6.0 Site stability, foundations, earthworks and retaining walls

In this section

6.1 Scope and performance requirements

6.1.1 General

This section presents design philosophy and design criteria for the:

- assessment of slope stability, liquefaction and lateral spreading in earthquakes
- design of mitigation measures for liquefaction and slope instability
- design of foundations
- design of embankments and cut and fill slopes
- design of earth retaining structures including mechanically stabilised earth (MSE).

It includes assessment and design under both non-seismic conditions and earthquake shaking and specifies minimum post-earthquake performance standards.

Acceptable methods of assessment are stated within this section. Alternative methods may be adopted subject to road controlling authority acceptance, but shall be suitably established, internationally recognised, widely used methods and shall be noted in the earthworks design report, structure options report or structure design statement.

The term "soil structures" used in this section includes cut and fill slopes (including stabilised slopes), embankments, retaining walls and earth retaining structures (including MSE).

The term "bridge" shall include bridges, major sign gantries and major culverts.

In this manual, as well as the serviceability limit state, the terminology adopted for the limit states relevant to earthquake resistance are the damage control limit state and the collapse avoidance limit state as outlined in 5.1.1.

6.1.2 Performance requirements

a. Settlement and displacement limits

Settlement and displacement limits that shall be applied to the performance of bridges and soil structures are given in table 6.1. The maximum settlement and horizontal displacement limits apply to the total cumulative settlements and displacements due to all actions including static soil pressures, inertial response to seismic actions, and ground movement due to liquefaction and/or slope instability.

In addition to the settlement limits, bridges shall also satisfy the performance requirements of 5.1.2 and 6.1.2(c), which may be more limiting.

The effects of 200% of the seismic displacement arising from one damage control limit state (DCLS) design intensity earthquake and, separately, of a collapse avoidance limit state (CALS) event displacement on any affected bridge structures shall be assessed and compared against the performance criteria specified in 5.1.2(c) for the CALS major earthquake event.

b. Serviceability limit state following a seismic event

Soil structures shall satisfy the following serviceability limit state requirements:

i. Where the serviceability of structures (bridges, major culverts, major sign gantries, etc) is dependent on, or influenced by associated or adjacent soil structures, the soil structures shall be designed to ensure that their performance does not deleteriously affect the structure from satisfying its serviceability requirements, as specified elsewhere within this manual.

ii. All soil structures associated with roads shall remain undamaged with no detriment to the road following earthquake events with an annual exceedance probability of 1/25.

6.1.2 continued

    iii. The operational continuity of routes shall not be significantly impeded following earthquake events of relatively high annual exceedance probabilities. The road controlling authority shall be consulted and should define the operational performance expectations for the section of road to be designed, taking into consideration the redundancy in the regional road network, and the resilience required for the proposed road to ensure the desired functionality of the road network. This should provide the access resilience expectations in terms of degree of access required on the road after different levels of events and the time for restoration of access.

Table 6.1: Total settlement, differential settlement and horizontal displacement limits

Structure, wall and slope scenario	Structure, retaining structure and slope type	SLS load combinations (including seismic events detailed in 6.1.2(b)(iii))		DCLS load combinations		
		Maximum total settlement	Maximum differential settlement	Maximum total settlement	Maximum differential settlement	Maximum total horizontal displacement*
Simply supported bridge	All types	Refer to 2.1.8†		100mm†	1/300†	N/A
Continuous bridge	All types			$Z<0.4$: 40mm†‡ $Z\geq0.4$: 50mm†‡	1/500†	N/A
Soil structure supporting or containing bridge abutments or piers	All types	As per bridge structure†				$Z<0.3$: 25mm‡ $0.3\leq Z<0.4$: 50mm‡ $Z\geq0.4$: 100mm‡
Soil structures above road level supporting structures belonging to other parties within 2H§ of any wall face at the top of the wall or bottom of the slope	All types	30mm	1/500	Refer to table 6.2		25mm
Soil structures supporting road carriageway with AADT** ≥ 2500	Rigid wall	50mm†	1/500†	Refer to table 6.2		100mm
	Flexible wall or slope capable of displacing without structural damage	50mm††	1/100††			150mm
Soil structures supporting road carriageway with AADT < 2500	Rigid wall	50mm†	1/300†	Refer to table 6.2		100mm
	Flexible wall or slope capable of displacing without structural damage	50mm††	1/100††			200mm

Notes:

*   The designer shall ensure that the displacements will not cause damage to adjacent structures or services.

†   Any settlement of bridges or rigid walls occurring prior to pilecap or foundation construction may be neglected.

‡   Subject to also satisfying the requirements of 6.6.9(b).

§   H is the height of the retaining structure including the height of any slope above, or the height of the slope.

**   AADT is the annual average daily traffic count.

††   Any settlement of flexible walls or slopes occurring prior to pavement construction or utilities installation may be neglected.

6.1.2 continued

The following default values are provided in the absence of such considered definition:

- 1/100 for routes of importance level 4, importance level 3+ and importance level 3 (see tables 2.2 and 2.3)
- 1/50 for routes of importance level 2
- 1/25 for minor routes of importance level 1.

Operational continuity is defined as:

- full live load capacity is maintained
- the road shall be useable by emergency traffic (as defined in 5.1.2)
- full vehicle access is restorable within 24 hours
- any necessary repairs shall be of such a nature that they can be completed within one month.

c. Damage control limit state following a seismic event

Soil structures not affecting bridges or major culverts shall satisfy the performance requirements given in table 6.2 when subjected to their design annual probability of exceedance earthquake event.

Soil structures affecting bridges or major culverts shall be designed to ensure that their performance does not deleteriously affect the structure from satisfying the requirements of 6.1.2(a).

Table 6.2: Seismic performance requirements for soil structures not affecting bridges after a design (DCLS) event

Post-earthquake – immediate	Slope stability factor of safety (FoS) > 1.1 for post-seismic stability with residual shear strengths and zero peak ground acceleration
Post-earthquake function – short term	Usable by emergency traffic (as defined in 5.1.2) and capable of supporting two lanes of HN (normal) loading with a slope stability FoS > 1.3 under static strength conditions
Post-earthquake function – after reinstatement	Feasible to reinstate for all design (ULS/DCLS) level actions
Acceptable damage	a. Damage possible: capable of permanent repair. b. The detailing of barriers, foundations formed within or upon soil structures and facing panels should be such that the predicted DCLS displacements do not result in damage to these elements beyond repair. Necessary reinstatement works should be limited to removal of facing panels and barriers, reconstruction of panel footings, reinstatement of barriers and panels to original levels or other lessor level acceptable to the road controlling authority and reconstruction of road pavement.

In addition to the structure performance requirements of 5.1.2 the following performance requirements shall also be met by structures exposed to the effects of liquefaction and site instability resulting from a damage control limit state design intensity earthquake:

i. the design specified required clearances over underlying roads, railway lines and design flood levels shall be maintained or be able to be readily reinstated without deleteriously affecting the functioning of underlying infrastructure

6.1.2 continued

ii. an acceptable quality of road surface rideability and vertical alignment, suitable for the design speed of the road, shall be maintained or be able to be readily reinstated. Any application of an overlay shall not reduce the structure's live load capacity to below the specified required design live load capacity or erode the performance of concrete barrier systems that have been constructed integral with the bridge deck

iii. the movement capacity of elements of the structure designed to accommodate seismic, thermal, and shrinkage and creep movements within the structure shall retain or be able to readily have reinstated sufficient movement capacity to accommodate the movements due to one further damage control limit state design intensity earthquake in addition to the design movements due to thermal, shrinkage and creep effects.

d. Departures from the specified standards

Where settlement and/or horizontal displacement limits are impractical or uneconomic to satisfy, or where the cost of ground improvement or other mitigation measures is very high in comparison with the project or structure cost, consideration should be given to making a request to the road controlling authority for accepting a lower design standard. Factors to be taken into account in making this request include:

– the route importance

– the value of the structure

– the likelihood and extent of disruption to the route and the consequences of the route disruption

– the ground improvement or other mitigation cost

– the extent to which the performance requirements will be satisfied by the proposed solution

– consequences of lower design standards and poorer performance of the structure.

In making a request for a lower design standard it should be noted that any compromise on the requirement for a structure to not collapse after a CALS major earthquake event (see 5.1.2(c)) is unlikely to be accepted.

Where a departure request involves displacements of structures or parts of structures exceeding the specified standards, there is likely to be a requirement that:

– the superstructure remains elastic in the design (DCLS) seismic event

– the superstructure is able to be jacked and relevelled to the original design levels

– major reconstruction of primary substructure elements shall not be required

– after the earthquake event, the bridge shall be usable by emergency traffic (as defined on 5.1.2)

– after reinstatement, the design level actions to be accommodated shall include the effects of any permanent deformation of the structure.

6.2 Design loadings and analysis

6.2.1 General

Design loads to be considered shall be as specified in section 3 of this manual. In particular, earth loads are specified in 3.4.12 and load combinations in 3.5.

6.2.2 addendum

Recent research (Cubrinovski et al[64] and Bradley et al[65]) indicates that for some locations the peak ground accelerations (PGA) and effective magnitudes detailed herein are significantly lower than the PGAs and mean earthquake magnitudes obtained from site-specific probabilistic seismic hazard analysis (PSHA). Consequently, the following interim measures shall be adopted whilst relevant research continues.

For projects in the six "principal locations" (ie Gisborne, Wellington, Palmerston North, Napier, Whanganui and Blenheim) and their neighbouring regions ("locations") identified in Cubrinovski et al[64] figure 11 where the value of the project soil structures and earthworks to be designed for earthquake resistance is:

- greater than or equal to $3.5 million – a site-specific seismic hazard study shall be undertaken as a special study, as per this clause 6.2.2

- less than $3.5 million – PGA and magnitude values shall be obtained from Cubrinovski et al[64] table 1, unless a site-specific seismic hazard study is undertaken.

Note that the PGAs and magnitudes given in this section have been developed specifically for the design of foundations, earthworks, soil structures and slopes associated with highway structures. They have not been developed for any other purposes and therefore such use is at the user's risk.

6.2.2 Earthquake loads and analysis for the assessment of liquefaction and of the stability and displacement of soil structures

The design earthquake loading to be applied to soils, rock and independent soil structures shall be derived as set out herein.

Methods for the assessment of liquefaction, slope stability, and soil structure displacements referred to within this section require the application of peak ground accelerations in combination with a corresponding earthquake magnitude. The peak ground accelerations (PGA) to be applied shall be 'unweighted' and derived for the relevant return period as follows:

$$PGA = C_{0,1000} \times \frac{R_u}{1.3} \times f \times g$$

Where:

$C_{0,1000}$ = 1000 year return period PGA coefficient for a subsoil Class A or B rock site or Class C shallow soil site derived from figure 6.1(a), or for subsoil Class D deep or soft soil site or Class E very soft soil site from figure 6.1(b). Alternatively, for the locations listed, PGA coefficients may be taken from table C6.1 contained in C6 in the *Bridge manual commentary*

R_u = DCLS return period factor derived from table 3.5 of NZS 1170.5 *Structural design actions* part 5 Earthquake actions – New Zealand[1], or table 5.3 of this manual as appropriate, corresponding to the design return period determined from tables 2.2 or 2.3, as appropriate (see 5.2.2(b))

f = Site subsoil class factor, where

f = 1.0 for a Class A, B, D and E soil sites

f = 1.33 for a Class C shallow soil site[*]

The earthquake magnitude shall be derived for the relevant return period from figures 6.2(a) to (f) or table C6.1 contained in C6 in the *Bridge manual commentary*[†].

As a lower bound, the damage control limit state effects to be designed for shall not be taken to be less than those due to a 6.5 magnitude earthquake at 20km distance, for which the peak ground acceleration coefficients shall be derived from table 6.3.

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[*] Note that this is the 1.33 factor referred to in the notes to figure 6.1(a) and table C6.1 to determine $C_{0,1000}$ for a Class C shallow soil site. This factor should not be applied twice when calculating PGA for a Class C shallow soil site using the equation above.
[†] The relationship between PGAs and effective magnitudes is under review. Consequently the figure 6.2 series of maps have not yet been updated to reflect all return periods in tables 2.1 to 2.3. Table C6.1 should therefore be used where necessary as an interim measure.

6.2.2 continued

Table 6.3: Peak ground acceleration coefficients corresponding to a magnitude 6.5 earthquake at 20 km distance

Site subsoil class	Class A/B rock	Class C shallow soil	Class D deep or soft soil	Class E very soft soil
PGA coefficient (g)	0.14	0.19	0.16	0.16

Note that PGAs derived using NZS 1170.5[1] are magnitude weighted to correspond to an earthquake magnitude of 7.5. Given that the performance of soils, earth structures, slopes and retaining walls exhibit a step-wise behaviour (where a critical acceleration results in a sudden loss of stability, ie dramatic change in behaviour), use of these values may be unconservative. Therefore unweighted PGAs are to be used in the assessment and design of these soil structures for earthquakes.

Unweighted PGAs are to be derived as specified herein. They are not to be back-calculated from NZS 1170.5[1] magnitude weighted PGAs as doing so will give rise to inconsistencies due to the approximations that are inherent in the NZS 1170.5[1] site hazard spectra.

Caution should be exercised in the use of the effective magnitudes from figures 6.2(a) to (f) or table C6.1 contained in C6 in the *Bridge manual commentary* as appropriately matching peak ground accelerations with earthquake magnitudes, representing the duration of shaking, lacks precision. Peak ground accelerations attenuate rapidly with distance and so their estimation is influenced most by local sources. Shaking duration may often be indicated in spectra by longer period components of motion, which are often contributed to most by larger magnitude more distant earthquakes.

For liquefaction analysis, an earthquake magnitude should be used in combination with the PGA.

Dependent on the value of the project soil structures and earthworks to be designed for earthquake resistance, a site-specific seismic hazard study shall be undertaken as a special study, as follows:

- less than $3.5 million – a site-specific seismic hazard study is not required
- $3.5 million to $8.5 million – a site-specific seismic hazard study is advisable
- more than $8.5 million – a site-specific seismic hazard study is mandatory (values at December 2021*)

Where the site is formed by potentially liquefiable materials, Waka Kotahi NZ Transport Agency may instruct the designer to carry out a site-specific seismic hazard study for projects with values of less than $8.5 million, especially for soil structures with importance levels 3 and 4.

Deaggregation of seismic hazard shall be carried out as part of a site-specific seismic study. The individual sources contributing the most to the seismic hazard of the site should be considered. The PGA and magnitude values representing realistic ground motions that could actually occur at the site due to known active faults in the area should be used in the assessment of liquefaction. This process may yield more than one magnitude-PGA pair for liquefaction analysis in some areas of New Zealand. Each magnitude-PGA pair should be evaluated individually in the liquefaction analysis. If liquefaction is estimated for any given magnitude-PGA pair, the evaluation of that pair should be continued through the slope stability and lateral deformation evaluation processes.

The effects to be designed for shall not be less severe than those due to the lower bound event of a magnitude 6.5 earthquake at 20km distance.

Where site-specific seismic hazard studies are undertaken as permitted or required for any of the cases described above, such studies shall comply with the requirements of 5.2.5, except that magnitude weighting shall not be applied.

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* Values shall be adjusted to current value. For the relevant cost adjustment factor refer to the Waka Kotahi NZ Transport Agency *Procurement manual*, Procurement manual tools, Infrastructure and public transport contract price adjustment indexes, Latest index values for infrastructure cost indexes – original 1991 series, Waka Kotahi NZ Transport Agency Construction index[2]

6.2.2 continued

Loads derived in accordance with this clause are also applicable to the design of MSE walls and slopes for both their external and internal stability and for the design of their facing panels.

Bridge foundations and soil structures shall be checked for stability using the relevant damage control limit state load combinations specified in table 3.3 and the appropriate strength reduction factors.

The stability of the supporting ground to bridge structures (either slopes or retained ground) shall be such that:

- the performance requirements specified in 5.1.2 for a minor earthquake, a design level earthquake and a major earthquake are satisfied
- the collapse of bridge structures is avoided under a collapse avoidance limit state (CALS) event.

Where a site-specific seismic hazard study has been undertaken, the magnitude associated with maximum considered motions (ie those associated with 1.5 times the design peak ground accelerations derived as above) shall be adopted as the CALS event. In the absence of such a study, the figure shown in the relevant coloured region of figure 6.3 shall be adopted as the default CALS event magnitude to be used in conjunction with 1.5 times the design peak ground accelerations.

For the assessment of the seismic performance of soil structures, and in the assessment of liquefaction potential, a structural performance factor (S_p) or any other reduction factor shall not be applied to the design earthquake loads unless otherwise specified herein.

Where time-history analysis is applied to the analysis of the response of features and structures falling within the scope of this section 6, the requirements pertaining to the application of time-history analysis set out in 5.3.11(d) shall be complied with.

6.2.3 Design earthquake loads for the structural design of earth retaining structures

a. Design earthquake loads given in this clause should not be used for the assessment of liquefaction, lateral spreading, deep-seated instability, seismic settlements and displacements of soil structures. Seismic loads for these cases are given in 6.2.2.

For the structural design of earth retaining structures comprised of concrete, steel or timber elements, the design horizontal ground acceleration to be used in computing seismic inertia forces of non-integral abutments and independent walls and of the soil acting against them shall be as follows:

Design acceleration $\qquad C_o g = C_h(T_0) Z R_u g$

Where:

C_o	=	design ground acceleration coefficient
g	=	acceleration due to gravity
$C_h(T_0)$	=	spectral shape factor at T=0 from NZS 1170.5[1] table 3.2, (given in brackets in the table), for the appropriate site subsoil category
Z	=	hazard factor, determined from 5.2.2 and NZS 1170.5[1] clause 3.1.5
R_u	=	DCLS return period factor from 5.2.2(b)

For the damage control limit state, the product ZR_u shall not be taken as less than 0.13.

Non-integral abutments and independent walls are defined in 6.6.1(a).

b. All structural components of abutments and walls shall have a design strength not less than the forces calculated using the relevant ultimate limit state/damage control limit state load combinations specified in 3.5.

Figure 6.1(a): Unweighted peak ground acceleration coefficients, $C_{0,1000}$, corresponding to a 1000 year return at a subsoil Class A or B rock site or Class C shallow soil site

Note: For Class C sites a scale factor of f=1.33 needs to be applied to the PGA coefficients derived from this figure (see 6.2.2).

Figure 6.1(b): Unweighted peak ground acceleration coefficients, $C_{0,1000}$, corresponding to a 1000 year return at a subsoil Class D or E deep or soft soil site

Figure 6.2(a): Effective magnitudes for use with unweighted peak ground accelerations (2500 year return period)
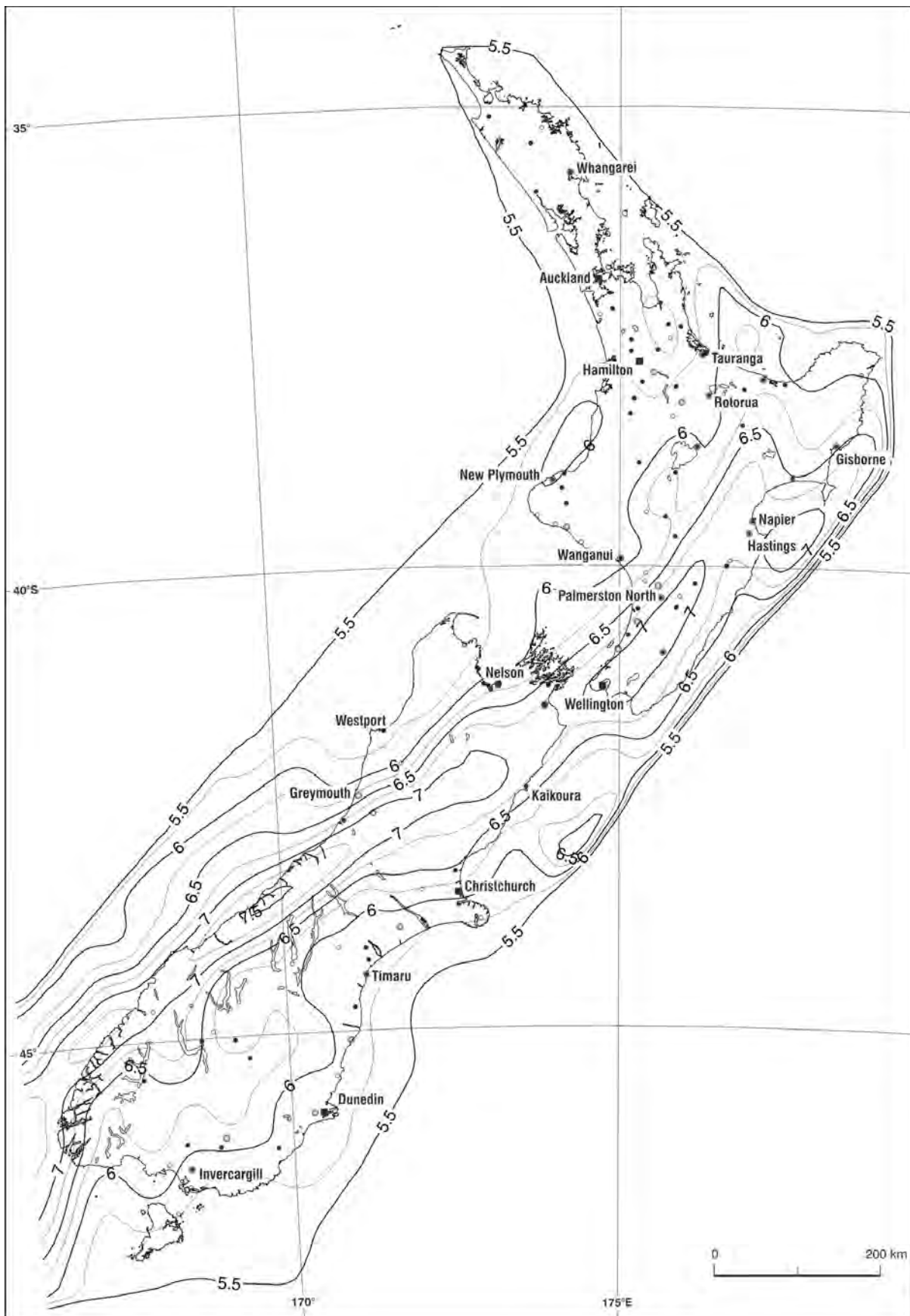
Figure 6.2(b): Effective magnitudes for use with unweighted peak ground accelerations (1500 year return period)

Figure 6.2(c): Effective magnitudes for use with unweighted peak ground accelerations (1000 year return period)

Figure 6.2(d): Effective magnitudes for use with unweighted peak ground accelerations (500 year return period)

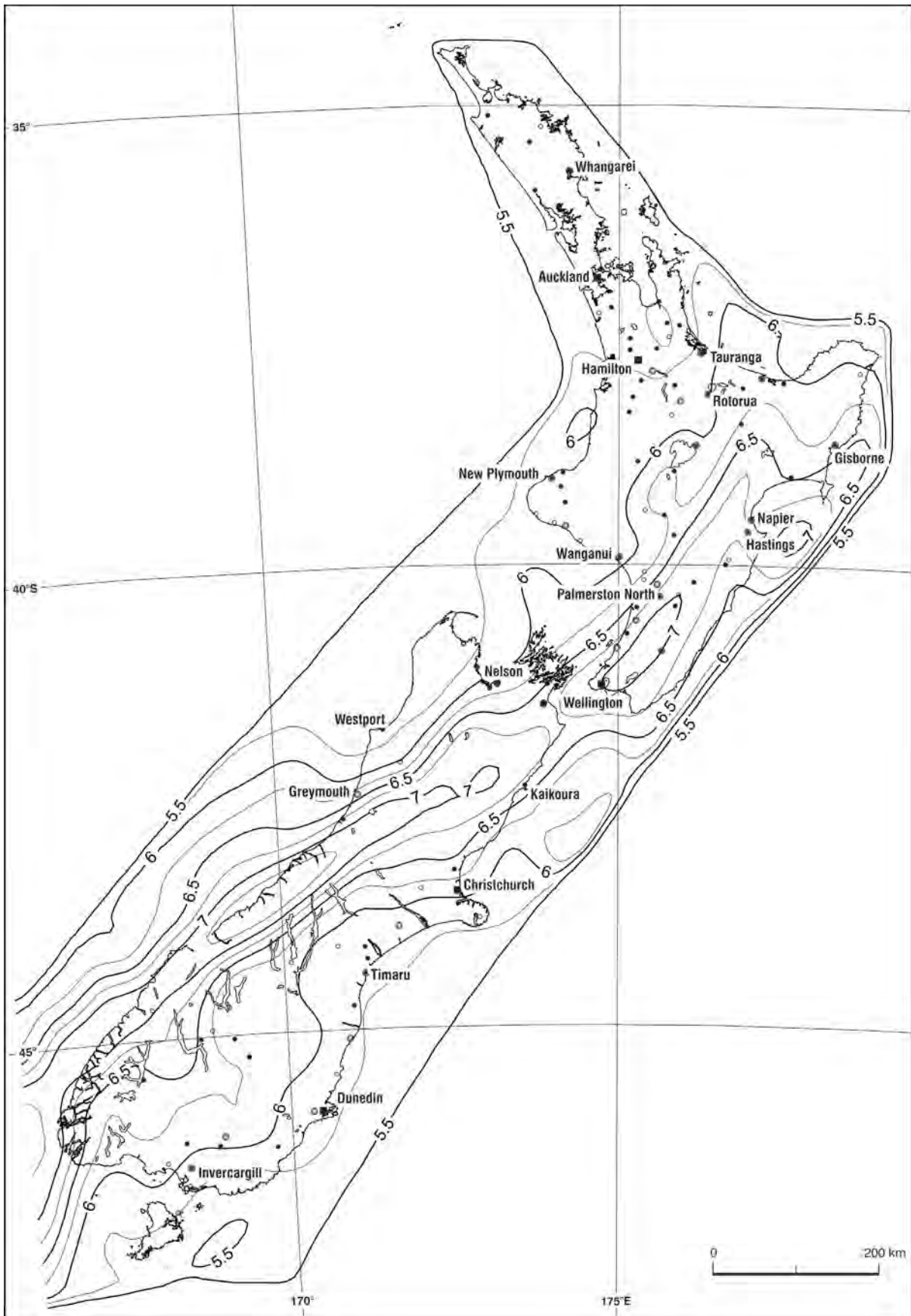**Figure 6.2(e):** Effective magnitudes for use with unweighted peak ground accelerations (100 year return period)

Figure 6.2(e): Effective magnitudes for use with unweighted peak ground accelerations (100 year return period)

Figure 6.2(f): Effective magnitudes for use with unweighted peak ground accelerations (50 year return period)
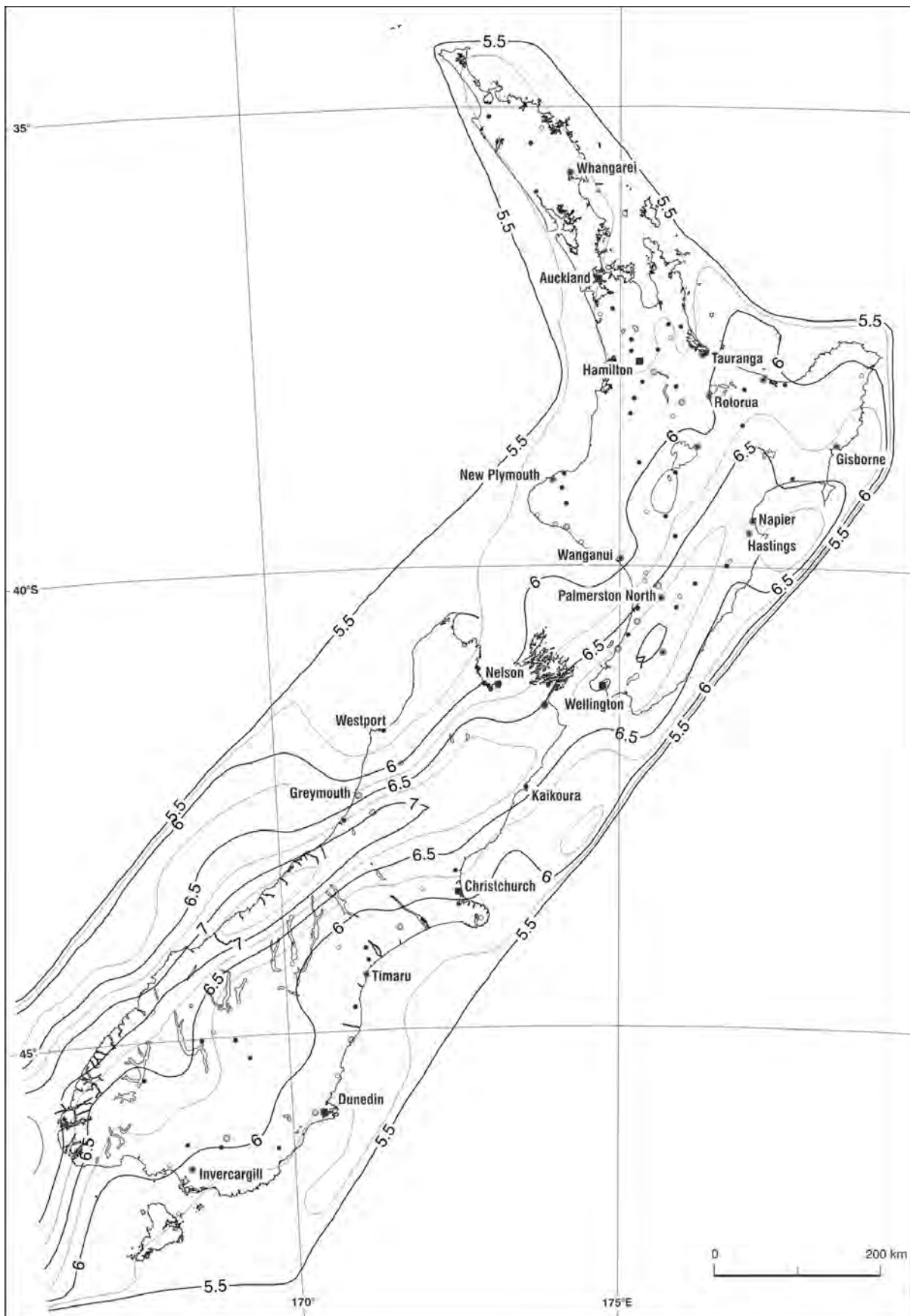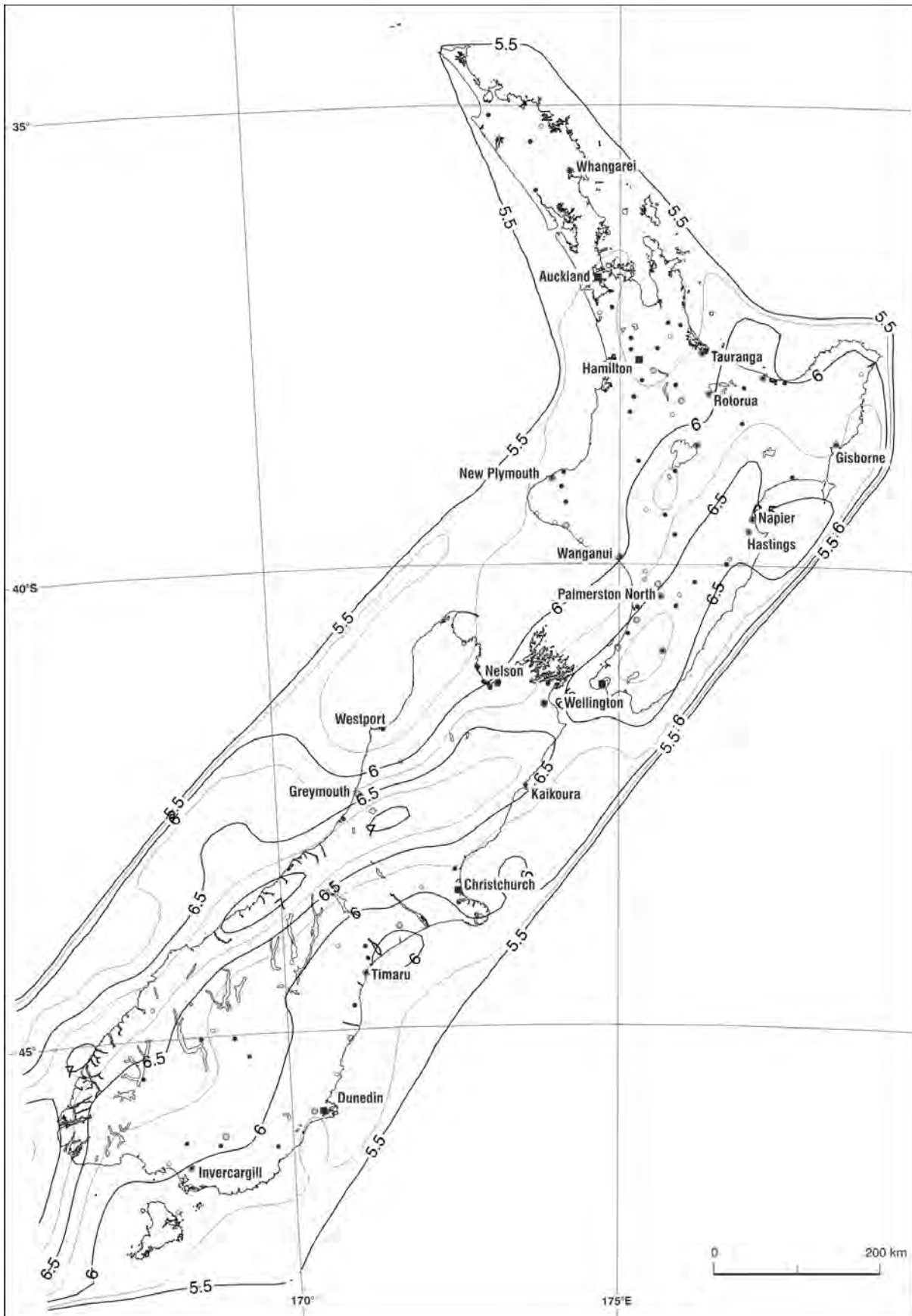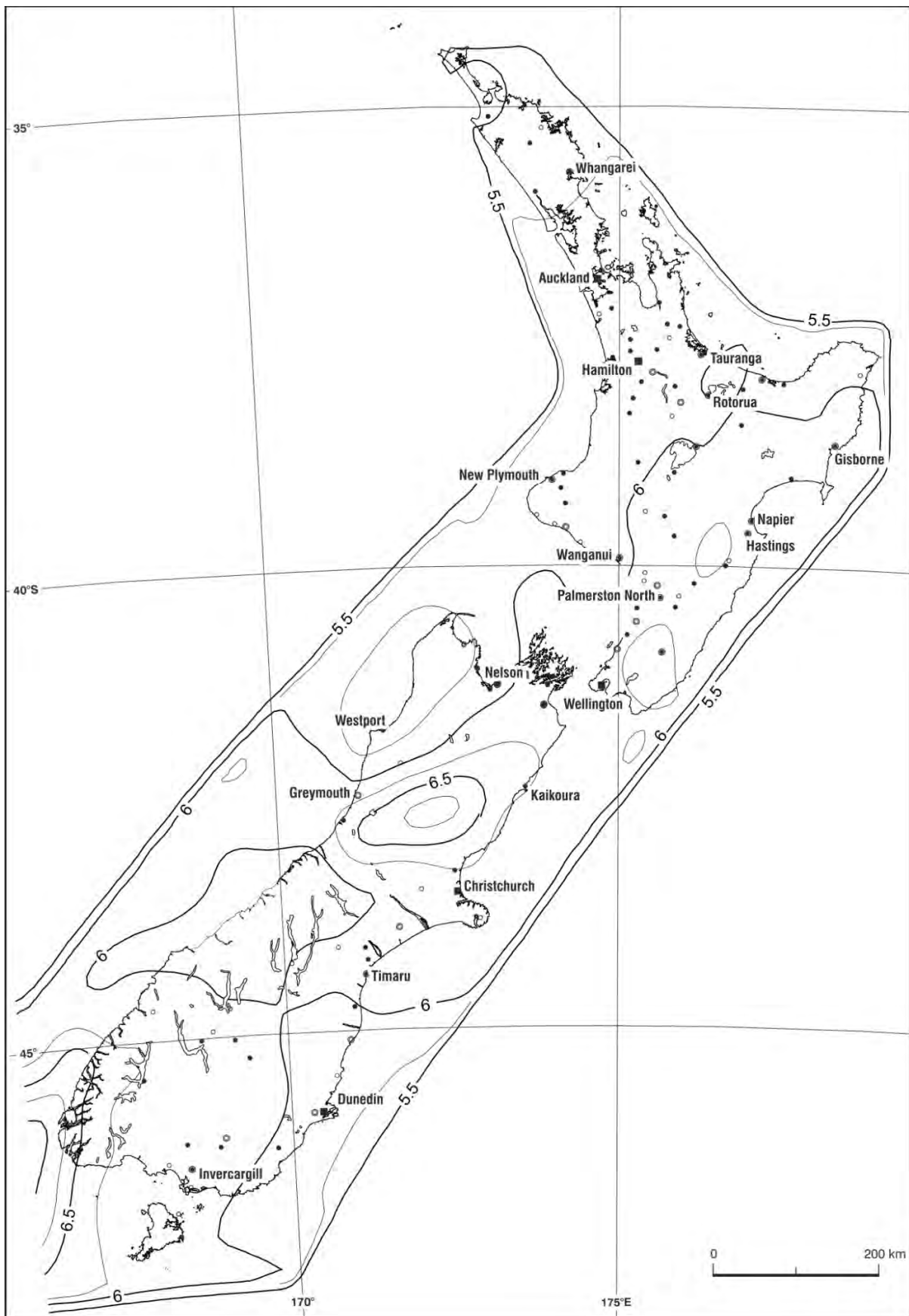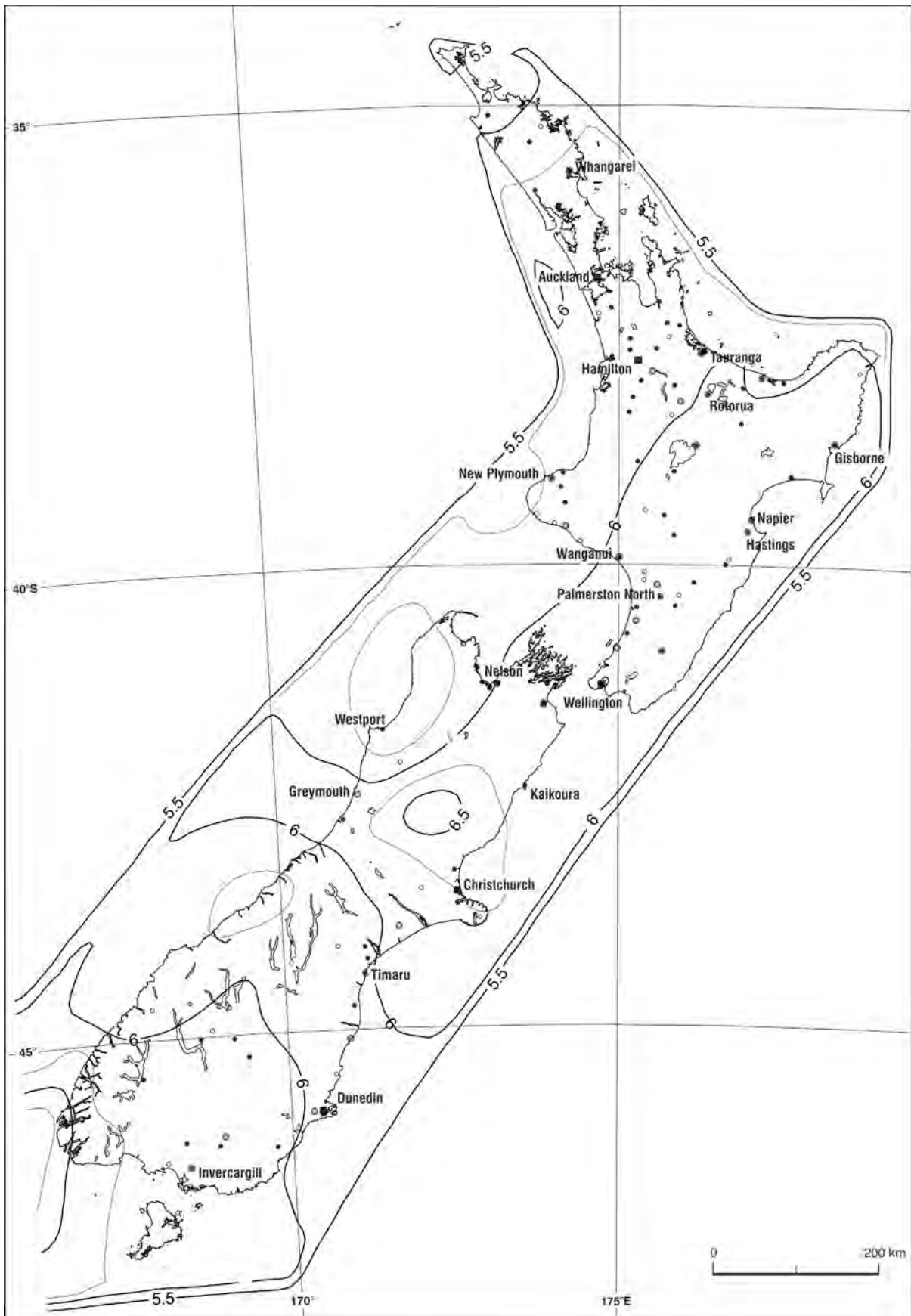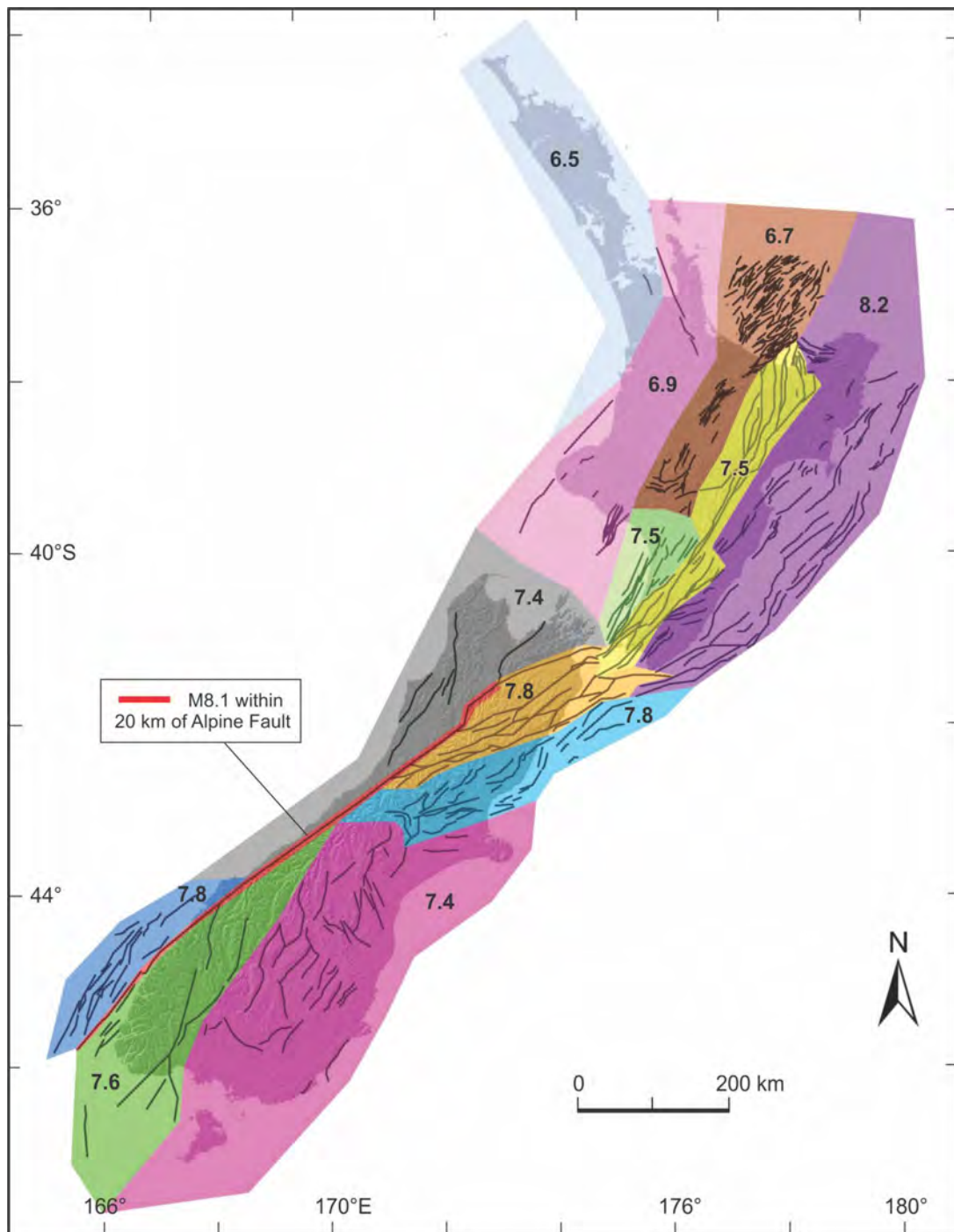
Figure 6.3: Default magnitudes for application in considering bridge structure collapse avoidance in the absence of magnitude deaggregation analysis from a site-specific hazard analysis

Notes:

(i)   Ideally, magnitudes associated with the maximum motions to be considered for collapse avoidance should be derived from deaggregation analyses performed as part of a site-specific seismic hazard study. The magnitudes from figure 6.3 shall be used in the absence of such a study.

(ii) The magnitudes in each region have been derived from consideration of the magnitudes associated with faults in the region that have estimated average recurrence intervals of rupture less than 10,000 years, the approximate return period associated with collapse avoidance for the most important (ie importance level 4) structures. They are likely to be larger than derived from site-specific analyses, especially for structures of lower importance level than importance level 4. The magnitudes may also be lower in locations remote from those faults associated with the magnitudes assigned to the region as a whole.

(iii) The northern part of the North Island has been assigned the magnitude of 6.5 associated with the event that governs the minimum damage control limit state ZR_u factor of 0.13.

6.2.4 Earth pressures and structure inertia forces on earth retaining structures

The forces discussed in 6.2.4(a) and (b) are illustrated in figure 6.4.

a. The following earth pressure effects shall be taken into account:

P_s - force due to static earth pressure (including compaction force, where appropriate).

ΔP_E - increment or decrement in earth pressure due to earthquake.

P_F - increment of force on wall due to its displacement towards the static backfill (force by the seismic response from a bridge superstructure).

In assessing earth pressure effects, due account shall be taken of the relative stiffnesses of the wall, backfill, foundations and any tie-back anchors.

The earthquake increment of earth pressure (ΔP_E) shall be derived using the 'rigid', 'stiff' or 'flexible' wall pressure distributions, where the wall classifications of 'rigid', 'stiff' or 'flexible' depend on the wall movements, all as given in the Road Research Unit bulletin 84, volume 2 *Seismic design of bridge abutments and retaining walls*[3]. As recommended in the document, the widely used Mononobe-Okabe earthquake pressure increments shall be used only when there is sufficient wall movement for the wall to be 'flexible'. Passive earth pressure decrements due to earthquake shaking shall be applied for the earthquake load case where passive pressures are relied on to provide stability, and these can be derived using the approach provided in the bulletin.

b. The structural inertia forces to be taken into account shall include:

P_I - The inertia force on the abutment or wall due to ground acceleration acting on the wall, and the soil block above the heel of the wall;

P_B - The force, if any, transmitted between the superstructure and the abutment. This force is the sum of that transmitted by the bearings, and that transmitted by a load limiting device if any.
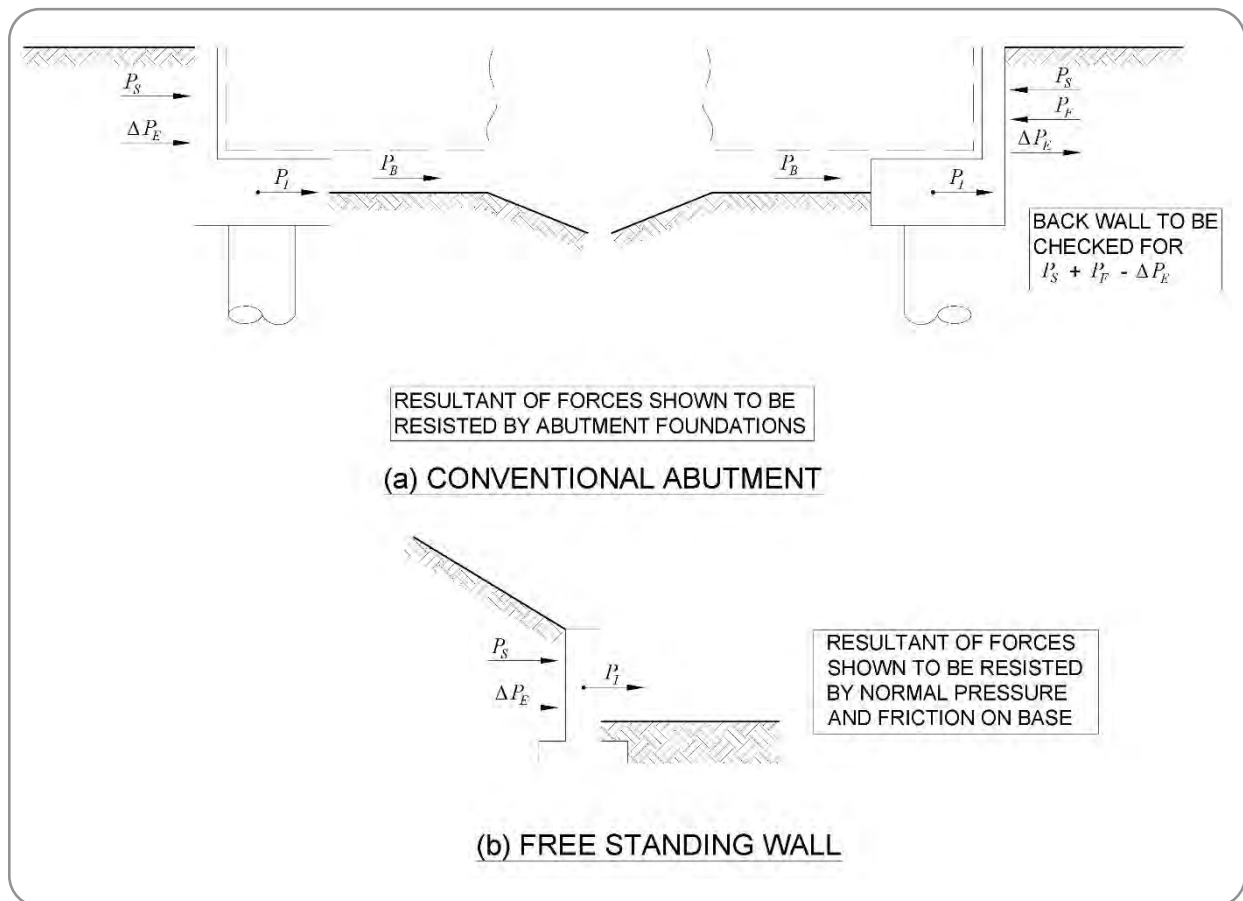
The force due to sliding bearings shall be calculated assuming the maximum likely friction coefficient. A value of at least 0.15 shall be assumed as specified by 5.3.9(b). The force due to other bearings shall be the product of the total support stiffness and the seismic displacement, Δ. The calculation of Δ shall take account of the relative stiffness of the various supports, and the relative stiffness of the abutment bearings and foundations.

c. The appropriate forces shall be combined as shown in figure 6.4. The structure shown in (a) represents extremes of relative resistance provided by the abutment piles and the backfill. Designs shall take account of intermediate conditions applying as appropriate. In both abutment cases the probability of P_B being out of phase with $\Delta P_E + P_I$ may be taken account of by applying the square root of the sum of the squares of the forces.

6.2.5 Groundwater levels, pressures and effects

Groundwater pressures, and the effect of groundwater on the stability and seismic performance of the site and soil structures, such as liquefaction, shall be considered. These should be based on groundwater levels recorded from site investigations with consideration given to the seasonal fluctuation of groundwater levels; potential for higher groundwater conditions in storm or rainfall events; the fluctuation of groundwater levels with river levels and tidal conditions; artesian water heads; and any potential for changes to groundwater level as a result of the construction or other anticipated changes during the life of the structures or slopes. The effects of climate change or other local changes in the area shall be taken into account together with any regional or local authority recommendations on seasonal groundwater level variation.

Figure 6.4: Seismic force combinations on non-integral abutments and retaining walls

(a) CONVENTIONAL ABUTMENT

RESULTANT OF FORCES SHOWN TO BE RESISTED BY ABUTMENT FOUNDATIONS

BACK WALL TO BE CHECKED FOR
$P_S + P_F - \Delta P_E$

RESULTANT OF FORCES SHOWN TO BE RESISTED BY NORMAL PRESSURE AND FRICTION ON BASE

(b) FREE STANDING WALL

6.3 Earthquake induced liquefaction, slope instability and ground deformation

6.3.1 Causes

Ground rupture, instability and deformation can result from:

- earthquake shaking
- earthquake induced liquefaction or cyclic softening
- lateral spreading with or without associated liquefaction or cyclic softening
- fault rupture associated with earthquakes
- subsidence from other causes, such as groundwater changes, mining, etc. These effects are not considered in this section, but should be assessed where they could occur.

Earthquakes can give rise to ground rupture, slope instability, liquefaction or cyclic softening induced deformation, tectonic movement (subsidence or uplift) and lateral spreading induced subsidence. The potential for such effects to occur, and their effect on the road and the associated structures, should be considered.

Supplementary to the requirements set out in the following clauses, guidance on the design of bridges for liquefaction and lateral spreading effects may be found in the Waka Kotahi NZ Transport Agency research report 553 *Development of the design guidance for bridges in New Zealand for liquefaction and lateral spreading effects*[4].

6.3.2 Assessment of slope or land stability in earthquakes

Permanent seismic displacement of soil associated with slope instability and ground movement can occur during earthquakes due to inertial effects associated with ground accelerations and/or weakening of the soil induced by the seismic shear strain.

In many cases, inertial slope instability and ground movement can occur as a result of temporary exceedance of the soil strength by dynamic earthquake stresses, even if there is no substantial reduction in soil strength due to pore pressure build up, liquefaction or soil sensitivity. Instability is mostly the result of soil becoming progressively weaker as shaking occurs such that the soil strength becomes insufficient to maintain a stable slope. Weakening may occur due to factors including shaking, pore pressure build up, liquefaction and sensitive fine grained soils.

Two types of slope instability can occur: flow failure and lateral spreading.

Flow failures are driven by static shear stresses that lead to large deformation or flow. Such failures are characterised by sudden initiation, rapid failure, and the large distances over which the failed materials move. Flow failures typically occur near the end of strong shaking or shortly after shaking. However, delayed flow failures caused by post-earthquake redistribution of pore water pressures can also occur, particularly if liquefiable soils are capped by soil layers with low permeability. Flow ceases only when the driving shear forces are reduced (such as by slope reduction) to values less than the residual shear resistance of the flowing material.

The potential for liquefaction and increased porewater pressure induced flow failures can be evaluated from limit equilibrium slope stability analyses, using residual undrained shear strength parameters for the liquefied soil and zero PGA (ie no seismic inertia loads are applied). Residual liquefied soil strength values for the flow failure analysis can be determined from empirical relationships in Waka Kotahi NZ Transport Agency research report 553[4] or from laboratory test results. If the limit equilibrium factor of safety is less than 1.05, flow failure is considered to be likely.

Lateral spreading can occur when the shear strength of the liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake or when soil strength and stiffness degrade sufficiently to produce substantial permanent strain in the soil. The result of lateral spreading is the horizontal movement of a non-liquefied crust over liquefied soils combined with the movement of liquefied soils themselves. Where the limit equilibrium factor of safety is greater than 1.05 for liquefied conditions and zero PGA, deformation and stability shall be evaluated using a lateral spreading analysis. As opposed to flow failures, lateral spreading analysis directly considers the effect of seismic acceleration.

Potential slope instability and displacements shall be assessed using geotechnical principles, as follows:

- The factor of safety against instability shall be assessed using conventional slope stability analysis with load and strength reduction factors of one, and the seismic coefficient associated with the relevant earthquake accelerations as set out in 6.2.2. Average groundwater conditions or maximum tide levels should be assumed for this assessment.

- If the factor of safety is less than 1 and the failure mechanism is not brittle (such as in rocks where the initiation of failure could substantially reduce the strength of the materials), then the critical seismic coefficient associated with the ground acceleration at which the factor of safety is one shall be assessed using large strain soil parameters consistent with the likely displacements due to earthquake shaking.

6.3.2 continued	• The displacement likely at the design damage control limit state and at the collapse avoidance limit state seismic response, shall be assessed using moderately conservative soil strengths consistent with the anticipated stress-strain behaviour and relevant strain levels and a Newmark sliding block displacement approach. Displacements shall be assessed using the methods described by Ambraseys and Menu[66], Ambraseys and Srbulov[5] and Jibson[6] without liquefaction and lateral spreading, and Bray and Travasarou[7] for displacements due to liquefaction and lateral spreading using the relevant peak ground accelerations (see 6.2), and the distance to the dominant earthquake sources in the area. Where a Newmark sliding block method is applied, the 50th percentile displacement shall be derived for both the damage control limit state and collapse avoidance limit state events. The range of predicted displacements (rather than a single value) should be used in the design process. In general, the upper bound values should be adopted unless a lesser value is otherwise justified and accepted by the road controlling authority. • The serviceability limit state requirements of 6.1.2(a) shall be satisfied.

6.3.3 Earthquake induced liquefaction

Further to the requirements set out below, additional guidance on the assessment of liquefaction and cyclic softening may be obtained from *Earthquake geotechnical engineering practice,* module 3 – Identification, assessment and mitigation of liquefaction hazards[8].

a. Soils susceptible to liquefaction and cyclic softening

The liquefaction of saturated predominantly cohesionless soils (generally sand, silt and loose sandy gravels) and cyclic softening of clays and plastic silts during strong earthquake shaking shall be taken into consideration in the design of structures, including highway bridges and their approaches, soil structures and the stability assessment of natural slopes.

b. Site investigations

Site investigations (including field and laboratory testing) should be adequately scoped to provide sufficient information for the assessment of seismic behaviour of the soils including:

– assessment of liquefaction susceptibility of the site soils

– assessment of the soils' potential for cyclic softening

– analysis of triggering of liquefaction and cyclic softening (using the design earthquake loading specified in 6.2)

– evaluation of consequences of liquefaction and cyclic softening.

The following investigation procedures can be used for the assessment of liquefaction potential of soils and their susceptibility to lateral spreading:

– historical evidence of liquefaction

– geological assessment of the site to identify whether the site soils might be susceptible to liquefaction

– cone penetration test (CPT or CPTU where pore water pressure is measured)

– boreholes with associated standard penetration tests (SPT) and sampling

– shear wave velocity tests (such testing should be carried out in accordance with ASTM D4428/D4428M *Standard test methods for crosshole seismic testing*[9] or ASTM D5777 *Standard guide for using the seismic refraction method for subsurface investigation*[10])

6.3.3 continued

- – in-situ and laboratory shear vane tests
- – measurement of groundwater table using piezometers and assessment of seasonal fluctuation of groundwater table
- – assessment of artesian aquifers' regime
- – monotonic and cyclic triaxial tests
- – monotonic and cyclic simple shear tests
- – laboratory testing including grading and Atterberg limit tests
- – topographical survey of the site including survey of river and stream banks, as well as river, stream and sea bed levels where the site is within 200m of a free face.

Within granular soil layers (including granular materials with fines) that may have potential for liquefaction, SPTs should be performed at 1m or 1.5m centres. Also, push tube samples should be taken within cohesive soil layers.

It should be noted that the results of the cyclic triaxial and cyclic simple shear tests are very sensitive to the quality of soil samples. Sampling techniques minimising disturbance of soil samples, eg as described by Taylor, Cubinovski and Haycock[11] or Hofmann and Robertson[12] should be used.

Where liquefaction susceptible soils containing gravels are present, where static cone penetration tests and standard penetration test results would be influenced by gravel particles, the use of in-situ shear wave velocity tests should be considered.

Additional dynamic triaxial tests should be considered if the potential for liquefaction or cyclic softening is uncertain but is critical to the performance of a significant structure.

CPT testing shall be carried out in accordance with ASTM D5778 *Standard test method for electronic friction cone and piezocone penetration testing of soils*[13].

SPT testing should be carried out generally in accordance with ASTM D1586 *Standard test method for standard penetration test (SPT) and split-barrel sampling of soils*[14] and ASTM D6066 *Standard practice for determining the normalized penetration resistance of sands for evaluation of liquefaction potential*[15]. All SPT tests shall be carried out using a hammer with a measured energy efficiency ratio.

Except when gravel particles are present (as noted above), shear wave velocity testing can be employed only as a testing procedure additional to SPT and CPT tests.

The results of the site investigations are to be documented in a site investigation factual report.

c. Liquefaction and cyclic softening triggering assessment

Materials such as sands, non-plastic silts, gravels and their mixtures may be susceptible to liquefaction. Clays and plastic silts are not susceptible to liquefaction but can be prone to softening under cyclic load.

The analysis of whether the soil is likely to experience sand-like behaviour (liquefaction) or clay-like behaviour (cyclic softening) should be carried out and the effects of cyclic softening should be assessed in accordance with recommendations given in Boulanger and Idriss (2006[16] and 2007[17]) and Bray and Sancio[18].

To ensure that the assessment of liquefaction and cyclic softening of soils is carried out in a uniform and consistent manner the following procedures shall be followed:

- – Ground motion parameters (unweighted earthquake PGAs and respective earthquake magnitudes) shall be derived as described in 6.2.2.

6.3.3 continued

– Triggering of liquefaction for materials with sand-like behaviour shall be carried out in accordance with the simplified procedure described in Boulanger and Idriss (2014[19]) unless specified otherwise in the principal's requirements or the requirements and minimum standards. The PGA required for the assessment of liquefaction triggering shall be derived as described in 6.2.2. The potential for liquefaction of the site at each of the limit states shall be assessed.

It should be noted that the data used for the development of the simplified procedure relates to depths of less than 15m. Extrapolation of the simplified procedure to greater depths should account for the increased uncertainties at depths greater than 15m.

Special care, expertise and additional calculations are required when the simplified liquefaction triggering procedure is employed for depths greater than 15 – 20m. For depths greater than 20m, an alternative to the simplified procedure is to perform a nonlinear, total or effective stress site response analysis utilising a computer code capable of modelling non-linear soil behaviour and, for effective stress analysis, pore water pressure generation and dissipation.

If effective stress site response analysis is used, the geotechnical designer shall provide evidence that their model has been appropriately validated and calibrated with field data, laboratory data and sensitivity analyses. Due to the specialised nature of this more sophisticated liquefaction assessment method, the road controlling authority's approval to the use of non-linear effective stress methods for liquefaction evaluation shall be obtained.

Most of the case studies used for the development of the simplified procedure for the assessment of liquefaction triggering based SPT and CPT data relate to sites formed by Holocene-age alluvial or fluvial sediment. Assessment of liquefaction potential of pumice soils based on CPT and SPT tests has been proven to be unreliable. Cyclic triaxial and cyclic simple shear tests on high quality undisturbed samples and shear wave velocity profiling should be considered for such materials; other methods of soil characterisation listed in 6.3.3(b) can also be used.

Each liquefaction or cyclic softening assessment procedure recommended in this *Bridge manual* shall only be used with the same analysis framework (and all its components) that was used to develop the procedure from the case history database.

In order to minimise discrepancies in assessments of liquefaction potential of soils for the tender design by different designers and enable a fair evaluation process, clear instructions and guidance shall be given in the principal's requirements or the requirements and minimum standards. All parameters adopted and to be applied in the liquefaction assessment shall be fully reported in the specimen design report covering the geotechnical design philosophy.

Where index values measured in the field (eg CPT cone resistance and sleeve friction) are used to calculate the fines correction factors, the fines content determined from the index values shall be checked against soil sample descriptions and laboratory test data, and where there is a discrepancy, the values from the laboratory test data should be adopted.

K_α factors applied to cyclic resistance ratios shall not be greater than 1.0 but factors less than one shall be used where in-situ stress conditions will reduce the soil's resistance to liquefaction. This factor is related to different modes of sloping ground deformation (as opposed to level ground condition), which affects pore pressure development and consequent liquefaction.

Changes to in-situ ground pressures from the construction of cuttings or embankments shall be considered in the assessment of liquefaction triggering.

6.3.3 continued

Where significant embankments are proposed on liquefaction susceptible ground, the potential for liquefaction with and without the overburden contribution of the embankment shall be assessed. The construction of significant embankments would result in increased overburden pressure on the foundation ground and may also induce some initial shear stresses in the soil. Such effects need to be considered in the assessment of liquefaction potential of the foundation ground.

Where embankments are to be constructed over potentially liquefiable soils, soil index values (eg CPT cone resistance, SPT blow count, shear wave velocity) measured prior to the construction of the embankment shall not be factored to account for densification unless the adopted factors are proven with field measurements for trial embankments or post-construction field measurements.

Fines correction factors, calculated using soil index values measured prior to the construction of the embankment, should not be adjusted by changes that may be made to the index values based on assessed level of possible or actual densification.

The residual shear strengths of liquefied soils for geotechnical analysis and design shall be estimated from empirical correlations to SPT or CPT data based on the recommendations of section 6.3.4 of *Earthquake geotechnical engineering practice, module 3*[8]. It should be noted that these correlations are approximate. As there are discrepancies in the residual shear strengths assessed from different correlations, designers shall evaluate how the use of different correlations would affect the residual shear strengths of liquefied soil and the expected seismic performance of structures, natural slopes as well as soil structures.

The assessment of liquefaction and cyclic softening, together with all the assumptions, methods, and analysis results shall be documented. Values and correlations adopted as the basis for the design shall be justified and include sensitivity analysis as described above. Values determined from sensitivity analyses for design shall be subject to acceptance of the road controlling authority.

d. Assessment of the effects of liquefaction and cyclic softening

The following effects of liquefaction and cyclic softening shall be taken into consideration in the development of design concepts and design of the project:

–   Loss of or substantial reduction in the foundation bearing capacity.

Bearing failures can lead to large total and differential settlement, tilt and overturning of structures and soil structures. The possibility of punching failures through a surface crust shall be considered in the design. Liquefaction in the soil zone adjoining or near the zone of influence (the soil zone loaded by a structural foundation or soil structure) may also reduce foundation bearing capacity and therefore should also be considered in the design process.

–   Reduction in soil strength and stiffness due to liquefaction and cyclic softening resulting in a reduction in the foundation's (especially pile foundation) lateral capacity and stiffness leading to modification of the flexibility of the structure and an increase in damping and the fundamental period of the structure.

–   Loss or substantial reduction in pile foundation capacity for uplift or lateral loading.

–   Subsidence of the ground.

Subsidence shall be assessed using the methods of Ishihara and Yoshimine[20] or Zhang, Robertson and Brachman(2002)[21]. Such subsidence is also referred to as liquefaction-induced free-field settlement or settlement due to re-consolidation of liquefied soils.

6.3.3 continued

– Soil-structure-interaction-induced settlement (associated with volumetric strains due to groundwater flow in response to transient gradients), large shear deformation of liquefied ground as well as structure rocking and ratcheting effects.

This settlement is additional to the free field settlement and can be quite substantial for structures founded on shallow foundations and non-piled soil structures on liquefiable sites. It is therefore important to assess this settlement for structural foundations and soil structures. While there are no well-established methods for the assessment of soil-structure-interaction-induced settlement associated with liquefaction, Bray and Dashti[22] and Murashev, Keepa and Tai[23] provide useful information.

– Large lateral movements from ground oscillation.

An empirical procedure proposed by Tokimatsu and Asaka[24] can be used for the assessment of cyclic displacements of liquefied ground.

– Large lateral movements as a result of lateral spreading and flow failure of natural ground towards free surfaces such as river banks and of approach embankment slopes.

Lateral spreading displacements can be assessed by either empirical or Newmark-type methods described by Youd, Hansen and Bratlett[25], Tokimatsu and Asaka[24], Zhang, Robertson and Brachman (2004)[26], Jibson[6] and Olson and Johnson[27]. There is a substantial uncertainty associated with the assessment of lateral spreading displacements. For both the damage control limit state design earthquake event and the collapse avoidance limit state event, 50th percentile displacements shall be derived and the upper bound result from not less than three of the methods shall be used as the basis for design unless use of a lower value is otherwise justified and accepted in writing by the road controlling authority.

– Natural or fill slope instability due to strength reduction in liquefied soil layers or lenses (this slope instability is not necessarily associated with lateral flow or lateral spreading).

– Negative skin friction or down-drag on piles and buried structures or their elements due to subsidence associated with liquefaction, and the downward movement of liquefiable soil and any overlying or interbedded liquefaction resistant layers.

– Uplift and flotation of buried structures (eg culverts and tunnels) and soil structures (eg geofoam) or structural members such as piles and anchors.

– Possible ground loss underneath foundations or foundation elements due to ejecta. Ishihara[28] developed correlations between surface manifestation of liquefaction (such as surface rupture and sand boils), thicknesses of the liquefied layer and the overlying non-liquefied crust. Ishihara's correlations can be used to assess the risk of ground loss associated with ejecta.

6.3.4 Mitigation of liquefaction and site instability hazards

a. General

The design shall mitigate the risks associated with potential damage to the highway and associated structures, including soil structures, from liquefaction, cyclic softening or site instability, through ground improvement or provision of sufficient strength or ductility in the structures to resist liquefaction and site instability effects. Such mitigation shall ensure that the performance requirements of 5.1.2 for structures at the serviceability limit state, damage control limit state and collapse avoidance limit state and 6.1.2(b) for soil structures at the serviceability limit state and 6.1.2(c) for both structures and soil structures at the damage control limit state are achieved, unless agreed with the road controlling authority to be impractical or uneconomic.

6.3.4 continued

b. Liquefaction mitigation by ground improvement

Measures to mitigate liquefaction hazard by ground improvement, such as using densification by dynamic compaction or vibroflotation, deep mixing, drainage, or combined densification and drainage using vibro-replacement or stone columns, shall be considered to reduce the risk to the highway from liquefaction of the soils. Where the ground is densified, testing shall be undertaken following construction to confirm that the required level of ground improvement has been achieved. Consideration should be given to possible effects of pore water migration from untreated soil zones to treated soil zones and ground motion amplification. Pore water migration may occur during and after shaking, particularly when the improved zone consists of densified soil. Ground motions tend to be amplified by treated zones, particularly those extending completely through liquefiable deposits, resulting in higher supported structure accelerations and inertial forces. Where improved ground extends to stronger soil or bedrock at depth, the site subsoil class may need to be adjusted.

Timber poles shall not be used for ground improvement in association with any structure or part thereof. The road controlling authority may however consider a departure to this requirement. Such a departure request shall be in the form and address the factors detailed in 6.1.2(d). In addition, the departure will be subject to confirmation of durability (design working life) for the particular ground conditions and confirmation of acceptable structural behaviour for the entire structure comprising ground improvement and supported structure.

c. Foundations in liquefiable soils

Shallow foundations shall not be founded in liquefiable layers, or within a zone above liquefied layers equivalent to twice the width of the foundation, or where liquefaction will prevent the performance requirements for the foundation being satisfied. Additional reinforcement of the foundation subgrade shall be incorporated to minimise differential subsidence effects on the foundations.

Foundations below liquefiable layers shall be located to ensure that under liquefaction they continue to achieve acceptable performance and that the bearing capacity is not diminished or settlements increased to unacceptable levels. Piles shall be founded a minimum of three pile diameters below the base of soil layers prone to liquefaction. The reduction in deep foundation, including piles, pile lateral capacity, bearing or uplift capacity (such as due to reduction in pile shaft and foundation friction) shall be taken into consideration in the design.

d. Isolation of the structure from ground displacement

Where separation of the structure from the ground (eg by the sleeving of piers or piles) is adopted to isolate the structure from the effects of permanent ground displacement the degree of separation provided shall be sufficient to protect the structure from the more severe of:

– the cumulative displacement effects of at least two successive damage control limit state design earthquake events, to be taken as 200% of displacement arising from one DCLS design intensity earthquake; and

– the displacement effects due to one CALS event.

6.3.5 Design scenarios to be considered

When evaluating the effects of liquefaction, cyclic softening and lateral spreading on the performance of a structure (eg pile foundations) using equivalent or pseudo static analysis, it is necessary to conduct separate analyses for different stages of the response. Scenarios to be considered are:

6.3.5 continued

i. Peak structural and ground response preceding liquefaction developing, in which inertial loads that would occur in the absence of liquefaction are considered. (Embankment and slope movements may occur in this scenario and should be taken into account.)

ii. Ground response with liquefaction or cyclic softening developing, and with the structural response modified for period shift due to softening of the foundation stiffness. This will involve estimating the potential and consequences of liquefaction, and considering simultaneous kinematic loads (due to cyclic ground displacements) and structural inertial loads while accounting for stiffness and strength degradation of the soils due to excess pore water pressures. For equivalent or pseudo static analysis, not less than 80% of the design structural inertial loading plus 100% of the cyclic kinematic loads shall be assumed to act concurrently with full degradation of the soil strength and stiffness due to liquefaction[*]. (Embankment and slope movements may occur in this scenario and should be taken into account.)

iii. Lateral soil spreading due to liquefaction having developed. This analysis requires estimating the potential for liquefaction and consequences of lateral spreading including substantial stiffness and strength degradation and the kinematic loads due to large spreading displacements. Inertial loads of the structure are in general of secondary importance in the spreading phase and may be ignored except in the following case. When, for DCLS or CALS events at the site, the percentage of the hazard contributing to the peak ground acceleration by a magnitude 7.5 or greater earthquake is more than 20%, the lateral spreading/flow failure forces on foundations shall be combined with the plastic hinge force or 25% of the structure inertial forces, whichever is less.

The design earthquake response spectrum for scenario (i) shall also be applied for scenarios (ii) and (iii). Scenario (ii) shall be considered if triggering of liquefaction is predicted, and scenario (iii) if lateral spreading is anticipated.

If liquefaction does not occur, or only occurs in small localised lenses that have no effect on ground displacements (kinematic loads), stiffness and strength degradation in the equivalent static analysis, scenarios (ii) and (iii) will not arise to any significant extent and do not need to be considered.

Where liquefaction and/or soil lateral spreading may occur or become more severe in earthquake events greater than the damage control limit state design intensity event but less than or equal to the collapse avoidance limit state intensity event, such that the soil loads on the structure dramatically increase and/or the nature of soil restraint to the structure is significantly reduced compared to that applying during an damage control limit state design intensity event, consideration shall be given to implications of these changes in the actions acting on the structure or restraint conditions applying to the structure for the avoidance of collapse of the structure.

Existing methods used for the assessment of liquefaction triggering and the consequences of liquefaction are not very accurate. Materials that have a factor of safety against liquefaction of less than 1.1 should be considered to be liquefiable in the design analysis unless otherwise agreed with the road controlling authority.

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[*] The peak cyclic ground displacement and superstructure inertial force are transient conditions occurring momentarily during the course of strong shaking. They may or may not occur at the same instant. For pseudo-static analysis of seismic problems, the load combination producing the critical (peak) pile response in liquefying soils cannot be predicted with any high degree of certainty. Therefore there is no commonly accepted strategy on how to combine these loads in pseudo-static analysis. Tamura and Tokimatsu[29] suggested that the phasing of the kinematic and inertial demands varies, and depends primarily on the natural frequency of the superstructure and soil deposit. Ashford, Boulanger and Brandenberg[30] suggested a simplified expression allowing for different combinations of kinematic and inertial loads on the pile while accounting for the frequency content characteristics of the ground motion. In the absence of commonly accepted strategy, the proposed 80% of the design structural inertial loading acting concurrently with kinematic load is considered to be an acceptable design approach.

6.3.5 continued	Even if full liquefaction is not predicted to occur, partial pore pressure build-up can result in deterioration of strength and stiffness of soils. The potential pore pressure build-up shall be assessed for damage control limit state and collapse avoidance limit state events and, where factor of safety against liquefaction is higher than 1.1 but less than 1.4, reduced stiffness and strength shall be used for the assessment of slope stability and foundation bearing capacity affecting the support of structures and soil structures.

The magnitude of the pore pressure generation and associated degradation of soil strength and stiffness are a function of the factor of safety against liquefaction and soil type. The magnitude of the pore pressure build up can be assessed as recommended by Marcuson, Hynes and Franklin[31]. In soil layers that experience pore pressure generation, the shear strength of the soil should be reduced by using a reduced effective stress due to pore pressure build up. More detailed recommendations on the reduction of soil strength and stiffness due to pore pressure build up are given by Ardoino et al[32].

6.3.6 Optimisation of ground improvement

Ground improvement is costly. Where liquefaction or cyclic softening problems are identified as potentially causing lateral spreads that may damage the structure (including soil structures), the following options should be considered:

- For new structures: relocate the structure to another less vulnerable site. This option should be considered at the concept design stage. If the risk of liquefaction or cyclic mobility is identified for a proposed route, alternative routes with better ground conditions at structure sites should be considered.

- For new and existing structures on liquefiable sites: soil-foundation structure interaction analysis should be undertaken to determine whether the deformation and load capacity of the foundation/structure system is adequate to accommodate the ground deformation demands and meet the performance criteria specified by 5.1.2 as well as the serviceability criteria specified by 6.1.2(b) (assuming no ground improvement); and where the foundation/structure system is found to be inadequate the most cost-efficient of the following options should be used:
    - foundation/structure system should be strengthened to accommodate the predicted liquefaction and related ground deformation demands
    - ground improvement should be undertaken to reduce liquefaction potential of soils and minimise ground displacement to acceptable levels
    - possible combination of the above two options.

This analysis will require close interaction between the structural and the geotechnical designers and shall be undertaken in accordance with Waka Kotahi NZ Transport Agency research report 553[4] guidelines, *Analysis of piled bridges at sites prone to liquefaction and lateral spreading in New Zealand*[33], *Earthquake geotechnical engineering practice*, module 5 – Ground improvement of soils prone to liquefaction[34] or similar methodology approved by the road controlling authority.

Where ground improvement is specified by the designer, the road controlling authority may require the designer to submit evidence of ground improvement optimisation analysis in accordance with this methodology. For projects where the cost of ground improvement is more than $1.2 million (price at December 2021*), consideration should be given to the use of inelastic time history finite element analysis of soil-foundation structure interaction to optimise the extent of ground improvement.

For design and construct type projects on sites prone to liquefaction, cyclic mobility or lateral spreading, assessment of liquefaction and optimisation of ground improvement should be carried out at the stage of specimen design and clear requirements should be included in the principal's requirements for the project.

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* Values shall be adjusted to current value. For the relevant cost adjustment factor refer to the Waka Kotahi NZ Transport Agency *Procurement manual*, Procurement manual tools, Infrastructure and public transport contract price adjustment indexes, Latest index values for infrastructure cost indexes – original 1991 series, Waka Kotahi NZ Transport Agency Construction index[2]

6.4 Design of earthworks

6.4.1 Design of embankments

a. Philosophy

The design of embankments shall be based on adequate site investigations and shall ensure acceptable performance of the embankment under gravity, live and earthquake loads, under flood and post-flood drawdown conditions, under conditions of changing groundwater levels and where water mains are present under the eventuality of them rupturing. Embankments considered to be dams, as defined in the Building Act 2004, shall satisfy all necessary requirements for the applicable dam definition. Appropriate measures shall be specified to ensure that post-construction settlements will be within acceptable limits compatible with the performance expectations for the road. Such limits shall be agreed with the road controlling authority.

b. Static behaviour

Under static conditions (including appropriate live load surcharge) completed embankments shall have a minimum design long term factor of safety against all modes of failure of 1.5 based on moderately conservative effective stress soil strengths under moderately conservative design operating piezometric conditions. This shall apply unless specific justification for a lower value has been accepted in writing by the road controlling authority.

A suitable monitoring programme shall be implemented by the designer to check embankment performance during and after construction. The designer shall specify acceptable limits for monitoring measurements.

During construction, embankments shall possess a minimum design static short term factor of safety against all modes of failure of 1.2 based on moderately conservative effective stress soil strengths or undrained shear strength parameters, under moderately conservative design operating piezometric conditions.

Where preloading, surcharging, staged loading, vertical drains or other techniques are required to permit construction of embankments or to accelerate settlement, a suitable monitoring programme shall be specified and the results shall be reviewed by the designer.

Factors of safety shall be calculated using loads and combinations for the serviceability limit state as specified in table 3.2.

c. Behaviour in seismic and flood events

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

Unless it is accepted by the road controlling authority to be impractical or not economically viable to significantly reduce the risk of embankment failure due to earthquake or flooding, the following design criteria shall apply:

6.4.1 continued

i.   For seismic events

For the assessment of the stability of embankments using pseudo-static seismic analysis the peak ground acceleration to be applied shall be derived in accordance with 6.2 for the annual probability of exceedance associated with the importance of the slope as defined in 2.1.3. In applying the pseudo-static analysis, the PGA shall not be factored down by a structural performance factor or any other factor.

Where embankments are designed on the basis of permitting displacement under earthquake response, the requirements of 6.6.9 shall also be satisfied.

For the following situations the minimum factors of safety and supporting requirements shall be satisfied under the damage control limit state earthquake event:

– Embankment stability not affecting bridges

In situations where liquefaction either does not occur or prior to liquefaction occurring:

Factor of safety ≥ 1.0, or

Factor of safety < 1.0 and permanent displacements less than the limits given in table 6.1.

– Embankment stability and liquefaction not affecting bridges

Where liquefaction is anticipated under the design seismic event, the design criteria for embankment stability not affecting bridges are to be met with post-liquefaction soil strengths and the embankment subjected to the design damage control limit state PGA.

– Embankment stability affecting bridges

In situations where liquefaction either does not occur or prior to liquefaction occurring:

Factor of safety ≥1.0; and

The seismic performance requirements for structures as specified in this manual are met.

Where a factor of safety of 1.0 cannot be achieved, the bridge shall be either isolated from the ground movement or designed to withstand the loads and effects imposed on it by the ground movement so that the seismic performance of this manual are met.

– Embankment stability and liquefaction affecting bridges

Where liquefaction is anticipated under the design seismic event, the design criteria for embankment stability affecting bridges are to be met for either the factor of safety ≥1.0 or the factor of safety <1.0, whichever is appropriate, with post-liquefaction soil strengths and the embankment subjected to the design damage control limit state PGA.

All displacements referred to in this item (i) should be assessed as described in 6.3.5.

ii.   For flood events

Where embankments may act as water retaining structures during flooding, the embankment shall remain stable under the lateral pressure and the ability of the embankment to sustain the effects of seepage and drawdown shall be examined. In such cases the embankment shall have a minimum factor of safety against failure of 1.25 unless there is potential for significant downstream damage or loss of life, in which case a minimum factor of safety of 1.5 shall apply. The *Dam safety guidelines*[35] provides guidance on embankments that may act as water retaining structures.

6.4.1 continued	Where lightweight embankments are constructed utilising geofoam materials (polystyrene or similar) a factor of safety of not less than 1.1 against flotation shall be provided under the ultimate limit state design flood event.

Factors of safety shall be calculated using loads and combinations for serviceability limit state as specified in table 3.2.

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

Where it is not practical or economically justifiable to significantly reduce the risk of embankment failure due to earthquake or flooding, and the effect of such failure on the performance of the road network, considering the required levels of service and lifeline requirements, is acceptable to the road controlling authority, then the design may allow for failure to occur in such large events (unless there are significant potential downstream effects on property, public/private infrastructure or the effects are potentially life threating). In such cases the manner and extent of such failure shall be assessed and, where bridges are affected, the bridge structure and foundations shall be designed to accommodate the embankment failure without damage to the structure.

Where it is proposed to accept failure of the embankment under the design earthquake, or under flood conditions, in order to adopt a factor of safety of less than specified above, justification for doing so shall be set out in a design statement for the road controlling authority's consideration and acceptance in writing, before the proposal is adopted.

d. Loadings on associated bridge structures

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

6.4.2 Design of cuttings	Cuttings shall be designed in accordance with recognised current highway design practice with provision of benches, and appropriate measures to mitigate the effects of rock fall and minor slope failures. As appropriate, design shall generally comply with 6.4.1 and the factors of safety for embankments given in 6.4.1 shall also be applied to the global stability of cuttings. Slope geometry shall be designed to ensure that any slope failure material will not be deposited against or over any bridge, gantry or soil structure. Where this is not practicable, provision shall be made in the design of these structures for additional dead load or earth pressure to represent the effect of slope failure material.

Where it is proposed to accept a significant risk of instability, justification for doing so shall be set out in the design statement for the road controlling authority's consideration and acceptance in writing before the proposal is adopted.

Where cutting slopes are designed on the basis of permitting displacement under earthquake response, the requirements of 6.6.9 shall also be satisfied.

6.4.3 Natural ground instability	Where any structure, soil structure or the highway can be affected by instability or creep of natural ground, measures shall be taken to isolate the structure or highway, remedy the instability, or design the structure or highway to accommodate displacements and loads arising from the natural ground. As appropriate, design shall generally comply with 6.4.1, and the factors of safety given in 6.4.1 shall be used.

Where slopes are designed on the basis of permitting displacement under earthquake response, the requirements of 6.6.9 shall also be satisfied.

6.5   Foundations

6.5.1 Loads on foundations

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

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

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

6.5.2 Design standards for foundations

a. Foundation design shall be based on appropriate sound design methods and shall satisfy the *Building code*[36].

b. The following standards and codes of practice provide guidance on the design of foundations:

   – *New Zealand building code* verification method B1/VM4[37].

   – BS EN 1997-1 *Eurocode 7 Geotechnical design* part 1 General rules[38], plus BS EN 1998-5 *Eurocode 8 Design of structures for earthquake resistance* part 5 Foundations, retaining structures and geotechnical aspects[39].

   – S6 *Canadian highway bridge design code*[40].

   – AASHTO *LRFD Bridge design specifications*[41].

   – AS 2159 *Piling – Design and installation*[42].

c. The Waka Kotahi NZ Transport Agency *Bridge manual* shall take precedence where there is a conflict.

6.5.3 Strength reduction factors for foundation design

Strength reduction factors shall be applied in the strength design of foundations for their bearing capacity and resistance to sliding. The strength reduction factors for sliding of shallow foundations shall be derived from table 1 of B1/VM4[37]. The strength reduction factors for bearing capacity of shallow and pile foundations shall be derived using the risk based methodology set out in AS 2159[42] section 4.3, and shall be presented in the structure design statement for acceptance by the road controlling authority.

The strength reduction factors adopted for bearing capacity of shallow foundations shall be taken as $\phi_g = \phi_{gb}$, where ϕ_{gb} is defined in AS 2159[42] and shall not exceed a maximum value of $\phi_g = 0.6$ for all load combinations, excluding earthquake overstrength where higher strength reduction factors, up to $\phi_g = 0.75$, may be adopted.

Strength reduction factors adopted for bearing capacity of both shallow and piled foundations shall not exceed a maximum value of $\phi_g = 0.75$, regardless of whether static, dynamic, or gravitational loading, or seismic loading induced by overstrength capacities developing are being considered.

6.5.4 Capacity design of foundations

The principles of capacity design are outlined in section 5.

The foundations should not compromise the seismic performance of the superstructure (above foundation level structure). Increasing flexibility in the foundations generally has the effect of increasing the curvature ductility demand imposed on plastic hinges or the element ductility demand on elements such as mechanical energy dissipating devices in the superstructure and exceeding the capacity of these elements needs to be avoided.

The foundations must be capable of transmitting the largest feasible actions to the supporting soil, and the soils must be capable of resisting the pressures applied by the foundations, otherwise the intended seismic response of the superstructure cannot eventuate. For structures designed using capacity design principles, the capacity of the footings, piles or caissons shall be such that deformations developed in the supporting soil under actions corresponding to the over-strength of the superstructure are limited in terms of their magnitude, so that the intended seismic response of the superstructure can eventuate.

In general, foundation systems shall be designed to preclude foundation failure, or uplift of an entire foundation element, at loadings corresponding to yielding of the earthquake energy dissipating elements, taking concurrency effects into account where applicable. Where it is intended to allow the rocking of foundations, inelastic time history analyses or a simplified analysis based on equilibrium consideration, as described in appendix A of the AASHTO *Guide specifications for LRFD seismic bridge design*[43] shall be performed to study the structure's behaviour as required by 5.6.12 and bearing areas within the foundation shall be so proportioned as to protect the soil against excessive plastic deformations that would be difficult to predict and which may result in premature misalignment of the otherwise undamaged superstructure.

Since there is greater uncertainty in the strength and stiffness properties of the ground, and their contribution to either increased loads or reduced resistance, as compared to other structural materials and depending on the case, it is not appropriate to use a single factored down strength for the soils and rocks. Upper and lower bound strength and stiffness properties of the soils shall be applied in order to assess the most adverse performance likely of the structure, which is to be adopted as the basis for its capacity design.

6.5.5 Foundation capacity determination

The load capacity of foundations shall be assessed using geotechnical parameters from geotechnical investigations and tests, and soil/rock mechanics theory or semi-empirical geotechnical methods. The ultimate capacity of foundations shall be assessed in accordance with the recommendations of the *New Zealand building code* verification method B1/VM4[37]. The capacity shall be confirmed during construction as specified in 6.5.6.

6.5.6 Confirmation of foundation conditions during construction

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

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

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

6.5.6 continued	• Inspection, logging and possibly testing of the ground by a geotechnical engineer or engineering geologist.

• Inspection, logging and possibly testing of the ground by a geotechnical engineer or engineering geologist.

• Plate bearing tests.

• Static pile load tests.

• Pilot hole drilling and testing.

• Down-hole inspection of pile shaft, particularly in bedrock.

• Dynamic pile load tests.

• Pile capacity assessment based on pile driving analysis.

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

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

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

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

6.6 Earth retaining systems

6.6.1 General

a. Scope

This section covers:

i. Non-integral bridge abutments (as compared with integral or semi-integral abutments defined in 4.8) and independent retaining walls associated with bridges. An abutment is defined as a substructure system that incorporates earth retaining members, and also supports part of the superstructure. Wing walls are part of the abutment if they are integral with it. Independent walls that are associated with bridges are defined as those walls that are not integral with the bridge abutment and which retain ground that provides support to bridge substructure elements and also walls that support approach fills at the bridge.

ii. Retaining walls not associated with bridges.

iii. Earth retaining structures (including MSE walls and slopes).

iv. Slopes designed on the basis of undergoing displacement.

b. Loads, displacements and settlement

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

6.6.1 continued

The designer shall derive the design loads on the structure in accordance with 6.2, taking into consideration the flexibility and likely deformation of the structure, and the allowable displacement or deformation of the system. Careful consideration shall be given to the interaction between the structure, the ground and foundations, under static, dynamic, earthquake and construction conditions. The deformation and displacement of the structure shall be compatible with the performance requirements for the structure and its interaction with adjacent or supported structures and facilities. Earthquake displacement criteria are specified in 6.6.9.

The design of all types of retaining wall shall consider the effects of total and differential settlement and designs shall accommodate all resulting effects. In particular, movement gaps and other measures may be necessary to prevent structural damage or to prevent unsightly cracking or spalling.

c. Side protection

   i. Road safety barriers adjacent to roads

   Barriers for traffic and vulnerable road users shall be provided for retaining walls supporting road carriageways, footpaths, cyclepaths and equestrian paths as required in appendix B.

   Where road safety barriers are positioned close to tops of earth retaining structures and can affect the performance of or apply additional load to the earth retaining structure, the earth retaining structure and its facing shall be designed to withstand the forces imposed on it by the design barrier loading (see B6.3).

   Road safety barriers shall not be fixed to the face of reinforced soil walls.

   ii. Safety fences and barriers remote from roads

   Safety from falling protection shall be provided at the top of retaining structures (and slopes) that are within the highway reserve but remote from the road that are adjacent to other public areas where people could fall 1.0m or more. The form of the protection provided shall meet the requirements of the *New Zealand building code* acceptable solution F4/AS1[44].

   Consideration shall also be given, through a risk assessment approach, to the need for safety from falling protection at the top of retaining structures (and slopes) within the highway reserve but remote from the road, where there may be the occasional presence of people (see B2.9) and a serious falling hazard exists within 1.5m of the edge of the path or working area. This shall take into account the frequency and nature of inspection and maintenance activities, and also the likelihood of public access to the area. For these purposes a serious falling hazard may be considered as defined in clause 5.19.1(c) of the *Highway structures design guide*[45].

   Where a barrier is considered to be necessary, as a minimum, safety from falling barriers 1000mm high, complying with the requirements of clause 1.2.2 of *New Zealand building code* acceptable solution F4/AS1[44] (excepting the reduced height) shall be provided. Barriers 1100mm high shall be provided where children less than six years of age are expected to frequent a public area.

   In both the above situations, the structural design of the barriers shall be in accordance with section 5.19.2 of the *Highway structures design guide*[45].

6.6.2 Design standards for earth retaining systems

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

- Road Research Unit bulletin 84, volume 2[3]
- BS EN 1997-1 *Eurocode 7 Geotechnical design* part 1 General rules[38], plus BS EN 1998-5 *Eurocode 8 Design of structures for earthquake resistance* part 5 Foundations, retaining structures and geotechnical aspects[39]
- AS 4678 *Earth-retaining structures*[46]
- S6 *Canadian highway bridge design code*[40]
- AASHTO *LRFD Bridge design specifications*[41]
- FHWA NHI-99-025 *Earth retaining structures*[47]
- FHWA-NHI-10-024 *Design and construction of mechanically stabilized earth walls and reinforced soil slopes* – volume I[67]
- FHWA-NHI-10-025 *Design and construction of mechanically stabilized earth walls and reinforced soil slopes* – volume II[68]
- CIRIA C580 *Embedded retaining walls – guidance for economic design*[48].

Road Research Unit bulletin 84[3] shall be used in preference to the other documents, particularly for evaluating seismic actions on walls.

The Waka Kotahi NZ Transport Agency *Bridge manual* shall take precedence over all other documents.

6.6.3 Strength reduction factors and factors of safety for earth retaining systems

Free-standing retaining structures shall be designed using loads and combinations as specified in tables 3.2 and 3.3 and section 6.2. The strength reduction factor for sliding shall be derived from table 1 of B1/VM4[37]. The strength reduction factors for bearing capacity of retaining structure foundations shall be derived using the risk based methodology set out in AS 2159[42] section 4.3, and shall be presented in the structure design statement for acceptance by the road controlling authority.

The strength reduction factors adopted for bearing capacity of retaining structure foundations shall be taken as $\phi_g = \phi_{gb}$, where ϕ_{gb} is defined in AS 2159[42] and shall not exceed a maximum value of $\phi_g = 0.6$ for all load combinations, excluding earthquake overstrength where higher strength reduction factors, up to $\phi_g = 0.75$, may be adopted.

Strength reduction factors adopted for bearing capacity shall not exceed a maximum value of $\phi_g = 0.75$, regardless of whether static, dynamic, or gravitational loading, or seismic loading induced by overstrength capacities developing are being considered.

The load combinations and load factors referred to in FHWA-NHI-10-024[67] shall be applicable for the design of mechanically stabilised earth walls, unless another method is agreed with the road controlling authority or is detailed in the principal's requirements for design and construct type contracts.

Potential deep-seated failure surfaces behind retaining and MSE structures extending below the toe of the structure shall be analysed. Both circular arc and sliding wedge methods shall be used.

A number of slope stability analysis computer programs are available most of which use unfactored loads and soil parameters. Therefore, unfactored loads and soil parameters shall be used for the deep-seated failure analysis.

The minimum acceptable factors of safety against deep-seated failure shall be as follows:

- for static conditions: factor of safety = 1.5
- for seismic conditions: factor of safety = 1.25

| 6.6.3 continued | If a retaining or MSE structure is designed for permanent displacement under earthquake loads, the above recommendations on the reduction factors and factors of safety will not apply. Requirements for structures designed for permanent displacement are specified in 6.6.9. |

6.6.4 Common highway earth retaining systems

Earth retaining systems commonly used for highway construction can be divided into different types as per table 6.4.

MSE structures comprise reinforcement elements in the ground to stabilise the soil against failure. For MSE walls a facing (eg reinforced concrete panels or blocks) connected to the reinforcement is generally provided.

MSE structures can be divided into two types:

- soil-nailed structures, where the reinforcement is inserted into the ground, with top-down construction as excavation proceeds
- reinforced soil structures, where the reinforcement is incorporated within fill as the fill is placed and compacted, using bottom-up construction.

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

Table 6.4: Earth retaining system categories

Earth retaining category	Earth retaining systems
Gravity, embedded and reinforced concrete cantilever walls	- gravity walls (concrete, gabion, crib) - embedded wall - reinforced concrete cantilever walls
Anchored walls and slopes	- anchored gravity walls - anchored cantilever walls - anchored soldier pile walls - anchored slopes
Mechanically stabilised earth (MSE) walls and slopes	- soil-nailed walls and slopes - reinforced soil walls and slopes – inextensible reinforcement – extensible reinforcement

6.6.5 Gravity and reinforced concrete cantilever walls

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

These walls may be designed to undergo limited sliding displacement under strong earthquake shaking as specified in 6.6.9.

6.6.6 Anchored walls

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

Anchors transfer the loads into the ground through:

- deadman structures
- grouting anchors into drilled holes
- mechanical systems.

6.6.6 continued

b. Anchors shall only be allowed to intrude into property outside the road reserve, if subsurface rights for the design life of the structure are obtained to prevent disturbance of the reinforced soil block by future subsurface (eg foundation, drainage) construction activities.

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

d. Ground anchors shall generally be designed and installed in accordance with established design standards such as BS 8081 *Code of practice for grouted anchors*[49], BS EN 1537 *Execution of special geotechnical work – ground anchors*[50] and FHWA-IF-99-015 *Ground anchors and anchored systems*[51], except as provided in this document.

e. The anchor system shall be designed to ensure a ductile failure of the wall, under earthquake overloads as discussed in 6.6.9.

f. The anchor system shall be corrosion protected to ensure its durability over the design working life of the structure.

Two classes of protection are provided for general use for anchors, as defined in table 6.5.

The class of corrosion protection shall be chosen based on the decision tree shown in figure 6.5.

In figure 6.5, a 'serious' consequence of failure shall arise when failure of the anchor and wall could:

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

In figure 6.5 'aggressive' shall be defined as where:

–   the maximum pitting corrosion rate of unprotected steel is greater than 0.1mm/year or,
–   soil resistivity is less than 2000ohm-cm or,
–   pH of either the groundwater or soil is less than 5.0 or,
–   total SO_3 in the soil is greater than 1% or,
–   sulphates in groundwater is above 2500ppm, or
–   chlorides in the groundwater are above 2000ppm.

Table 6.5: Class of corrosion protection for anchors and soil nails

| Class of protection	Corrosion protection standard
Class I	Double corrosion protection by encapsulation of the tendon or bar pre-grouted under factory conditions inside a corrugated plastic sheath to minimise crack widths in the pre-grouted grout as defined in BS 8081[49]. The whole assembly is grouted into the anchor hole.
Class II	Single corrosion protection using a galvanized (to AS/NZS 4680[52]) or fusion bonded epoxy-coated (to ASTM A934/A934M[53] or ASTM D3963/D3963M[54]) bar grouted into the anchor hole.

Note :
A higher class of protection, such as using multiple sheaths, may be chosen, depending on the aggressiveness of the environment, the consequences of anchor failure and the importance of the structure.

6.6.6 continued

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

h. On-site suitability tests shall be carried out on a selected number of initially installed special anchors or production anchors in accordance with BS EN 1537[50] to confirm the performance of the anchors and their suitability to ensure performance of the wall. A small number of representative full-scale anchors shall be installed and tested to confirm the suitability and performance of the anchors, prior to installation of the remainder of the anchors.

i. On-site acceptance tests shall be carried out on all anchors installed in accordance with BS EN 1537[50].

j. If there is any reason for concern about the long-term performance of anchors supporting structures, and/or their ability to achieve the required design working life, the designer shall provide for future inspection, re-testing and replacement.

k. The designer shall consider the long term maintenance of anchors and all elements forming part thereof. Where individual components (eg within the head assembly) have a working life less than the design working life, the designer shall detail replacement methodology in the inspection and maintenance requirements section of the structure design statements or soil structures report and within safety in design documentation. The long term monitoring and instrumentation should be carried out in accordance with FHWA-RD-97-130 *Design manual for permanent ground anchor walls*[55].

Figure 6.5: Guide to selection of corrosion protection for ground anchors
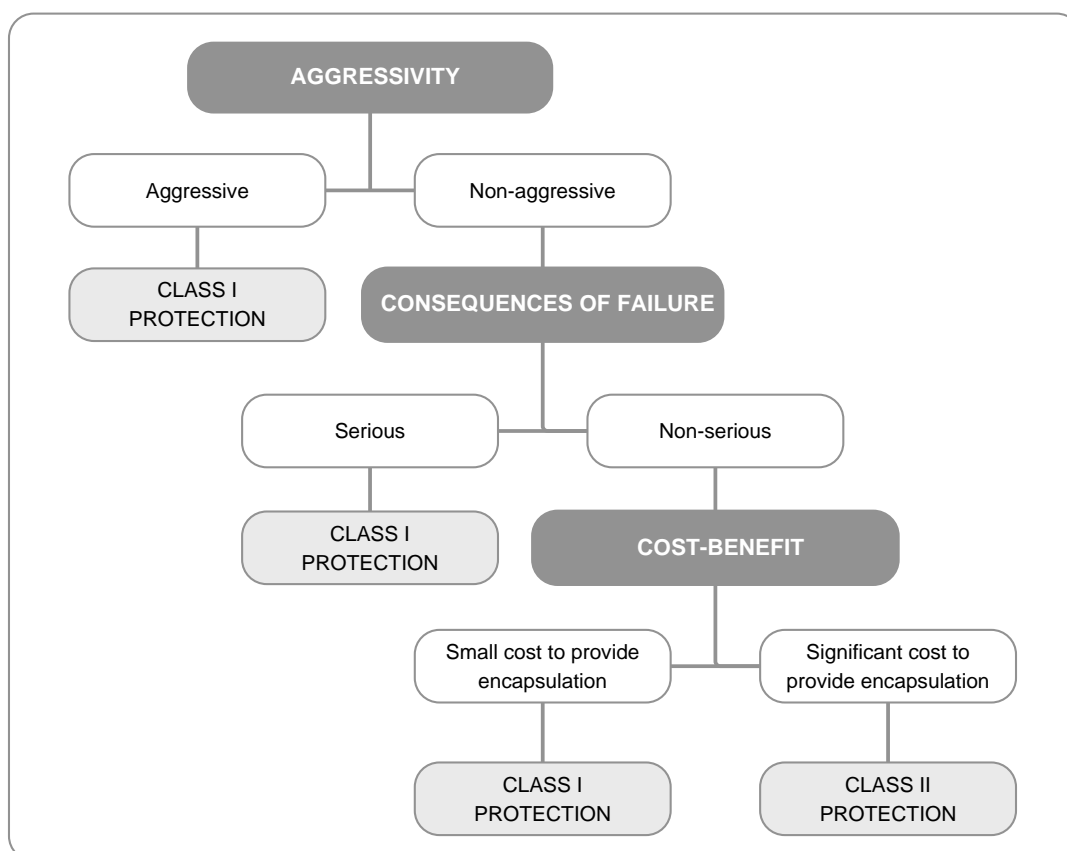
6.6.7 Soil-nailed walls

Soil-nailed walls shall be designed and constructed in accordance with appropriate design codes such as FHWA-SA-96-069R *Manual for the design and construction monitoring of soil nailed walls*[56], except as provided for in this *Bridge manual*.

Soil-nailed walls are acceptable subject to the following criteria:

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

b. Soil-nailed walls shall not support abutments of bridges, except where it can be demonstrated that the deformation associated with mobilisation of the soil nail capacities, or any displacements associated with earthquakes can be tolerated or catered for in the design of the bridge structure.

c. Overall limited block displacement in strong earthquakes may be allowed subject to the criteria in 6.6.9.

d. Soil nails shall only be allowed to intrude into property outside the road reserve, if subsurface rights for the design life of the structure are obtained to prevent disturbance of the reinforced soil block by future subsurface (eg foundation, drainage) construction activities.

e. The soil nail reinforcement shall be subject to the corrosion protection requirements specified in 6.6.6 for anchors.

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

g. On-site suitability tests shall be carried out on a selected number of production soil nails as per BS EN 1537[50] to confirm the performance of the soil nails and their suitability to ensure performance of the wall. A small number of representative full-scale soil nails shall be installed and tested to confirm the suitability and performance of the soil nails, prior to installation of the remainder of the soil nails.

h. On-site acceptance tests shall be carried out in accordance with BS EN 1537[50] on at least 25% of all installed soil nails. A higher proportion of nails shall be tested if the ground conditions are variable and the consequences of failure are high.

If there is any reason for concern about the long-term performance of soil nails and/or their ability to achieve the required design working life, the designer shall provide for future inspection, re-testing and replacement.

The designer shall consider the long term maintenance of soil nails and all elements forming part thereof. Where individual components (for example any associated nail head assembly) have a working life less than the design working life, the designer shall detail replacement methodology in the inspection and maintenance requirements section of the structure or soil structures report and within safety in design documentation.

6.6.8 Reinforced soil walls and slopes

Reinforced soil walls and slopes usually comprise either 'inextensible' (usually steel) or 'extensible' (usually geogrid) reinforcement. Reference should also be made to requirements with respect to earthquake design given in 6.6.9.

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

a. Inextensible (steel) reinforcement shall be used for reinforced soil walls and slopes supporting bridge abutments or where limiting the deformation of the wall is critical due to the presence of adjacent structures. Geogrid reinforcement may be used, provided that the bridge abutment seat is supported on piles, and the design, including pile design, takes into account the expected deformation of the wall system.

b. Design of geosynthetic-reinforced structures shall comply with appropriate design codes or manuals such as the recommendations of Waka Kotahi NZ Transport Agency research report 239 *Guidelines for design & construction of geosynthetic-reinforced soil structures in New Zealand*[57], except as otherwise provided in this manual.

c. The long-term durability, strength and creep performance of the reinforcement, and the environmental conditions associated with the site, backfill and groundwater shall be considered in the selection and use of appropriate types of reinforcement and backfill.

   In considering the rate of corrosion of the buried steel reinforcement, guidance may be obtained from AS/NZS 2041.1 *Buried corrugated metal structures* part 1 Design methods[58], and from the New Zealand Heavy Engineering Research Association (HERA) report R4-133 *New Zealand steelwork corrosion and coatings guide*[59].

d. Steel reinforcement shall have an adequate level of corrosion protection and/or sacrificial steel content to ensure the required performance over the design working life of the structure.

e. The strength of the connections between the soil reinforcement and the facing panels or blocks of reinforced soil walls shall exceed by a suitable margin the upper bound pull-out strength of the reinforcement through granular fill, or the post-yield over-strength capacity of the reinforcement, whichever is lower. Design shall ensure that brittle failures of the connections will not occur.

f. Any capping blocks provided shall be adequately fixed in position to resist dislodgement due to earthquake actions or vandalism.

6.6.9 Design performance of earth retaining structures and slopes

a. Permanent displacement of earth retaining structures and slopes in earthquakes

   Earth retaining structures and slopes may be designed to remain elastic under the design earthquake load specified in 6.2.2 or to allow limited controlled permanent outward horizontal displacement under strong earthquake shaking.

   Walls designed on the basis of permissible permanent outward horizontal displacement under strong earthquake shaking shall comply with all relevant recommendations of the Road Research Unit bulletin 84[3].

   The horizontal displacement likely at the design damage control limit state seismic response, and under the collapse avoidance limit state seismic response, shall be assessed using moderately conservative soil strengths consistent with the anticipated strain and a Newmark Sliding Block displacement approach. Displacements shall be assessed using the methods described by Ambraseys and Menu[66], Ambraseys and Srbulov[5] and Jibson[6] without liquefaction and lateral spreading, and Bray and Travasarou[7] for displacements due to liquefaction and lateral spreading using the relevant peak ground accelerations (see 6.2), and the distance to the dominant earthquake sources in the area.

6.6.9 continued

Where a Newmark sliding block method is applied, the 50th percentile displacements shall be derived for both the damage control limit state and the collapse avoidance limit state events. The range of predicted displacements (rather than a single value) should be used in the design process. In general, the upper bound values should be adopted unless a lesser value is otherwise justified and accepted by the road controlling authority.

Vertical accelerations shall be taken into consideration in the design of retaining structures. The energy and frequency content of earthquake shaking as well as the vertical earthquake motions (which tend to be high particularly in near field situations) have a significant effect on retaining wall performance in strong earthquakes. The effects of vertical shaking have been observed in recent earthquakes as well as in research sponsored by the Earthquake Commission Research Foundation (Brabhaharan et al, 2003[60]). (Vertical accelerations of up to 2.2g were observed in the Christchurch earthquake of 22 February 2011.)

The uncertainty in the assessment of wall and slope horizontal displacements using peak ground accelerations shall be taken into consideration in the assessment of likely wall and slope displacements, although the peak ground acceleration based estimates remain the only quantitative estimation methods currently available.

In the design of earth retaining structures and slopes that are allowed limited permanent outward horizontal displacement in the design earthquake:

i.   The soil strength parameters used for assessment of sliding horizontal displacement shall be large strain soil strength parameters (and not peak strengths), consistent with large soil strains from the predicted displacements.

ii. The probable ranges of soil parameters shall be considered when estimating the upper and lower bounds of threshold acceleration to cause wall or slope horizontal displacement.

iii. Walls shall be proportioned to ensure sliding, rather than overturning or internal instability (in the case of MSE structures).

iv. The expected horizontal displacement due to the design earthquake shall not encroach into minimum clearances from road carriageways and railway tracks or infringe property boundaries, or cause damage to services that may exacerbate movements or cause instability.

v.   It shall be recognised that, in near-field situations, the vertical accelerations associated with strong earthquake shaking would lead to larger horizontal displacements than assessed using peak ground accelerations alone. The design shall cater for larger horizontal displacements than those predicted using horizontal peak ground accelerations alone, to account for the effect of vertical seismic accelerations.

vi. The assessed likely horizontal displacements and settlements of the structure or slope that would arise from sliding due to the design earthquake shall not exceed the values given in 6.1.2.

6.6.9 continued

b. Walls and earth structures (including slopes) supporting abutments or piers

Subject to obtaining the agreement of the road controlling authority, where the bridge abutment and superstructure can be designed to remain serviceable with limited abutment displacement and without damage to the bearings or piles, and can retain adequate allowance for temperature change, vibration etc, walls or earth retaining structures supporting abutments or piers may be designed on the basis of sustaining permanent displacement not exceeding the limits specified in 6.1.2 under the design damage control limit state earthquake event subject to the limitations below. This shall be substantiated in the structure design statement, which shall include quantification of the damage due to the movements and the consequences for the use of the bridge and its permanent repair to full capacity for design loading and movements.

These displacements limits shall apply to displacements determined in accordance with 6.3.2 and 6.6.9(a).

In addition:
- where the structure has non-integral abutments, the clearance between the abutment back wall and the end of the end span main girders in the longitudinal direction shall be sufficient to accommodate 2.0 times the full damage control limit state seismic movement of the bridge superstructure plus one half of the thermal movement, plus 200% of the assessed 50th percentile permanent abutment displacements. The abutment back wall shall be designed to accommodate reinstatement of the knock-off element at a position displaced further back from its original position by an amount of 200% of the abutment permanent displacement due to the design damage control limit state earthquake event, without need to strengthen or reconstruct the abutment back wall. The same general principles shall be applied during consideration of the transverse earthquake loads.

- Where the bridge is supported by piles and columns at the abutments, the piles and columns shall be protected from displacement of the wall, earth retaining structure or slope, for example by use of a sleeve with adequate space to accommodate the greater of the 50th percentile collapse avoidance limit state event displacement and 200% of the assessed 50th percentile seismic displacement arising from one damage control limit state design intensity earthquake. Alternatively the bridge shall be capable of withstanding the greater of the force applied by soil translating past the piles and columns due to a CALS event and the forces applied by two sequential DCLS design earthquake events. CALS major earthquake (ie an event with a return period significantly greater than the design event) performance criteria shall apply to structures subjected to these loading conditions.

c. Gravity and reinforced concrete cantilever walls

Gravity and reinforced concrete cantilever walls may be designed so that either:
- the wall remains elastic and does not suffer any permanent displacement under the design earthquake load specified in 6.2.2 and 6.2.3, or
- limited permanent outward movement due to soil deformation is accepted (see 6.6.9(a)) and the wall is designed to avoid yielding of the structural elements wherever practicable. In this case provision shall be made to accommodate the calculated displacement with minimal damage, and without encroaching on clearances. Walls other than those supported on piles shall be designed to slide rather than rotate. Due account shall be taken of the probable range of soil strength when estimating the upper and lower bounds of the threshold acceleration to cause wall displacement. The resistance to overturning shall be greater than 1.25 times the overturning moment derived from the upper bound combination of forces that act to cause sliding.

6.6.9 continued

d. Anchored walls

Anchored walls shall be designed to remain elastic under the seismic loading specified in 6.2.2 and 6.2.3. Consideration shall be given to the consequences of anchor and wall flexibilities under design conditions. Walls shall be detailed to ensure that under seismic overload, controlled displacement of the wall will occur through yielding of the anchor material, and sudden failure will be avoided. All anchors bars shall have ductile post-yield behaviour up to at least 5% strain.

Particular attention shall be given to the post-earthquake effectiveness of the anchor corrosion protection.

e. Mechanically stabilised earth walls

Design of MSE walls shall comply with appropriate design codes or guides as listed in 6.6.2 and follow the recommendations for load and resistance factors as detailed in 6.6.3.

Section 6.6.7 provides guidance on the design of soil-nailed walls and design codes such as FHWA-SA-96-069R[56] also provide guidance.

A wall required to avoid permanent displacement shall be designed to remain elastic and stable under the design loading specified in 6.2.2.

The connection strengths between the reinforcements and the facing shall be such that the failure under earthquake overload is always ductile, that is, by either pull out of the reinforcement through granular materials without loss of pull-out capacity with displacement, or by yielding or deformation of the reinforcement, and not by failure of the connections. The strength margin over connection failure shall be at least 1.3.

Connections to facings, whether or not the facing is technically part of the retaining structure, shall be designed to ensure the reinforcing elements do not become separated from the facing during vertical movements for the seismic design accelerations.

A wall intended to undergo permanent displacement shall be designed so that the outward movement results from block sliding of the reinforced block as a whole and not due to internal instability or pull out of the reinforcement.

Using strip reinforcement, under earthquake overload, deformation shall preferably be by pull out of the reinforcement strips or, where this is impractical, by ductile extension of the reinforcement strips.

Where design is for pull-out, the nominal strength of the connection between the reinforcement and the wall facing shall be at least twice the pull-out force calculated from the probable apparent coefficient of friction. Upper and lower bounds of the threshold acceleration required to produce incipient failure shall be calculated by considering the reinforcement acting both horizontally and along the failure surface and allowing for probable variations in the pull-out resistance and yield strength of the reinforcement. Stability shall be checked under the upper bound acceleration. Design displacements shall not encroach on required clearances.

Using grid reinforcement, particularly geogrids with closely spaced transverse members, under earthquake overload, any internal deformation shall be through ductile elongation of the reinforcement rather than pull out of the reinforcement through the soil.

6.7 Geofoam road embankments

Geofoam is any manufactured geosynthetic material produced by an internal expansion process that results in a material with a texture of numerous, closed, gas-filled cells using either a fixed plant or an in situ expansion process. Expanded polystyrene-block (EPS-block) geofoam is a material that is widely used as lightweight fill in road construction.

The following documents provide guidance on the design of geofoam applications on roading projects:

- *Guidelines for geofoam applications in slope stability projects*[61]
- *Guideline and recommended standard for geofoam applications in highway embankments*[62]
- *Geofoam applications in the design and construction of highway embankments*[63].

6.8 Geosynthetic soil reinforcement

6.8.1 Product approval

Geosynthetic soil reinforcement and systems employed in the reinforcement of soil structures (embankments, slopes, reinforced soil walls, etc) are relatively new materials with widely varying properties and a relatively limited history of application and proven performance.

Where geosynthetic soil reinforcement is proposed to be used, the specific geosynthetic reinforcement material and supplier shall be subject to the approval of the road controlling authority. Documentation to be submitted in support of an application for approval shall include the following:

- For geosynthetic reinforced soil wall (GRS) systems and their components (including geosynthetic reinforcement):
  - GRS system or component development and the year it was commercialised.
  - GRS system or component supplier organisational structure, engineering and construction support staff.
  - Limitations and disadvantages of the system or component.
  - A representative list of previous and current projects with the same application in areas with similar seismicity, as well as the names of the project owners, including names, addresses and telephone numbers of representatives of the owners who hold the authority to provide references on behalf of the owner.
  - Sample material and control specifications showing material type, quality, certification, test data, acceptance and rejection criteria and placement procedures.
  - A documented field construction manual.
  - Design calculations and drawings for the proposed application.
- For geosynthetic reinforcement, the following additional information is also required:
  - Polymer and additive composition of the geosynthetic material, including polymer and additive composition of any coating materials.
  - Past practical applications of the geosynthetic material use with descriptions and photos.

6.8.1 continued

- Limitations and disadvantages of the geosynthetic material.
- Sample long-term design strength and interaction values, and index property specifications.
- Laboratory test results documenting creep performance over a range of load levels, for a minimum duration of 10,000 hours.
- Laboratory test results, along with a comprehensive literature review, documenting extrapolation of creep data to a 100-year design life.
- Field and laboratory test results, along with a literature review, documenting reduction factors for installation damage.
- Laboratory test results and extrapolation techniques, along with a comprehensive literature review, documenting chemical resistance of all material components of the geosynthetic and reduction factors for chemical degradation.
- Susceptibility of the geosynthetic to degradation by hydrolysis, which may lead to premature failure.
- Where a potential for biological degradation exists, laboratory test results, extrapolation techniques, along with a comprehensive literature review, documenting biological resistance of all material components of the geosynthetic and reduction factors for biological degradation.
- Laboratory test results documenting the test method and the value of short-term strength.
- Laboratory test results documenting joint (seams and connection) strength and values for reduction factor for joints and seams.
- Laboratory tests documenting long-term pull-out interaction coefficients for the project site-specific soils.
- Laboratory tests documenting the direct sliding coefficients for various soil types or for the project site-specific soils.
- Robustness of the geosynthetic against damage during construction, including test results for use with similar reinforced fill materials as proposed, and adequate junction strength in the case of geogrids.
- The manufacturing quality control programme and data indicating minimum test requirements, test methods, test frequency etc. Minimum conformance requirements shall be indicated. Data shall be from a laboratory qualified and registered by IANZ for the testing. Data from an equivalent international laboratory may also be accepted, at the discretion of the road controlling authority.
- The reduction factors applied in the design. These shall be as recommended by the manufacturer/supplier (based on product-specific testing) or a combination of manufacturer/supplier recommended values and default values recommended by Waka Kotahi NZ Transport Agency research report 239[57].

The approval by the road controlling authority should include establishment of a set of index criteria for the purpose of quality assurance testing during construction (refer to 6.8.3).

Parameter values stated on product approval certificates shall be exclusively used in design.

6.8.2 Material properties

The geosynthetic reinforcement shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geosynthetic reinforcement structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced. The geosynthetic reinforcement shall be sufficiently durable to ensure that it continues to fulfil its intended function throughout the design life of 100 years.

The geosynthetic reinforcement shall have a design tensile strength, pull-out and direct shear parameters and other properties adequate to satisfy the performance requirements of earth structure in which it is being utilised.

The permeability of the geosynthetic reinforcement shall be greater than the permeability of the fill soil in which it is being placed.

6.8.3 Quality assurance and index properties

Each roll of product shall be labelled with unique identifiers that allow traceability back to the manufacturer and thereby the manufacturing process. Records of product used including the location, level and batch/lot number shall be kept and incorporated into maintenance documentation,

Quality assurance (QA) procedures shall be included to ensure that each product satisfies the design parameters and values required by the design and approval certificates.

Testing procedures for measuring design properties require elaborate equipment, tedious set-up procedures and long durations for testing. These tests are inappropriate for QA testing of geosynthetic reinforcements received on site. A series of index criteria shall be established for QA testing of geosynthetic reinforcement materials received on site during construction. These index criteria should include mechanical and geometric properties that directly impact the design strength and soil interaction behaviour of the geosynthetics.

6.9 References

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