in each layer of each orthotropic direction. The reinforcement ratio shall be determined using the effective depth of slab, d , being the distance from the extreme compression fibre (excluding any wearing surface) to the centroid of the tension reinforcement. For the purposes of a layer of isotropic reinforcement, d shall be the average of the effective depths, at the midspan of the slab, in the two reinforcing directions.
- the maximum spacing of the reinforcement shall be 300mm
- bars shall be spliced by lapping or by butt welding only
- for skew angles greater than 20° the end regions of each span shall be reinforced with 0.6% isotropic reinforcement in two layers. The span end regions are as defined in Figure 4.1.

The longitudinal bars of the isotropic reinforcement may be assumed to resist negative moments at an internal support in continuous structures.

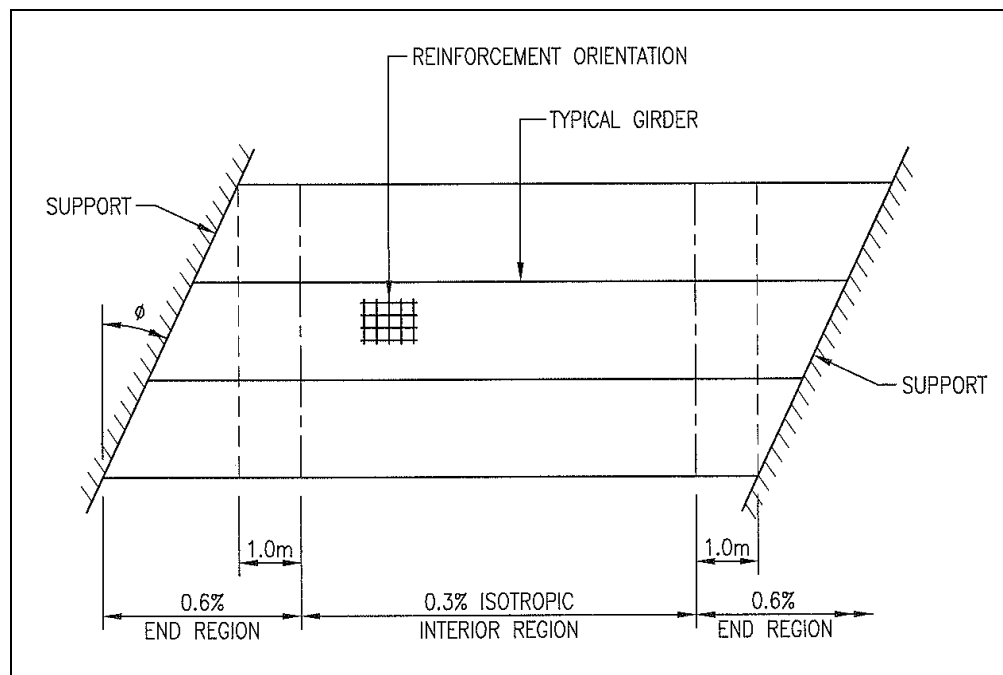


Figure 4.1 : Reinforcement of Skewed Slabs by the Empirical Method

(b) Design Based on Elastic Plate Bending Analysis

The moments in deck slabs due to the local effects of wheels shall be determined by an elastic analysis, assuming the slab to act as a thin plate. Adequate allowance shall be made for the effects of the rotation of the edges of slabs monolithic with beams, due to torsional rotation of the beams, and the effects of relative displacement of beams.

Where the deck slab resists the effects of live load in two ways (e.g., the top flange of a box girder also functioning as the deck slab, or a transverse distribution member integral with the deck slab) the slab shall be designed for the sum of the effects of the appropriate loading for each condition.

Where slabs are haunched at fixed edges, allowance for the increase in support moment due to the haunch shall be made either by modifying the moments determined for slabs of uniform thickness, or by a rational analysis that takes into account the varying section.

4.3 Structural Steel and Composite Construction

4.3.1 General

Design shall be in accordance with NZS 3404 *Steel Structures Standard*⁽⁵⁾ for all bridge types falling within the scope of the standard, with the provisos set out in 4.3.2.

Steel box girders are not within the scope of NZS 3404⁽⁵⁾. Design for these bridge elements shall be in accordance with the AS 5100: *Bridge Design*⁽⁴⁾, Part 6, Steel and Composite Construction, for box girders with composite concrete decks, or otherwise to BS 5400: *Steel, Concrete and Composite Bridges*⁽⁶⁾, Parts 3, 5 and 10.

Design of deck slabs for composite bridges shall be in accordance with 4.2.2.

4.3.2 Application of NZS 3404

(a) Design Loadings (Clause 3.2.3)

The design load combinations for the ultimate and serviceability limit states shall be those specified in this document.

(b) Seismic Design Structural Performance Factor (CI 12.2.2.1)

The structural performance factor, S_p , shall be as specified in this document.

(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 *Transit New Zealand 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 *Transit New Zealand Bridge Manual*. For bridges it is not generally necessary to consider seismic response at the serviceability limit state.

4.3.3 Application of BS 5400

In clause 4.3.3 of BS 5400⁽⁶⁾: Part 3, and clause 4.1 of Part 5, the value of the product $\gamma_{fl} \times \gamma_{f3}$, for the ultimate limit state for each load effect shall be taken as equal to the product $a.b$. These shall be taken from Table 3.2 of this document, where a is the factor outside the bracket, and b is the factor attached to the load effect in each case.

In design of composite sections, the value of f_{cu} in BS 5400⁽⁶⁾: Part 5 shall be taken as $1.4 f_c'$. The term f_c' is the specified strength of the concrete, as defined in NZS 3101⁽¹⁾.

4.3.4 Seismic Resistance

Where materials design codes other than NZS 3404⁽⁵⁾ 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, *Steel Structures Standard*⁽⁵⁾, shall be followed. The recommendations of the NZNSEE study group on *Seismic Design of Steel Structures*⁽⁷⁾ shall also be followed where applicable.

4.3.5 Fatigue Design

Assessment of the fatigue resistance of steel structures shall be based on NZS 3404: Parts 1 and 2:1997⁽⁵⁾. For comment on the fatigue loading, see 3.2.6 of this document.

4.4 Timber

4.4.1 General

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

NZS 3603 *Timber Structures Standard*⁽⁸⁾, for the timbers that it covers.

AS 1720.1 *Timber Structures*, Part 1: “Design Methods”⁽⁹⁾, for the timbers that it covers that are not covered by NZS 3603.

Characteristic stresses adopted for design to AS 1720.1⁽⁹⁾ shall be in accordance with AS 1720.2 *SAA Timber Structures Code*, Part 2: “Timber Properties”⁽⁹⁾ and AS 2878 *Timber - Classification Into Strength Groups*⁽¹⁰⁾.

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 1720.1⁽⁹⁾, Table 2.5, corresponding to the type of timber product (e.g. sawn timber, round timbers, etc.) and type of grading (e.g. visually graded, machine graded, proof graded, etc.).

For bridges carrying less than 2500 vehicles per day, the duration of load factor may be increased to 0.94 for the Group 1A load combination of Table 3.2.

For the grid system or parallel support system modification factor (k_4 , k_5 or k_6 in NZS 3603⁽⁸⁾, or k_9 in AS 1720.1⁽⁹⁾) 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 possesses smooth sealed approaches, the live load impact factor may be taken as follows:

$$\text{Impact factor} = 1.0 + (I - 1.0) \times 0.7$$

Where I is the impact factor defined by 3.2.5.

4.4.3 Seismic Resistance

Design shall comply with NZS 3603⁽⁸⁾ 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 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: 1997 *Aluminium Structures*, Part 1: "Limit State Design"⁽¹¹⁾, 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⁽¹¹⁾ 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 General Manager, Transit New Zealand.

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 the AS 5100: *Bridge Design*⁽⁴⁾, Part 4: Bearings and Deck Joints except as modified herein. Where there may be conflict between the requirements of AS 5100.4⁽⁴⁾ and this document, this document shall take precedence.

(b) 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.

(c) 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 60 m and skews of less than 30° should have integral abutments. It is accepted deck joints may be necessary in larger bridges to cater for periodic changes in length.

4.7.2 Modifications and Extensions to the AS 5100: *Bridge Design, Part 4: Bearings and Deck Joints* Criteria for Bearings

(a) Limit State Requirements and Robustness

Pot bearings shall be designed for both the serviceability and ultimate limit states. Elastomeric bearings shall be designed for serviceability limit state effects, with the bearing fixings and overall bridge structure stability checked at the ultimate limit state.

Particular consideration shall be given to the robustness of bearings and their fixings to damage or loss of stability due to earthquake actions.

(b) Design Loads and Load Factors

Reference in the AS 5100 *Bridge Design*⁽⁴⁾, Part 4, “Bearings and Deck Joints” to design loads and load factors given in AS 5100.2 shall be replaced by reference to Chapter 3 of this manual.

(c) Anchorage of Bearings

Bearings, other than thin elastomeric strip bearings less than 25 mm in thickness, shall be positively anchored to the bridge structure above and below to prevent their dislodgement during response to the ultimate 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.

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. 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.

(d) 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.

(e) Elastomeric Bearings

Elastomeric bearings shall conform with the requirements of either AS 1523 *Elastomeric Bearings for Use in Structures*⁽¹²⁾, or BE 1/76, *Design Requirements for Elastomeric Bridge Bearings*⁽¹³⁾, except that steel reinforcing plates may be a minimum of 3 mm 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 effects, 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 ultimate limit state design intensity earthquake, plus other prevailing conditions such as shortening effects, 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, 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 creep shortening of the superstructure due to the prestress on the bearings.

4.7.3 Modifications to the AS 5100: *Bridge Design, Part 4: Bearings and Deck Joints* Criteria for Deck Joints**(a) General Requirements**

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

(b) Design Loads

Deck joints and their fixings shall be designed at the ultimate limit state for the following loads in place of those specified by the AS 5100: *Bridge Design*⁽⁴⁾, Part 4: Bearings and Deck Joints:

(i) Vertical

The vehicle axle loads defined in 3.2.2 together with a dynamic load factor of 1.60. The ultimate limit state load factors to be applied shall be 2.25 to an HN axle load, and 1.49 to an HO axle load

(ii) 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 5.5.1(b), and to otherwise satisfy the requirements of 5.5.1(b).

(ii) Design for longitudinal movements shall include the effect of beam end rotation under live load

(d) Anchorage

The second paragraph of AS 5100.4⁽⁴⁾, Clause 17.4 shall be replaced by the following:

Where the deck joint is attached by bolts fixing into a concrete substrate or screwed into cast-in anchor ferrules, fully tensioned high tensile bolts 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 500 kN per metre length on each side of the joint.

(e) Drainage

The AS 5100.4⁽⁴⁾, Clause 17.5 shall be replaced by the following:

Deck joints shall be watertight unless specific provision is made to collect and dispose of the water. 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 the movement across the deck joints of the bridge of not less than one quarter of the calculated relative movement under the ultimate limit state design earthquake conditions, plus long term shortening effects where applicable, and one third of the 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.

(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).

4.7.4 Additional Criteria and Guidance for Deck Joints

(a) Joint Type and Joint System Selection

Deck joints shall be designed to provide for the total design range and direction of movement expected for a specific installation. The design engineer shall consider the guidance provided by the UK Highways Agency publication: BD33/94 *Expansion Joints for Use in Highway Bridge Decks*⁽¹⁴⁾ 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. The design engineer shall specify all dimensional and performance requirements, including movement capacity, 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 design engineer shall consider information provided in Transfund New Zealand Research Report No.186 *Performance of Deck Expansion Joints in New Zealand Road Bridges*⁽¹⁵⁾ and Burke, (1989), *Bridge Deck Joints*⁽¹⁶⁾ with respect to the performance history of deck joints.

Proprietary deck joint suppliers shall provide a warranty on the serviceability 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 (e.g. 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. 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 200 mm 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 75 mm and a minimum width of bond with the structure on either side of the joint gap of 200 mm. 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 Transit 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 Foundations

4.8.1 Loads on Foundations

Foundations shall be designed for bearing capacity and stability to resist combined horizontal and vertical loadings with acceptable displacements and settlement. Consideration shall be given to the behaviour of the founding soils under static and dynamic loading and during construction.

Foundations shall be designed to resist loads that may arise from negative friction (down-drag) associated with settlement or ground subsidence. Lateral loads associated with slope movements shall be considered, although wherever practicable the designer shall isolate the structure and foundations from such forces.

The effects of live load may normally be ignored in the evaluation of foundation settlement, except in special cases where the live load is sustained over long periods of time. The repetitive nature of live load shall be taken into consideration, where it has the potential to affect foundation performance.

4.8.2 Design Standards

- (a) Foundation design shall be based on appropriate sound design methods and shall satisfy *The New Zealand Building Code*⁽¹⁷⁾.
- (b) The following standards and codes of practice provide guidance on the design of foundations:
- *New Zealand Building Code Handbook*⁽¹⁷⁾, Verification Method B1/VM4
 - BS 8004: *Code of Practice for Foundations*⁽¹⁸⁾ (it should be noted that this document does not cover earthquake resistant design aspects).
 - CAN/CSA-S6-00 *Canadian Highway Bridge Design Code*⁽¹⁹⁾
 - AASHTO *Standard Specification for Highway Bridges*, Code HB-17⁽²⁰⁾.
 - AS 2159: 1995 Piling – Design and Installation⁽²¹⁾

The Transit New Zealand *Bridge Manual* shall take precedence where there is a conflict.

4.8.3 Strength Reduction Factors for Foundation Design

Strength reduction factors for ultimate limit state design shall lie within the ranges in Table 4.2 unless specific conditions justify lower or higher values. Adoption of lower or higher values shall be justified in the design statement, for Transit New Zealand's acceptance.

**Table 4.2: Ranges of Strength Reduction Factor, Φ ,
for Foundation Design**

Method of Assessment of Ultimate Geotechnical Strength	Strength Reduction Factor Φ
Plate bearing strength (to failure)	0.55 – 0.65
Pile static load tests (to failure)	0.65 – 0.80
Static proof load tests (not to failure)	0.55 – 0.65
Geotechnical analyses using in situ test results and correlations	0.45 – 0.55
Geotechnical analyses using laboratory test results for cohesive soils	0.45 – 0.55
Geotechnical analyses based on visually assessed parameters from borehole logs or engineering geology description of rock	0.40 – 0.45
Dynamic pile load tests using wave analyses	0.50 – 0.75
Dynamic pile formula (Hiley)	0.45 - 0.50

The strength reduction factors for design shall be carefully chosen by the designer from within the ranges specified in Table 4.2, giving consideration to:

- The extent of redundancy in the foundations
- Complexity of the geology and variability of the ground conditions
- Extent and quality of site investigations carried out
- Level of uncertainty as to the ground conditions and strength parameters
- Proximity of site investigation to particular foundations
- Mode of failure
- Design method adopted
- Experience with similar conditions
- The method of design analyses and method of acceptance testing
- The number of load tests or acceptance tests carried out
- Level of construction control
- Level of performance monitoring during and after construction
- Consequences of foundation failure to the structure, the road link, the road users and the community.

Particular care should be taken in the use of the high end of the strength reduction factors used for pile static load tests and dynamic tests using wave analyses, depending on the variability of the ground and number of tests carried out, and the confirmation of dynamic tests using static load tests.

Strength reduction factors for assessment or confirmation methods not included in the above table shall not be higher than that given in the table for the closest method, and shall be highlighted in the Design Statement for consideration by Transit New Zealand.

4.8.4 Capacity Design of Foundations

Capacity design shall be applied to the design of foundations to resist forces induced in the structure by yielding structural elements developing their over-strength capacity under the action of earthquake acting in combination with gravity loads. The strength of the foundations to be provided shall be determined on the basis of the nominal ultimate strength of the foundation (S_n) factored by the strength reduction factor derived from 4.8.3, and divided by 0.85, i.e.:

$$\text{Foundation forces mobilized by over-strength actions} \leq \frac{\Phi S_n}{0.85}$$

4.8.5 Foundation Capacity Determination

The load capacity of foundations shall be assessed using geotechnical parameters from geotechnical investigations and tests, and soil / rock mechanics theory or semi-empirical geotechnical methods. The capacity shall be confirmed during construction as specified in Section 4.8.5.

4.8.6 Confirmation of Foundation Conditions during Construction

The designer shall clearly state on the drawings and the in the specifications, the foundation conditions assumed in the design, or ensure that the designer is consulted during construction to ensure that the design requirements are being met.

The foundation conditions shall always be verified during construction, against the ground conditions assumed in the design, as site investigations cannot fully define the actual ground conditions at each foundation. The designer shall specify measures to be used to verify the ground conditions.

Appropriate measures to confirm foundation conditions may comprise one or more of the following, depending on the particular situation:

- Inspection, logging and possibly testing of the ground by a geotechnical engineer or engineering geologist.
- Plate bearing tests
- Static pile load tests
- Pilot hole drilling and testing
- Down-hole Inspection of pile shaft, particularly in bedrock
- Dynamic pile load tests

The Hiley formula has traditionally been used to confirm the pile capacities in cohesionless soils, by relating the pile driving energy and the pile set (displacement per hammer blow) to pile capacities. The limitations of this method are now recognised. A more sophisticated method involves analysis of the pile response to hammer driving, using a Pile Driving Analyser. Usually the pile response data should be further interpreted using a signal matching program such as CAPWAP or similar. Such methods still have their limitations, and these should be recognised by the designer.

- Pile integrity tests to confirm the structural integrity of the pile, the relative shape of the pile shaft or the continuity of the pile.

Pile integrity tests shall be specified where the piles are not permanently cased and where there is a risk of collapse of the ground during construction of bored piles, particularly below the water table, or where there is significant potential for damage to the pile shaft during pile driving.

The type of pile integrity testing to be used shall be specified by the designer.

4.9 Earth Retaining Systems

4.9.1 Loads

Earth retaining systems shall be designed to ensure overall stability, internal stability and bearing capacity under appropriate combinations of horizontal and vertical loads, with acceptable displacements and settlement.

The designer shall derive the design loads on the structure, taking into consideration the flexibility and likely deformation of the structure, and the allowable displacement or deformation of the system. Careful consideration shall be given to the interaction between the structure, the ground and foundations, under static, dynamic, earthquake and construction conditions. The deformation and displacement of the structure shall be compatible with the performance requirements for the structure and its interaction with adjacent or supported structures and facilities. Earthquake displacement criteria are specified in Section 5.7.

4.9.2 Design Standards

The following standards and codes of practice provide guidance on the design of retaining structures:

- Road Research Bulletin 84, *Bridge Design and Research Seminar 1990*, Volume 2: Seismic Design of Bridge Abutments and Retaining Walls. (Transit New Zealand, 1990)⁽²²⁾.
- BS 8002: 1994, *Code of Practice for Earth Retaining Structures*⁽²³⁾ (note that this does not cover earthquake resistant design issues).
- AS 4678-2002 *Earth-Retaining Structures*⁽²⁴⁾. (Standards Australia, 2002).
- CAN/CSA-S6-00 *Canadian Highway Bridge Design Code*⁽¹⁹⁾.
- AASHTO *Standard Specification for Highway Bridges*, Code HB-17⁽²⁰⁾.
- FHWA NHI-99-025 *Earth Retaining Structures*⁽²⁵⁾.
- CIRIA C580 *Embedded Retaining Walls – Guidance for Economic Design*⁽²⁶⁾.

Road Research Bulletin 84⁽²²⁾ shall be used in preference to the other documents, particularly for earthquake resistant design. The Transit New Zealand *Bridge Manual* shall take precedence over all other documents.

4.9.3 Factors of Safety

The minimum factors of safety for a free-standing retaining structure shall be as set out in Table 4.3.

The factor of safety for sliding is the ratio of forces resisting sliding to those causing sliding.

The factor of safety for overturning is calculated as the ratio of restoring moments to disturbing moments, for overturning of the wall.

Table 4.3: Minimum Factors of Safety for Stability

	Static including Flooding	Seismic
Sliding	1.5	1.2*
Overturning	2.0	1.5
Overall Stability against soil failure	1.5	1.25

* Unless the wall is designed for permanent displacement under earthquake loads, see Section 5.7.

4.9.4 Common Highway Retaining Structures

Different common retaining wall systems used for highway construction are listed in Table 4.4.

Table 4.4: Retaining Wall Categories

Retaining Wall Category	Retaining Wall Systems
Gravity and Reinforced Concrete Cantilever Walls	<ul style="list-style-type: none"> ▪ Gravity walls (concrete, gabion, crib) ▪ Reinforced concrete cantilever walls
Anchored Walls	<ul style="list-style-type: none"> ▪ Anchored gravity walls ▪ Anchored cantilever walls ▪ Anchored soldier pile walls
Mechanically Stabilised Earth (MSE) Walls	<ul style="list-style-type: none"> ▪ Soil nailed walls ▪ Reinforced soil walls <ul style="list-style-type: none"> - inextensible reinforcement - extensible reinforcement

Mechanically stabilised earth (MSE) walls comprise reinforcement elements in the ground to stabilise the soil against failure. A wall face (eg reinforced concrete panels or blocks) connected to the reinforcement is generally provided.

MSE walls can be divided into two types :

- Soil nailed walls, where the reinforcement is inserted into the ground, with top-down construction as excavation for the wall face proceeds.
- Reinforced soil walls, where the reinforcement is incorporated within fill as the fill is placed and compacted, to build the wall using bottom-up construction.

Specific requirements for different retaining wall systems in common use are specified in the following sections.

4.9.5 Gravity and Reinforced Concrete Cantilever Walls

Gravity and reinforced concrete cantilever walls are relatively rigid and are less tolerant of settlements. Therefore they shall be founded on an appropriate stratum which will minimise settlements.

These walls may be designed to undergo limited sliding displacement under strong earthquake shaking as specified in Section 5.7.

4.9.6 Anchored Walls

- (a)** Walls that are restrained using anchors, are designed to transfer some of the loads on walls to the ground outside the zone of influence of the wall.

Anchors transfer the loads into the ground through :

- Deadman structures
- Grouting anchors into drilled holes
- Mechanical systems

- (b)** Anchored walls are generally rigid systems, and shall be designed to resist the full ground, groundwater and earthquakes forces on the walls. They shall not be designed to allow outward displacement by sliding in earthquakes or other conditions. An exception may be when the wall is anchored to a deadman that is designed to undergo limited displacement under strong earthquake shaking.

- (c)** Ground anchors shall generally be designed and installed in accordance with established design standards such as BS 8081:1989 *Code of Practice for Ground Anchors*⁽²⁷⁾, BS EN 1537; 2000 *Execution of Special Geotechnical Work – Ground Anchors*⁽²⁸⁾, and FHWA-IF-99-015 *Ground Anchors and Anchored Systems*⁽²⁹⁾, except as provided in this document.

- (d)** The anchor system shall be designed to ensure a ductile failure of the wall, under earthquake overloads as discussed in Section 5.7.

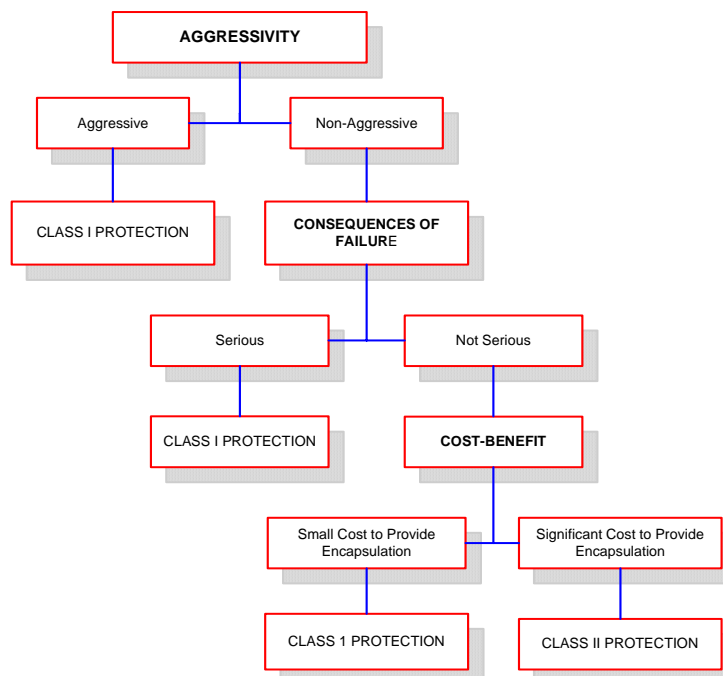
- (e)** The anchor system shall be corrosion protected to ensure its durability over the design working life of the structure.

Two classes of protection are provided for general use for anchors, as defined in Table 4.5 below.

Table 4.5: Class of Corrosion Protection for Anchors and Soil Nails

Class of Protection	Corrosion Protection Standard
Class I	<p>Double corrosion protection by encapsulation of the tendon or bar pre-grouted under factory conditions inside a corrugated plastic sheath to minimise crack widths in the pre-grouted grout as defined in BS 8081. The whole assembly is grouted into the anchor hole.</p> <p>A higher class of protection, such as using multiple sheaths, may be chosen, depending on the aggressiveness of the environment, the consequences of anchor failure and the importance of the structure.</p>
Class II	<p>Single corrosion protection using a galvanized (to AS/NZS 4680⁽³⁰⁾) or fusion bonded epoxy-coated (to ASTM A934⁽³¹⁾ or ASTM D3963⁽³²⁾) bar grouted into the anchor hole.</p>

The class of corrosion protection shall be chosen based on the decision tree shown in Figure 4.2.



Guide for Selecting the Class of Corrosion Protection for a Ground Anchor (after FHWA, 1998)

Figure 4.2: Guide to Selection of Corrosion Protection for Ground Anchors

In the decision tree, a “serious” consequence of failure shall arise when failure of the anchor and wall could:

- Affect nearby buildings or other structures
- Lead to closure of one or more lanes of the road causing major traffic disruption
- Lead to disruption to a road that has a lifeline function
- Lead to destabilisation of a landslide or slope that has experienced past instability.

(f) In Figure 4.2 ‘Aggressive’ shall be defined as where:

- the maximum pitting corrosion rate of unprotected steel is greater than 0.1mm/year or,
- soil resistivity is greater than 2000 ohm-cm or,
- pH of either the groundwater or soil is less than 5.0 or,
- total SO₃ in the soil is greater than 1% or,
- sulphates in groundwater is above 2500 ppm or,
- chlorides in the groundwater are above 2000 ppm.

(g) Pull-out tests shall be specified to be carried out on trial anchors to be installed prior to the final wall anchors being constructed. The pullout tests shall be used to confirm the design grout-ground bond strengths. The number of tests shall be chosen and specified by the designer based on the variability of the ground conditions and the number of anchors required.

(h) On-site suitability tests shall be carried out on a selected number of initially installed special anchors production anchors in accordance with BS EN 1537⁽²⁸⁾, to confirm the performance of the anchors and their suitability to ensure performance of the wall. A small number of representative full-scale anchors shall be installed and tested to confirm the suitability and performance of the anchors, prior to installation of the remainder of the anchors.

(i) On site acceptance tests shall be carried out on all anchors installed in accordance with BS EN 1537⁽²⁸⁾.

(j) The designer shall provide for future inspection, re-testing and replacement of the anchors supporting structures, if there is any reason for concern about their long-term performance.

4.9.7 Soil Nailed Walls

Soil nailed walls shall be designed and constructed in accordance with appropriate design codes such as FHWA-SA-96-069R *Manual for the Design and Construction Monitoring of Soil Nailed Walls*⁽³³⁾, except as provided for in this Bridge Manual.

Soil nailed walls are acceptable subject to the following criteria:

(a) Soil nailing shall be carried out only on drained slopes free of groundwater, or with an adequate level of drainage to ensure that the facing and the soil nailed block are fully drained.

- (b) Soil nailed walls shall not support abutments of bridges, except where it can be demonstrated that the deformation associated with mobilisation of the soil nail capacities, or any displacements associated with earthquakes can be tolerated or catered for in the design of the bridge structure.
- (c) Overall limited block displacement in strong earthquakes may be allowed subject to the criteria in Section 5.7.
- (d) Soil nails shall only be allowed to intrude into property outside the road reserve, if sub-surface rights for the design life of the structure are obtained to prevent disturbance of the reinforced soil block by future sub-surface (eg foundation, drainage) construction activities.
- (e) The soil nail reinforcement shall be subject to the corrosion protection requirements specified in Section 4.9.6 for anchors.
- (f) Pull-out tests shall be specified to be carried out on trial soil nails to be installed prior to the final wall anchors being constructed. The pullout tests shall be used to confirm the design grout-ground bond strengths. The number of tests shall be chosen and specified by the designer based on the variability of the ground conditions and the number of anchors required.
- (g) On-site suitability tests shall be carried out on a selected number of production soil nails as per BS EN 1537⁽²⁸⁾, to confirm their performance of the soil nails and their suitability to ensure performance of the wall. A small number of representative full-scale soil nails shall be installed and tested to confirm the suitability and performance of the soil nails, prior to installation of the remainder of the soil nails.
- (h) On site Acceptance tests shall be carried out in accordance with BS EN 1537⁽²⁸⁾, on at least 25% of all installed soil nails. A higher proportion of nails shall be tested, if the ground conditions are variable and the consequences of failure is high.

The designer shall provide for future re-testing of the soil nails supporting structures, if there is any concern about their long-term performance.

4.9.8 Reinforced Soil Walls

Reinforced soil walls usually comprise either “inextensible” (usually steel) or “extensible” (usually geogrid) reinforcement. Reference should also be made to requirements with respect to earthquake design given in Section 5.7.

The following criteria shall be used in the design and construction of reinforced soil walls:

- (a) Inextensible (steel) reinforcement shall be used for reinforced soil walls supporting bridge abutments, or where limiting the deformation of the wall is critical due to the presence of adjacent structures. Geogrid reinforcement may be used, provided that the bridge abutment seat is supported on piles, and the design takes into account the expected deformation of the wall system.
- (b) Design of geosynthetic-reinforced structures shall comply with appropriate design codes or manuals such as the recommendations of the Transfund New

Zealand Research Report No 239 *Guidelines for Design & Construction of Geosynthetic-Reinforced Soil Structures in New Zealand*⁽³⁴⁾, except as otherwise provided in this manual.

- (c) The long-term durability, strength and creep performance of the reinforcement, and the environmental conditions associated with the site, backfill and groundwater shall be considered in the selection and use of appropriate types of reinforcement and backfill.
- (d) Steel reinforcement shall have an adequate level of corrosion protection and sacrificial steel content to ensure the required performance over the design life of the structure.
- (e) The strength of the connections between the soil reinforcement and the facing panels or blocks shall exceed by a suitable margin the upper bound pullout strength of the reinforcement through granular fill, or the post-yield over-strength capacity of the reinforcement, whichever is lower. Design shall ensure that brittle failures of the connections will not occur.

4.10 Design of Earthworks

4.10.1 Design of Embankments

(a) Philosophy

The design of approach embankments shall be based on adequate site investigations and shall ensure acceptable performance of the embankment under gravity, live and earthquake loads and under flood and post-flood drawdown conditions. Appropriate measures shall be specified to ensure that post-construction settlements will be within acceptable limits compatible with the performance expectations for the road. Such limits shall be agreed with Transit New Zealand.

(b) Static Behaviour

Under static conditions (including appropriate live load surcharge) the completed embankment shall have a minimum calculated factor of safety of 1.5, unless specific justification for a lower value set out in the geotechnical engineering design report for the embankment has been accepted by Transit New Zealand. A suitable monitoring programme shall be implemented by the designer to check embankment performance during and after construction. The designer shall specify acceptable limits for monitoring measurements.

During construction, factors of safety less than the long-term values may be accepted but the value shall generally exceed 1.2. Where preloading, surcharging, staged loading, vertical drains or other techniques are required to permit construction of embankments or to accelerate settlement, a suitable monitoring programme shall be specified and the results shall be reviewed by the designer.

Factors of safety shall be calculated using loads and combinations for the serviceability limit state as specified in Table 3.1.

(c) Behaviour in Seismic and Flood Events

Assessments shall be made of the potential of embankment materials and underlying foundation materials to lose strength during or after flooding or earthquake. The presence of liquefiable, collapsible, sensitive or erodible materials shall be determined by appropriate site investigations and testing. Where such materials are present, assessments shall be made of the risk presented by them and the feasibility and cost of eliminating or reducing risks and/or damage.

Where it is not practical or economically justifiable to significantly reduce the risk of embankment failure due to earthquake or flooding, and the effect of such failure on the performance of the road network is acceptable to Transit New Zealand, considering the required levels of service and lifeline requirements, then the design may allow for failure to occur in such large events. This shall have the written acceptance from Transit New Zealand.

Where Transit New Zealand accepts that mitigation of failure of the road embankment in specified large events is acceptable, then the manner and extent of such failure shall be assessed and the bridge structure and foundations shall be designed to accommodate the embankment failure without damage to the structure.

Where embankments may act as water retaining structures during flooding, the ability of the embankment to sustain effects of seepage and drawdown shall be examined. In such cases the embankment shall have a minimum factor of safety against failure of between 1.25 and 1.5, depending on the consequences of failure in terms of potential downstream damage or loss of life. The *Dam Safety Guidelines*⁽³⁵⁾ published by the New Zealand Society on Large Dams provides guidance on embankments that may act as water retaining structures.

Factors of safety shall be calculated using loads and combinations for serviceability limit state as specified in Table 3.1.

Adequate protection from erosion during flooding or from adjacent waterways shall be incorporated into the design of embankments.

Where it is proposed to accept failure of the embankment under the design earthquake or flood conditions, or to adopt a factor of safety less than 1.5, justification for doing so shall be set out in a design report for Transit New Zealand's consideration and acceptance in writing, before the proposal is adopted.

(d) Loadings on Associated Bridge Structures

Earth pressure loadings, lateral loads due to ground deformation or displacement and negative friction effects on foundations that arise from the presence of the embankment shall be taken into account. Appropriate load factors shall be applied in accordance with 3.4 and 3.5.

4.10.2 Design of Approach Cuttings

Approach cuttings shall be designed in accordance with recognised current highway design practice with provision of benches, and appropriate measures to mitigate the effects of rock fall and minor slope failures as appropriate. Factors of safety given in 4.10.1 may be used as appropriate. Slope geometry shall be designed to ensure that any slope failure material will not be deposited against or over the bridge structure. Where this is not practicable, provision shall be made in the bridge design for additional dead load or earth pressure to represent the effect of slope failure material.

Where it is proposed to accept a significant risk of instability, justification for doing so shall be set out in the Design Statement for Transit New Zealand's consideration and acceptance before the proposal is adopted.

4.10.3 Natural Ground Instability

Where the bridge and associated structures can be affected by instability or creep of natural ground, measures shall be taken to either isolate the structure, or remedy the instability, or design the structure to accommodate displacements and loads arising from the natural ground. Factors of safety given in 4.10.1 may be used, as appropriate.

4.11 Integral and Semi-Integral Abutments

4.11.1 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. 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 not translation, between the superstructure and the supporting piles.

4.11.2 Design Criteria

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

- (a) Length over abutments not exceeding:
- | | |
|---|------|
| with concrete superstructure- | 70 m |
| with steel superstructure main members- | 55 m |
- 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, at least 2 m long, shall be attached to the back face of the abutment, sloped to divert surface water from flowing down the abutment/soil interface. The slab shall be deep enough below the road surface at the end remote from the bridge to distribute soil strains due to length changes without significant surface cracking.

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.

4.12 Buried Corrugated Metal Structures

Design of these structures shall be in accordance with the relevant following standards as defined by their statement of scope:

- AS/NZS 2041 *Buried Corrugated Metal Structures*⁽³⁶⁾
 AS 1761 *Helical Lock-Seam Corrugated Steel Pipes*⁽³⁷⁾ and
 AS 1762 *Helical Lock-Seam Corrugated Steel Pipes – Design and Installation*⁽³⁸⁾
 (adopted by Standards New Zealand as NZS 4405 and NZS 4406)
 AS 3703 *Long-Span Corrugated Steel Structures*⁽³⁹⁾,
 Part 1: “Materials and Manufacture”
 Part 2: “Design and Installation”

Consideration shall be given to the effect of possible earthquake induced ground deformation or liquefaction on the structure.

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 corrugated metal structure shall be determined as follows:

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

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

4.13 Miscellaneous Design Requirements

4.13.1 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.

4.13.2 Settlement Slabs

A settlement slab shall be provided at every abutment supporting earth filling. 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. Slabs shall be at least 2 m in length, and the top surface of a slab shall not be above formation level at the end remote from the bridge. The effects set out in 2.5 shall be considered.

4.13.3 Deck Drainage

On spans wholly over water, stormwater may be discharged over the edge of the deck unless there is a particular reason for not doing so. In all other situations (including spans over batter slopes) the stormwater shall be collected and specific provision made for its disposal. Stormwater channels and pipes shall be designed to be self-cleaning where possible, and shall include access for cleaning.

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 20-year return period. Guidance on the design for surface drainage may be obtained from the National Roads Board publication: *Highway Surface Drainage: Design Guide for Highways with a Positive Collection System*⁽⁴⁰⁾, except that more up-to-date sources of information to that referenced should be drawn on for the estimation of rainfall precipitations, such as:

- Tomlinson, A.I., 1980, *The Frequency of High Intensity rainfalls in New Zealand, Part 1*, Water and Soil Technical Publication No. 19, Ministry of Works and Development, Wellington.⁽⁴¹⁾
- Thompson, C.S, 2002, *HIRDS High Intensity Rainfall Design System*, (Version 2), National Institute of Water and Atmospheric Research, Wellington (A computer software system)⁽⁴²⁾
- Databases that may be held by the Regional Council responsible for the locality under consideration.

Detailing shall ensure that water does not leak onto significant visible structural surfaces to cause staining or corrosion, or onto bearings or energy dissipating devices. Drip grooves shall be provided to achieve this where necessary. 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.

Deck expansion joints shall be watertight unless specific provision is made to collect and dispose of the water.

4.13.4 Services

Agreement shall be reached with network utility operators of services, over support conditions required for 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.

Designers shall consider the implications of possible bridge overloading due to leakage or rupture of pipes carrying water or other fluids inside a box girder, and shall provide adequate drainage.

Special approvals and conditions apply to the installation of pipelines carrying flammable fluids (including gas). Refer to Section 6 of *State Highway Policy and Procedure Manual*⁽⁴³⁾. Such pipelines shall not be carried inside box girders.

4.13.5 Date and Loading Panels

All bridges shall have displayed details of date of construction and design live loading.

Each bridge designed to HN-HO-72 loading shall have this information displayed on two panels, as shown in Figure 4.2. The panels shall be of bronze or other approved material of equivalent durability.

The panels shall be located one at each end of the bridge 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 roadway face of concrete barriers, or on the deck behind the line of the guardrail clear of any subsequent sealing work.

Bridges designed to other loadings shall have similar panels.

4.13.6 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

(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 5.4.9.

(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.

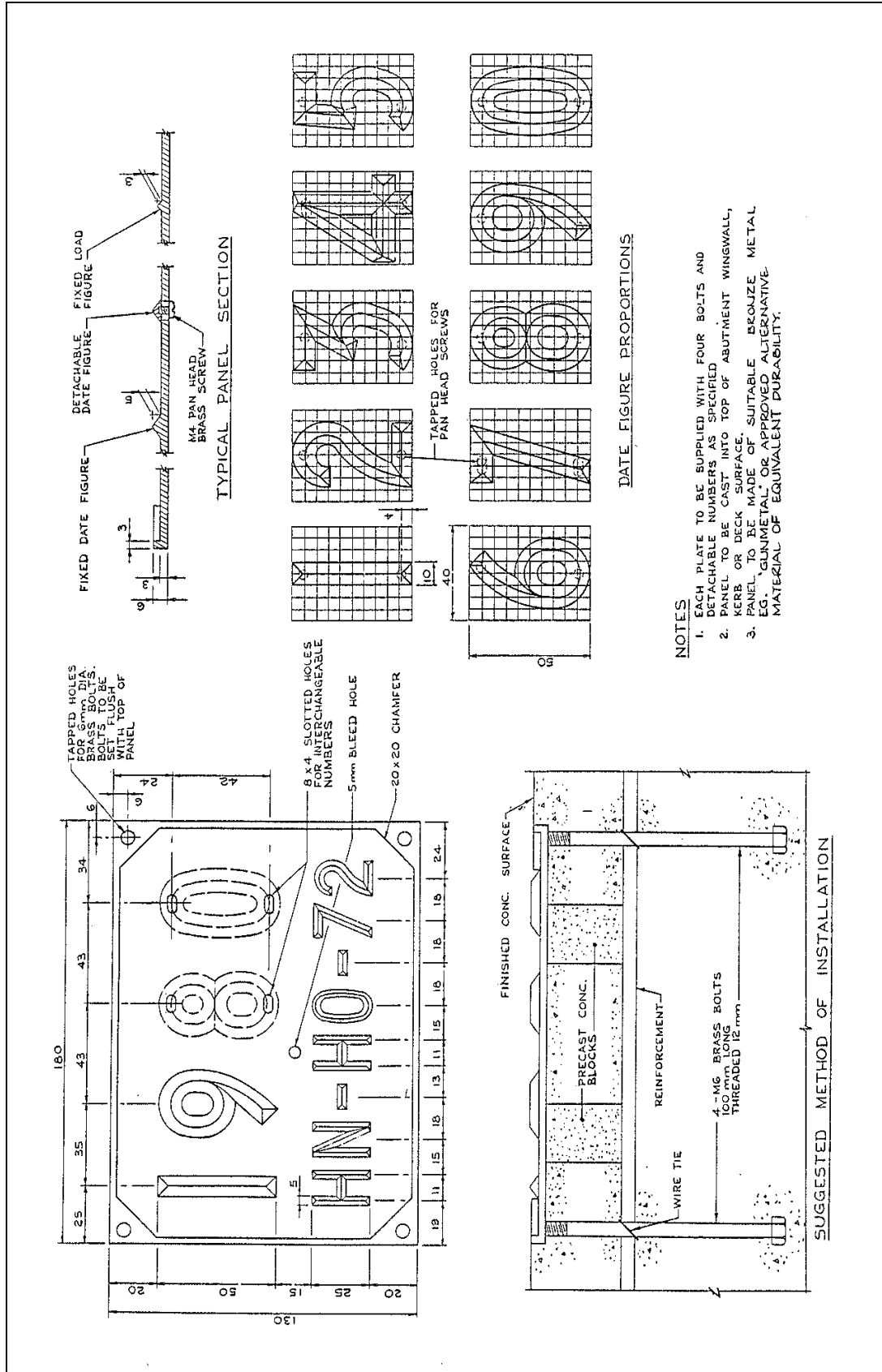


Figure 4.2: Date and Loading Panel: HN-HO-72 Loading

4.14 References

- (1) NZS 3101:1995, *Concrete Structures Standard*, Standards New Zealand.
- (2) NZS 3122:1995, *Specification for Portland and Blended Cements (General and Special Purpose)*, Standards New Zealand.
- (3) AS 3610:1995, *Formwork for Concrete*, Standards Australia.
- (4) AS 5100: 2004, *Bridge Design*,
Part 4: Bearings and Deck Joints
Part 5: Concrete
Part 6: Steel and Composite Construction
Standards Australia.
- (5) NZS 3404:1997, *Steel Structures Standard*, Standards New Zealand.
- (6) BS 5400, _____, *Steel, Concrete and Composite Bridges*
Part 3:1982, “Code of Practice for Design of Steel Bridges”
Part 5:1979, “Code of Practice for Design of Composite Bridges”
Part 10:1980, “Code of Practice for Fatigue”
- (7) NZNSEE, 1985, *Seismic Design of Steel Structures*, Deliberations of the Society's Study Group, Bulletin of New Zealand National Society for Earthquake Engineering, December 1985.
- (8) NZS 3603:1993, *Timber Structures Standard*, Standards New Zealand.
- (9) AS 1720: _____, *Timber Structures*
Part 1: 1997 “Design Methods”
Part 2: 1990 “Timber Properties”
Standards Australia.
- (10) AS 2878-1986, *Timber – Classification into Strength Groups*, Standards Australia.
- (11) AS/NZS 1664.1:1997, *Aluminium Structures*, Part 1: “Limit State Design”, Standards Australia and Standards New Zealand jointly.
- (12) AS 1523:1981, *Elastomeric Bearings for Use in Structures*, Standards Australia.
- (13) DOE(UK), 1976, BE 1/76, *Design Requirements for Elastomeric Bridge Bearings*, Technical Memorandum (Bridges), UK Department of the Environment, Highways Directorate.
- (14) _____, 1994, BD 33/94, *Expansion Joints for Use in Highway Bridge Decks*, UK Highways Agency.
- (15) Bruce, SM, and Kirkcaldie, DK, 2000, *Performance of Deck Expansion Joints in New Zealand Road Bridges*, Research Report No. 186, Transfund New Zealand, Wellington.
- (16) Burke, MP, 1989, *Bridge Deck Expansion Joints*, National Cooperative Highway Research program Synthesis of Highway Practice 141, Transportation Research Board, Washington DC.
- (17) BIA, 1992, *New Zealand Building Code Handbook*, Building Industry Authority, Wellington
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