Compliance Document for New Zealand Building Code Structure Clause B1

Amend 6 March 2005

Prepared by the Department of Building and Housing
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Department of Building and Housing

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Users should make themselves familiar with the preface to the New Zealand Building Code Handbook, which describes the status of Compliance Documents and explains alternative methods of achieving compliance.

Classified uses and defined words italicised in the text are explained in clauses A1 and A2 of the New Zealand Building Code.

Document Status
The most recent version of this document, as detailed in the Document History, is approved by the Department of Building and Housing. It is effective from 1 March 2005 and supersedes all previous versions of this document.

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# Clause B1 Structure

This Clause is extracted from the New Zealand Building Code contained in the First Schedule of the Building Regulations 1992.

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<td><strong>OBJECTIVE</strong></td>
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<td>B1.1 The objective of this provision is to:</td>
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<td>(a) Safeguard people from injury caused by structural failure,</td>
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<td>(b) Safeguard people from loss of amenity caused by structural behaviour,</td>
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<td>(c) Protect other property from physical damage caused by structural failure.</td>
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<td><strong>FUNCTIONAL REQUIREMENT</strong></td>
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<td>B1.2 Buildings, building elements and sitework shall withstand the</td>
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<td>combination of loads that they are likely to experience during construction or alteration and throughout their lives.</td>
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<td><strong>PERFORMANCE</strong></td>
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<td>B1.3.1 Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives.</td>
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<td>B1.3.2 Buildings, building elements and sitework shall have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during construction or alteration when the building is in use.</td>
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<td>B1.3.3 Account shall be taken of all physical conditions likely to affect the stability of buildings, building elements and sitework, including:</td>
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<td>(b) Imposed gravity loads arising from use,</td>
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<td>(c) Temperature,</td>
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<td>(d) Earth pressure,</td>
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<td>(e) Water and other liquids,</td>
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<td>(f) Earthquake,</td>
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<td>(m) Differential movement,</td>
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<td>(n) Vegetation,</td>
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<td>(o) Adverse effects due to insufficient separation from other buildings,</td>
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<td>(p) Influence of equipment, services, non-structural elements and contents,</td>
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<td>(q) Time dependent effects including creep and shrinkage, and</td>
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<td>(r) Removal of support.</td>
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</table>

**B1.3.4** Due allowance shall be made for:

(a) The consequences of failure,
(b) The intended use of the building,
(c) Effects of uncertainties resulting from construction activities, or the sequence in which construction activities occur,
(d) Variation in the properties of materials and the characteristics of the site, and
(e) Accuracy limitations inherent in the methods used to predict the stability of buildings.

**B1.3.5** The demolition of buildings shall be carried out in a way that avoids the likelihood of premature collapse.

**B1.3.6** Sitework, where necessary, shall be carried out to:
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<td>(a) Provide stability for construction on the site, and</td>
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<td>(b) Avoid the likelihood of damage to other property.</td>
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<td><strong>B1.3.7</strong> Any site work and associated supports shall take account of the effects of:</td>
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<td>(a) Changes in ground water level,</td>
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<td>(b) Water, weather and vegetation, and</td>
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<td>(c) Ground loss and slumping.</td>
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For the purposes of New Zealand Building Code compliance, acceptable reference documents include only the quoted edition and specific amendments as listed below.

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Definitions

This is an abbreviated list of definitions for words or terms particularly relevant to this Approved Document. The definitions for any other italicised words may be found in the New Zealand Building Code Handbook.

**Adequate**
Adequate to achieve the objectives of the building code.

**Alter**
in relation to a building, includes to rebuild, re-erect, repair, enlarge and extend; and **alteration** has a corresponding meaning.

**Baluster**
A post providing the support for the top and bottom rails of a barrier.

**Boundary joist**
A joist running along the outer ends of the floor joists.

**Building**
has the meaning ascribed to it by the Building Act 1991.

**Building element**
Any structural and non-structural component or assembly incorporated into or associated with a building. Included are fixtures, services, drains, permanent mechanical installations for access, glazing, partitions, ceilings and temporary supports.

**Chimney**
A non-combustible structure which encloses one or more flues, fireplaces or other heating appliances.

**Chimney back**
The non-combustible wall forming the back of a fireplace.

**Chimney base**
That part of a chimney which houses the fireplace.

**Chimney jamb**
The side walls of a fireplace.

**Combustible**
See non-combustible.

**Construct**
in relation to a building, includes to build, erect, prefabricate, and relocate; and **construction** has a corresponding meaning.

**Drain**
A pipe normally laid below ground level including fittings and equipment and intended to convey foul water or surface water to an outfall.

**Factor of safety**
in relation to any building means the ratio of resisting forces to applied forces for a given loading condition. It is generally expressed to two significant figures.

**Fireplace**
A space formed by the chimney back, the chimney jamb, and the chimney breast in which fuel is burned for the purpose of heating the room into which it opens.

**Fixture**
An article intended to remain permanently attached to and form part of a building.

**Flue**
The passage through which the products of combustion are conveyed to the outside.

**Gather**
That part of a chimney where the transition from fireplace to stack occurs.

**Good ground**
means any soil or rock capable of permanently withstanding an ultimate bearing pressure of 300 kPa (i.e. an allowable bearing pressure of 100 kPa using a factor of safety of 3.0), but excludes:

a) Potentially compressible ground such as topsoil, soft soils such as clay which can be moulded easily in the fingers, and uncompacted loose gravel which contains obvious voids,

b) Expansive soils being those that have a liquid limit of more than 50% when tested in accordance with NZS 4402 Test 2.2, and a linear shrinkage of more than 15% when tested, from the liquid limit, in accordance with NZS 4402 Test 2.6, and

c) Any ground which could forseeably experience movement of 25 mm or greater for any reason including one or a combination of:
land instability, ground creep, subsidence, seasonal swelling and shrinking, frost heave, changing ground water level, erosion, dissolution of soil in water, and effects of tree roots.

**COMMENT:**
Soils (excepting those described in a), b) and c) above) tested with a dynamic cone penetrometer in accordance with NZS 4402 Test 6.5.2, shall be acceptable as good ground for building foundations if penetration resistance is no less than:

a) 3 blows per 75 mm at depths no greater than the footing width.
b) 2 blows per 75 mm at depths greater than the footing width.

Depths shall be measured from the underside of the proposed footing.

**Hearth** The insulating floor under the fire and in front and at the sides of the fireplace.

**Intended use** of a building includes:

a) Any reasonably foreseeable occasional other use that is not incompatible with the intended use; and

b) Normal maintenance; and

c) Activities taken in response to fire or any other reasonably foreseeable emergency - but does not include any other maintenance and repairs or rebuilding.

**Nominal pile width** The least width of a pile in side view and is equal to the diameter in round piles.

**Non-combustible** Materials shall be classified as non-combustible or combustible when tested to:

AS 1530 – Part 1.

**Other property** means any land or buildings or part thereof which are:

a) Not held under the same allotment; or

b) Not held under the same ownership – and includes any road.

**Sitework** means work on a building site, including earthworks, preparatory to or associated with the construction, alteration, demolition or removal of a building.

**Specified intended life** has the meaning ascribed to it by section 39 of the Act as follows: “specified intended life” in relation to a proposed building, or any existing building proposed to be altered, and which is intended to have a use of not more than 50 years, means the period of time, as stated in an application for a building consent or in the consent itself, for which the building is proposed to be used for its intended use.

**Strength reduction factor** The factor by which the ultimate strength is multiplied to obtain the design strength.

**COMMENT:**

NZS 4203: 1992 uses the terms ideal strength in place of ultimate strength, and dependable strength in place of design strength.

**Surface water** All naturally occurring water, other than sub-surface water, which results from rainfall on the site or water flowing onto the site, including that flowing from a drain, stream, river, lake or sea.

**Territorial authority** has the meaning ascribed to it by section 2 of the Local Government Act 1974; and includes any organisation which is authorised to permit structures pursuant to section 12(1)(b) of the Resource Management Act 1991.
Verification Method B1/VM1

General
(Revised by Amendment 4)

1.0 Explanatory Note

1.0.1 This part of the Approved Document lists under category headings other Approved Documents and Standards suitable as methods of verification.

1.0.2 Elsewhere in this Approved Document a verification method is given for foundations. It is referred to in Paragraph 9.0.

1.0.3 Modifications to the Standards, necessary for compliance with the New Zealand Building Code, are given against the relevant clause number of each Standard.

1.1 Loadings of NZS 4203 to be used

1.1.1 For compliance with this verification method, compatible loadings and material standards shall be used.

1.1.2 Loadings from NZS 4203: 1992 shall be used where materials standards or documents listed below are written in limit state format.

1.1.3 NZS 4203: 1984 is retained as a reference document for use with those material standards which are not written in limit state format.

1.1.4 Neither of the NZS 4203 Standards addresses the problem of localised site effects such as enhanced earthquake ground motions due to unfavourable ground conditions or proximity to a fault. Where these are identified they shall be the subject of a special study.

2.0 Loadings

2.1 NZS 4203: 1984

2.2 NZS 4203: 1992 Volumes 1 and 2 subject to the following modifications:

Clause 6.6.1 Reword to read:

“A load factor of 1.6 shall be applied to lateral loads on earth retaining structures except for cases involving earthquake loading where the load factor shall be 1.0.

On earth retaining structures dead loads which promote stability shall be unfactored (i.e. load factor of unity).”

Clauses 6.6.3 and 6.6.4 Delete.

Clause C6.6 Delete.

3.0 Concrete

3.1 NZS 3101: Part 1 subject to the following modifications:

a) The word “may” where used to describe provisions of the Standard shall be taken to read “is permitted”.

b) The use of this Standard as a verification method does not extend to the use of any “other appropriate (or approved) loadings standard”. Further where this Standard has provisions that are in non-specific or unquantified terms (such as where provisions are required to be appropriate, adequate, suitable, relevant, satisfactory, acceptable, applicable or the like), then these also do not form part of the verification method and must be treated as an alternative solution.

c) Normative appendices A and B shall be read as an integral part of the text which means that they do not have the status of recommendation but are required to be followed. Accordingly, all references in these appendices to “recommended” (or “suggested”) are to be read as “required” and references to “should” are to be read as “shall”.

d) The word “should” is to be read as “shall” in Notes 1 and 2 of Table 5.1, and Clauses 10.3.1.2, 14.4.6 and 16.3.16.

e) In Clause 7.3.1.1 delete the words “unless there is a special reason for using plain bars”.

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3.1.2 Some provisions of NZS 3101 may exceed the requirements of the NZBC.

3.2 NZS 3106

4.0 Masonry

4.1 NZS 4230: Parts 1 and 2

5.0 Steel

5.1 NZS 3404: Part 1

5.2 AS/NZS 4600 subject to the following modifications:

a) Where the Standard refers to the “relevant loadings Standard” or the “appropriate limit state loading Standard” or other similar statements this shall be read as a reference to NZS 4203: 1992.

b) The term “normative” identifies a mandatory requirement for compliance with this Standard.

c) The term “informative” identifies information provided for guidance or background which may be of interest to the Standard’s users. Informative provisions do not form part of the mandatory requirements of the Standard.

d) Where this Standard has provisions that are in non-specific or unquantified terms then these do not form part of the verification method and the proposed details must be submitted to the territorial authority for approval as part of the building consent application. This includes, but is not limited to, special studies and manufacturer’s advice.

e) All stages of construction of a structure or part of a structure to which this Standard is applied shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.

f) The extent of the review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

g) At the end of the first paragraph of Appendix A add the words “Unless noted otherwise a document referred to below shall be the version of that document current at the date of issue of this Standard or if amendments are cited to this Standard in the “References” pages of Approved Document B1 at the latest date of those amendments.”

h) Appendix B shall be read as normative with “shoulds” changed to “shalls”.

6.0 Timber

6.1 NZS 3603 subject to the following modifications:

Clause 1.6 Reword to read:

“All stages of construction of a structure or part of a structure to which this Standard is applied shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.”

7.0 Aluminium

7.1 AS/NZS 1664.1 subject to the following modifications:

a) Design loadings must be in accordance with NZS 4203: 1992.

b) The terms “capacity factor” and “strength limit state” are to be read as “strength reduction factor” and “ultimate limit state” respectively.

c) Where this Standard has provisions that are in non-specific or unquantified terms then these do not form part of the verification method and the proposed details must be submitted to the territorial authority for approval as part of the building consent application. This includes, but is not limited to, special studies and manufacturer’s advice.
d) All stages of construction of a structure or part of a structure to which this Standard is applied shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.

e) The extent of the review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

f) Clause 1.2 to read “MATERIALS This Standard applies to aluminium alloys listed in Table 3.3(A) that comply with AS 1734, AS 1865, AS 1866, AS 1867 and AS 2748.1.”

g) At the end of the first paragraph of Clause 1.4 add the words “Unless noted otherwise a document referred to below shall be the version of that document current at the date of issue of this Standard or if amendments are cited to this Standard in the “References” pages of Approved Document B1 at the latest date of those amendments.”

7.2 AS/NZS 1664.2 subject to the following modifications:

a) Design loadings must be in accordance with NZS 4203: 1984.

b) The Standard applies to building type structures (see Clause 1.3.1) and bridge type structures (see Clause 1.3.2) only.

c) Where this Standard has provisions that are in non-specific or unquantified terms then these do not form part of the verification method and the proposed details must be submitted to the territorial authority for approval as part of the building consent application. This includes, but is not limited to, determination of type of structure (i.e. building type, bridge type or other), factors of safety other than those specified, special studies and manufacturer's advice.

d) All stages of construction of a structure or part of a structure to which this Standard is applied shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.

e) The extent of the review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

f) Clause 1.2 to read “MATERIALS This Standard applies to aluminium alloys listed in Table 3.3(A) that comply with AS 1734, AS 1865, AS 1866, AS 1867 and AS 2748.1.”

g) At the end of the first paragraph of Clause 1.4 add the words “Unless noted otherwise a document referred to below shall be the version of that document current at the date of issue of this Standard or if amendments are cited to this Standard in the “References” pages of Approved Document B1 at the latest date of those amendments.”

8.0 Earth Buildings

8.1 NZS 4297

9.0 Foundations

See B1/VM4 of this Approved Document.

10.0 Siteworks

10.1 NZS 4431
11.0 Drains

11.1 NZS/AS 3725 subject to the following modifications:

Clause 1(b) After the words “AS 1342 and AS 1392” add “or NZS 3107”.

Clause 3 Add to the list of reference documents:

“NZS 3101 The design of concrete structures.

NZS 3107 Specification for precast concrete drainage and pressure pipes.

NZS 4402 Methods of testing soils for civil engineering purposes:

Tests 2.4, 2.8, 4.1.1, 4.2.1, 4.2.2, 4.2.3 and 5.1.1.


New Zealand Geomechanics Society, Guidelines for the field description of soils and rocks in engineering use.”

Clause 4 In the paragraph headed “Bedding factor (F)”, after the words “AS 1342 and AS 1392” add “or NZS 3107”.

In the paragraph headed “(c) Select fill”, after the words “Appendix D of AS 1726” add “or the New Zealand Geomechanics Society guidelines”.

In the paragraph headed “Test load”, after the words “AS 1342” add “or NZS 3107”.

Clause 5 In definition of $P_o$, after the words “AS 1392” add “or NZS 3107”.

Clause 6.4 Replace the word “may” with “shall”.

Delete the words “Superimposed concentrated dead loads should be avoided.”

Clause 6.5.2.1 Delete the words “Unless otherwise specified by the relevant Regulatory Authority”.

Clause 6.5.2.2 In the last paragraph replace the words “may not apply” with “shall be reassessed”.

Clause 6.5.2.3 Add new text after note 3 to read:

“Alternatively, vehicle loads may be taken as HN-HO-72 loading as specified in the Transit New Zealand Bridge Manual for Design and Evaluation. The average live load intensity (q) due to those vehicles and their impact effects shall be calculated in accordance with Clause 6.5.2.2.

Notes:

1. For depths of cover less than 0.6 m, the wheel loads shall be considered to act directly on the pipe. However the length of pipe supporting the load shall be taken as not greater than Le (see Clause 6.5.2.4).

2. For single pipes, the effect of HN-HO-72 wheel loads may be neglected when the depth of cover (H) is greater than 2.4 m, and exceeds the pipe diameter (D).”

Clause 6.5.3 In the first paragraph delete the words “Unless otherwise specified by the relevant Regulatory Authority”.

In the last paragraph delete the words “unless specifically approved by the relevant Railway Authority”.

Clause 6.5.4 Replace the words “shall be obtained from the relevant Regulatory Authority” with “are not covered by this document”.

Clause 7 Replace the word “should” with “shall”.

Clause 8 Rerword 8(a) and (b) to read:

“a) Cohesive soils. The dry density ratio (RD) shall be determined either:

i) in accordance with AS 1289 E4.1, based on the field dry density in accordance with AS 1289 E3.2 and the maximum dry density in accordance with AS 1289 E1.1, or

ii) as the ratio of the field dry density in accordance with NZS 4402 Test 5.1 and the maximum dry density in accordance with NZS 4402 Test 4.1.1, expressed as a percentage.

b) Cohesionless soils. The density index (ID) shall be determined either:

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BUILDING INDUSTRY AUTHORITY
i) in accordance with AS 1289 E6.1, based on the maximum and minimum dry densities in accordance with AS 1289 E5.1 and the field dry density in accordance with AS 1289 E3.2 or AS 1289 E3.5, or

ii) as the relative density in accordance with NZS 4402 Test 4.2.3.”

Clause 9.2.2.2 Reword first paragraph to read:

“9.2.2.2 Types H1 and H2. For support types H1 and H2, select fill in both the bed and the haunch zones and which is not cement stabilized, shall have a particle size distribution, determined in accordance with either:

i) AS 1289 C6.1, preferably falling within the limits given in Table 3 with the fraction passing the 0.075 mm sieve being material of low plasticity, as defined in Appendix D of AS 1726, or

ii) NZS 4402 Test 2.8.1, falling within the limits given in Table 3 with the fraction passing the 0.075 mm sieve having a plasticity index of less than 15 as determined in accordance with NZS 4402 Test 2.4.

The fill shall be placed and compacted to the appropriate depths and densities given in Clause 9.3.”

Clause 9.2.2.3 After the words “AS 3600” add “or NZS 3101”.

Reword second paragraph to read:

“Vertical transverse construction joints, where required in the support, shall coincide with pipe joints. Between these joints the cross-section shall be cast in one operation, or if this cannot be achieved, shear keys shall be provided in any horizontal construction joint to ensure monolithic action of the cradle cross-section.”

Clause 9.2.3.2 After the words “AS 1289 C6.1” add “or NZS 4402 Test 2.8.1”.

Clause 10.1 After the words “AS 1342 or AS 1392, as appropriate” add “or NZS 3107”.

Clause 10.2(a) After the word “(Tc)” add “or proof load”.

Clause 10.2(b) Reword to read:

“The test ultimate load (Tu) or ultimate load shall be obtained from AS 1342 or NZS 3107 respectively, for the appropriate pipe size. A load class shall be determined from either AS 1342 or NZS 3107 corresponding to the value of Tc determined from (a) above.”

Clause 10.3 After the words “the test load” add “or proof load”.

Clause 10.4 After the words “(Tc)" add “or proof load”.

After the words “AS 1392” add “or NZS 3107”.

12.0 Windows

12.1 NZS 4211

References to air leakage, water leakage and operational effectiveness of opening sashes in NZS 4211, are non-structural considerations and do not apply to this Approved Document. The more substantial of such references are listed in Amendment No 2, July 1992.

13.0 Farm Buildings

13.1 NZS 1900: Division 11.2

14.0 Seismic Resistance of Building Services

14.1 NZS 4219
Acceptable Solution B1/AS1

General (Revised by Amendment 4)

1.0 Explanatory Note

1.1 This part of the Approved Document lists under category headings other Approved Documents and Standards, suitable as acceptable solutions.

1.2 In other parts of this Approved Document acceptable solutions are given for small chimneys and timber barriers. These are referred to in Paragraphs 8.0 and 9.0 respectively.

1.3 Modifications to the Standards, necessary for compliance with the New Zealand Building Code, are given against the relevant clause number of each Standard.

2.0 Masonry

2.1 NZS 4229

3.0 Timber

3.1 NZS 3604

4.0 Earth Buildings

4.1 NZS 4299

5.0 Stucco

5.1 NZS 4251

6.0 Drains

6.1 NZS 4452

6.2 NZS 7643 subject to the following modifications:

Clause 1.1 Add a sentence at the end with the words “Pipes below ground shall be laid in narrow trenches as defined in clause 1.2”.

Clause 1.2 Add new clause:

1.2 Definition

NARROW TRENCH means a trench with sides having a slope no flatter than 1 horizontal to 4 vertical and having dimensions of either:

a) $B \leq 2.0D$ and $H > 1.5B$, or
b) $2.0D < B < 3.0D$ and $H > 3.5B$

where

$B =$ trench width at the top of the pipe, but not greater than 600 mm.

$H =$ depth of cover over the pipe.

$D =$ outside diameter of the pipe.

The bottom width of the trench shall be $D + 200$ mm or $2D$, whichever is the lesser.”

Clause 5.1.1 Reword to read:

“5.1.1 This section sets out the rules for laying uPVC pressure pipelines underground in a narrow trench.”

<table>
<thead>
<tr>
<th>Location</th>
<th>Types A, B, C, and D</th>
<th>Type F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roads and streets</td>
<td>750</td>
<td>750</td>
</tr>
<tr>
<td>Residential driveways and similar areas rarely subject to heavy traffic</td>
<td>600</td>
<td>375</td>
</tr>
<tr>
<td>Footpaths, gardens and open country*</td>
<td>500</td>
<td>375</td>
</tr>
</tbody>
</table>

* Protected as required against mechanical damage from agricultural or similar operations
Clause 5.4 Reword heading to read:

“5.4 Pipe installation in a narrow trench.”

Clause 5.4.1 Reword text to read:

“5.4.1 Trench construction. Pipes shall be laid in narrow trenches that comply with the definition given in Clause 1.2. The bedding, surround and backfilling details shall comply with Type A, B, C, D or F as given in Appendix D.”

Tables 2 and 3. Amend as shown on previous page.

Clause 6.5.2 Delete.

Clause 6.5.3 Replace the words “Where permitted by the Engineer” with the words “Subject to specific design”.

Clause 6.13 Delete the words “Type F bedding shall be used where pipe is laid at depths less than those shown in Table 3”.

Appendix D Amend as follows:

Reword heading to read “Construction details for pipes laid in narrow trenches (see Clause 1.2)”.

Delete the dimension “D + 300” from the bottom of all figures and replace with “The lesser of 2D or D + 200 mm”.

8.0 Small Chimneys

See B1/AS3 of this Approved Document.

9.0 Timber Barriers

See B1/AS2 of this Approved Document.

7.0 Glazing

7.1 NZS 4223 subject to the following modifications:

Clause 1.1 Reword to read:

“This document provides an acceptable solution for the sizing of glass to carry loads resulting from wind and human impact. Part 3 contains the provisions for human impact and is subject to the modifications given in F2/AS1.

Clauses 104 onwards deal with glazing materials and systems, which are to be used with glass sized as above.”
Verification Method B1/VM2

Timber Barriers

No specific test methods have been adopted for verifying compliance of timber barriers with NZBC Performance B1.
1.0 Scope

1.0.1 This document applies to the construction of timber barriers for Housing. The solution described will have an expected life of 15 years.

COMMENT:
The 15 year life is based on the life expectancy of the nail plates and the circular tooth plate connectors described in this acceptable solution. These connectors can be fabricated from thicker steel, have different coatings or be made from other materials such as stainless steel, to give the barrier a greater life. If, however, a greater life is claimed it must be substantiated by supporting information.

1.0.2 Barriers complying with this document, satisfy NZBC F4 for the protection of children under the age of six years.

1.0.3 Timber used in the construction of the timber barriers shall be No. 1 framing grade Radiata Pine as specified in NZS 3631.

1.0.4 Cross-sectional dimensions of timber given in this document are call dimensions as specified in NZS 3601, unless otherwise noted.

COMMENT:
Actual timber dimensions will vary according to moisture content and level of finish e.g. roughsawn, gauged or dressed.

1.0.5 Barriers exposed to the weather shall have:

a) All timber treated to at least hazard class H3 in accordance with NZMP 3640.

b) Mild steel fixings hot-dip galvanised as specified in NZS/AS 1650 for nails, and AS 1214 for bolts and coach screws.

Comment:
Circular toothed plate connectors and nail plates, where required by this document, that are formed from 1.0 mm thick sheet mild steel with a zinc coating of at least 275 g/m² in accordance with NZS 3441.

2.0 Construction

2.1 General

2.1.1 Barriers shall comprise balusters, top and bottom rails, and palings. The supporting floor shall have joists no less in size than 125 mm x 50 mm. Where a boundary joist or blocking is used it shall have the same cross section as the joists.

2.2 Top rails

2.2.1 Top rails shall be Type 1 or Type 2 as shown in Figure 1. The size of the top rail depends on its span between balusters and is given in Table 1.

2.2.2 Top rails shall be fixed to each baluster with nails having a shank diameter of no less than 3.75 mm and penetrating the baluster by no less than 50 mm. The number of nails for each fixing shall be:

2 for spans up to and including 1400 mm
3 for spans greater than 1400 mm

2.3 Balusters

2.3.1 Baluster size depends on the type and span of the top rail and shall be as given in Table 2.

<table>
<thead>
<tr>
<th>Table 1: Top Rail Sizes – Type 1 and Type 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum spacing (c-c) of balusters for top rail (mm)</td>
</tr>
<tr>
<td>---------------------------------------------</td>
</tr>
<tr>
<td><strong>Type 1:</strong></td>
</tr>
<tr>
<td>1000</td>
</tr>
<tr>
<td>1500</td>
</tr>
<tr>
<td>1800</td>
</tr>
<tr>
<td>2000</td>
</tr>
<tr>
<td><strong>Type 2:</strong></td>
</tr>
<tr>
<td>1000</td>
</tr>
<tr>
<td>1400</td>
</tr>
<tr>
<td>1500</td>
</tr>
<tr>
<td>1600</td>
</tr>
<tr>
<td>1700</td>
</tr>
<tr>
<td>1800</td>
</tr>
</tbody>
</table>
**Figure 1:** Top Rail Types
Paragraph 2.2.1

<table>
<thead>
<tr>
<th>Type 1</th>
<th>Type 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top rail</td>
<td>Top rail</td>
</tr>
<tr>
<td>Refer Table 1 for size</td>
<td>Refer Table 1 for size</td>
</tr>
<tr>
<td>Baluster</td>
<td>Baluster</td>
</tr>
</tbody>
</table>

**Figure 2:** Baluster Sizes
Paragraph 2.3.1 and Figures 3 and 4

<table>
<thead>
<tr>
<th>Baluster size (mm)</th>
<th>Maximum baluster spacing (c-c) for top rail (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 1</td>
</tr>
<tr>
<td>50 x 100</td>
<td>400</td>
</tr>
<tr>
<td>75 x 50</td>
<td>600</td>
</tr>
<tr>
<td>75 x 75</td>
<td>800</td>
</tr>
<tr>
<td>100 x 50</td>
<td>1200</td>
</tr>
<tr>
<td>75 x 100</td>
<td>1200</td>
</tr>
<tr>
<td>100 x 75</td>
<td>1800</td>
</tr>
<tr>
<td>100 x 100</td>
<td>2000</td>
</tr>
</tbody>
</table>

Note:
The first dimension given for the baluster is the depth measured perpendicular to the line of the top rail as shown in Figures 3 and 4.
2.3.2 Balusters can be fixed to an intermediate joist, end joist or to a boundary joist. The three situations are shown in Figure 2, and the corresponding fixing details are shown in Figures 3 and 4.

2.3.3 Where a baluster is fixed to an intermediate joist, its fixings depends on the baluster spacing and the depth of the joist, and shall comprise either of M12 bolts or M12 bolts with double sided circular toothed plate connectors, as determined from Table 3.

2.3.4 Where a baluster is fixed to an end joist or a boundary joist, it shall be fixed with two M12 bolts in all situations, and the baluster shall be no further than 85 mm from the required blocking or adjacent joist as shown in Figure 4.

2.3.5 Boundary joists shall be fixed to each joist adjacent to a baluster with two 12 mm diameter coach screws, and with two nail plates (one at the top and one at the bottom) as shown in Figure 4. Coach screws shall have a minimum penetration of 100 mm and shall have a 50 x 50 x 3 mm washer. Nail plates shall comply with Paragraph 1.0.5 c) and shall be capable of carrying a tension force of 4.0 kN (capacity load as defined in NZS 3604). The boundary joist shall be fixed to all other joists with three nails of 3.75 mm shank diameter, penetrating 50 mm into the joist.

See Comment page 28.
Figure 3: Baluster Fixed to Intermediate Joist
Paragraphs 2.3.2, 2.4.1 and 2.5.3

NOTE:
See Table 4 for edge and end distances a, b and c.
Figure 4: Baluster Fixed to Boundary Joist or End Joist
Paragraphs 2.3.2, 2.3.4, 2.3.5, 2.3.6, and 2.4.1

(a) Plan
Baluster fixed to boundary joist

(b) Plan
Baluster fixed to end joist

Section A-A
Showing baluster fixing

Section B-B
Showing boundary joist or end joist fixing

NOTE:
Circular toothed plate connectors (CTP) are not required.
1. The nail plates may be of any type provided the above requirements are met. Acceptable types include plates with hammer-down claws or plates with pre-drilled holes for subsequent nailing.

2. The coach screws may be replaced with any fixing(s) of equal durability and capable of carrying a shear load of 4 kN (capacity load as defined in NZS 3604).

2.3.6 The end joist blocking required in Paragraph 2.3.4 shall be fixed with two 12 mm diameter coach screws, and with two nail plates (one at the top and one at the bottom) as shown in Figure 4. Coach screws shall have a minimum penetration of 100 mm and shall have a 50 x 50 x 3 mm washer. Nail plates shall comply with Paragraph 1.0.5 c) and shall be capable of carrying a tension force of 4.0 kN (capacity load defined in NZS 3604).

2.3.7 End and edge distances for bolts and for bolts with circular toothed plate connectors are given in Table 4.

<table>
<thead>
<tr>
<th>Baluster spacing (c-c) (mm)</th>
<th>125 x 50</th>
<th>150 x 50</th>
<th>200 x 50</th>
<th>225 x 50</th>
<th>250 x 50</th>
<th>300 x 50</th>
<th>350 x 50</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>B A A A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>B A A A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>B A A A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>B B A A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>B B B A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>B B B A A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1200</td>
<td>C B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1350</td>
<td>B B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1400</td>
<td>C B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1500</td>
<td>C B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1600</td>
<td>C B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1700</td>
<td>C B B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1800</td>
<td>C C B B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2000</td>
<td>C C C B A A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Connection types:
- A – Two M12 bolts
- B – Two M12 bolts with 50 mm double sided circular toothed plate connectors
- C – Two M12 bolts with 63 mm double sided circular toothed plate connectors

Note: All bolts shall have 50 x 50 x 3 mm washers each end.
2.3.8 Coach screws shall be fixed in predrilled holes located centrally in the width of the floor joist or blocking. The diameter of the predrilled holes shall be:

a) Over the shank length, no less than the shank diameter and no more than the shank diameter plus 1.5 mm, and

b) Over the threaded length, no more than the root diameter of the screw.

2.3.9 The depth of the hole drilled to the diameter for the threaded length shall be at least two diameters greater than the intended depth of the screw. Coach screws shall not be hammered into place but shall be turned with a wrench.

2.4 Bottom rails

2.4.1 Bottom rails (see Figures 3 and 4) shall span between balusters and be sized and fixed in accordance with Table 5.

2.5 Palings

2.5.1 Palings shall comprise 25 mm thick timber spanning between the top and bottom rails. Palings shall have a minimum width of 65 mm and be installed with a gap between palings of no more than 100 mm.

COMMENT:
Lightweight infills other than timber palings may be acceptable. The infill would require specific design.

2.5.2 Palings shall be fixed to the top and bottom rails with:

a) Two 60 mm x 2.8 mm shank diameter nails if the paling is 200 mm wide or less.

b) 60 mm x 2.8 mm shank diameter nails at 150 mm centres if the palings are wider than 200 mm.

2.5.3 With a Type 1 top rail the palings shall be fixed to the top rail using a 50 mm x 50 mm batten as shown in Figure 3. The batten shall be fixed to the top rail with 75 mm x 3.15 mm shank diameter nails at 300 mm centres.

2.6 Timber moisture content

2.6.1 Timber in balusters and elements to which they are connected should be installed at similar moisture contents. The moisture content shall not exceed 20%.
2.7 Alternative details

2.7.1 Where Figures in this acceptable solution show rails and top battens fixed to the outer face of balusters, it is also acceptable for them to be fixed to the inner face. It is also acceptable for the bottom rail and top batten to be cut between balusters.

2.7.2 Palings may be nailed to either side of the rails and top battens.
Verification Method B1/VM3

Small Chimneys

No specific test methods have been adopted for verifying compliance of small chimneys with NZBC Performance B1.
Acceptable Solution B1/AS3
Small Chimneys

Scope
This acceptable solution applies to small chimneys and to supporting hearth slabs for solid fuel burning domestic appliances. It is to be read in conjunction with the Approved Document for NZBC C1 which has additional requirements to prevent outbreak of fire.

1.0 Chimney Construction

1.1 General

1.1.1 Type
The acceptable solutions described in this document are for chimneys built of brickwork, concrete or precast pumice concrete, that are connected to timber frame or masonry buildings complying with NZS 3604 or NZS 4229.

1.1.2 Height
The height of any chimney measured from the top of the chimney foundation slab to the top of the chimney stack shall not exceed 9 m. Chimneys shall not cantilever more than 2.4 m above the fixing at roof level (refer Paragraph 1.7).

1.1.3 Size
The width (measured along the building line) and depth (measured perpendicular to the building line) shall not exceed:

- For the foundation and chimney base
  - precast pumice concrete 1600 mm wide x 1050 mm deep
  - brickwork or concrete 1200 mm wide x 1050 mm deep

- For a brick chimney stack
  - single skin (see Figure 2) 500 mm wide x 500 mm deep
  - double skin (see Figure 3) 1200 mm wide x 680 mm deep

- For a concrete or precast pumice concrete chimney stack
  - 1200 mm wide x 700 mm deep

1.1.4 Chimney liners
Where chimney liners are used they are to be separated from the chimney to ensure free thermal movement. This shall be achieved by coating the liner with a suitable debonding agent or by wrapping it in a combustible material no less than 0.25 mm thick.

1.2 Chimney wall thickness

1.2.1 Chimney wall thicknesses shall be no less than:

- Brick
  - single skin (see Figure 2) 155 mm
  - double skin (see Figure 3) 245 mm

- Concrete 170 mm

- Precast pumice concrete 85 mm

These thicknesses apply to the chimney stack, gather and chimney base.

1.3 Foundations

1.3.1 Chimneys shall be built on a foundation comprising walls and slab for suspended floors (see Figure 1(a)), or on a thickened slab for floor slabs on ground (see Figure 1(b)).

1.3.2 The chimney foundation slab shall be constructed in reinforced concrete, founded on good ground, and have:

- A thickness of no less than 200 mm, and be placed at a depth of no less than 300 mm below surrounding ground level.
- Reinforcement as shown in Figure 1.
- D12 starters at 400 mm maximum centres, to match vertical steel locations in the chimney.

1.3.3 The chimney foundation walls shall be 150 mm thick reinforced concrete, 190 mm thick concrete masonry, or brick construction complying with Figures 2 or 3. Vertical and horizontal reinforcing steel shall be as given in Paragraph 1.6.
Figure 1: Chimney Foundation
Paragraphs 1.3.1, 1.3.2 b) and 1.4.1, and Figures 2, 3, 4 and 5

- Outline of fireplace
- 75mm hearth slab
- D10 @ 225mm ctrs. each way
- Building line
- Timber floor (Double joists each side of hearth slab)
- D12 at 400mm ctrs. 300mm legs.
- R6 U bars at 200mm ctrs. in walls.
- 300mm into ground
- Chimney foundation slab on good ground.
- Chimney foundation slab. D12 at 300mm ctrs. each way or 661 mesh top and bottom.
- 300mm leg

(a) Suspended floor

- Chimney base
- Outline of fireplace and hearth
- See Note
- 400mm lap
- Ground floor slab
- Four D12 @ 400mm ctrs. (300mm leg) Lap 400mm with reinforcement in ground floor slab.
- Chimney foundation slab on good ground

(b) Slab on ground

NOTE:
Figures 1(a) and 1(b) have been drawn for the chimney being exterior to the building. The dotted line indicates the timber floor in Figure 1(a) and the ground floor slab in Figure 1(b) for the situation where the chimney is internal.
Figure 2: Brick Chimney with Liner

Paragraphs 1.1.3 b), 1.2.1 a), 1.3.3, 1.6.1, 1.7.2, 1.7.5 and 1.7.6

NOTES:

Figure 2 has been drawn for the chimney being exterior to the building. The dotted lines indicate the ceiling joists and floor for the situation where the chimney is internal.

Flues must be sized to ensure adequate draught. Sizing depends on many factors including the height of the flue but generally flue cross-sectional area needs to be a minimum of 1/12th of the fireplace opening. The flue also needs to be sufficiently large to enable cleaning. 195mm is regarded as a practical minimum diameter.
NOTES:
1. Figure 3 has been drawn for the chimney being exterior to the building. The dotted lines indicate the ceiling joists and floor for the situation where the chimney is internal.
2. Flues must be sized to ensure adequate draught. Sizing depends on many factors including the height of the flue but generally flue cross sectional area needs to be a minimum of 1/12th of the fireplace opening. The flue also needs to be sufficiently large to enable cleaning. 175 x 175 mm is regarded as a practical minimum size.
1.4 *Hearths*

1.4.1 Hearth slabs shall be of concrete no less than 75 mm thick, reinforced with D10 bars located centrally at 225 mm centres each way. See Figure 1.

1.5 *Chimney breasts*

1.5.1 The widths of openings in chimney breasts, and their supporting lintels, shall comply with Table 1.

### Table 1: Chimney Breast Openings and Lintels

<table>
<thead>
<tr>
<th>Opening width</th>
<th>Lintel reinforcing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick</td>
<td></td>
</tr>
<tr>
<td>1.0 m maximum</td>
<td>65 x 10 mm m.s. flat or 80 x 60 x 5 mm m.s. angle</td>
</tr>
<tr>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Up to 900 mm</td>
<td>Two D10 rods</td>
</tr>
<tr>
<td>900 – 1500 mm</td>
<td>D12 upper rod</td>
</tr>
<tr>
<td></td>
<td>D16 lower rod</td>
</tr>
<tr>
<td>Precast pumice</td>
<td></td>
</tr>
<tr>
<td>1.0 m maximum</td>
<td>Two D10 rods</td>
</tr>
</tbody>
</table>

Note: Horizontal reinforcing rods to concrete and precast pumice are to be placed one above the other at a spacing of 75 mm, and have R6 ties at 150 mm maximum centres.

1.6 *Reinforcing*

1.6.1 Reinforcing of foundation walls, chimney bases (including the gathers) and chimney stacks (see Figures 2 to 5 inclusive) shall comprise:

a) D12 bars at 400 mm maximum centres vertically. Laps in bars shall be no less than 300 mm.

b) R6 bars at 200 mm centres horizontally. These will be in the form of closed stirrups in the stack and U bars elsewhere.

c) Double horizontal reinforcing at any change in direction of the vertical steel (e.g. at the gather/stack intersection).

1.6.2 Bars which do not extend for the full height of the chimney shall be stopped in the gather:

a) In reinforced concrete and brick, by continuing these bars through to the far face of the gather and terminating with a 200 mm leg.

b) In precast pumice concrete, by anchoring the last 200 mm of the bar in a high strength cementitious grout. (See Figure 5.) Refer Paragraph 1.8.3 g) for grout details.

1.7 *Chimney restraint*

1.7.1 Chimneys which are not constructed integrally with the building shall be secured by floor and roof brackets. An acceptable alternative for brick and precast pumice concrete chimneys is that they be restrained by a roof tie used in conjunction with closely spaced wall ties. (Refer Paragraphs 1.7.5 to 1.7.16.)

1.7.2 Where a packer (see Figures 2, 3, 6 and 7(b)) is shown between the chimney and building it shall be:

a) Concrete, brick, steel (angle, channel or Z section), or any insulating material which has a long term operating temperature of no less than 150°C,

b) Secured in place to prevent it dislodging, and

c) Capable of withstanding a compressive force of 10 kN without shortening by more than 1.5 mm.

**COMMENT:**

C/AS1 Part 9 requires a 50 mm separation between the chimney and any combustible material. Where the chimney fixing described does not prevent the chimney moving within this gap, a packer is shown.

1.7.3 *Floor and roof brackets*

The brackets shall comprise a 50 mm x 4 mm hot dip galvanised steel strap placed around the chimney. Each leg of the strap shall be horizontal and shall be bolted to the joists with three M12 bolts at 75 mm centres as shown in Figure 6.
Note: Although drawn for the concrete chimney, the reinforcing details apply to brick chimneys, except for the lintel which is described in Table 1.
Figure 5: Reinforcing Details - Precast Pumice Concrete Chimney
Paragraphs 1.6.1 and 1.6.2 b)

- Four corner rods to be bonded 200mm min into gather with a high strength grout.
- Two outside stack rods pass continuously from foundation to base to gather to stack.
- Two inside stack rods to be fixed a minimum of 200mm into gather with a high strength grout.
- Gather on base.
- 50mm dia. ducts for reinforcing rods.
- Chimney stack units
- D12 grouted in 50mm diameter duct.
- 50mm dia. duct for reinforcing in gather.
- Gather
- D12 in high strength grout over length of 200mm.
- D12 grouted into 50mm dia. duct.
- Chimney base
- Chimney foundation and starters as Fig. 1.
1.7.4 Brackets shall be located so that the distance between the top of the chimney foundation slab and the first bracket, and the distance between adjacent brackets does not exceed 3.0 m. Where a chimney foundation wall is integral with a building foundation wall, then the height to the first bracket may be measured from the top of the building foundation wall.

1.7.5 Alternative fixing using roof tie and closely spaced wall ties

This alternative chimney fixing shall apply only from the gather to roof level. It requires that either the top of the chimney foundation slab or a floor bracket complying with Paragraph 1.7.3 be located within a distance of 2.5 m below the first of the closely spaced wall ties. (See Figures 2 and 3.) If the latter applies, the chimney below this bracket shall be fixed by floor brackets spaced in accordance with Paragraph 1.7.4.

1.7.6 Brick chimneys

Brick chimneys shall be restrained at roof level by a zinc coated 50 x 1.0 mm mild steel ‘U’ strap used in conjunction with closely spaced wall ties. The strap shall be:

a) Cast into the grout and wrap around the reinforcing steel (see Figures 2 and 3),

b) Placed at no more than 20° from the horizontal,

c) Used in conjunction with a packer (complying with Paragraph 1.7.2) placed at the same level, and

d) Fixed with twelve 30 x 3.15 mm galvanised nails to roof or ceiling framing.

1.7.7 Wall ties (see Figure 7(a)) shall be located in mortar joints at 225 mm maximum centres up each side of the chimney, except that pairs of ties shall be used for chimneys wider than 600 mm.

1.7.8 Wall ties shall be constructed from either 4 mm diameter galvanised bar or 25 x 1.5 mm zinc coated steel strip capable of withstanding a load of 1.2 kN without elongating or shortening by more than 1.5 mm.

1.7.9 Where zinc coating of components is required it shall be no less than 300 g/m² in accordance with NZS 3441.

1.7.10 Nails used to fix straps to roof or ceiling framing shall be spaced at no less than 35 mm in Radiata Pine, and 70 mm in other timbers.

1.7.11 Acceptable alternatives to the cast-in ‘U’ strap are:

a) Any proprietary bracing strip system of equal durability to the ‘U’ strap described in Paragraph 1.7.6, and capable of carrying a seismic force of 12 kN without elongating by more than 1.5 mm, or

b) A cast-in hot dip galvanised, deformed 6.0 mm reinforcing bar bent to a ‘U’ shape, with each end fixed to the roof or ceiling framing with six 50 x 4.0 mm galvanised fencing staples.

1.7.12 The ‘U’ strap or either of the acceptable alternatives may be wrapped around the outside of the chimney rather than be cast-in, provided that if strap is used it shall be painted with a zinc rich primer.

1.7.13 Precast pumice concrete chimneys

Precast pumice concrete chimneys shall be restrained at roof level either by a 50 x 1 mm ‘U’ strap wrapped around the chimney, or by a hot dip galvanised deformed 6 mm reinforcing bar placed into the grout around the reinforcing steel, together with either fixing brackets or fixing ties (see Figure 7(b)). Straps and bars shall satisfy the relevant requirements of Paragraphs 1.7.6 to 1.7.12.

1.7.14 Fixing brackets (see Figure 7(b)) shall be made from 5.0 mm thick mild steel angle and drilled with:

a) A 50 mm diameter hole to suit the reinforcing duct location, and

b) A 14 mm diameter hole for the 12 mm diameter coach screw fixing to the double stud.

1.7.15 Fixing brackets shall be located in mortar joints between the units, and be spaced at no less than 480 mm centres for stacks up to 600 mm wide, and no less than 320 mm centres for stacks wider than 600 mm.
Figure 6: Chimney Restraint - Floor and Roof Brackets
Drawn for Roof Restraint
Paragraphs 1.7.2 and 1.7.3

Note:
Although drawn for a chimney on an exterior wall, the bracket details also apply to an interior chimney.
Figure 7: Chimney Restraint
Paragraphs 1.7.2, 1.7.7, and 1.7.13

(a) Brick chimney

(b) Precast pumice concrete chimney

Notes:
1. Brackets shall be made from 5 mm thick m.s. angle drilled with 50 mm dia. hole to suit the
   reinforcing duct. Locate brackets in the mortar joints between units at no more than:
   480 mm maximum centres for stacks up to 600 mm wide
   320 mm maximum centres for stacks over 600 mm wide
2. Ties shall comprise 4 mm galvanised hairpins located in the mortar joint between units at
   no more than:
   320 mm maximum centres for stacks up to 600 mm wide
   160 mm maximum centres for stacks over 600 mm wide
1.7.16 Fixing ties shall comprise 4 mm galvanised wire hairpins, which are hooked behind the reinforcing ducts and secured to the required adjacent double studding with four heavy duty fencing staples. The ties shall be located in mortar joints between the units and be at no less than 320 mm centres for stacks up to 600 mm wide, and no less than 160 mm centres for stacks wider than 600 mm.

1.8 Materials and construction

1.8.1 Brickwork
Brick chimney construction shall conform to the relevant sections of NZS 4210.

1.8.2 Concrete
Chimneys, foundations and hearth slabs of reinforced concrete, shall comply with the relevant clauses of NZS 3109 for ordinary grade concrete.

1.8.3 Precast pumice concrete
Pumice concrete units for use in precast chimneys shall:

a) Have pumice aggregate which:
   i) is free of combustible and organic matter, and
   ii) has a maximum aggregate size of no greater than 19 mm, with at least 40% but not more than 60% of the aggregate retained by a 4.75 mm standard test sieve, and

b) Have a mix ratio by volume of no more than five parts of mixed pumice aggregate to one part of cement,

c) Have a compressive strength of no less than 7 MPa at 28 days when cured and tested in accordance with NZS 3112: Part 2,

d) After adequate curing, be air dried and kept under cover during storage, transport and on the site,

e) Be laid dry. (Work left unfinished should be protected from rain.)

f) Be joined with mortar which complies with NZS 4210, and

g) Have ducts filled with grout complying with NZS 4210, except over the last 200 mm where bars are anchored in the gather (refer Paragraph 1.6.2 b)). At these locations a non-shrinking cement-based grout, which attains a minimum compressive strength of 30 MPa at 7 days, shall be used.

1.8.4 Concrete masonry
Concrete masonry construction for chimney foundation walls shall conform to the relevant sections of NZS 4229.

1.8.5 Reinforcing steel
Reinforcing used in chimneys is to conform to NZS 3402 for steel bars and NZS 3421, NZS 3422 for steel wire, and shall:

a) For brick, be embedded centrally in the thickness of the grout,

b) For in-situ concrete, have cover to the steel in accordance with NZS 3109,

c) For precast pumice concrete, be placed with grout in the preformed ducts in the units.

1.8.6 Hot dip galvanising
Hot dip galvanising shall comply with NZS/AS 1650.

1.9 Systems to resist horizontal earthquake loadings

1.9.1 The bracing described in Paragraphs 1.9.2 to 1.9.6 shall be provided in those buildings where one or more of the following apply:

a) The area of the room containing the chimney exceeds 24 m²;

b) The length of the wall on which the chimney is located exceeds 3.5 m between supporting braced walls which are perpendicular to it. This length may be increased to 6.5 m where the wall is supported, at each floor level and at the roof or ceiling level, by either a structural diaphragm which conforms with the relevant requirements of NZS 3604 or by dragon ties. The dragon ties shall:
i) consist of a continuous length of 100 x 50 mm timber fixed in accordance with NZS 3604 clauses 8.3.3.3 and 8.3.3.4,

ii) be run as a pair, with one dragon tie going from the wall on which the chimney is located, back to each of the supporting braced walls. The enclosed angle between the wall on which the chimney is located and each dragon tie shall be 60°, and

iii) be located no more than 1.5 m out from each supporting braced wall.

c) The floor area on any level of the building, for a given chimney type (see Table 2), is less than:

i) 50 m² for chimney Type 1,

ii) 75 m² for chimney Types 2, 3 and 4,

iii) 150 m² for chimney Types 5, 6 and 7.

1.9.2 The building supporting the chimney shall contain bracing elements to resist earthquake loads from the chimney. These loads are applied at roof level and at each floor to which the chimney is connected. The bracing elements necessary are additional to those required by NZS 3604 or NZS 4229.

1.9.3 The number of bracing units to be provided for each chimney connection (see Paragraph 1.9.4) is given in Table 2. The number of bracing units to be provided at any level shall be the sum of the bracing units required at each of the chimney connections above the level being considered.

**COMMENT:**
As an example: for a standard precast pumice concrete chimney in a two storey building in Zone A, that is connected to the building by a roof bracket and by floor brackets at ground and first floor, the number of bracing units required are:

<table>
<thead>
<tr>
<th>Location</th>
<th>Bracing units required</th>
</tr>
</thead>
<tbody>
<tr>
<td>- J ust below roof level</td>
<td>60</td>
</tr>
<tr>
<td>- J ust below first floor level</td>
<td>60 + (60% of 60) = 96</td>
</tr>
<tr>
<td>- J ust below ground floor level</td>
<td>60 + (60% of 60) + 60</td>
</tr>
<tr>
<td></td>
<td>= 156</td>
</tr>
</tbody>
</table>

1.9.4 A chimney shall be considered as connected to the building when:

a) At roof level: it is held either by a roof bracket or by a roof tie,

b) At ground floor level: it is held by a floor bracket or the chimney base is integral with the building foundation wall,

### Table 2: Bracing Units Required for Each Chimney Connection to Resist Earthquake Loadings

<table>
<thead>
<tr>
<th>Chimney construction</th>
<th>Type</th>
<th>Max size of chimney: (Stack, Base, Zone A, Zone B, Zone C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast pumice</td>
<td>1</td>
<td>500 x 400, 1600 x 1050, 60, 50, 40</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1100 x 400, 1600 x 1050, 110, 90, 70</td>
</tr>
<tr>
<td>Brick</td>
<td>3</td>
<td>500 x 500, 1200 x 1050, 90, 70, 60</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>590 x 590, 1200 x 1050, 130, 100, 80</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1200 x 680, 1200 x 1050, 240, 200, 160</td>
</tr>
<tr>
<td>Concrete</td>
<td>6</td>
<td>590 x 590, 1200 x 1050, 210, 170, 140</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>1200 x 700, 1200 x 1050, 390, 320, 260</td>
</tr>
</tbody>
</table>

Note:
1. The number of bracing units required at floor connections other than the ground floor shall be taken as 60% of the value given in the table.
c) At an intermediate floor level: it is held either by a floor bracket or by closely spaced wall ties spanning the floor.

1.9.5 For earthquake ground movement in the direction perpendicular to the wall on which the chimney is located, structural diaphragms shall be provided at roof/ceiling level and at each floor level to which the chimney is connected. The diaphragms shall comply with all relevant clauses of NZS 3604 and NZS 4229.

1.9.6 For earthquake in the direction parallel to the wall on which the chimney is located, the bracing units required as determined from Paragraph 1.9.3 shall be provided solely by that wall.

2.0 Solid Fuel Burning Domestic Appliances

2.1 Chimneys

2.1.1 Chimneys for solid fuel burning appliances shall comply with Paragraph 1.0 or with the relevant sections of NZS 7401 and NZS 7421 for sheetmetal chimneys.

2.2 Hearth slab

2.2.1 Solid fuel burning domestic appliances weighing no more than 130 kg shall be supported on a 65 mm thick hearth slab that is:

- a) Reinforced with 665 mesh, or D10 rods at 300 mm centres each way, placed centrally in the slab thickness,

- b) Supported on a timber or concrete floor, or integral with a concrete floor. (The floor supporting the hearth slab shall comply with NZS 3604 or NZS 4229 as appropriate), and

- c) Comprised of ordinary grade concrete complying with the relevant clauses of NZS 3109.

2.2.2 Hearth slabs on a timber floor shall be held in position by supporting members on all four sides of the hearth. These members shall each be held by four screws with a minimum shank diameter of 4.88 mm that penetrate the floor framing by 50 mm.

2.2.3 Hearth slabs on concrete floors shall be secured in position by four D10 starter rods. The rods shall be located in each corner of the hearth slab and they shall terminate each end with standard hooks complying with NZS 3109.

Spread of fire

2.2.4 Paragraphs 2.2.1 to 2.2.3 provide an acceptable structural solution, but depending on the particular installation, different hearth dimensions may be necessary to meet the spread of fire requirements of NZBC Clause C1.3.2. Hearth slabs for solid fuel burning appliances shall comply with NZS 7421.
1.0 Scope and Limitations

1.0.1 This document covers the ultimate limit state design of foundations, including those of earth retaining structures. Methods are given for determining ultimate bearing and lateral sliding strengths.

1.0.2 This document does not describe a means of determining the value of the soil parameters used in the document (e.g. c', \(\phi'\), and \(s_u\)). The derivation of these parameters, which must be based on the most adverse moisture and groundwater conditions likely to occur, is outside of the scope of this verification method.

1.0.3 Serviceability limit state deformations are not covered in this document. The determination of such deformations and their acceptability to the design in question needs to be considered but is outside the scope of this document.

1.0.4 This document assumes general ground or slope stability and provides methods only for ensuring against local failure of the foundation. Overall ground stability needs to be verified before this document can be applied; this is outside the scope of this verification method.

1.0.5 This document shall not be used to design foundations on loose saturated sands or on cohesive soils having a sensitivity greater than 10.

1.0.6 This document shall not be used for foundations subject to continuous vibration.

1.0.7 The “Comments” and “Informative Appendices” of this document provide comment, background or general information but do not form part of this verification method.

2.0 General

2.0.1 Foundations shall be designed for the load combinations given in NZS 4203: 1992 as amended by B1/VM1. Strength reduction factors given in this document shall be used to determine the design strength of the foundation. The design loadings shall not cause the foundation’s design strength to be exceeded.

2.0.2 The design procedures of this document must be performed by a person who, on the basis of experience or qualifications, is competent to apply them.

2.0.3 The building’s foundation elements or the elements of earth retaining structures shall be designed in accordance with the appropriate material Standards, as given in B1/VM1.

2.0.4 Foundations may be shallow or deep. A shallow foundation is one in which the...
depth from the ground surface to the underside of the foundation is less than five times the width of the foundation. All other foundations are considered to be deep.

2.0.5 In assigning values for soil parameters the worst groundwater condition shall be considered.

**COMMENT:**
For cohesive soils the fully saturated condition will generally give the lowest strength and stiffness.

2.0.6 Foundation strength for cohesive soil depends on loading duration and whether consolidation can occur. For this reason the distinction is made between short term (e.g. initial load application, earthquake actions or wind gusts) and long term loading (e.g. permanent loads such as foundation dead load). For the short term case no consolidation occurs and the calculations shall be in terms of undrained shear strength (i.e. shear strength of the soil $s_u$) and total stress. For long term loading, full consolidation occurs and the calculations shall be in terms of drained shear strength and effective stress (i.e. soil parameters being cohesion, $c$, and the angle of shearing resistance $\phi'$).

2.0.7 For cohesionless soils consolidation occurs very quickly so drained strength shall be used in all cases.

2.0.8 Supervision and verification of soil parameters

Design assumptions and soil parameters shall be verified during construction. The designer shall nominate what supervision, including verification of soil parameters, will be undertaken during the construction period.

3.0 Shallow Foundations

3.1 General Provisions

3.1.1 The ultimate bearing strength shall be based on the most adverse moisture and groundwater conditions likely to occur.

3.1.2 Founding depths in clay soils known to exhibit swelling and shrinking behaviour shall be chosen so that the underside of the foundation is beneath the zone of soil affected by shrinking and swelling caused by seasonal weather changes, and the root systems of nearby trees and shrubs.

3.1.3 Consideration shall be given to the possibility of any surcharge adjacent to a shallow foundation being removed during the life of the foundation, so reducing the available ultimate bearing strength.

3.1.4 Foundations subject to moment loading shall not be proportioned such that the point of application of the reaction force on the underside of the foundation is closer to the edge than $B/6$, for a rectangular foundation, or $r/2$, for a circular foundation.

3.2 Ultimate and design bearing strength and design bearing pressure

3.2.1 The design bearing pressure $q_d$ shall be determined by dividing the design vertical forces (derived from combinations of factored vertical loads) by the effective area of the foundation. See Paragraph 3.3 for notation and the definition of effective area.

3.2.2 The ultimate bearing strength $q_u$ is that pressure, exerted on the ground by the building foundation, which causes the ground to fail by mobilisation of all available shear strength. It shall be evaluated using the provisions of Paragraph 3.3.

3.2.3 The design bearing strength $q_{uds}$ shall be determined by multiplying the ultimate bearing strength by the appropriate strength reduction factor (see Paragraph 3.5.1).

3.2.4 The design bearing pressure shall not exceed the design bearing strength.

3.3 Ultimate limit state bearing strength for shallow foundations

3.3.1 The procedures specified in the following text apply to foundations of any size. The formulae are limited to soil profiles that for a depth beneath the underside of the foundation of at least two times the foundation width can be represented with single values for the density, angle of shearing
resistance, cohesion, and if appropriate, undrained shear strength.

Notation:

- $A'$: effective foundation area ($m^2$). For a rectangular foundation $A' = B'L'$. For a circular foundation see Figure 2.
- $B$: foundation breadth ($m$).
- $B'$: the smaller of $2(X + e_b) / (B - X - e_b)$ (see Figure 1) ($m$).
- $D_e$: minimum horizontal distance from the edge of the underside of the foundation to the face of an adjacent downward slope ($m$).
- $D_i$: depth to the underside of the foundation ($m$).
- $H$: design horizontal load, the resultant of the factored horizontal forces applied to the foundation (kN).
- $H_{uf}$: unfactored horizontal foundation load (kN).
- $L$: foundation length ($m$).
- $L'$: the smaller of $2(Y + e_l) / (L - Y - e_l)$ (see Figure 1) ($m$).
- $M_b$: design moment applied about an axis parallel to the breadth direction of the foundation (kNm).
- $M_c$: design moment applied to a circular footing (kNm).
- $M_l$: design moment applied about an axis parallel to the length direction of the foundation (kNm).
- $N_c$, $N_q$, $N_{q/H}$: bearing strength factors.
- $P_p$: ultimate lateral resistance derived from passive earth pressure (kN).
- $R$: reaction on underside of foundation = $q_d A'$ (kN).
- $S$: ultimate shear strength between the base of the foundation and the ground (kN).
- $V$: design factored vertical foundation load (kN).
- $V_{uf}$: unfactored vertical foundation load (kN).
- $V'$: effective design factored vertical load = $V - u_r A'$ (kN).
- $X$: the distance from the edge of the foundation, along the $x$ axis, to the point of application of the design vertical foundation load $V$ ($m$).
- $Y$: the distance from the edge of the foundation, along the $y$ axis, to the point of application of the design vertical foundation load $V$ ($m$).
- $Z$: the distance from the edge of a circular foundation, along the $z$ axis, to the point of application of the design vertical foundation load $V$ ($m$).
- $c$: cohesion (kPa).
- $c'$: effective stress cohesion (kPa).
- $e_b$: $M_b / V$ (positive when $R$ is further along the $x$ axis than $V$, see Figure 1) ($m$).
- $e_c$: $M_c / V$ (positive when $R$ is further along the $z$ axis than $V$, see Figure 2) ($m$).
- $e_l$: $M_l / V$ (positive when $R$ is further along the $y$ axis than $V$, see Figure 1) ($m$).
- $q$: vertical total stress in ground adjacent to the foundation at depth $D_i$ (kPa).
- $q'$: vertical effective stress ($\sigma'_v$) in ground adjacent to the foundation at depth $D_i$ (kPa).
- $q_d$: design bearing pressure = $V/A'$ (kPa).
- $q_u$: ultimate bearing strength (kPa).
- $q_{db}$: design bearing strength = $q_{bc} q_u$ (kPa).
- $r$: radius of a circular foundation ($m$).
- $s_u$: undrained shear strength (kPa).
- $u$: pore water pressure at a given position in the soil profile (kPa).
- $u_r$: pore water pressure at depth $D_i$ (kPa).
- $x$: axis through design vertical foundation load $V$ in direction of foundation breadth. The axis starts at the foundation edge and is positive in the direction towards $V$.
- $y$: axis through design vertical foundation load $V$ in direction of foundation length. The axis starts at the foundation edge and is positive in the direction towards $V$.
- $z$: axis through the centre of a circular foundation and the design vertical foundation load $V$. The axis starts at the foundation edge and is positive in the direction towards $V$. 
\[ q_u = c \lambda_{cs} \lambda_{cg} N_c + q \lambda_{cp} \lambda_{cg} \lambda_{qp} N_q + \frac{1}{2} B \lambda_{cs} \lambda_{cg} \lambda_{dg} N_d \]

The bearing strength factors are obtained from Figure 3 or the following equations:

\[ N_c = e^{\tan \theta} \left( 45 + \frac{\phi}{2} \right) \]

where \( e \) is the mathematical constant = 2.7183

For undrained analysis (\( \phi = 0 \)) use the following form of the general equation:

\[ q_u = s \lambda_{cs} \lambda_{cg} N_c + \lambda_{cq} q \]

For drained analysis use the following form of the general equation:

\[ q_u = s \lambda_{cs} \lambda_{cg} N_c + \lambda_{cq} q \]

\[ N_c = (N_q - 1) \tan \phi \]

The \( \lambda \) factors in the above equation are:

a) Shape factors: \( \lambda_{cs}, \lambda_{cp}, \) and \( \lambda_{cd} \)

where:

\[ \lambda_{cs} = 1 + \left( \frac{B_i}{B} \right) \left( \frac{N_q}{N_c} - 1 \right) \]

\[ \lambda_{cp} = 1 + \left( \frac{B_i}{B} \right) \tan \phi \]

\[ \lambda_{cd} = 1 - 0.4 \left( \frac{B_i}{B} \right) \]

b) Depth factors: \( \lambda_{cd}, \lambda_{cq}, \) and \( \lambda_{cd} \)

where:

- For \( \phi = 0 \) and \( \frac{D_i}{B} \leq 1 \):

\[ \lambda_{cd} = 1 + 0.4 \left( \frac{D_i}{B} \right) \]

- For \( \phi = 0 \) and \( \frac{D_i}{B} > 1 \):

\[ \lambda_{cd} = 1 + 0.4 \tan^2 \left( \frac{D_i}{B} \right) \]

where \( \tan^2 \) is in radians
Figure 1: Bearing Strength Stress Block for a Shallow Rectangular Foundation Subject to Vertical Load and Moment

Paragraph 3.3.1

Notes:
1. Section (a) above drawn through foundation width. Section through foundation length similar.
2. B’ is the smaller of 2(X + e_b) and 2(B - X - e_b). Similarly L’ is the smaller of 2(Y + e_l) and 2(L - Y - e_l).
3. M can be applied anywhere on the foundation and does not have to be applied at the location of V.
Notes:

Effective area $A'$ shall be represented by an equivalent rectangle of length $L'$ and breadth $B'$, where:

$$A' = r^2 \left[ \cos^{-1} \left( \frac{\xi}{1 - \xi^2} \right) - \frac{\xi}{\sqrt{1 - \xi^2}} \right]$$  \hspace{1cm} (Note: $\cos^{-1}(\xi)$ is in radians.)

$$B' = \left( \frac{1}{1 + \xi} \right)^{0.25} \sqrt{A'}$$

$$L' = \left( \frac{1}{1 - \xi} \right)^{0.25} \sqrt{A'}$$

where:

$$\xi = \frac{e_c + Z - r}{r} \text{ when } e_c + Z \geq r, \text{ or } \frac{r - e_c - Z}{r} \text{ otherwise}$$

Figure 2: Effective Foundation Area for a Circular Foundation Subject to Vertical Load and Moment

Paragraph 3.3.1
\[ \lambda_{ci} = \frac{1 - \lambda_{ci}}{N_q \tan \phi} \]

- for \( \frac{D_I}{B_I} \leq 1: \)
  \[ \lambda_{ci} = 1 + 2\tan \phi(1 - \sin \phi) \left( \frac{D_I}{B_I} \right) \]

- for \( \frac{D_I}{B_I} > 1: \)
  \[ \lambda_{ci} = 1 + 2\tan \phi(1 - \sin \phi)^2 \tan \left( \frac{D_I}{B_I} \right) \]
  where \( \tan \phi \) is in radians

- for all cases \( \lambda_{ci} = 1 \)

\( c) \) Load inclination factors: \( \lambda_{ci}, \lambda_{qi} \) and \( \lambda_{qi} \) where:

- for \( \phi = 0 \)
  \[ \lambda_{ci} = 0.5 \left( 1 + \sqrt{1 - \frac{H_{ud}}{A'S_{ud}}} \right) \]
  \[ \lambda_{qi} = 1 \]

- for \( \phi > 0 \)

  - for horizontal loading parallel to \( L^I \)
    \[ \lambda_{qi} = \lambda_{ci} = 1 - \frac{H_{ud}}{(V_{ud} + A'c \cot \phi')} \]
  
    where \( \lambda_{ci} = \lambda_{qi} N_q - 1 \)
    
    \[ \lambda_{qi} = \left( 1 - \frac{0.7H_{ud}}{V_{ud} + A'c \cot \phi'} \right)^3 \]

  - for horizontal loading parallel to \( B^I \)

\( d) \) Ground inclination factors: \( \lambda_{cq}, \lambda_{qg} \) and \( \lambda_{qg} \)

- for horizontal ground \( \lambda_{cq} = \lambda_{qg} = \lambda_{qg} = 1 \)

- for inclined ground, the permitted slope (angle \( \omega \) below the horizontal) depends on soil angle of shearing resistance \( \phi \) and the distance \( D_e \) between the foundation and the slope face:

  - where \( \phi > 0 \) (drained analysis)
    \[ \omega \text{ shall not be } > \phi \]
  
  - where \( \phi = 0 \) (undrained analysis)
    \[ \omega \text{ shall not be } > 45^\circ \]

The ground inclination factors shall be:

- for \( D_e \leq 2B \)
  \[ \lambda_{cq} = \lambda_{qg} = \lambda_{qg} = 1 \]

- for \( D_e < 2B \)
  \[ \lambda_{cq} = 1 - \omega (1 - D_e/2B)/150 \]
  
  \[ \lambda_{qg} = \lambda_{qg} = (1 - \tan(\omega (1 - D_e/2B)))^2 \]

\( 3.3.3 \) Local shear

For sands with relative densities less than 40% and clays having liquidity indices greater than 0.7, the bearing strength shall be evaluated using 0.67c for cohesion and \( \tan^{-1}(0.67\tan \phi) \) for the angle of shearing resistance.

\( \text{COMMENT:} \)

The formulae in Paragraph 3.3.2 assume a general shear failure of the soil but for the soils specified in this Paragraph a local shear failure is likely.
3.4 Ultimate limit state sliding resistance

3.4.1 When the loading is not normal to the foundation base, foundations shall be checked for failure by sliding.

3.4.2 The ultimate sliding resistance shall comprise the sum of the ultimate sliding strength between the base of the foundation and the ground, and any available passive earth pressure in the direction of sliding at the side of the foundation.

3.4.3 Passive earth pressure shall not be considered if:

a) For foundations in clay soils, it is possible that the clay could shrink away from the vertical faces of the foundation, or

b) The possibility exists that the soil in front of the foundation may be removed by erosion or by building or landscaping work in the future.

3.4.4 For drained conditions, the ultimate sliding strength shall be:

\[ S = c' A' + V \tan \phi \]

The value of \( \phi \) shall be taken as the angle of shearing resistance \( (\phi) \) of the foundation soil for cast-in-situ concrete foundations and 0.67(\( \phi \)) for smooth precast foundations.

3.4.5 For undrained conditions, the ultimate sliding strength shall be:

\[ S = A's_u \]
3.4.6 Design sliding resistance

The design horizontal load \( H \) shall not exceed the design sliding resistance, that is:

\[
H < \Phi_s S + \Phi_p P_p
\]

3.5 Strength reduction factors

3.5.1 Strength reduction factors to be applied to shallow foundation design shall be within the range given in Table 1.

The designer shall nominate in the design the strength reduction factors chosen along with substantiation as to why the values chosen are considered appropriate. The values chosen shall be to the approval of the territorial authority.

**COMMENT:**

The value of the strength reduction factor used in design will depend on the designer’s knowledge of the site and the investigations undertaken. As a guide the lower end of the range will generally be appropriate when a limited site investigation is undertaken, average geotechnical properties are used, published correlations are used to obtain design parameters or there will be minimal construction control. The upper end of the range will generally be appropriate when a comprehensive site investigation and laboratory testing is undertaken, geotechnical properties are chosen conservatively, site specific correlations are used for design parameters and there will be careful construction control.

4.0 Pile Foundations

4.0.1 The ultimate axial compressive pile strength for a single pile shall be determined using either or both of the following methods:

a) Geotechnical calculation,

b) Static load testing.

**COMMENT:**

Where piles are driven, a driving formula, such as the Hiley Formula, may provide a useful means of assessing the comparative strength of the individual piles at a particular site.

4.0.2 Using geotechnical calculation, the ultimate axial compressive pile strength is the sum of the ultimate pile point-bearing resistance and the shaft resistance.

4.0.3 When determined by static load testing, the ultimate axial compressive pile strength shall be taken as no more than that load which produces a penetration or pile settlement of 0.1 times the:

a) Nominal pile width for driven piles,

b) Bell diameter for belled piles,

c) Estimated minimum bulb diameter for bulbed piles.

Suitable procedures for static load testing are described in AS 2159 Section 8, ASTM D1143 and BS 8004 Section 7.5.

4.0.4 The design pile vertical or lateral strength of a single pile or pile group shall be determined by multiplying the ultimate strength by the appropriate strength reduction factor (see Paragraph 4.7.1). The design strength shall be greater than the applied factored loads.

**Table 1: Strength Reduction Factors for Shallow Foundation Design**

<table>
<thead>
<tr>
<th>Load combination</th>
<th>Strength reduction factor range</th>
</tr>
</thead>
</table>
| For bearing \((\Phi_s)\) and passive earth pressure \((\Phi_p)\):
  Load combinations involving earthquake overstrength | 0.80 – 0.90                     |
  All other load combinations                    | 0.45 – 0.60                     |
| For sliding \((\Phi_s)\):
  All load combinations, including earthquake overstrength | 0.80 – 0.90                     |
4.1 Ultimate vertical strength of single piles

4.1.1 Notation

- $A_b$: area of pile base (m$^2$).
- $B_G$: width (between pile extremities) of a pile group (m).
- $C$: circumference of the pile shaft (m).
- $D_b$: diameter of the pile base (m).
- $D_s$: diameter of the pile shaft (m).
- $H$: design horizontal load applied to the pile head (factored applied loads) (kN).
- $H_u$: ultimate lateral strength of a pile (kN).
- $K_o$: the coefficient of earth pressure at rest $= 1 – \sin \psi$ for loose sand and normally consolidated clay, and $(1 – \sin \psi) / \sqrt{OCR}$ for over-consolidated soils.
- $K_p$: coefficient of passive earth pressure $= (1 + \sin \psi)/(1 - \sin \psi)$.
- $K_s$: factor that expresses the horizontal effective stress at the pile/soil interface in terms of the vertical effective stress (see Table 2).
- $L$: length of the pile shaft (m).
- $L_G$: length (between pile extremities) of a pile group (m).
- $M$: design moment applied to the pile head (factored applied moments) (kNm).
- $M_{ult}$: ultimate moment strength of the pile shaft (kNm).
- $OCR$: over-consolidation ratio being the previous maximum effective stress/current effective stress.
- $R_d$: relative density as measured in accordance with Test 4.2.3 of NZS 4402.
- $V_1$: ultimate strength of an individual pile in the group (kN).
- $V_{bl}$: ultimate strength of the block of soil enclosed within the pile group (kN).
- $V_G$: ultimate strength of the group (kN).
- $V_{bu}$: ultimate base resistance (kN).
- $V_{su}$: ultimate shaft resistance (kN).
- $V_u$: vertical pile strength (kN).
- $W$: pile weight (part of the dead load) (kN).
- $c_a$: the undrained adhesion (total stress) at the soil/shaft interface in a clay soil, or the adhesion at the boundary of a pile group $= \alpha_s \sigma_u$ (kPa).
- $c'_a$: drained (effective stress) adhesion at the soil/shaft interface in a cohesive soil, or the adhesion at the boundary of a pile group (kPa).
- $f$: for a free head pile, the distance above the ground surface at which the horizontal shear is applied ($= M/H$); and for a restrained head pile, the distance above the ground surface at which the restraint is applied (m).
- $f_o$: length of pile shaft assumed to be unsupported in cohesive soil $= 1.5D_s$ (m).
- $g_c$, $g_n$, $g_s$: position along the pile shaft at which yielding occurs for piles in over-consolidated clay, normally consolidated clay, and sand respectively (m).
- $n$: number of piles in the group.

### Table 2: Values of $\delta$ and $K_s$ for Pile Shafts

<table>
<thead>
<tr>
<th>Pile Material</th>
<th>$\delta$</th>
<th>$K_s$</th>
<th>$R_d &lt; 40%$</th>
<th>$R_d &gt; 40%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>$20^\circ$</td>
<td></td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>$3\delta/4$</td>
<td></td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Timber</td>
<td>$2\delta/3$</td>
<td></td>
<td>1.5</td>
<td>4.0</td>
</tr>
</tbody>
</table>
q vertical stress in the soil at a depth equal to the base of the pile shaft, total stress for undrained analysis and effective stress for drained analysis (kPa).

$s_u$ undrained shear strength (kPa).

$\phi_{pc}$ strength reduction factor for pile strength (for both vertical and lateral strength) (see Paragraph 4.7.1).

$\alpha$ adhesion factor (see Figure 5).

$\gamma$ unit weight of the soil in which the pile is embedded, chosen to give the total stresses for undrained loading in cohesive soil and effective stresses for drained loading ($\gamma'$ beneath the water table) (kN/m$^3$).

$\Gamma$ $\gamma$ when the water table is deeper than 2B beneath the underside of the foundation and $\gamma'$ when the water table is above this.

$\delta'$ drained angle of shearing resistance at the soil/shaft interface (see Table 2) (degrees).

$\phi$ angle of shearing resistance (degrees).

$\phi'$ effective stress angle of shearing resistance (degrees).

$\chi$ rate of increase in undrained shear strength with depth (kPa/m).

( )$_{\text{average}}$ the average value of the parameter in the brackets taken over the length of the pile shaft.

4.1.2 Vertical strength

The vertical pile strength is:

$$V_u = V_{su} + V_{bu}$$

4.1.3 Base resistance

The undrained base resistance of piles in cohesive soil is:

$$V_{bu} = (9s_u + q) A_b$$

The drained base resistance, when the soil is sufficiently uniform to be represented by single values of $c'$, $\phi'$, $s_u$, and $\gamma$ for a distance of three pile shaft diameters above and below the pile base, shall be:

$$V_{bu} = (9c' + qN_q + 0.6D_b \Gamma N_s) A_b$$

The values of $N_q$ are taken from Figure 4 and $N_s$ from Figure 3.

4.1.4 Shaft resistance

a) For undrained loading of piles in cohesive soils:

$$V_{su} = (c_a)_{\text{average}} CL$$

where $c_a = \alpha s_u$ and values for $\alpha$ are given in Figure 5 for both driven and bored piles.

b) For drained loading of piles in cohesive soils:

$$V_{su} = \left\{ (c'\phi_{\text{average}} + (r\phi\tan\delta')_{\text{average}}) \right\} CL$$

The value of $\delta'$ is taken from Table 2.

c) For drained loading of driven piles in cohesionless soils:

$$V_{su} = \left( r\phi K_s \tan\delta' \right)_{\text{average}} CL$$

Values for $K_s$ are given in Table 2.

4.2 Column action

4.2.1 Piles which stand unbraced in ground, water, or other material incapable of providing lateral support, shall be designed as columns.

4.2.2 For a column partly embedded in the ground, the effective length is dependent upon the position of end restraint, which in turn is dependent upon the nature of the ground. End restraint shall be assumed at a depth of no less than:

a) 3 times the nominal pile width in very stiff soil. (For clays an undrained shear strength greater than or equal to 100 kPa, and for sands a relative density greater than or equal to 50% shall be regarded as very stiff soil.)

b) 6 times the nominal pile width in firm soil. (For clays an undrained shear strength between 50 and 100 kPa, and for sands a relative density between 30 and 50% shall be regarded as stiff soil.)

c) 9 times the nominal pile width in other soil conditions.
Figure 4: $N_q$ Values for Pile Foundations
Paragraph 4.1.3

Figure 5: Adhesion Factor for Piles in Cohesive
Paragraphs 4.1.1 and 4.1.4 a)
4.3 Ultimate lateral strength of single piles

4.3.1 In the following paragraphs the terms “free head” and “restrained head” pile are used. Free head piles are classified as short and long. Restrained head piles are classified as short, intermediate and long. These terms are explained as follows:

a) A free head pile has no restriction against head rotation when lateral displacement occurs. For a short free head pile the magnitude of the maximum bending moment in the embedded shaft is less than the ultimate moment strength of the pile shaft, and the ultimate strength is controlled by the embedment length of the pile shaft. The strength of a long free head pile is controlled by the ultimate moment strength of the pile shaft and not by the embedded length.

b) For a restrained head pile subject to lateral displacement, the head rotation is constrained at the pile head by a fixing moment. A short pile is one in which the head moment and the maximum pile shaft moment are less than the ultimate moment strength of the pile section. For an intermediate length restrained head pile the head moment is equal to the ultimate strength of the pile shaft and elsewhere the shaft moments are less than $M_{ult}$. For a long restrained head pile the head moment and the maximum pile shaft moment each have a magnitude of $M_{ult}$.

4.3.2 Undrained lateral strength of piles in cohesive soil having a constant undrained shear strength with depth

a) Free head piles

i) short free head piles

The ultimate lateral strength of a short free head pile is given by:

$$H_u = 9s_uD_s \left\{ \sqrt{2(2f + L)^2 + (f + f_o)^2} - (L + 2f + f_o) \right\}$$

The location, measured from the ground surface, of the maximum pile shaft moment is:

$$g_c = \frac{H_u}{9s_uD_s} + f_o$$

The maximum moment in the pile shaft is:

$$M_{max} = H_u \left( f + f_o + \frac{H_u}{18s_uD_s} \right)$$

If $M_{max}$ is greater than $M_{ult}$, the strength must be evaluated as for a long free head pile.

ii) long free head piles

The ultimate lateral strength of a long free head pile is:

$$H_u = 3s_uD_s \left( 9(f + f_o)^2 + \frac{2M_{ult}}{s_uD_s} - 3(f + f_o) \right)$$

The location of the maximum pile shaft moment ($M_{ult}$) is obtained from the same equation as for the short pile.

b) Restrained head piles

i) short restrained head piles

The ultimate lateral strength of a short restrained head pile is:

$$H_u = 9s_uD_s (L - f_o)$$

The pile head moment is:

$$M_{max} = 0.5H_u (L + 2f + f_o)$$

If $M_{max}$ is greater than $M_{ult}$, then the intermediate length case, ii) below, is appropriate.
ii) intermediate restrained head piles

The ultimate lateral strength of an intermediate length restrained head pile is:

\[ H_u = 9s_uD_s \sqrt{(L + 2f + f_o)^2 + (L - f_o)^2 + \frac{4M_{ult}}{9s_uD_s} - (L + 2f + f_o)} ]

The location, measured from the ground surface, of the maximum pile shaft moment is:

\[ g_c = \frac{H_u}{9s_uD_s} + f_o \]

The pile shaft moment at this depth is:

\[ M_{max} = H_u \left[ \frac{H_u}{18s_uD_s} + f + f_o \right] - M_{ult} \]

If \( M_{max} \) calculated from this equation is greater than \( M_{ult} \), then the long case, iii) below, is appropriate.

iii) long restrained head piles

The ultimate lateral strength of a long restrained head pile is:

\[ H_u = \frac{9s_uD_s}{H_{20844}} \left( f + f_o \right)^2 + \frac{4M_{ult}}{2(f + L)} - (f + f_o) \]

The location of the maximum pile shaft moment \( (M_{ult}) \) is obtained from the same equation as for the intermediate length pile.

4.3.3 Undrained lateral strength of piles in normally consolidated cohesive soil

Normally consolidated cohesive soils have a linear increase in undrained shear strength with depth, starting with a value of zero at ground surface level.

**COMMENT:**

Only the long free head pile and intermediate and long restrained head piles are considered. Short piles are not normally used in such material.

The rate of increase in undrained shear strength with depth is denoted by \( \chi \) (kPa/m).

a) **Long free head pile**

The ultimate lateral strength of a long free head pile is obtained by solving:

\[ H_u \left[ \frac{2}{3} \sqrt{\frac{2H_u}{9D_s\chi}} + f \right] - M_{ult} = 0 \]

The location, measured from the ground surface, of the maximum pile shaft moment \( (M_{ult}) \) is:

\[ g_c = \sqrt{\frac{2H_u}{9D_s\chi}} \]

b) **Restrained head pile**

i) intermediate restrained head piles

The ultimate lateral strength of an intermediate length restrained head pile is:

\[ H_u = \frac{3D_sL_s^3}{2(\chi + L)} + \frac{M_{ult}}{f + L} \]

The location of the maximum pile shaft moment \( (M_{ult}) \) is obtained from the same equation as for the long free head pile.

The pile shaft moment at this depth is:

\[ M_{max} = H_u \left[ \frac{2}{3} \sqrt{\frac{2H_u}{9D_s\chi}} + f \right] - M_{ult} \]

If \( M_{max} \) calculated from this equation is greater than \( M_{ult} \), then the long case, ii) below, is appropriate.

ii) long restrained head piles

The ultimate lateral strength of a long restrained head pile is obtained by solving:
4.3.4 Drained lateral strength of piles in cohesionless soil

a) Free head piles

i) short free head piles

The ultimate lateral strength of a short free head pile is:

\[ H_u = \frac{K_p D_s L^2 y}{2(f + L)} \]

The location, measured from the ground surface, of the maximum pile shaft moment is:

\[ g_s = \sqrt{\frac{2H_u}{3K_p D_s y}} \]

The maximum pile shaft moment is:

\[ M_{max} = H_u \left[ 2 \sqrt{\frac{2H_u}{3K_p D_s y}} + f \right] - M_{ult} = 0 \]

ii) long free head piles

The ultimate lateral strength of a long free head pile is obtained by solving the following equation:

\[ H_u \left[ \frac{2}{3} \sqrt{\frac{2H_u}{3K_p D_s y}} + f \right] - M_{ult} = 0 \]

The location of the maximum pile shaft moment \( (M_{ult}) \) is obtained from the same equation as for the short pile.

b) Restrained head piles

i) short restrained head piles

The ultimate lateral strength of a short restrained head pile is:

\[ H_u = 1.5K_p D_s L^2 y \]

The magnitude of the maximum pile head moment is:

\[ M_{max} = H_u \left( \frac{2}{3}L + f \right) \]

If \( M_{max} \) is greater than \( M_{ult} \) then the intermediate length case, ii) below, is appropriate.

ii) intermediate restrained head piles

The ultimate lateral strength of an intermediate length restrained head pile is:

\[ H_u = \frac{K_p D_s L^2 y}{2(f + L)} + \frac{M_{ult}}{f + L} \]

The location, measured from the ground surface, of the maximum pile shaft moment is:

\[ g_s = \sqrt{\frac{2H_u}{3K_p D_s y}} \]

The pile shaft moment at this depth is:

\[ M_{max} = H_u \left[ 2 \sqrt{\frac{2H_u}{3K_p D_s y}} + f \right] - M_{ult} \]

If \( M_{max} \) calculated from this equation is greater than \( M_{ult} \) then the long case, iii) below, is appropriate.
iii) long restrained head piles
The ultimate lateral strength of a long
restrained head pile is obtained by solving:

$$H_u \left| \frac{2}{3} \sqrt{\frac{2H_u}{3K_pD_s^3}} + f \right| - 2M_{uk} = 0$$

The location of the maximum pile shaft
moment is obtained from the same
equation as is used for the intermediate
length case.

4.4 Pile groups
4.4.1 Ultimate vertical strength of pile groups
The undrained vertical strength of a pile
group considered as a single block in a
cohesive soil is:

$$V_B = (9s_u + q) B_G L_G + 2 (B_G + L_G) L (c_a)_{average}$$

The drained strength of a pile group considered
as a single block of soil is given by:

$$V_B = (c' + q' N_q + 0.6 B_G N_G) B_G L_G +
2 (B_G + L_G) L \left\{ (c_a)_{average} +
\left( n' K_o \tan \alpha \right)_{average} \right\}$$

The ultimate vertical strength of the group is
determined from:

$$\frac{1}{V_G} = \frac{1}{nV_i} + \frac{1}{V_B^n}$$

4.4.2 If only part of an embedded friction pile
length is in satisfactory material, the surface
area calculated as providing frictional
resistance shall be limited to the surface areas
in contact with that material.

4.5 Downdrag
4.5.1 Downdrag may be generated when a
pile shaft passes through a compressible soil
layer. Downdrag shall be considered as dead
load applied to the parts of the pile below the
compressible layer. It shall be added to the
imposed loadings and factored accordingly.

4.6 Ultimate lateral strength of pile
groups
4.6.1 If piles are spaced at centre to centre
intervals of less than 4.0 times the nominal
pile width, the ultimate lateral pile strength
shall be reduced. The reduced value shall be
calculated as a percentage of the ultimate
lateral pile strength for an isolated pile by
linear interpolation between the two values
given in Table 3.

4.7 Strength reduction factors
4.7.1 Strength reduction factors for design of
ultimate vertical and lateral strengths in pile
foundations shall be within the range given in
Table 4.

The designer shall nominate in the design the
strength reduction factors chosen along with
substantiation as to why the values chosen
are considered appropriate. The values chosen
shall be to the approval of the territorial authority.

COMMENT:
The value of the strength reduction factor used in
design will depend on the designer's knowledge of the
site and the investigations undertaken. As a guide the
lower end of the range will generally be appropriate
when a limited site investigation is undertaken, average
geotechnical properties are used, published correlations
are used to obtain design parameters or there will be
minimal construction control. The upper end of the
range will generally be appropriate when a comprehensive
site investigation and laboratory testing is undertaken,
geotechnical properties are chosen conservatively, site
specific correlations are used for design parameters and
there will be careful construction control.

5.0 Pile Types
5.1 Concrete piles
5.1.1 Precast concrete piles, including
prestressed piles, shall withstand without
damage or significant cracking, the stresses
arising from manufacture, handling and
transportation, in addition to those arising
from driving and imposed loadings.
5.1.2 Belled bases of cast-in-situ concrete piles shall be no less than 100 mm thick at the edge of the required base and, unless the bell is reinforced, the conical surfaces shall slope at an angle from the horizontal of no less than 60°.

5.2 Steel piles

5.2.1 The design of steel piles shall be based on the nett steel section after deducting an appropriate thickness for future loss by corrosion.

This verification method does not describe a means of determining the amount of corrosion and proposals must be submitted to the territorial authority for approval.

COMMENT:
The amount deducted needs to take account of the aggressiveness of the soil. Further guidance can be found in AS 2159 Section 6.3 or the HERA Design and Construction Bulletin No 46.

5.2.2 Allowance for corrosion loss need not be made for steel encased in concrete provided cover to the steel is no less than:

a) 30 mm for prestressed concrete,
b) 50 mm for precast concrete,
c) 75 mm for cast-in-situ concrete.

5.3 Timber piles

5.3.1 Timber piles shall comply with NZS 3605 or NZS 3603 as applicable, and be naturally durable or treated to the appropriate hazard level as recommended by NZMP 3640.

Table 3: Closely Spaced Piles, Design Lateral Resistance
Paragraph 4.6.1

<table>
<thead>
<tr>
<th>Pile spacing</th>
<th>% of isolated pile lateral resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0 x nominal pile width</td>
<td>100</td>
</tr>
<tr>
<td>1.0 x nominal pile width (palisade type wall)</td>
<td>25</td>
</tr>
</tbody>
</table>

Table 4: Strength Reduction Factors for Deep Foundation Design
Paragraph 4.7.1

<table>
<thead>
<tr>
<th>Method of assessment of ultimate geotechnical strength for load combinations not involving earthquake overstrength</th>
<th>Range of values of $\phi_{pe}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static load testing to failure</td>
<td>0.65 – 0.85</td>
</tr>
<tr>
<td>Static proof (not to failure) load testing</td>
<td>0.70 – 0.90</td>
</tr>
<tr>
<td>Static analysis using CPT (Cone Penetrometer Test) data</td>
<td>0.45 – 0.65</td>
</tr>
<tr>
<td>Static analysis using SPT (Standard Penetrometer Test) data in cohesionless soils</td>
<td>0.40 – 0.55</td>
</tr>
<tr>
<td>Static analysis using laboratory data for cohesive soils</td>
<td>0.45 – 0.55</td>
</tr>
<tr>
<td>Method of assessment of ultimate geotechnical strength for load combinations including earthquake overstrength</td>
<td>0.80 – 0.90</td>
</tr>
</tbody>
</table>
Appendix A (Informative)

A1.0 Site Investigation

A1.1 General

A1.1.1 No specific site investigation procedures are given in this document. The following information is provided for guidance only.

A1.1.2 The ground conditions at the building site should be investigated to the extent considered necessary, by a person with appropriate expertise and experience, to provide essential site data for design of the proposed building. Both preliminary and detailed investigations may need to be undertaken.

A1.2 Preliminary investigation

A1.2.1 The preliminary site assessment may include investigation of:

a) General land form, geology and any conditions likely to facilitate landslip, soil creep, shrinkage and expansion, or subsidence.

b) Information available from records of previous constructions, excavations, fillings, drains and concealed works, on and adjacent to the site.

c) History and behaviour of neighbouring buildings and details of their foundation types, depths and loadings.

d) Potential for flooding (see also NZBC E1) and seasonal changes of soil characteristics.

e) Seasonal, tidal or other natural groundwater changes.

f) Presence of corrosive soil, groundwater and effluents (see also F1/VM1).

A1.3 Detailed investigation

A1.3.1 Detailed investigation may include:

a) Test bores and excavations.

b) Visual inspection.

c) Laboratory and field testing of soil and rock samples.

d) Advice from other people with relevant expertise.

A1.4 Recording information

A1.4.1 The description of the foundation material should be recorded. A suitable method for describing soil and rock is contained in “Guidelines for the field description of soils and rocks in engineering use” published by the New Zealand Geotechnical Society.

A1.4.2 The site investigation record should include a site plan showing the locations of the test bores and excavations.
Appendix B (Informative)

B1.0 Serviceability Limit State Deformations (Settlement)

B1.0.1 No specific method is given for determining foundation settlement. The following information is provided for guidance only.

B1.0.2 Foundation design should limit the probable maximum differential settlement over a horizontal distance of 6 m to no more than 25 mm under serviceability limit state load combinations of NZS 4203: 1992, unless the structure is specifically designed to prevent damage under a greater settlement.

B1.0.3 The basis for analysing settlement should be stated in the design. The analysis shall pay due consideration to:

a) Size, shape and depth of the foundations,
b) Proximity and influence of proposed and existing foundations,
c) Variability of the ground,
d) The presence of compressive or expansive materials,
e) Rate of consolidation,
f) Groundwater level,
g) Extent of fill placed and ground removed when constructing the foundation, and
h) Likelihood of liquefaction, internal erosion, soil collapse or other special feature.
C1.0 Description of Wall, Limit States and Soil Properties

A gravity retaining wall backfilled with dry sand and founded on clay. In addition to static loading the effect of a horizontal earthquake acceleration of 0.20g is considered (there is no vertical acceleration component to the earthquake). The additional active thrust generated by the earthquake loading is assumed to act 0.6 times the height of the wall above foundation level. The proportions of the wall and the various soil properties are given in Figure C1.

There are six ultimate limit states to be considered:
1st  Short term bearing capacity failure of the foundation beneath the wall,
2nd  Short term sliding at foundation level,
3rd  Short term bearing capacity failure under earthquake loading,
4th  Short term sliding under earthquake loading,
5th  Long term bearing capacity failure of the foundation beneath the wall,
6th  Long term sliding at foundation level.

Short term analysis (both for the initial static loading of the foundation and the earthquake loading) is performed in terms of total stress and uses the undrained shear strength ($s_u$) of the clay whilst the long term analysis is done using effective stresses and uses the strength parameters $c'$ and $\phi'$ for the clay (see Paragraph 2.0.6). The thrust from the sand backfill is based on effective stresses and is the same for all cases (see Paragraph 2.0.7).

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<table>
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<tr>
<th>Clay Properties for Short Term, Total Stress Analysis</th>
<th>Clay Properties for Long Term, Effective Stress Analysis</th>
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<td>$s_u = 75$ kPa, $\gamma = 18$ kN/m$^3$</td>
<td>$c' = 12.5$ kPa, $\phi' = 25^\circ$, $\gamma' = 8.2$ kN/m$^3$</td>
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Dry sand: $\phi = 30^\circ$, $\gamma = 16$ kN/m$^3$, $\delta = 15^\circ$

---

Figure C1: Wall Details and Soil Properties for the Short Term, Long Term, and Earthquake Loading Cases
C2.0 Earth Pressure Coefficients

Active pressure coefficient for the sand backfill ($\delta = \phi/2$) 

$\alpha = 0.30$

Active pressure coefficient for the sand backfill under EQ (0.20g) 

$\alpha = 0.45$

Passive earth pressure coefficient for clay ($\phi = 25^\circ$ and $\delta = \phi/2$) 

$\phi_s = 3.50$

C3.0 Load Factors and Strength Reduction Factors

Load factor for dead loads that improve stability 

$\lambda = 1.0$

Load factor for static active earth thrusts 

$\lambda = 1.6$

Load factor for active earth thrusts under earthquake 

$\lambda = 1.0$

Strength reduction factor for static and EQ bearing failure ($\phi_{bc}$) 

$\phi_{bc} = 0.45$

Strength reduction factor for passive earth pressure ($\phi_{pp}$) 

$\phi_{pp} = 0.45$

Strength reduction factor for static and EQ sliding failure ($\phi_{sl}$) 

$\phi_{sl} = 0.80$

C4.0 Notation

The notation in Figure C2 is used to identify the weights and active thrusts on the wall, whilst in Figure C3 the actions on the foundation are shown.

Figure C2: Notation for the Wall Loads

![Diagram of wall loads and notation](image)
C5.0 Loadings

C5.1 Active thrusts

Unfactored active thrusts:

- Static active thrust per metre from backfill (kN/m) \( \times 16 \times 4 \) = 38.40
- Static active thrust per metre from surcharge \( \times 6 \times 4 \) = 7.20
- Total static active thrust per metre of wall \( \times 38.4 + 7.2 \) = 45.60
- EQ active thrust per metre from backfill \( \times 16 \times 4 \) = 57.60
- EQ active thrust per metre from surcharge \( \times 6 \times 4 \) = 10.80
- Total EQ active thrust per metre of wall \( \times 57.60 + 10.80 \) = 68.40
- Difference between static and EQ active thrust from backfill \( \times 57.60 - 38.40 \) = 19.20

Factored static active thrusts and find location of resultant:

- Factored static active thrust from backfill (kN/m) \( \times 1.6 \) = 61.44
- Factored static active thrust from surcharge \( \times 1.6 \) = 11.52
- Total factored active thrust per metre of wall \( \times 61.44 + 11.52 \) = 72.96

Take moments about heel to get location of active thrust (m)

\[ h_{pa, static} = \left( \frac{38.40}{3.0} + 7.2 \times 0.5 \right) \times 4/45.60 = 1.44 \]

Horizontal component of static thrust (kN/m) \( H = P_{ah, static} = 72.96 \times \cos15 = 70.47 \)

Vertical component of static thrust (kN/m) \( P_{av, static} = 72.96 \times \sin15 = 18.88 \)
Factor EQ active thrusts and find location of resultant:

Factored EQ active thrust ......................... 57.60 x 1 + 10.80 x 1 = 68.40

Take moments about heel to get location of active thrust (m)

\[ h_{pa,EQ} = \frac{38.40}{3.0} + 10.80 \times 0.5 + 19.20 \times 0.6 \times 4}{68.40} = 1.74 \]

Horizontal component of EQ thrust (kN/m) ................ P_{ah,EQ} = 68.40 \times \cos 15 = 66.07

Vertical component of EQ thrust (kN/m) ................ P_{av,EQ} = 68.40 \times \sin 15 = 17.70

---

**Figure C4:** Horizontal Components of the Active Thrust During Earthquake Loading

C5.2 Weights (load factor 1.0) and resultant vertical forces on the wall

Weight of wall stem ........................................... \( W_s = 3.6 \times 0.3 \times 25 = 27.00 \)

Weight of wall foundation .................................. \( W_f = 2.65 \times 0.4 \times 25 = 26.50 \)

Weight of backfill above the heel of the wall ............ \( W_{bf} = 3.6 \times 1.35 \times 16 = 77.76 \)

Vertical force from surcharge above heel ................. \( W_{sur} = 3.5 \times 1.35 = 4.73 \)

Static vertical force on foundation (kN/m)

\[ V_{static} = 18.88 + 27.00 + 26.50 + 77.76 + 4.73 = 154.87 \]

EQ vertical force on foundation \( k_v = 0.2 \) and \( k_n = 0 \) (kN/m)

\[ V_{EQ} = 17.71 + 27.00 + 26.50 + 77.76 + 4.73 = 153.69 \]
C5.3 Horizontal forces applied to the foundation

For the first, second, fifth and sixth ultimate limit states the horizontal force is:

\[ H = P_{ah\ static} = 70.47 \text{ kN/metre length of wall} \]

For the third and fourth ultimate limit states the inertia of the wall is added to the active thrust to determine \( H \):
- Horizontal inertia force on the weight of the wall stem (\( W_s \)) \( \ldots \ldots \ldots \ldots 27.00 \times 0.20 = 5.40 \)
- Horizontal inertia force on the weight of the wall foundation (\( W_f \)) \( \ldots \ldots \ldots \ldots 26.50 \times 0.20 = 5.30 \)
- Horizontal inertia force on the weight of fill over heel (\( W_{bf} \)) \( \ldots \ldots \ldots \ldots 77.76 \times 0.20 = 15.55 \)
- Horizontal inertia force on the surcharge over heel (\( W_{sur} \)) \( \ldots \ldots \ldots \ldots 4.73 \times 0.20 = 0.95 \)

\[ H_{inertia} = 5.40 + 5.30 + 15.55 + 0.95 = 27.20 \]

Take moments about heel to get location of resultant inertia force (m):

\[ h_{inertia} = \frac{(15.55 + 5.40) \times (0.4 + 3.6/2) + 5.30 \times 0.2 + 0.94 \times 4}{27.20} = 1.87 \]

\[ H_{EQ} = P_{ah\ EQ} + H_{inertia} = 66.07 + 27.20 = 93.27 \text{ kN/metre of wall} \]

The weights of the various components of the wall and the horizontal inertia forces generated by the earthquake horizontal acceleration of 0.20g are given in Figure C5.

**Figure C5:** Weights of the Wall Components, Horizontal 0.20g Inertia Forces and the Location of the Resultant Inertia Force

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C6.0 Surcharge Pressures at Toe

Total stress surcharge pressure in front of wall (kPa) \( \ldots \ldots \ldots \ldots q = \gamma_{\text{clay}}D_t = 18 \times 0.4 = 7.2 \)

Effective stress surcharge pressure in front of wall (kPa) \( \ldots \ldots \ldots \ldots q' = \gamma_{\text{clay}}'D_t = 8.2 \times 0.4 = 3.3 \)
C7.0 First Ultimate Limit State (short term static foundation bearing failure)

Find X (location of V) by taking moments about heel.

Moment of the vertical forces:

\[(77.76 \times 1.35/2 + 4.73 \times 1.35/2 + 27.00 \times (1.35 + 0.15) + 26.50 \times 2.65/2) = 131.29\]

\[X = 131.29/154.87 = 0.848\]

Eccentricity: \[e = 70.47 \times 1.44/154.87 = 0.655\]

\[B1' = 2 \times (0.848 + 0.655) = 3.01\]

\[B2' = 2 \times (2.65 - 0.848 - 0.655) = 2.29\]

\[B' \text{ is the smaller of } B1' \text{ and } B2':\]

\[B' = 2.29 \text{ (Distance from R to foundation edge) = } B'/2 = 1.15 > B/6 \text{ .} \quad \text{ok (Paragraph 3.1.4)}\]

Design bearing pressure: \[q_d = V/B' = 154.87/2.29 = 67.6 \text{ kPa}\]

Determine ultimate bearing strength \[q_u = \frac{q_u}{H9261} = \frac{s_u\lambda_c\lambda_d\lambda_a\lambda_g N_c + \lambda_g q}{H9261}\]

For this case \[\phi = 0\], so \[N_c = 5.14\]. \[\lambda_c\] shall be taken as 1.0 as foundation is assumed to be long compared to its width. Also \[\lambda_g = \lambda_q = 1.0\] as the foundation is horizontal. Thus we need only to evaluate \[\lambda_{cd}\] and \[\lambda_{ci}\].

\[\lambda_{cd} = 1 + 0.4 \times D_f/B' = 1 + 0.4 \times 0.4/2.29 = 1.07\]

\[\lambda_{ci} = 0.5(1 + \sqrt{(1 - B/6s_p})) = 0.5 \times (1 + \sqrt{(1 - 70.47/2.29 \times 75)}) = 0.88\]

\[q_u = s_u N_c\lambda_d\lambda_c\lambda_g = 75 \times 5.14 \times 1.07 \times 0.88 + 7.2 = 370.19\]

\[q_{dbs} = q_u\Phi_{ci} = 370.19 \times 0.45 = 166.6 \quad q_u = 67.6 \quad \text{Thus OK as } q_{dbs} > q_u\]

**Figure C6:** Wall and Foundation Loads for the First and Second Ultimate Limit States
**C8.0 Second Ultimate Limit State (short term static foundation sliding failure)**

The design sliding resistance is derived from the shear strength on the base and the passive resistance from the clay in front of the embedded part.

Ultimate shear strength: \( S = s_u B' = 75 \times 2.29 = 171.75 \)

Passive resistance: \( P_p = 2 s_u T_f + 0.5 \gamma_{clay} T_f^2 \text{ where } T_f \text{ is the foundation thickness} \)
\[ = 2 \times 75 \times 0.4 + 0.5 \times 18 \times 0.4^2 = 61.44 \]

Design sliding resistance: \( S \Phi_{sl} + P_p \Phi_{pp} = 171.75 \times 0.8 + 61.44 \times 0.45 = 165.1 \)

\( H = 70.5 \)

Thus OK as \( S \Phi_{sl} + P_p \Phi_{pp} > H \)

**C9.0 Third Ultimate Limit State (short term foundation bearing failure under EQ)**

Find \( X \) (location of \( V \)) by taking moments of vertical forces about heel.

Moment, as for the first ultimate limit state = 131.29

\( X = 131.29/153.69 = 0.854 \)

To get eccentricity, we need to add the moment of the horizontal inertia forces to that of the lateral thrust from the backfill:

\( e = (66.07 \times 1.74 + 27.20 \times 1.87)/153.69 = 1.079 \)

\( B' = 2 \times (0.854 + 1.079) = 3.87 \)

\( B' = 2 \times (2.65 - 0.854 - 1.079) = 1.43 \)

\( B' \) is the smaller of \( B'1 \) and \( B'2 \):

\( B' = 1.43 \) (Distance from \( R \) to foundation edge = \( B'/2 = 0.72 > B/6 \) \( \therefore \) ok (Paragraph 3.1.4)

Design bearing pressure: \( q_d = V/B' = 153.69/1.43 = 107.46 \text{ kPa} \)

---

**Figure C7:** Wall and Foundation Loads for the Third and Fourth Ultimate Limit States

---

Virtual back of wall

\( V_{EQ} = 153.69 \text{ kN} \)

\( P_{an, EQ} = 66.07 \text{ kN} \)

\( h_{inertia} = 1.87 \text{ m} \)

\( h_{pa, EQ} = 1.74 \text{ m} \)

\( q_d = 107.5 \text{ kPa} \)

\( B' = 1.43 \text{ m} \)

\( (H_{EQ} = P_{an, EQ} + H_{inertia} = 93.27 \text{ kN}) \)
Determine ultimate bearing strength:
For this case $\phi = 0$, so $N_c = 5.14$. $\lambda_{cs}$ shall be taken as 1.0 as foundation is assumed to be long compared to its width. Also $\lambda_{cg} = \lambda_{cG} = 1.0$ as the foundation is horizontal. Thus we need only to evaluate $\lambda_{cd}$ and $\lambda_{ci}$.

$$
\lambda_{cd} = 1 + 0.4 x \frac{Df}{BI} = 1 + 0.4 x 0.4/1.43 = 1.11 \\
\lambda_{ci} = 0.5(1 + \sqrt{1 - \frac{H}{B's_{u}}}) = 0.5 x (1 + \sqrt{1 - 93.29/1.43 x 75}) = 0.68
$$

$$
q_u = s_u N_c \lambda_{cd} \lambda_{ci} + q = 75 x 5.14 x 1.11 x 0.68 + 7.2 = 298.17
$$

$$
q_{bs} = q_u \Phi_{bc} = 298.17 x 0.45 = 134.2 \quad q_d = 107.5 \quad \text{Thus OK as } q_{bs} > q_d
$$

C10.0 Fourth Ultimate Limit State (short term foundation sliding failure under EQ)

The design sliding resistance is derived from the shear strength on the base and the passive resistance from the clay in front of the embedded part.

Ultimate shear strength: $S = s_u B' = 75 x 1.43 = 107.25$

Passive resistance: $P_p = 2 s_u T_r + 0.5 \gamma_{clay} T_f^2 = 2 x 75 x 0.4 + 0.5 x 18 x 0.4^2 = 61.44$

Design sliding resistance: $S \Phi_{si} + P_p \Phi_{pp} = 107.25 x 0.8 + 61.44 x 0.45 = 113.5$

$H = 93.3 \quad \text{Thus OK as } S \Phi_{si} + P_p \Phi_{pp} > H$

C11.0 Fifth Ultimate Limit State (long term foundation bearing failure)

For this case we work in terms of effective stress.

The strength parameters for the clay become: $c' = 12.5$ kPa and $\phi' = 25^\circ$. Furthermore, the water table is at the ground surface in front of the wall so the buoyant density ($18 - 9.81 = 8.2$ kN/m$^3$) controls the effective stresses.

![Figure C8: Wall and Foundation Loads for the Fifth and Sixth Ultimate Limit States](ARCHIVED)
In addition there is a small positive water pressure acting on the underside of the wall which reduces the vertical load applied to the foundation.

\[ u = 0.4 \times 9.81 = 3.92 \] and \[ V_{\text{drained}} = 154.87 - 3.92 \times 2.65 = 144.48 \]

This has the effect of changing slightly \( X \) and \( e \), hence \( B' \) and \( q_d \). We have from the first ultimate limit state the moment about the heel of the wall of the vertical forces = 131.29 kNm per metre length of the wall, so:

\[ X = (131.29 - 3.92 \times 2.65 \times 2.65/2)/144.48 = 0.813 \]

Eccentricity of load:

\[ e = 70.47 \times 1.44/144.48 = 0.703 \]

Design bearing pressure:

\[ V_{\text{drained}} / B' = 144.5/2.27 = 63.7 \text{ kPa} \]

For \( \phi \) equal to 25° the bearing capacity factors are: \( N_c = 21 \), \( N_q = 11 \) and \( N_i = 9 \).

Determine ultimate bearing strength:

\[ q_{d,\text{drained}} = c_{\text{drained}} N_c + q'_{\text{drained}} N_q + 0.5 B' \gamma_{\text{dr}} \gamma_{\text{ch}} \gamma_{\text{g}} N_i \]

Shape factors \( \lambda_{cs}, \lambda_{qp}, \) and \( \lambda_{qs} \) shall be taken as 1.0 as foundation is assumed to be long compared to its width. Also ground inclination factors \( \lambda_{cg}, \lambda_{qg}, \) and \( \lambda_g = 1.0 \) as the foundation is horizontal. Thus we need only to evaluate depth and load inclination factors.

Depth factors:

\[ \lambda_{qd} = 1 + 2 \tan \phi (1 - \sin \phi)(D_f / B) = 1 + 2 \tan(25)(1 - \sin(25)) \times (0.4/2.27) = 1.05 \]

Load inclination factors:

\[ \lambda_{ci} = (1 - 0.7 H / (V_{\text{drained}} + c_{\text{dr}} B \tan \phi))^{1/3} = 0.46 \]

\[ \lambda_{si} = (1 - H / (V_{\text{drained}} + c_{\text{dr}} B \tan \phi))^{1/3} = 0.28 \]

\[ q_{d,\text{drained}} = c_{\text{drained}} N_c + q'_{\text{drained}} N_q + 0.5 B' \gamma_{\text{dr}} \gamma_{\text{ch}} \gamma_{\text{g}} N_i \]

\[ = 12.5 \times 21 \times 0.46 + 3.3 \times 11 \times 0.46 + 0.5 \times 9 \times 2.27 \times 8.2 \times 0.31 = 152.70 \]

\[ q_{d,\text{drained}} = 152.70 \times 0.45 = 68.7 \quad q_d = 63.7 \]

Thus OK as \( q_{d,\text{drained}} > q_d \)

**C12.0 Sixth Ultimate Limit State (long term foundation sliding failure)**

The design sliding strength is derived from the sliding resistance on the base and the passive resistance from the clay in front of the embedded part.

Sliding resistance:

\[ S_{\text{drained}} = c' B' + V_{\text{drained}} \tan \phi = 12.5 \times 2.27 + 144.48 \times \tan(25) = 95.75 \]

Passive resistance:

\[ P_{p,\text{drained}} = 0.5 K_p \gamma T_i^2 + 2 c T_i \sqrt{K_p} \]

\[ = 0.5 \times 3.5 \times 8.2 \times 0.4^2 + 2 \times 12.5 \times 0.4 \times \sqrt{3.5} = 21.00 \]

Design sliding strength:

\[ S \phi_i + P_p \phi_p = 95.75 \times 0.8 + 21.00 \times 0.45 = 86.05 \]

\[ H = 70.5 \]

Thus OK as \( S \phi_i + P_p \phi_p > H \)
C13.0 Comments

The above calculations reveal that, for static loading, it is the long term case that is critical. Also for the short term cases the sliding strength derived from passive earth pressure in front of the embedded foundation is significant.

If the horizontal earthquake acceleration is increased much above 0.2g the third ultimate limit state becomes the limiting case as bearing failure is initiated. However, as explained in clauses 4.11.2.4 and C4.11.2.4 of NZS 4402: 1992, controlled sliding and tilting of the foundation during the passage of an earthquake is possible if the resulting post-earthquake permanent displacements are acceptable. The procedures and criteria for this approach are beyond the scope of this document.
Acceptable Solution B1/AS4

Foundations

(Revised by Amendment 4)

No specific acceptable solution for foundations has been adopted for complying with the Performances of NZBC B1.
All references to Verification Methods and Acceptable Solutions are preceded by **VM** or **AS** respectively.

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