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Theme 6 – Recommendations for earthquake resilient design of earthworks

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1 Introduction

1.1 Research overview

The 14 November 2016 M_w 7.8 Kaikōura earthquake triggered over 30,000 landslides, hundreds of significant landslide dams and damaged hillslopes that are now susceptible to failure during rainstorms and aftershocks (Massey *et al.*, 2018). The damage caused by the earthquake included landslides, debris flows, rock falls, failure of retaining walls and bridges, fault rupture and slumping of embankments located over a 200 km long stretch of land in the northeastern part of the South Island. This resulted in severe disruption to transport infrastructure in the North Canterbury and Marlborough regions, with the Main North Line railway (MNL) closed for 10 months and State Highway 1 (SH1) closed for over a year.

The Ministry of Business Innovation and Employment (MBIE) has funded a programme of research into earthquake-induced landscape dynamics (EILD) following the Kaikōura earthquake under its Endeavour research programme. This is being led by GNS Science, with 7 research themes addressing different aspects of the landslides and sediment cascades triggered by that earthquake.

Under this research programme, WSP has been commissioned by GNS to investigate and analyse the performance of engineered and modified slopes along transport routes in the earthquake, and to develop recommendations for resilient slope design and landslide hazard management.

The objectives of this research theme are to:

Step 1: Map the locations and extents of failures of cut slopes, natural slopes, fill embankments, and retaining systems along the transport corridors affected by the 2016 Kaikōura earthquake;

Step 2: Carry out site investigations at selected key slope failures triggered by the Kaikōura earthquake;

Step 3: Analyse selected landslides from the Kaikōura event to characterise the slope failure mechanisms and relate these to the observed impacts;

Step 4: Identify critical factors that contributed to the slope failure impacts and develop recommendations for best practice measures for the resilient design of earthworks;

Step 5: Disseminate the recommendations amongst the engineering profession.

1.2 Outputs

This report presents the results of Step 4 above.

The following reports have been prepared as outputs from Steps 1 to 3:

Step 1: Landslide inventory mapping report and accompanying GIS data (Mason and Brabhaharan, 2023a);

Step 2: Factual site investigation report (Mason and Brabhaharan, 2023b);

Step 3: Assessment report (Mason and Brabhaharan, 2023c).

1.3 Scope of this report

The work carried out under this step consists of the following:

(a) Carry out a literature review of relevant standards and guidelines for the design of earthworks for seismic loading, and consider whether any aspects of current design practices would have significantly mitigated the likelihood or impacts of the observed slope failures;

(b) Review the findings from the mapping, investigation, and analysis of slope failures in Steps 1 to 3 to identify the critical factors in the performance of slopes in the Kaikōura earthquake that contributed to the observed failures and damage impacts;

(c) Integrate the results of (a) and (b) and make recommendations for resilience-based design of earthworks under seismic loading in New Zealand for roading projects.

1.4 Study area

This study focuses on the key transport corridors through the Kaikōura earthquake-affected region, as shown in Figure 1. These include State Highway 1 (SH1) and the Main North Line railway (MNL) between Picton and Waipara, State Highway 7 (SH7) between Hanmer Springs and Waipara, the Inland Route 70 between Culverden and Kaikōura, and Awatere Valley Road in Marlborough. The geology and geomorphology of the study area, and the distribution of slope failures triggered by the Kaikōura earthquake, are described in more detail in Mason and Brabhaharan (2023a).

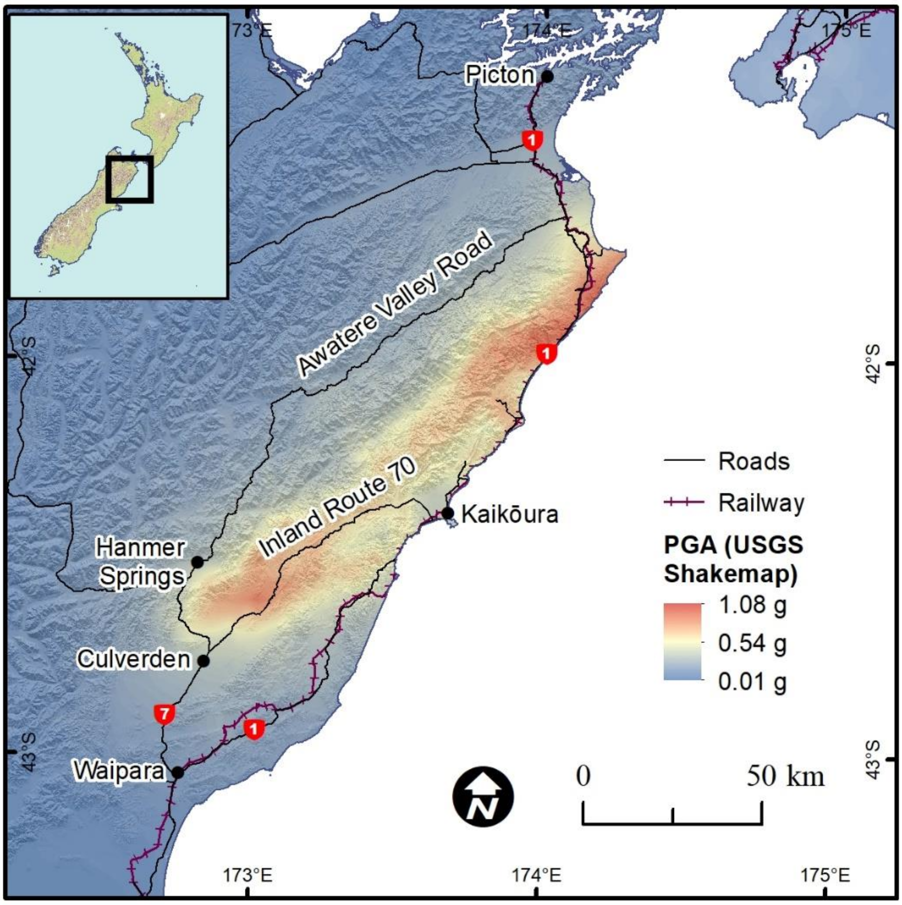

Figure 1: Study area

2 Review of standards and guidelines for design of earthworks

Relevant design standards and guidelines for the design of earthworks under seismic loading have been collated and reviewed. This included New Zealand, European and USA standards.

2.1 NZS 1170.5 (2004) Structural design actions – Earthquake actions

In New Zealand, NZS 1170.5: 2004 Earthquake actions – New Zealand (SNZ, 2004) provides guidance on the selection of earthquake loading for structural design actions. Unfortunately, it specifically excludes the parts of the built environment associated with slopes including soil retaining structures, slope stability, liquefaction, dams and bunds.

2.2 New Zealand Geotechnical Society – Ministry for Business Innovation and Employment (2021) Geotechnical Module 1

Geotechnical Module 1 provides seismicity parameters for assessment and design of geotechnical aspects of developments. This can be used to select peak ground accelerations and effective magnitudes for geotechnical assessment and design.

2.3 New Zealand Seismic Hazard Model 2022

A new national seismic hazard model has been developed for New Zealand over the past few years and was released in October 2022. This gives much higher levels of seismic loads than previous standards for many areas of New Zealand. However, design standards to incorporate this into design practice are still under development.

2.4 NZS 4431 (2022) Engineered fill construction for lightweight structures

NZS 4431: 2022 *Engineered fill construction for lightweight structures* (SNZ, 2022) describes processes to be adopted for geotechnical investigation, design, construction, quality assurance testing, and certification of engineered fill as foundation support for lightweight buildings and associated infrastructure. The standard was updated in 2022 to incorporate advancements since the previous edition in 1989 in the areas of geotechnical investigation, design and construction practice (amongst others). The standard now requires safety in design and sustainability in design issues to be assessed and incorporated into the design and construction specification, as well as for future operation, maintenance, and deconstruction or renewal activities.

Design requirements for engineered fill are set out in the first section of the standard. For the earthquake design of engineered fills, NZS 4431 requires the designer to assess the stability of the site and adjoining land to ensure an adequate factor of safety for the construction period and the long term. Factors of safety, design seismic load cases, design methodologies or acceptance criteria for displacement-based design are not specified, as the geotechnical design requirements are only covered at a high level. Instead, reference is made to published guidance or standards relevant to New Zealand conditions for expected seismic hazards at the site, but no specific standards or guidelines are mentioned. For engineered fills supporting state highways, the requirements of the Bridge Manual would take precedence.

In conjunction with the review of NZS 4431, the New Zealand Geotechnical Society prepared a generic earthworks specification (NZGS, 2022) to be used for most residential or light commercial development projects. The NZGS document specifies the requirements for:

- Acceptable type and condition of materials for use in fill embankments;
- Particle size criteria for material types;
- Construction activities, including excavation and formation and compaction of fill bodies.

2.5 Waka Kotahi NZ Transport Agency Bridge Manual

2.5.1 Scope of manual

Waka Kotahi NZ Transport Agency's Bridge Manual (NZTA, 2022) provides criteria for the design and evaluation of bridges, bridge abutments, culverts, underpasses, subways, retaining walls, reinforced soil and unreinforced embankments and cut slopes on the state highway network in New Zealand. The overall design philosophy of the Bridge Manual is based on limit state principles. The document provides design loadings, load combinations and load factors, together with criteria for earthquake resistant design. As the Bridge Manual is primarily for state highway infrastructure design, the design loading levels that structures are designed for is dependent on the importance of the route. Importance level and annual probabilities of exceedance for earthquake actions for earth retaining structures and earth slopes are given in Table 2.2 and Table 2.3 of the manual, respectively.

Design requirements for the design of bridge foundations, retaining walls and earthworks were gradually incorporated into the Bridge Manual from the early 2000s, and the principles behind those are presented by Brabhaharan (2006) and Kirkcaldie *et al.* (2009). Section 6 of the Bridge Manual now sets out the design philosophy and design criteria for the design of embankments, cut and fill slopes, foundations, and retaining structures.

2.5.2 Design of earthworks – Scope and performance requirements (Section 6.1)

The Bridge Manual defines "soil structures" as cut and fill slopes, stabilised slopes, embankments, retaining walls and earth retaining structures. The performance requirements for soil structures are given in terms of settlement and displacement limits for damage control limit state events (Table 6.1 in the manual), operational continuity requirements for serviceability limit state events, and emergency accessibility and damage reinstatement requirements for damage control limit state events (Table 6.2 in the manual).

2.5.3 Seismic design parameters (Section 6.2)

The Bridge Manual provides guidance on the selection of seismic design parameters for slopes associated with state highways by classifying the roads depending on their level of importance. No allowance is made in the Bridge Manual for changes to seismic design parameters, either to allow for amplification (such as due to topographic effects) or reductions (to allow for incoherence of motions where the slopes are of significant height).

In the current edition of the Bridge Manual (3rd edition, amendment 4), charts and formulae are provided for the derivation of the peak ground acceleration and effective magnitude for geotechnical design. These are derived using the hazard factor (based on the seismicity of the location), the return period factor (based on the importance and the road form) and the site subsoil class (A, B, C, D or E based on NZS 1170.5). The Bridge Manual refers to the paper published by Cubrinovski *et al.* (2022) for earthquake design parameters for six areas in central New Zealand (Gisborne, Napier, Whanganui, Palmerston North, Wellington and Blenheim and their neighbouring areas), the same reference used in the NZGS-MBIE Geotechnical Module 1 (NZGS-MBIE, 2021a) for earthquake design parameters for central New Zealand. A comprehensive study to update the national seismic hazard model for New Zealand was published in October 2022, and the results of this are yet to be incorporated into the Bridge Manual approach.

2.5.4 Assessment of slope or land stability in earthquakes (Section 6.3)

Potential slope instability is assessed using conventional slope stability analysis with load and strength reduction factors of 1, average groundwater conditions, and the seismic coefficient associated with the relevant earthquake accelerations as described above. Where the pseudo-static factor of safety is less than 1 and the failure mechanism is not brittle, potential slope displacements are assessed using the Newmark sliding block approaches of Ambraseys and Menu (1988), Ambraseys and Srbulov (1995), Jibson (2007) and Bray and Travasarou (2007).

2.5.5 Design of earthworks (Section 6.4)

Embankments (Section 6.4.1)

Under static conditions, embankments are required to have a minimum design long term factor of safety against all modes of failure of 1.5, using moderately conservative effective stress soil strengths under moderately conservative design operating piezometric conditions.

For seismic events, the stability of embankments is assessed using pseudo-static slope stability analysis. Embankments require a FOS ≥1.0, except where embankment stability does not affect bridges, in which case the FOS can be less than 1.0 with slope displacements allowable up to the following limits (summarised from Table 6.1 of the manual):

- Soil structures supporting a road carriageway with AADT <2500: 100 mm for a rigid wall, 200 mm for a flexible wall or slope capable of displacing without causing structural damage.
- Soil structures supporting a road with AADT ≥2500: 100 mm for a rigid wall, 150 mm for a flexible wall or slope.

The methodology used for establishing these displacement limits is not referenced, and there is no differentiation of allowable displacement to account for different types, heights or importance of soil structures.

Where slopes are to be designed for permitting displacement under earthquake loading, reference is also made to section 6.6.9 of the manual, which provides guidance on the performance of earth retaining structures and slopes.

Particular mention is made of the need to assess the potential for embankment materials and the underlying foundation materials to lose strength during or after an earthquake, the associated risks to the embankment, and the feasibility and cost of eliminating or reducing those risks.

Cut slopes (Section 6.4.2)

The manual does not provide specific guidance on the design of cut slopes other than that they shall be designed in accordance with recognised highway design practice, with the provision of benches and appropriate measures to mitigate effects of rock fall and minor slope failures.

Reference is made to the design of cut slopes generally complying with the requirements specified for embankments, with the minimum factors of safety for embankments applying to global stability of cuttings. Given that cut slopes are almost always in natural geological materials, compared to engineered retaining walls or engineered embankments, there is a greater level of uncertainty as to the materials and the mechanisms influencing cut slope stability. However, these distinctions for cut slopes are not differentiated in the manual. Similarly, the allowable displacement limits given for slopes would not apply to brittle natural materials such as rock masses that lose significant strength in the initial stages of failure.

It should also be noted that for a given level of importance of the route, the Bridge Manual requires cut slopes to be designed to a much lower level of earthquake hazard compared to other soil structures. This could lead to poor performance of cut slopes in earthquakes compared to other structures including embankments, and this issue is discussed in more depth by Brabhaharan *et al.* (2018).

Natural ground instability (Section 6.4.3)

The Bridge Manual includes provision for assessing the effects of natural slope instability on highways and associated structures. The manual requires measures to isolate any structure, soil structure or highway that could be affected by natural slope instability, remedy the instability, or design the structure or highway to accommodate the potential displacements and loads. The manual does not provide specific guidance on the assessment of natural slope instability; reference is made to the required factors of safety provided for the design of embankments, but the mechanisms of failure or the level of design event to be considered in the analysis of natural slopes are not covered.

2.6 NZ Transport Agency Research Report 613 (2018) – Seismic design and performance of high cut slopes

NZ Transport Agency commissioned Opus International Consultants (now WSP) to carry out research into and develop guidance for the design and performance of high cut slopes. The outcome of the research and guidance for design were published by the NZ Transport Agency as guidance for the seismic design of high cut slopes (Brabhaharan *et al.*, 2018). The guidance proposes a resilience-based approach to design where the importance level and resilience expectations of the route are used to determine the design approach for new cut slopes.

A four-level design approach is presented, from simplified design methods (suitable for low height cuts along low importance routes) through to detailed design methods considering complex failure mechanisms, topographic amplification of ground motions, and assessment of slope displacements, performance of the cuttings and the resilience consequences to the route. More detailed methods are proposed for high cut slopes or complex ground conditions where slope performance is critical for continued functionality.

The design approaches are consistent with the geotechnical limit state principles in the Bridge Manual, although more detail is provided on methods of analysis for different importance level slopes or geological complexity. Design methodologies are principally based on pseudo-static limit equilibrium analysis, with dynamic stress-deformation analysis suggested for situations where slope displacements, topographic amplification effects, or the effects of vertical accelerations need to be analysed. Assessment criteria such as minimum factors of safety or maximum allowable displacements are not provided, and the Bridge Manual requirements would therefore apply.

The design methodology of Brabhaharan *et al.* (2018) also makes the following key recommendations to the Bridge Manual approach:

- Peak ground accelerations for design are to be derived from the Bridge Manual, however higher hazard levels are proposed for selecting design ground accelerations, given the current edition of the Bridge Manual requires cut slopes to be designed to a lower level of hazard compared to fill slopes, regardless of the cut slope height or consequence of failure to the road.
- Topographical amplification of ground motion is included as a topographic amplification factor (TAF) for use in pseudo-static limit equilibrium analyses. The TAF varies from 1 to 3 depending on the type of slope (ridge or terrace), the height of the slope, and the slope angle.
- A reduction factor is applied to the design ground accelerations for large failure mechanisms of high slopes to account for the spatial incoherence of ground motions experienced by deep-seated failure mechanisms.

Embankments are not covered as this was outside the scope of that research report.

2.7 New Zealand Geotechnical Society earthquake geotechnical engineering modules

A series of guideline documents (modules) have been developed for the earthquake geotechnical engineering practice in New Zealand and published by the New Zealand Geotechnical Society (NZGS) and the Ministry for Business, Innovation and Employment (MBIE). The modules were published in 2016 and were revised in 2021. The modules address the following areas:

- Module 1: Overview and ground motion parameters

- Module 2: Geotechnical investigations
- Module 3: Assessment and mitigation of liquefaction hazards
- Module 4: Earthquake-resistant foundation design
- Module 5: Ground improvement
- Module 6: Earthquake-resistant retaining wall design

Module 1 (NZGS-MBIE, 2021a) presents an overview of the modules and provides ground motion parameters for geotechnical design (PGA and magnitude). Peak ground accelerations for Site Class C are recommended in the NZGS-MBIE guidelines to be used for all site classes, following the findings of Bradley *et al.* (2022) and Cubrinovski *et al.* (2022). These recommendations apply to the geotechnical design of cut slopes and engineered fills, until the update to the national seismic hazard model is finalised and adopted in seismic design standards.

Module 6 (NZGS-MBIE, 2021b) covers the design of retaining walls for earthquake loading. Whilst the design of retaining walls is outside the scope of this study, the method for deriving the design horizontal coefficient of acceleration is relevant to the design of cut and fill slopes. Module 6 specifies the design horizontal coefficient of acceleration (k_h) for pseudo-static limit equilibrium analysis to be derived as follows:

$k_h = \alpha_{max} A_{topo} W_d$

Where α_{max} = unweighted horizontal peak ground acceleration, A_{topo} = topographic amplification factor, and W_d = wall displacement factor.

The topographic amplification factors in Module 6 were adapted from Eurocode 8 (discussed below) and vary from 1.0 to 1.4. The wall displacement factor is a reduction factor to account for the acceptability of limited displacements for residential structures, as well as inertia and damping of the retained soil and wave scattering effects. The factor varies from 0.3 to 0.7.

None of the current NZGS-MBIE earthquake design modules cover the design of earthworks – cuttings, embankments, or slopes. Recognising the gap, NZGS is currently in the process of developing a slope stability guideline series with the support of EQC and MBIE.

2.8 International standards and guidelines

2.8.1 Eurocode 8

Eurocode 8 (EC8) provides standards for the design of structures for earthquake resistance. Individual countries supplement this with their own specific information in the national annexes appended to the Eurocode.

EC8 Part 1 (CEN, 2004a) provides the basis for derivation of seismic loads, based on:

- The importance of the structure and associated importance factor.
- The reference peak ground acceleration and reference return period chosen by the national authorities for each seismic zone, corresponding to a no-collapse requirement.
- Ground type (A to E, S1 and S2).
- A topographical amplification factor, for important structures.

It also provides for representation of earthquake motions as a time history.

Part 5 (CEN, 2004b) provides for geotechnical structures, including consideration of slope stability associated with natural or artificial slopes, for structures on or near such slopes. It provides for analyses either by means of:

- Established dynamic analyses such as finite element or rigid block models, or
- Simplified pseudo-static methods, provided that:
    - Surface topography and soil stratigraphy do not present very abrupt irregularities.
    - The soil is not capable of developing high porewater pressures or significant degradation of stiffness under cyclic loading.

In the pseudo-static analyses method, EC8 proposes the following methods of derivation of seismic inertia forces:

- $F_H = 0.5\alpha * S * W$
- $F_V = 0.5F_H$, if vertical acceleration / design horizontal acceleration ratio is greater than 0.6
- $F_V = 0.33F_H$, if vertical acceleration / design horizontal acceleration ratio is less than 0.6

Where α = peak ground acceleration on rock, S = ground type factor, W = weight of the sliding mass.

A topographic amplification factor is provided for situations where the importance factor is > 1. Annex A of EC8 provides guidance on derivation of the topographical amplification factors. These amplification factors are proposed when slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m. For average slope angles of less than about 15°, the topographic effects may be neglected. The topographical amplification factors proposed in EC8 are summarised in Table 1.

Table 1: Topographical amplification factors from Eurocode 8

Topographical situation	Topographical amplification factor				
	Sites near top edge			Sites in between top and base	Sites at base of slope
	Slope angle				
	< 15°	15°-30°	> 30°		
Isolated cliffs and slopes	1	≥ 1.2		Linear interpolation between base and top edge of slope	1
Ridges with crest width significantly less than base width	1	≥ 1.2	≥ 1.4		1

Notes:

(1) In the presence of a loose surface layer, the smallest value of the factor given in the table should be increased by 20%

(2) Seismic amplification also decreases rapidly with depth within the ridge. Therefore, for deep seated failures surfaces passing near to the base, the topographical amplification factor may be neglected in pseudo-static analyses.

The bibliographical background of the recommended values for topographical amplification factor proposed in the EC8 is not clear. Following literature review and communications with international experts, our conclusion is that they were derived as an average of the values proposed in a number of precedent studies and published analyses.

2.8.2 NCHRP Report 611 (2008) – Seismic analysis and design of retaining walls, buried structures, slopes and embankments

The US National Cooperative Highway Research Programme developed analysis and design methods and recommended load and resistance factor design (LRFD) specifications for the seismic design of retaining walls, slopes, embankments and buried structures under research project 12-70, described in Report 611 (NCHRP, 2008). Section 8.3 of the report contains proposed approaches for the seismic design of embankments and slopes, summarised as follows:

(1) Limit equilibrium approach

   a. Conduct static slope stability analysis to confirm that performance meets static loading requirements.

   b. Establish the upper bound of the seismic coefficient k_{max} = F_{pga} PGA (where F_{pga} = AASHTO peak ground acceleration site factor and PGA = USGS mapped acceleration coefficient for site class B conditions).

   c. Modify k_{max} to find the average peak acceleration in the potential failure mass accounting for slope height effects $k_{av} = \alpha\, k_{max}$ (where α = slope height reduction factor#).

   d. Reduce k_{av} by a factor of 0.5 to find k_s (assuming permanent displacements of 25 to 50 mm are permissible).

   e. Conduct a conventional slope stability analysis using $k_s = 0.5\, k_{av}$. If the factor of safety is ≥1.1 the slope meets seismic stability requirements.

   # Based on parametric wave propagation analyses conducted to evaluate the variation in average ground acceleration behind retaining walls and within slopes, as a function of slope height.

(2) Displacement-based method

   a. Conduct static slope stability analysis to confirm that performance meets static loading requirements.

   b. Establish the site peak ground acceleration coefficient k_{max} = F_{pga} PGA (where F_{pga} = AASHTO peak ground acceleration site factor and PGA = USGS mapped acceleration coefficient for site class B conditions)

   c. Modify k_{max} to account for slope height effects for full slope or embankment height stability analyses $k_{av} = \alpha\, k_{max}$ (where α = slope height reduction factor).

   d. Determine the yield acceleration k_y using a pseudo-static stability analysis for the slope using undrained strength parameters.

   e. Establish the earthquake slope displacement potential corresponding to the value of k_y/k_{max} using Newmark displacement charts provided.

   f. Evaluate the acceptability of the displacement based on performance criteria established by the project owner.

NCHRP (2008) note that Newmark displacements provide an index of probable seismic slope performance, and that previous studies suggest that displacement-based analyses of slopes are very sensitive to the frequency and amplitude characteristics of earthquake acceleration time histories and to earthquake duration. The report also notes that dynamic response of the sliding mass on slopes and embankments greater than c. 10 m height may influence displacement magnitudes, and therefore modifications to calculated Newmark displacements may be required. The report refers to the Southern California Earthquake Centre guidelines (Blake *et al.*, 2002) for examples.

2.8.3 FHWA (2011) – LRFD seismic analysis and design of transportation geotechnical features and structural foundations

The US Federal Highway Association developed a reference manual for load and resistance factor design of geotechnical features and structural foundations (FHWA, 2011). The manual serves as a technical resource for seismic analysis and design of geotechnical features such as soil and rock slopes, earth embankments, retaining structures and buried structures, and structural foundations.

Seismic stability analysis methods include pseudo-static and displacement-based analysis approaches are described in Section 6.2 of the manual, which are based on NCHRP (2008). Section 6.2.2 of the manual states that most slopes can accommodate a limited amount of seismically induced movement (typically on the order of 25 to 50 mm) and therefore the seismic coefficient used in pseudo-static analysis should be equal to or less than 50% of the site-specific PGA. Section 7.3 of the manual presents example slope stability design approaches, primarily based on displacement-based analysis.

Section 7.3.4 of the manual proposes strategies for developing acceptable displacement criteria, noting that it is the responsibility of the asset owner to decide on what magnitude of displacement is acceptable (and the consequent risk). Considerations in this process include:

- The location of the slope (e.g. urban vs rural settings)
- Importance level of the route or traffic volume
- Proximity of nearby utilities or facilities and their vulnerability to permanent slope displacement
- Consequences of slope movement in terms of risk to public safety
- Type of failure mechanism and response of the slope to earthquake loading
- Material properties (material type, strength, and stiffness characteristics, whether the materials are brittle or ductile, potential for liquefaction or cyclic softening etc.)
- Aesthetics, vegetation and drainage of the slope.

A decision tree is provided to assist asset owners in determining acceptable slope displacement limits, as shown in Figure 2. The flowchart highlights that the issues associated with displacement needs to be considered, but provides no real guidance on the selection of acceptable levels of displacement.

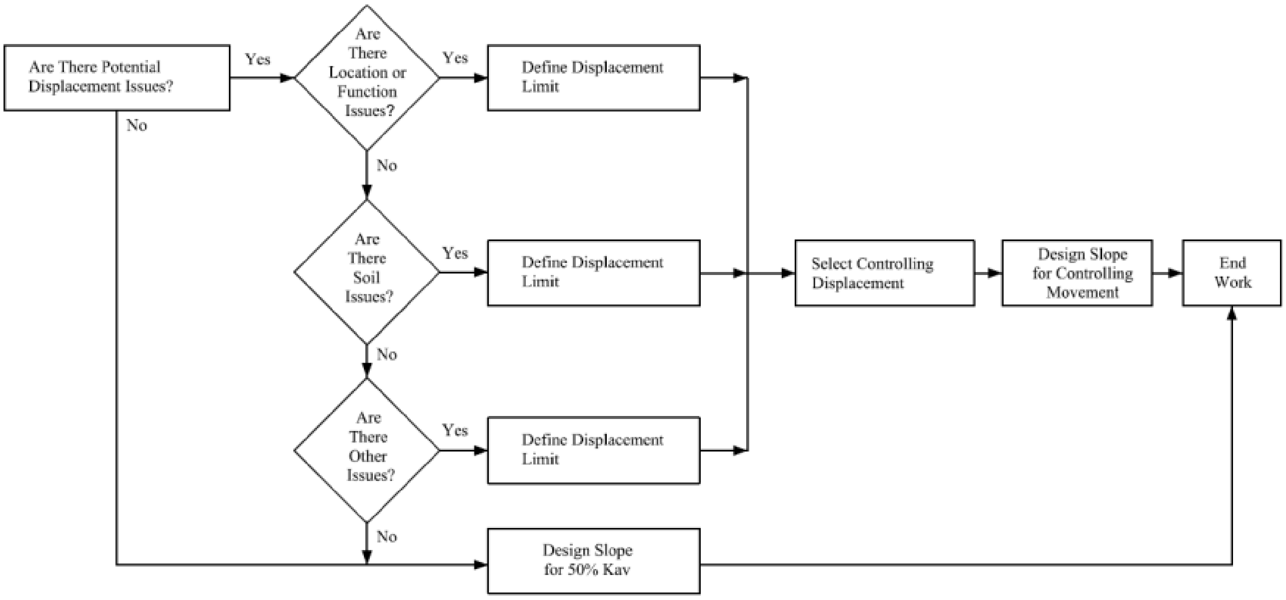

Figure 2: FHWA (2011) procedure for establishing acceptable slope displacement limits

2.9 Discussion

The selected slope failures that were investigated and assessed in Step 3 of this research project (Mason and Brabhaharan, 2023c) have been reviewed in the context of available guidelines, to assess what aspects of currently available guidance for the design of earthworks may have helped mitigate the failures, or to identify gaps. This is described in Table 2.

Table 2: Discussion of slope failures and relevant standards/guidelines

Site details	Contributing factors to observed performance	Discussion
Culvert 55 • 10 m high embankment for gully crossing in alluvial terrace terrain. • Underlain by late Quaternary alluvial gravels. • Constructed ~2000-2001. • Translational slump triggered by 2016 earthquake with 5.8 m displacement.	• Large displacement probably due to strength degradation of the fill materials during long duration ground shaking. • Elevated groundwater level within fill due to blocked culvert, upstream pond, and lack of subsoil drainage measures within embankment. • Lack of construction quality assurance, with topsoil or weak materials possibly not undercut and removed prior to embankment construction.	Back analysis of the slope using a lower groundwater level would satisfy Bridge Manual FOS requirements. More effective maintenance of the culvert (or enhancing the design capacity of the culvert to transmit sediment) is therefore likely to have reduced the damage. The embankment is unlikely to have been designed for a long duration of strong shaking, as the design was carried out in 1998, although the specific seismic design parameters have not been able to be confirmed from the literature review. Similarly, the material specification and construction controls at the time the fill was formed are also not known. Notwithstanding this, given developments in understanding of seismic hazard in NZ over the c. 25 years since the slope was designed, analysis of the slope using current best practice would include higher design ground motions (e.g. NZGS-MBIE Module 1) and dynamic response of the potential sliding mass (e.g. NCHRP). Given the observed magnitude of displacement was far in excess of the estimated Newmark displacements, limiting the design displacements to small values would be required to ensure adequate performance, such as use of better fill materials or inclusion of geogrid reinforcement within the fill slope. Design of a similar 10 m high embankment using current standards would not require a topographic amplification factor to be applied to the seismic coefficient. However, we note that amplification of PGA was observed in the decoupled slope analysis, suggesting that dynamic response of embankments 10 m or more in height is an important consideration (NCHRP, 2008).
Hundalee Forest • 2-5 m high sidling fill embankments and 1-3 m high gabion walls in hilly terrain. • Underlain by Neogene siltstone. • Originally formed in early 1900s; thickened and widened in 2009-2010. • Widespread cracking and displacement of the fills and walls in the 2016 earthquake.	• Progressive thickening of the fill slopes and construction of unreinforced gravity retaining walls for road widening to incorporate guardrails without geotechnical engineering design. • Weak soils below embankment and retaining wall foundations and lack of subsoil drainage measures led to excessive slope displacement and retaining wall failures.	Stability analysis considering the weak materials in gullies or on slopes below embankments and retaining walls is required by the current edition of the Bridge Manual. Suitable investigation and analysis of these slopes would identify the presence of weak/soft materials and therefore measures such as undercutting of unsuitable soils from the subgrade, replacement with appropriately compacted fill, and inclusion of geogrid reinforcement behind the retaining walls would have limited the deformation, loss of support and consequent damage that occurred in the 2016 earthquake.

Site details	Contributing factors to observed performance	Discussion
The Sandpit • 3-5 m high sidling fill embankments and low height (1-3 m) gabion walls in hilly terrain. • Underlain by late Quaternary aeolian sand. • Originally formed in late 19th Century; progressively thickened and widened from 1960-2015. • Translational slide triggered by 2016 earthquake with 7.8 m displacement.	• Lack of design (static and seismic) as embankments formed over existing slope underlain by loose to medium dense dune sand. • Progressive realignments resulted in incremental thickening of the fills using material cut from the uphill side of the road. • The fill slopes and walls in this area are unlikely to have been designed for seismic loading and experienced strong ground shaking over a significant duration in the Kaikōura earthquake.	Lack of engineering input into the design of the fills is a legacy of historical construction practice given the road corridor through this area was first constructed in the late 19th Century. Application of the current Bridge Manual criteria would require suitable investigations and analysis to confirm the factor of safety or slope displacements. Given the in situ soils consist of loose sand deposits, it is considered likely that the pseudo-static FOS would be less than 1 and a displacement-based design approach would be utilised. Engineered slope stabilisation, reinforcement or retention measures to limit displacements would be required by the Bridge Manual; analysis using NCHRP or FHWA analysis approaches may result in lower calculated displacements as these incorporate reduction factors to account for the spatial incoherence of ground motions for slopes greater than c. 10 m height. (We note that the NCTIR recovery works at this site were also not designed to meet full Bridge Manual requirements – a departure from the Bridge Manual was approved for a 2–5-year design life with accordingly lower design seismic loading.)
Awatere Gorge • Steep northwest-facing hillslope ~170 m high. • Awatere Valley Road crosses the hillslope approximately 100 m below the top of the slope. • Underlain by Pahau Terrane greywacke. • Wedge and joint step-path sliding failures were triggered by the 2016 earthquake. • Disintegration of the landslide mass and runout as a rock/debris avalanche completely buried the road.	• Strong ground shaking potentially amplified due to topographic and/or directivity effects. • Weathered surface layer of bedrock due to alpine environment resulting from weakening/dilation of rock mass. • Indurated and tectonically deformed rock mass with high intensity of short persistence joints providing multiple degrees of freedom for propagation of failure surfaces as well as persistent defects (bedding, sheared zones) providing potential failure planes for deeper seated structurally-controlled failures. • Steep, high hillslope. • Long runout path below failure area allowed transition of rock slide into debris avalanche.	The Bridge Manual requires the stability of natural slopes to be assessed if structures, soil structures or the highway could be impacted. However, there is very little guidance on the assessment methodology, performance criteria, or philosophy for determining what type of mitigation would be suitable. Under the Bridge Manual, the design seismic loading is determined by the importance level of the route. The remote location and low traffic volume of Awatere Valley Road only require consideration of earthquake loads at 1/100 AEP, which corresponds to a PGA of 0.2 using the current Bridge Manual or 0.28 using NZGS-MBIE (2021) Module 1. Back analysis of the slope resulted in a critical acceleration of 0.26. Therefore, a pre-earthquake assessment of the slope using the Bridge Manual would satisfy the FOS criteria and no mitigation would have been required. The Bridge Manual also makes no provision for topographic amplification which was observed in the slope analysis carried out in Step 3 of this study. Consideration of the resilience importance of the route, allowance for topographic amplification (e.g. NZTA-RR613), and the recent update to the national seismic hazard model would result in a significantly higher design PGA and consequently lower FOS that would highlight the vulnerability of the site to earthquake-induced landsliding. However, guidance for selecting appropriate types and extent of mitigation measures for this situation is lacking, and business case approaches are often required to justify risk mitigation. At this site, avoidance of the hazard would

Site details	Contributing factors to observed performance	Discussion
	• Narrow road corridor with no catch capacity for debris completely buried by landslide, with 8-week recovery to clear debris and reopen the road.	require extensive realignment of the road, and passive mitigation measures such as landslide barriers would not be practical or cost effective. Slope stabilisation (active mitigation) such as rock bolting could be feasible, but the low traffic volume would make the cost disproportionate to the level of risk reduction, and therefore unlikely to satisfy affordability criteria. However, access to remote communities needs consideration with a design that allows quicker restoration of access.
Kahutara Bridge • Moderately steep east-facing hillslopes 105 m high. • SH1 lies at the base of the hillslope with cut slopes up to 25 m high. • Underlain by Pahau Terrane greywacke. • Translational block slide on the hillslope and shallow wedge/avalanche failures on the cut slope were triggered by the 2016 earthquake.	• Steep cut slope in closely jointed, dilated, brittle rock mass resulted in shallow disaggregated rock mass failures that inundated the state highway. • Persistent outward-dipping bedding planes at depth in the lower half of the hillslope formed a kinematically admissible mechanism for the translational block slide. Cutting the toe of the hillslope for the road corridor reduced the potential resistance to sliding (legacy of historic construction practice).	Static stability of the slope was assessed to be >1.5, with a critical acceleration of 0.21. Design PGA for the site is 0.78 using NZGS-MBIE Module 1, as the proximity of the landslide to the Kahutara Bridge requires design seismic loading for a 1/1000 AEP design event compared to 1/500 for a slope not affecting a bridge. The Bridge Manual requires measures to isolate any structure, soil structure or highway that could be affected by natural slope instability, remedy the instability, or design the structure or highway to accommodate the potential displacements and loads. For this site, avoidance of the hazard such as realigning the road to avoid the block slide (for example constructing the road on an embankment on the shore platform or benching and partially removing the potential slide mass) would have major cultural, archaeological and ecological obstacles at this site. Engineered stabilisation or buttressing of potential block slides would be difficult to achieve in practice, and rock bolting slope would only be effective to limit the shallow wedge and disaggregated failures on the cut slope rather than the deeper seated structurally-controlled failure. Design of the cut slope using NZTA-RR613 would determine a suitable overall slope angle and bench configuration to minimise the impact to the state highway. However, assessment of the slope using RR613 would indicate the upper part of the ridge was more susceptible to failure because the topographic amplification factor would be applied to potential failures in this part of the slope and not the lower slope where the failure actually occurred. Engineering geology mapping and site investigations suggest that structurally-controlled failures were kinematically inadmissible in the upper half of the hillslope and therefore the feasible mechanisms of failure need to be carefully identified in an engineering geological model that underpins the assessment of slope response and stability. Measured actual displacements were far in excess of back-calculated displacements using semi-empirical methods and decoupled analysis. Further research and analysis are needed to help establish acceptability criteria for calculated displacements for similar structurally controlled rock slope failure mechanisms, given that the empirical relationships based on the Newmark sliding block model were from more ductile fill materials used to form dams and

Site details	Contributing factors to observed performance	Discussion
		embankments and assumes rigid-plastic behaviour of the landslide mass (Jibson, 2011; Newmark, 1965). Allowing displacements in brittle materials such as rock should be avoided or kept to very low values.
Okiwi Bay • Very steep to vertical bluffs 120 m high. • SH1 and MNL lie at the base of the hillslope. • Underlain by late Quaternary fan gravels and Pahau Terrane greywacke. • Block slide (combined rock wedge slide and breakout through rock mass) triggered by the 2016 earthquake.	• Steep, high, wave-cut coastal cliff. • Strong ground shaking likely to have been amplified by topographic and near-fault / hanging wall effects. • Evidence of prehistoric landslide activity. • Weak siltstone rock material with closely spaced incipient fractures as well as persistent outward-dipping sheared zones forming release for propagation of failure surface.	Assessment of natural slope instability is required by the Bridge Manual for the design of structures, soil structures and the highway if these could be impacted by slope movement. In this situation, assessment of the slope at Okiwi Bay would be for a 1/500 AEP design event. The PGA for this level of event is 0.56 g (from NZGS Module 1), which is very similar to the modelled PGA in the 2016 earthquake (0.57-0.6). Back analysis of the slope using the modelled PGA for the Kaikōura event resulted in FOS of ~0.7. The Bridge Manual requirement to avoid or remedy the instability or ensure the highway/structure is designed for the load applied by the slope failure would all be impractical for the scale of slope failure. Displacements assessed using decoupled and semi-empirical methods and the Kaikōura ground motions were between 0.1 m and 1.5 m. These were significantly smaller than the actual displacement of 50 m, highlighting the need to recognise the inappropriateness of using the semi-empirical displacement assessments based on the Newmark sliding block approach for structurally-controlled landslides in brittle materials.

3 Slope failure mechanisms

3.1 Identification of mechanisms of failure

A fundamental step in the assessment and design of slopes for earthquake performance is understanding the potential mechanisms of slope failure. This will enable the identification of the different factors that influence slope failure and the consequences of the failures. Here we combine observations from the classification and analysis of slope failure mechanisms on a corridor/regional scale (Mason and Brabhaharan, 2023a; Massey *et al.*, 2018) and local scale (Mason and Brabhaharan, 2023c; Singeisen *et al.*, 2022) to summarise the key observed slope failure mechanisms that will be important to account for in design of new slopes and assessment of existing slopes.

3.2 Rock slope failures

Rock slope failures were observed at 963 locations along the road and rail corridors and consisted of landslides on cut slopes as well as natural hillslopes within the transport corridor. The mechanisms of rock slope failures were classified using the schemes of Glastonbury and Fell (2000, 2010) and Hungr *et al.* (2014). The primary focus was to classify the initial failure mechanism in the source area so these can be used to inform future slope assessments. The transport mechanisms and velocity of landslide runout have not been assessed or captured in the inventory however the damage impacts have been classified (which relate strongly to landslide runout) and these are discussed in Section 4. The key failure mechanisms of engineering importance are summarised in Table 3.

3.3 Fill slope failures

Failure of fill embankments was observed at 424 locations. The modes of fill slope failures in the inventory were classified using fill slope deformation mechanisms described in previous large earthquakes by Rogers (1992) and Stewart *et al.* (2001); (2004), as well as the landslide classification scheme of Hungr *et al.* (2014). The key fill slope failure mechanisms are summarised in Table 4.

3.4 Retaining wall failures

Deformation of retaining wall structures occurred at 45 locations. Retaining walls which were tied back with geogrid reinforcement or ground anchors included gabion basket walls and timber pole walls up to 5 m retained height. Deformation of these structures consisted of minor displacement and rotation, causing subsidence of the fill materials behind the wall and consequential settlement-induced cracking of the road surface. Gravity retaining structures included gabion basket walls and crib walls ranging in height from 1 m to 3 m. These structures tended to perform poorly, with frequent overturning failures and a number of instances of translation observed of single basket-high walls. Pavement cracking and loss of shoulder support to the carriageway typically occurred at walls which exhibited this type of failure. The principal modes of retaining wall failures are summarised in Table 5.

Table 3: Rock slope failure mechanisms

Failure mechanism	Description	Contributing factors	Significance	Example
Topple / rock fall	Toppling of rock blocks bound by steeply-dipping defects. Shallow failure, typically with low volumes of debris (c. 1 m^3 to 100 m^3).	Unfavourable defects (joints).	Slope stabilisation often required because of unacceptable residual risk – lengthened the duration of outage of the corridor.	 Te Ana Pouri (NCTIR slip no. NS11), SH1
Wedge failure	Sliding along the line of intersection of two sets of persistent defects. Variable failure volumes, from low (<100 m^3) to high (c. 100,000 m^3) volume depending on the height of the slope.	Unfavourable defects (joints, sheared zones, bedding) with low strength.	Slope stabilisation often required because of unacceptable residual risk – lengthened the duration of outage of the corridor. Large scale wedge failures from slopes 100 m high generated large volumes of debris and correspondingly long outage. E.g. Mangamaunu (pictured) and Okiwi Bay.	 Mangamaunu (NCTIR slip no. NRP1B), SH1/MNL
Planar slide	Planar and rough translational sliding on continuous defects that daylight in the slope face. Deep-seated translational rock slides in Neogene sedimentary rocks remained coherent with 10^1 to 10^2 m displacement. Failures transitioned into debris avalanches where the downslope topography allowed.	Structural geological control on failure: Persistent defects such as bedding planes, fault zones, sheared zones with unfavourable orientation with respect to topography. Common in Neogene sedimentary rocks. Rare in greywacke given the high degree of fracturing.	Deep seated, structurally-controlled slides in inland hills generated large volumes of debris, damming rivers and causing sediment aggradation. Planar failures along the transport corridor extended over significant slope heights and required lengthy outage to clear debris and implement slope stabilisation or risk mitigation measures.	 Whales Back dip slope, Inland Road
Step-path en echelon slide	Unfavourable outward-dipping joints with m-scale persistence separated by short cm-scale persistence, steeply inclined joint steps between the m-scale discontinuities. Limited displacement of the failure mass leads to brittle failure, disaggregation and avalanche-type runout.	Unfavourable defects (joints) with close spacing and low strength. Brittle rock mass. Steep slope.	Characteristic failure mechanism in greywacke. Larger failure volumes than simple planar failures on similar height slopes. Involved in c. 40% of cut slope failures. Engineered mitigation measures often required.	 Punchbowl Corner (NCTIR slip no's SR23-24), SH1 (photo credit University of Canterbury)
Irregular or compound slides in jointed rock mass	Sliding along a rough, irregular path of short-persistence, interconnecting, outward-dipping defects. Limited displacement of the failure mass led to brittle failure, disaggregation and avalanche-type runout.	Jointed rock mass. Steep slope. Low rock mass strength.	Outage lengthened by unstable, dilated condition of rock mass in head scarp area requiring stabilisation or risk mitigation measures. Initiating mechanism of rock mass (avalanche-type) failures.	 NCTIR slip no. SR6, SH1 near Kahutara (photo credit J. Claridge)

Failure mechanism	Description	Contributing factors	Significance	Example
Combined defect-rock mass	Kinematic release on persistent planar defects and shear failure through the rock mass along the base by a secondary mechanism, e.g. shear failure through weak rock mass or sliding along a secondary defect. Generally coherent slide blocks, with some internal deformation.	Unfavourable persistent defects. Low rock mass strength. Over-steepened toe of slope.	Deep-seated failures, often extended over the full height of the slope. Mechanism of some of the largest failures along the transport corridor.	 Mason River, Inland Road (photo credit R. Ridl)
Disaggregated rock mass failure	Sliding and tensile release along a rough, irregular path of short-persistence, interconnecting, non-systematically oriented defects. Disaggregation of the landslide mass due to closely spaced fractures and lack of confining pressure at shallow depths within the slope, leading to complete strength loss and runout as a rock avalanche.	Jointed/dilated and low strength rock mass. Upper parts of steep slopes most susceptible.	Common failure type. Although generally shallow-seated these generated significant volumes of debris and caused the most damage and disruption to the transport corridor (e.g. Ohau Point).	 Iron Gate (NCTIR slip no. NRP2), SH1
Rotational/coherent slides	Deep-seated rotational slumps in massive, weak rock. Sliding mass generally remains coherent with little internal distortion due to limited displacement of the landslide.	Low strength, homogeneous rock mass. Oversteepened slopes e.g. coastal cliffs, river banks and road cuts.	Deep-seated failures, often extended over the full height of the slope.	 Lake Grassmere, Kaparu Road

Table 4: Embankment failure mechanisms

Failure mechanism	Contributing factors	Form of damage	Significance/transport impacts	Example
Subsidence	Loose fill density	Settlement, vertical displacement, cracking.	Cracking of road surface, difficulty in driving across.	 Hundalee Hills, SH1 (photo credit D. Coll)
Translational	Steep fill slope	Cracking, vertical and lateral displacement. Disaggregated runout.	Wide cracking of road surface, vertical steps, slumping and loss of (part or full) carriageways, loss of lane(s).	 The Sandpit, SH1
Rotational (or semi-rotational)	Weak surfaces within fill or foundations, relic surfaces. Perched or high groundwater.	Vertical and lateral displacement. Gross failure, slumping. Debris flow, fluidised flow.	Wide cracking of road surface, vertical steps, slumping. Loss of (part or full) carriageways, impassable. Deformation and displacement of buried structures (culverts, tunnels).	 Culvert 55, Hawkswood, SH1 (photo credit C. Parkes)
Lateral displacement	Poor foundations. Amplification of shaking.	Cracking, vertical and lateral displacement.	Wide cracking of road surface, vertical steps, impassable.	 Hundalee Hills, SH1 (photo credit C. Parkes)
Thrust	Load from structure	Cracking, vertical and lateral displacement.	Wide cracking of road surface, vertical steps, impassable.	
Liquefaction	Loose saturated cohesionless fill / foundation materials.	Subsidence, lateral spreading. Vertical and lateral displacement.	Wide cracking of road surface, vertical steps, impassable.	 Wairau Diversion stopbank, near Blenheim (photo credit Marlborough District Council)

Table 5: Retaining wall failure mechanisms

Failure mechanism	Contributing factors	Form of damage	Significance/transport impacts	Example
Overturning	Founded on top of slope, not tied back.	Wall detached and rolls down slope.	Loss of lane(s).	 Overturning of unreinforced gabion wall, Hundalee Hills, SH1 (photo credit M. Broughton)
Displacement	High earthquake load or designed for displacement.	Cracking of surface above.	Cracking of road, vertical steps in road pavement.	 Displacement of unreinforced crib wall within gully, Hundalee Hills, SH1 (photo credit M. Broughton)
Rotational slip with bulging of the wall	Overloaded due to surcharge and earthquake loads.	Damage to wall due to bulging and rotation.	Cracking / slumping of road pavement.	 Rotational slump and bulging of crib wall supporting bridge abutment, Parnassus, SH1 (photo credit C. Parkes)
Global rotational failure of wall through foundation	Weak foundation materials.	Complete failure.	Loss of road platform (lanes).	
Structural failure of abutment walls	High earthquake loads and structural thrust.	Cracking / displacement of structure.	Cracking, reduced load capacity of bridge/structure.	 Cracking of Oaro rail overbridge abutment, SH1 (photo credit G. Saul)

3.5 Fault-proximal landslides

In the wider earthquake-affected region the largest landslides were deep-seated structurally controlled slides that also occurred on faults that ruptured the ground surface in the earthquake. These include the Seafront (28 M m^3), Hapuku (17.5 M m^3), Leader (16 M m^3), Stanton (6 M m^3) and Linton (2 M m^3) landslides. Detailed information about the failure mechanisms of these landslides has not yet been published, however we understand that the modes of failure of these landslides were consistent with those in the wider region, including translational, compound/bi-planar, and rotational mechanisms (C. Gasston, A. Wolter, pers. comm., 2022). Reduced rock mass quality around the fault zones, ground surface displacement across the weak fault zone during the fault rupture, and forward directivity or pulse-like ground motions are likely to have been significant factors for these large volume landslides.

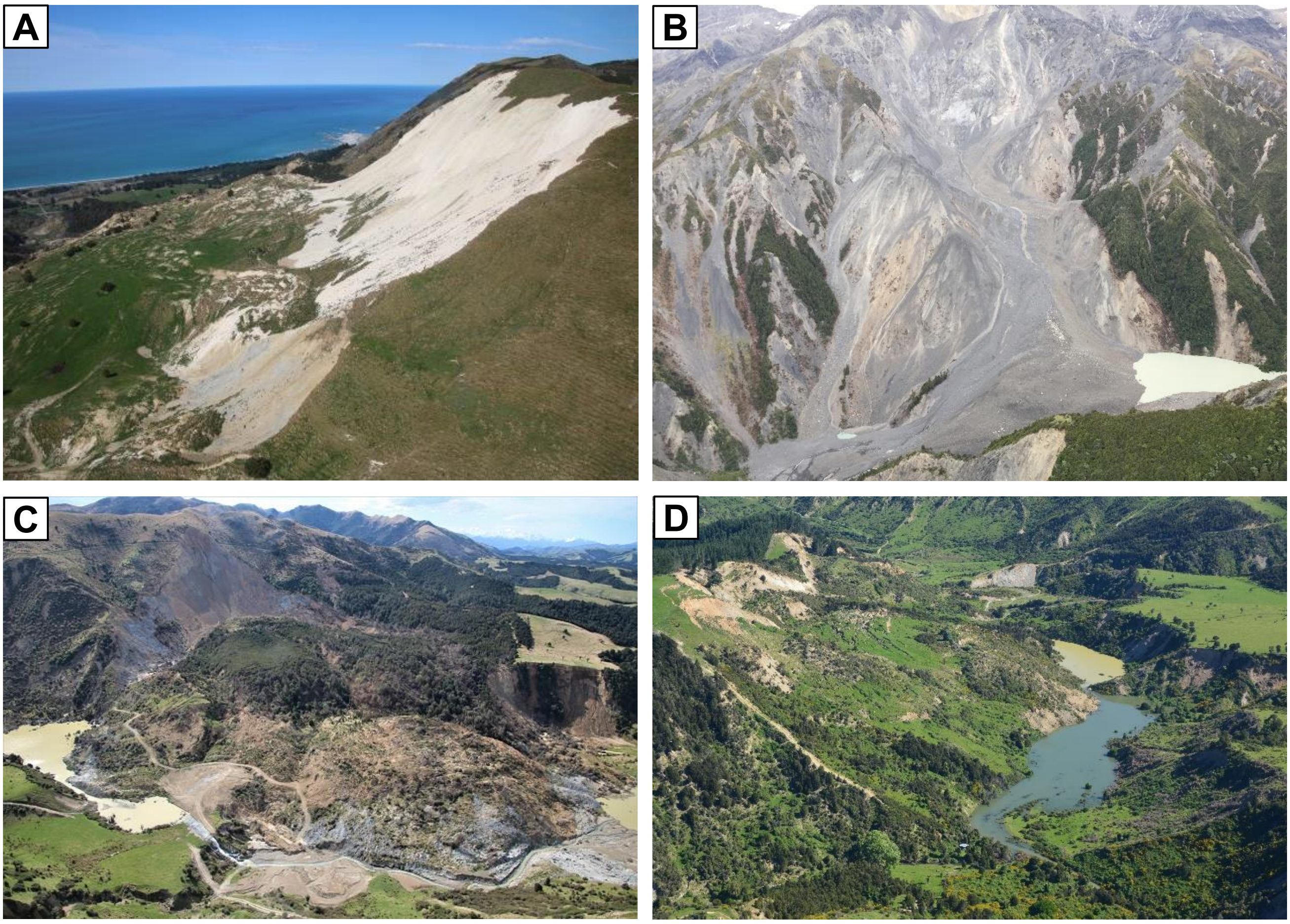

Figure 3: Examples of very large, fault-proximal landslides
(A) Seafront landslide, photo credit: Petley (2017a)
(B) Hapuku rock avalanche, photo credit: ECan (2017)
(C) Leader landslide, photo credit: Petley (2017b)
(D) Stanton landslide, photo credit: GeoNet (2017)

4 Direct damage impacts of slope failures

The Kaikōura earthquake was notable for the large number of faults that ruptured, the complexity of the fault rupture propagation, and the large area affected by strong ground shaking (Bradley *et al.*, 2017; Litchfield *et al.*, 2018; Stirling *et al.*, 2017). The strongest ground shaking occurred in the mountainous part of the northeastern South Island, and consequently the primary form of ground damage was from landsliding. As the earthquake-affected region is a relatively sparsely populated part of the country, the principal damage effects were to horizontal infrastructure such as transport networks (Davies *et al.*, 2017), telecommunications infrastructure (Giovinazzi *et al.*, 2017), and stopbanks (Bastin *et al.*, 2018). The most significant impact was the closure of the arterial transport routes along the east coast of the South Island, with the MNL railway, SH1, and the local road networks all severely damaged by slope failures (Davies *et al.*, 2017; Mason and Brabhaharan, 2021; Massey *et al.*, 2018).

The direct damage impacts of the slope failures can be classified using 'performance states' (Brabhaharan *et al.*, 2006) to assess the resilience of transport infrastructure. The 'availability state', which defines the level of access after the earthquake (representing the reduced level of service), and the 'outage state', which represents the duration of reduced access at a given availability state have been assessed for each slope failure in the inventory. The availability states have been represented as the following levels given in in Table 6.

Table 6: Availability state

Availability level	Availability state	Availability description
1	Full	Full access (perhaps with driver care).
2	Poor	Available for slow access, but with difficulty by normal vehicles due to partial lane blockage, erosion or deformation.
3	Single lane	Single lane access only with difficulty for normal vehicles due to poor condition of remaining road.
4	Difficult	Road accessible single lane by only 4x4 off road vehicles.
5	Closed	Road closed and unavailable for use.

The outage states have been represented as the following levels given in Table 7.

Table 7: Outage state

Outage level	Outage state	Outage description
1	Open	No closure, except for maintenance
2	Minor	Condition persists for up to 1 day
3	Moderate	Condition persists for 1 day to 3 days
4	Short term	Condition persists for 3 days to 2 weeks
5	Medium term	Condition persists for 2 weeks to 2 months
6	Long term	Condition persists for 2 months to 6 months
7	Very long term	Condition persists for greater than 6 months

Capturing the damage impacts and linking these to the types of failures enables the identification of the key slope failure modes of engineering importance. The classification of slope failure impacts on the availability of the road and rail corridors immediately after the earthquake is summarised in Table 8.

The principal rock slope failure mechanisms that cause the most significant damage and outage of the transport corridor were rock avalanches and structurally-controlled translational or compound slides. Rock and debris avalanches on high hillslopes are one of the main causes of the severe damage and long outage, as observed along the coastal transport corridor through Kaikōura (Mason and Brabhaharan, 2021). These can have the cumulative impact of large volumes of debris blocking the transport corridor and significant areas of perched debris and disrupted rock mass at height above the corridor posing a safety hazard to the transport corridors. Sites with these combined factors require a lengthy outage for initial helicopter sluicing and abseil scaling to reduce the safety risks to earthworks personnel and machinery to manageable levels before earthmoving to clear the debris could commence. Post-earthquake rainstorms could remobilise the perched debris into debris flows and cause regression of the head scarps (thus releasing additional debris onto the slope), which can cause regular and ongoing delays to the recovery works.

Deep-seated translational or combined defect-rock mass failures can be comparatively rare along the transport corridor but can cause severe damage and outage impacts. These failures typically extend the full height of the slope and cause a slower recovery rate than other sites due to (a) the larger volumes of debris generated by these deep-seated failures, and (b) the unknown condition and stability of the rock mass underlying the failure surface, which require a longer period for investigation and scoping of mitigation measures before these could be designed and implemented.

Small scale failure mechanisms such as joint-controlled step-path slides and wedge or toppling failures of slopes near the road and rail cause less significant damage impacts, but often require engineered stabilisation or risk mitigation measures during the recovery phase because of unacceptable residual risks from over-steepened head scarps or destabilised areas of the adjacent slope. This can lengthen the recovery time for the transport corridor and result in more significant disruption from traffic management restrictions along the narrow transport corridors often encountered in the mountainous or hilly regions of New Zealand.

Extensive damage to earth fill embankments can be caused by strong ground shaking, which can result in difficult access for the initial emergency response. Slow access for 4WD vehicles may still be available along the transport corridors, which can somewhat mitigate the impacts of the fill slope failures. Where road embankments are damaged by settlement and limited shallow-seated displacement of the fill materials, the damage may be able to be quickly reinstated by repairs to the pavement and resurfacing of the damaged sections. Deeper seated failure mechanisms (such as rotational slumps and translational displacement of embankment slopes and retaining walls) can result in much longer outage times, to carry out investigations, design and to excavate and reconstruct the failed sections of embankments.

Very large, deep-seated landslides such as those described in Section 3.5, can occur adjacent to faults that can rupture the ground surface, such as the Seafront (28 M m^3), Hapuku (17.5 M m^3), Leader (16 M m^3), Stanton (6 M m^3) and Linton (2 M m^3) landslides. These and other similar deep-seated landslides can lead to extensive landscape impacts, particularly the formation of landslide dams and addition of large volumes of debris into the river system (Massey *et al.*, 2018), which can pose a risk to infrastructure and the built environment.

Table 8: Slope failure impacts on the availability of the road and rail corridors

Availability State[1] and direct damage impacts

(1) Full access: No damage, or slope failures did not encroach into road/rail corridor sufficiently to affect access




(2) Poor: Slow access. Minor cracking of road pavement (limited or no vertical displacement across the cracks), minor deformation of rail tracks (able to run normal traffic possibly under reduced speed limits), inundation of the live traffic corridor by small volumes of landslide/rock fall debris




(3) Single lane (road corridor only): Loss of half the road width due to (a) failure of fill embankments or retaining walls on slopes below the road, or (b) blockage of one lane by rock fall or landslide debris from slopes above the road




(4) Difficult: Road only accessible in a single lane by 4WD vehicles, rail only accessible with temporary speed/weight restrictions. Damage impacts consist of subsidence and deformation of the road or rail from slope failure extending beneath the whole embankment




(5) Closed: Corridor unavailable for use. Damage impacts consist of complete blockage by slip debris, or extensive deformation of the road/rail from fault rupture or underslips through the whole embankment




[1] Availability state refers to the level of accessibility of the transport corridor immediately following the earthquake, after Brabhaharan *et al.* (2006)

5 Recommendations for best practice in design of earthworks

5.1 Introduction

Design for earthquakes requires the twin considerations of life safety and resilience. Infrastructure and our built environment primarily provide functionality for societies. They need to be designed for earthquakes (and other perils) so that they do not pose an unacceptable safety hazard to users, but also provide resilience in terms of their functionality. The design of earthworks is no exception in the need for safety and resilience.

The existing design standards commonly have focussed on life safety, and do not adequately consider the need for resilience and functionality. Life safety will be a critical consideration in the design of earthworks for residential development and buildings.

The focus of these recommendations is on transportation routes, but the same resilience principles are applicable in the design of earthworks for other aspects of our infrastructure and built environment, and can be adapted for such applications. The Australian Geomechanics Society guidelines (AGS, 2007a, 2007b, 2007c, 2007d) provide a comprehensive approach to the assessment of life safety risk.

5.2 Life safety considerations

Life safety in transportation route applications will depend on the mode of failure, consequences of failure (which are considered in these recommendations) as well as the likelihood and presence of vehicles at the location of failure during earthquakes. Well-designed embankments generally fail in a manner which does not impose life safely issues. However, high cut slopes can give rise to life safety considerations during normal conditions, and not just in earthquakes. Life safety considerations in earthquakes will be of particular importance in the design of transport routes in urban areas with high traffic volumes.

The avoidance or mitigation of large-scale failures will mitigate life safety and resilience risks. However, rock falls and small failures can also lead to life safety risks. It is common practice to mitigate these using rock fall management practices and rock fall, slip and debris flow barriers. When these are designed, these structures also need to be designed taking into consideration earthquake loading and consequences.

These are addressed in the existing literature, and hence not repeated here. For guidance on rock fall management systems refer to the guidance published by the Ministry of Business, Innovation and Employment (MBIE, 2016) and Waka Kotahi (WK, 2023).

5.3 Resilience principles

The resilience of infrastructure such as roads is defined as the ability to recover readily and return to original functionality following an adverse event. Brabhaharan *et al.* (2006) applied this concept to transport networks, using the metrics of 'availability state' to represent the degree of damage or loss of functionality and 'outage state' to represent the time required to restore the pre-event level of service. These are illustrated in Figure 4.

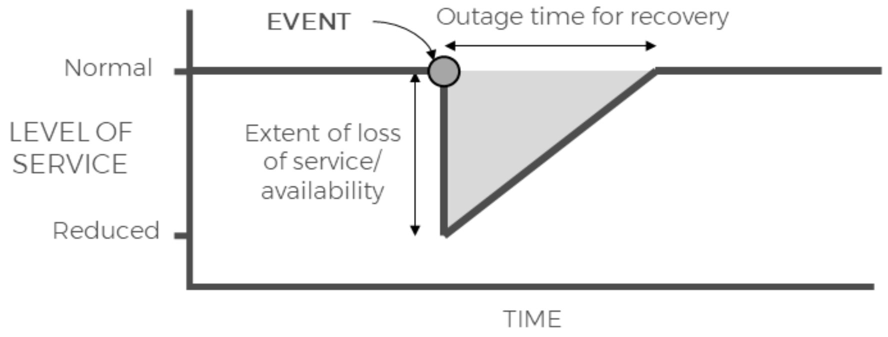

Figure 4: Resilience metrics for transport networks (Brabhaharan et al., 2006)

The resilience of infrastructure can be maximised by reducing the shaded triangular area in Figure 4. This can be achieved by developing infrastructure in a way that (a) reduces damage, and the consequent loss or reduction in functionality, and (b) enhances the ability to recover quickly from such a reduction, including strategies for organisations and communities to cope with or contain losses effectively, and emergency response strategies to enable rapid recovery (Bruneau *et al.*, 2003).

5.4 Resilience-based approach to slope assessment and design

Based on the resilience principles described above, and taking into account the observations of disruption to transport routes in the Kaikōura earthquake and the review of NZ and international literature, a best practice approach to designing resilient earthworks would involve the following:

(a) Adopting a **resilience-based design approach** for new cut and fill slopes, that have a low vulnerability to failures and closure of the route by minimising the size and nature of failures, and developing earthworks in a form that can be returned to functionality quickly after major events.

(b) Adopting a **proactive approach to resilience management** along corridors where earthquake-induced landslides from natural hillslopes or pre-existing earthworks slopes pose a threat to the route's security.

Particular aspects of slope assessment and design are described further in the following sections. Although the focus of this research is the earthquake performance of slopes along transport corridors, these considerations also readily apply to different types of infrastructure where earthquake-induced slope failures pose a threat to the built environment.

5.5 Importance of earthworks

5.5.1 Purpose

NZTA research report 613 (Brabhaharan *et al.*, 2018) provides a valuable approach for the classification of the importance of earthworks from a resilience perspective.

Classification of earthworks is used to:

- Adopt an appropriate approach for design
- Select appropriate levels of earthquake shaking

This enables earthworks to be designed in accordance with the resilience expectations for the facility it affects; and use a level of design which is consistent with the importance.

An example is provided for a transportation route but this can be modified for other infrastructure facilities or the built environment.

It would be useful to consider the importance of the transportation route as well as the criticality of the earthworks component for the performance of the route.

The Australian/New Zealand Standard Structural Design Actions AS/NZS 1170.0 classifies structures in accordance with their importance levels for selection of the design earthquake motions. The Bridge manual also classifies highways in terms of importance levels which aims to be consistent with AS/NZS 1170.0.

Classification of the importance of transportation routes in accordance with these New Zealand standards and design manuals ensures that cut slopes (including those which are part of other facilities) are designed and constructed in general accordance with the principles adopted for design in New Zealand. The Bridge Manual provides more definitive definitions of importance levels for highways and can be used in conjunction with this guideline. However, there is an anomaly as to how cut slopes are currently considered in the Bridge Manual (see section 2.5.5), and the recommendations from report RR 613 attempts to address this.

5.5.2 Importance level

The importance levels of transportation corridors for use in design are given in Table 9.

Table 9: Selection of importance level

Importance level	Situation
IL 1	Transportation facilities of low importance, e.g. rural farm roads
IL 2	Transportation routes of normal importance, e.g. urban roads other than key arterial and collector routes
IL 3	Transportation routes of high importance which are lifelines, or carry significant traffic volumes of more than 2,500 vpd
IL 4	Transportation routes providing access to post-disaster facilities and that are expected to provide a service in a post-disaster scenario

Notes:

1) Importance levels (IL) referred to are those defined in AS/NZS 1170.0, supplemented by the Bridge manual for highways and arterial roads.
2) Though rural roads would be classified as IL1, some roads may be the only access for communities, and this needs to be considered in the Resilience Importance category at a regional level (see next section)

The aim of this selection has been to use the existing importance level framework in the New Zealand standards and the Bridge manual.

5.5.3 Resilience importance category

Once the importance level is selected based on existing criteria, it is recognised that the resilience expectations even for the same importance level of transportation route vary significantly depending on the regional context of the routes. For example, IL3 routes in Christchurch or Auckland would have a lot more redundancy because of the many routes given the terrain, whereas in places like Wellington, Dunedin or Central Otago there is very little redundancy.

Resilience importance categories provide a basis for incorporating the local context and resilience expectations into the design process.

The resilience importance category (RIC) of earthworks along transportation corridors for use in design are given in Table 10. Resilience importance categories take into consideration the importance level of the transportation route as well as the resilience expectations of the section of route from a regional network context. This recognises that sections of the transportation network may have a higher resilience importance because of the nature of the regional network, its resilience and the availability or lack of alternative routes in the event of incidents.

It is envisaged that the transportation authority would consider the regional context and specify the resilience importance category to be used in the design of earthworks for a particular transportation corridor.

Table 10: Selection of resilience importance category

	Resilience importance category			
	Importance level			
Resilience expectations	1	2	3	4
Low Resilience expectations low from the regional context, such as due to availability of many secure alternative routes of adequate capacity.	I	II	III	– [(A)]
Medium Resilience expectations moderate from a regional context, due to some alternative routes but may not have adequate capacity or security in events.	II	III	IV	V
High Resilience expectations high from a regional perspective because of limited or no alternative routes and/or their capacity or security in events is poor.	– [(A)]	IV	V	V

[(A)] IL4 routes are not likely to have a low resilience expectation and IL1 routes are not expected to have a high resilience expectation.

Notes:

(adopted from Research report RR 613)

1) Importance levels are defined in Table 9.
2) Resilience expectations will depend on importance of route and availability of secure alternatives.
3) The resilience of access for rural communities or businesses (eg milk processing plants or tourism) where this is the only form of access needs to be taken into consideration.

The resilience importance category is a function of the transportation route rather than the cut slope or fill embankment.

5.6 Design approach

5.6.1 Outline of design approach

A four-level design approach is presented, to suit the design importance level of earthworks, adapted from research report RR613:

- Design approach 1 is a simplified design approach suitable for use by practitioners for simple relatively low height earthworks slopes of relatively low importance.
- Design approach 2 is a standard design approach for use where performance is important for continued functionality on relatively moderate height earthworks slopes in simple geotechnical conditions.
- Design approach 3 is for use where performance is important for continued functionality on relatively higher earthworks slopes in moderately complex geotechnical conditions.
- Design approach 4 is for use where performance is critically important for continued functionality, with very high earthworks slopes, or in complex geotechnical conditions.

A fundamental difference of the design approach given compared with that stipulated in AS/NZS 1170 is that the design is for resilience and life safety rather than life safety alone.

Earthworks slopes most commonly affect the functionality of transportation routes, although they can also be critical to life safety in some circumstances.

5.6.2 Selection of design approach

The design approach suitable for a particular earthworks slope design is presented in Table 11. A higher-level design approach may be adopted for design, but a lower level than indicated is not considered appropriate. Here the height of the slope is used as a proxy for potential consequences to the transportation corridor due to failure and blockage or safety hazards from rock fall or failures, and the level of difficulty in restoring access in the event of failures.

Table 11: Selection of design approach

Resilience importance category (RIC)	Earthworks slope height			
	< 10 m	10 – 30 m	30 – 50 m	> 50 m
I	DA1	DA2	DA3	DA3
II	DA1	DA2	DA3	DA3
III	DA2	DA2	DA3	DA4
IV	DA2	DA3	DA4	DA4
V	DA2	DA3	DA4	DA4

Notes:

1) Resilience importance categories (RIC) referred to are those defined in Table 10.
2) The situation or cut height could trigger a design approach.
3) The design approaches are a minimum expectation, and a higher-level design approach may be chosen if the risks are higher, for example due to presence of any structures within the zone of influence either above or below the slopes.

5.6.3 Description of design approach

The various levels of design approach developed depending on the importance of the route and the scale and complexity of the cut slopes is described in Table 12.

Table 12: Design approaches

Design approach / situation		Investigations	Analysis
DA1	Low height slopes, low importance routes where failure will not affect structures. Consequences of failure leading to closure would be acceptable and the route can be readily opened by clearance of debris. Embankments can be designed to prevent failure accepting some deformation.	Desk study and engineering geology mapping to establish the precedent behaviour of slopes in the area in similar ground conditions.	Consideration of precedent behaviour of slopes in the area, consideration of critical kinematically feasible failure mechanisms. Kinematic or limit equilibrium slope stability analysis as required. No specific earthquake design would be required for cuts. Embankments can be designed by pseudo-static limit equilibrium slope stability analysis, and to limit deformation in earthquakes.
DA2	Standard approach for moderate height slopes in simple geology. Performance for continued functionality is considered beneficial.	As for DA1, plus rock slope survey by UAV or LiDAR, supplemented by detailed logging of rock mass characteristics and defects. Characterise soil strength for soils.	Consideration of precedent behaviour of cut slopes in earthquakes and storms, consideration of critical kinematically feasible failure mechanisms, supplemented by pseudo-static limit equilibrium slope stability analysis. Consideration of simple earthquake motions from standards or the Bridge Manual.

Design approach / situation		Investigations	Analysis
DA3	Detailed approach for moderate to high slopes, routes where performance is important in hazard events such as earthquakes and storms, or for complex ground conditions.	As for DA2, plus logging of ground conditions and drilling of cored boreholes. Characterisation of rock defects by field mapping and downhole televiewer (ATV/OTV) surveys. Laboratory classification and strength testing.	Assessment of kinematically feasible failure mechanisms controlled by defects, rock mass stability and combined defect-rock mass mechanisms. Consideration of earthquake motions will include assessment of topographical amplification effects. Soil or rock slope stability analysis will consider complex failure mechanisms and assessment of displacements and performance of the slope. Pseudo-static limit equilibrium and decoupled or coupled permanent displacement analysis. Assessment of topographical amplification effects and stability may be aided by numerical modelling (stress-deformation analysis) where appropriate.
DA4	Detailed design approach suited to high slopes, routes where performance is important in hazard events such as earthquakes and storms, or for complex ground conditions. Adjacent structures may be present.	As for DA3, plus downhole or surface geophysical surveys. Laboratory classification and strength testing.	Assessment of kinematically feasible failure mechanisms controlled by defects, rock mass stability and combined defect-rock mass mechanisms. Consideration of earthquake motions from standards or the Bridge Manual will include assessment of topographical amplification effects. Slope stability analysis will consider complex failure mechanisms and topographical amplification effects and will include assessment of displacements and performance of the slope. Decoupled or coupled permanent displacement analysis supplemented by dynamic stress-deformation analysis where appropriate. Assessment of the consequences of slope displacement or failure to the resilience of adjacent infrastructure.

Note: The design approaches are a minimum expectation, and a higher-level design approach or design actions may be chosen if the risks are higher, for example due to presence of any structures within the zone of influence either above or below the slope.

5.7 Selection of ground motions for design

5.7.1 Introduction

It is noted that the current design standards are based on the old seismic hazard model, and do not yet take into consideration the new National Seismic Hazard Model (NSHM) that was released in October 2022. Seismic loads can be derived from the NSHM, and future updates to the models, together with updated design standards that reflect these new models. Site specific seismic hazard can be considered for important or high value projects, and guidance is provided in the Bridge Manual. The new NSHM presents the uncertainty associated with the estimate of seismic ground motions, and the uncertainties need to be taken into consideration in the design. A technical advice note was jointly prepared by the NZ Society for Earthquake Engineering, the Structural Engineering Society and the NZ Geotechnical Society on “Designing for Uncertainty” (NZSEE-SESOC-NZGS, 2022) to provide advice on these issues.

Seismic coefficients for design should take into account the slope geometry, failure mechanisms, and importance of the route. As earthworks assets such as retaining walls, slopes and embankments are all components of the transportation network, using the same importance criteria as for bridges provides a common basis for judging risk to the transportation system.

Modification of the seismic coefficient to account for topographic amplification and the spatial incoherence of ground motions should be applied using appropriately referenced modification factors.

5.7.2 Peak ground accelerations for design

The peak ground accelerations for design are selected based on the Bridge manual, to obtain peak ground accelerations that are not weighted by their relevant magnitudes. Spectral accelerations may be considered if there is a dominant period for the site. It should be noted that the Bridge Manual has not yet adopted appropriate changes arising from the development and release of the new National Seismic Hazard Model in late 2022. Until that is incorporated, it may be appropriate to consider the results of the NSHM to derive design parameters, to avoid underestimation of seismic design parameters.

The Bridge Manual provides maps providing hazard factors to derive peak ground accelerations across New Zealand. These represent free-field accelerations before any topographic effects are taken into consideration.

As discussed in section 2.5.5, there is an anomaly in the current Bridge Manual in that it provides for different hazard levels for different types of structures – bridge, retaining wall, embankments and cut slopes – on transportation routes with a selected importance level. For example, for an important transportation route, to design a bridge or wall for a 2,500-year return period earthquake, but only considering the cut slopes (regardless of height) for earthquakes with 1/5th the return period of 500 years, is considered to be inappropriate. It would be prudent for a transportation route importance level to drive the level of hazard it is designed for regardless of the type of structure.

Having set a hazard level, different structures may be designed for a consistent level of resilience, and this may mean accepting a level of performance consistent with that resilience expectation, and perhaps the cost of remediation.

For example, if the resilience expectation is to be able to reopen the route within say three days for a selected ultimate limit state event, then all components forming the route will be designed to perform adequately in the same event, which means that:

- The bridges on the route would be designed to minimise settlement of abutments to a level that can be readily repaired and ensure only minimal repairable damage occurs to critical structural members, which would not compromise access.
- The retaining walls would be designed to minimise displacement and consequent damage to the transportation facility and related structures, which could be readily repaired within the required timeframe of three days.
- The embankments would be designed to minimise displacements to enable 4WD access after the events and enable them to be repaired within a short time.
- The cut slopes would be designed to minimise the size of failures so they can be cleared within a few days, with the remaining cut slope being stable and measures in place to manage the rock fall hazards.

In line with the above discussion on the importance of designing all components of the route for the same event, the return periods for limit state design of cut slopes in Table 13 should be consistent with the hazard levels specified in the Bridge Manual for bridges. Obviously this would need to be discussed at a wider level.

Table 13: Return period for limit state design of earthworks

Resilience importance category (RIC)	Return period (years)		
	Operational continuity	Ultimate limit state (ULS)	Maximum considered earthquake (MCE)
I	100	250	-
II	150	500	-
III	250	1,000	1.5 x ULS
IV	500	2,500	1.5 x ULS
V	500 min with site specific consideration	2,500	1.5 x ULS

Notes:

1) Resilience importance categories (RIC) referred to are those defined in Table 10.
2) Ultimate limit state (ULS) and maximum considered earthquake (MCE) are as defined in the Bridge Manual.
3) Operational continuity as defined in the Bridge Manual.

The earthquake peak ground accelerations for the relevant hazard levels can be derived from the Bridge Manual.

The proposed hazard levels are higher than currently provided for in the Bridge manual, for cut slopes. As discussed, these are too low and will lead to transportation routes that have poor resilience. Therefore, higher hazard levels, but still consistent with the hazard levels for bridges, are proposed to achieve a consistent level of hazard for which the route is designed, regardless of the road form.

Operational continuity is very important from a resilience perspective, and therefore a higher hazard level is proposed in Table 13, depending on the resilience importance category. This should be considered together with the 'design for resilience' approach noted in section 5.8. These levels would need to be discussed at a wider level. It is noted that seismic performance requirements in chapter 5 of the Bridge Manual and the operational continuity requirements in chapter 6 are different, and it would be useful to ensure these are aligned.

While the design hazard level is proposed to be consistent for all elements of the transportation route, the consequences to ensure resilience can be different, for example small cut slope failures and small embankment deformations can still be accommodated without seriously compromising the resilience of the route, and this approach will help manage the cost of construction to an acceptable level.

5.7.3 Topographical amplification

The numerical analyses in research report RR 613 as well as this research and evidence from observations of earthquakes clearly indicate topographical amplification is a key issue that needs to be addressed in design. Understanding and quantifying topographical amplification is an area of recent research and development, and there is more research yet required to develop a good understanding of the issues. However, topographical amplification factors to use in design are given based on up-to-date research and current knowledge, so they can be applied to the design of transportation infrastructure involving earthworks slopes, currently happening or planned for the near future in New Zealand.

Slopes in steep topography or formed at steep or high slopes are likely to lead to topographic amplification of the ground shaking. The amplification factor based on the numerical analyses and the literature is higher on ridges than on terrace slopes.

The proposed topographical amplification factor, based on information to date, can be derived from Table 14 for ridge slopes and Table 15 for terrace slopes.

Table 14: Topographical amplification factors for ridge slopes and embankments

Slope angle	Slope height				
	< 5 m	5 m – 10 m	10 m – 50 m	50 m – 100 m	> 100 m
0° – 15°	1	1	1	1	1
15° – 30°	1	1.2	1.4	1.6	2
> 30°	1	1.2	2	2.5	3

Notes:

1) Where the cut slope extends only over part of the ridge slope height, then an appropriate topographical amplification factor for the ridge crest should still be considered, assuming a reduced height ridge.

Table 15: Topographical amplification factors for terrace slopes

Slope angle	Slope height			
	< 10 m	10 m – 50 m	50 m – 100 m	> 100 m
0° – 15°	1	1	1	1
15° – 30°	1	1.1	1.2	1.6
> 30°	1.1	1.2	1.4	2

Notes:

1) Where the cut slope extends only over part of the terrace slope height, then an appropriate topographical amplification factor for the cut crest should still be considered, assuming a reduced height slope.

The topographical amplification factor values proposed here only take into account the effect of topography type, slope and height and inclination. It is clear through the literature research there is vast experimental and theoretical evidence that:

- Topographical amplification factors are also affected by the predominant wavelength of seismic excitation.
- A parasitic vertical seismic motion may develop from purely horizontal excitations.

However, from a practical perspective it will be difficult take into consideration the frequencies or wave lengths, as the dominant earthquake frequencies are likely to change depending on the different earthquakes that may affect a particular site, and in a particular earthquake there are likely to be different frequencies.

A similar table could be provided for vertical accelerations unless there is strong evidence through further research that the vertical and horizontal components are not synchronous. At the present time, there is inadequate research information to allow specification of vertical ground motions for design. However, this could be considered for very important earthworks through numerical time history analyses using both horizontal and vertical accelerations.

5.7.4 Presence of soil overburden

The presence of overburden on parent ground such as bedrock could lead to ground accelerations that are much higher than peak ground accelerations from either:

a) The presence of overburden on flat ground, or

b) Steep rock slopes with soil overburden.

The combined effects of soil overburden and topography appear to lead to the much higher accelerations observed in practice, and these are also indicated by the limited numerical analyses reported in the literature as well as that carried out as part of this research. Such situations need to be considered with care and the sensitivity of performance checked with higher ground accelerations.

5.7.5 Design ground accelerations for slope design

The numerical analyses and evidence from observations in earthquakes clearly indicate that topographical amplification is present and highest at the crest of slopes. The amplification further down the slope at mid-height or below is much lower, and even de-amplification may be encountered. The evidence of slope failures from earthquakes also suggest that slope failures are predominant at the top of slopes, both ridges and terraces.

It is also clear that the ground accelerations, whether amplified by topography or not, are likely to be different along the height of slopes and the peak acceleration is not expected to be encountered at the same point in time during an earthquake along the height of the slope. Therefore, a lower average acceleration is appropriate for pseudo-static design when large failure mechanisms are considered. To reflect this, design ground accelerations for pseudo-static design are proposed based on consideration of different size mechanisms affecting the slope. These are presented in Table 16.

Table 16: Application of ground accelerations for pseudo-static earthquake design of slopes greater than 20 m height

Situation of failure mechanism	Design acceleration for pseudo-static design
Upper quartile of slope	PGA x TAF
Upper half of slope	PGA
Full slope	PGA x 0.65

Notes:

1) PGA = peak ground accelerations, derived from the Bridge Manual.
2) The topographical amplification factor is derived from consideration of the height of cut slopes and slope angles as suggested in Table 14 and Table 15, depending on the situation (ridge or terrace).

Makdisi and Seed (1978) provide average accelerations for embankment dams considering potential depths of the sliding mass. Unlike embankments, steep slopes often involve shallow failures, and this approach has therefore not been adopted for considering cut slopes.

5.7.6 Time history records for design

If numerical analyses using time histories are used in the design, then appropriate time histories that reflect the following should be considered:

a) Seismo-tectonic regime.

b) Near-fault time history records, where the design ground motions at the site is expected to be experienced during nearby earthquake faults or sources.

c) Frequency content of records representative of expected events that could affect the site.

d) Time histories should be scaled as provided for in NZS 1170.5.

It is also important to monitor the free-field accelerations during numerical analyses to ensure that these are reflective of what is proposed for the site.

5.8 Earthquake design of earthworks

5.8.1 Introduction

The choice of analysis method should depend on the slope height, the complexity of the ground conditions and failure mechanisms, and the importance of the route. Cut slopes for transport corridors in New Zealand have traditionally been designed based on a combination of precedent (observation of the performance of similar cut slopes in similar geology) and simplified quantitative analyses, typically comprising two-dimensional pseudo-static limit equilibrium slope stability analyses or kinematic analyses of rock slopes with persistent defects. These methods are considered appropriate for low to moderate height slopes in simple geology where the consequences of slope failure would be acceptable, and the route can be readily opened by clearance of debris.

For higher slopes, more complex ground conditions, and where slope performance in earthquakes is important for route security, the analyses would be extended to include decoupled or coupled permanent displacement analysis, and assessment of slope stability and response (including topographical amplification effects) using dynamic stress-deformation analysis where appropriate.

Design of fill embankments is typically carried out by pseudo-static limit equilibrium analysis in conjunction with displacement-based design.

5.8.2 Design for resilience

In the earthquake design of cut slopes, it would be important to set appropriate performance criteria to achieve:

a) A level of performance that is consistent with the resilience objectives set for the transportation route or project, and

b) An economical solution.

Unlike made structures such as bridges, and made earth structures such as embankments, cut slopes are mostly formed in natural materials with their inherent variability and in situ ground characteristics. Therefore, smaller failures, such as small wedge failures in rock, are difficult to prevent, unless a significant expenditure is incurred to protect/stabilise the slope against such small failures.

A resilience-based design would be suitable, such as consideration of the effect of any failures on the level of service or performance of the transportation route, but the time it would take to restore the level of service or access also needs to be considered. It would be more economical to accept such small failures in large events but design the cut slope to avoid or minimise the risk of large-scale failures that would affect the performance of the route for a significant period of time, see Figure 5.

Similarly for embankments, a resilience-based design approach would be to design for elastic behaviour, i.e. no deformation in small to moderate events, but accept deformation of the embankment (but not gross failure) that can be readily repaired after an earthquake. Acceptable levels of deformation are provided in the Bridge Manual.

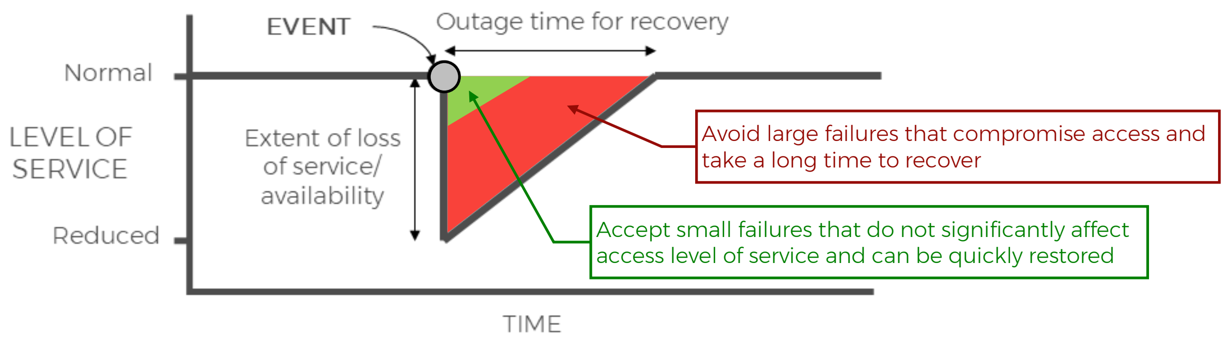

Figure 5: Design for resilience of route/network

5.9 Performance levels

5.9.1 Slope displacement assessment methods

Assessment of slope displacement is widely used as an index of slope performance. Estimation of permanent slope displacements may use a simplified Newmark sliding block approach using correlations between Newmark displacements and the ratio of yield acceleration to peak ground acceleration, decoupled analysis of numerical integration of time histories of shaking within a slide mass, or more complex dynamic stress-deformation analyses (Jibson, 2011; Massey *et al.*, 2022).

5.9.2 Semi-empirical displacement assessment methods

Semi-empirical displacement assessment methods have been developed by statistical analyses of displacements observed from past earthquakes. Common methods used include:

a) Ambraseys and Srbulov (1995)

b) Jibson (2007)

c) Bray and Travasarou (2007)

Ambraseys and Srbulov (1995) consider the peak ground acceleration, magnitude and epicentral distance of potential earthquake sources, but a smaller database of earthquakes available at the time and gives larger displacements for large magnitude earthquakes. Jibson (2007) considered a much larger statistical database, and gives lower displacement values.

5.9.3 Acceptability of slope displacements

There are few published criteria for acceptable slope displacements; the Bridge Manual provides limits on acceptable slope and wall displacements but does not differentiate between variable slope heights, failure modes, or importance level of the slope/asset.

The Southern California Earthquake Centre landslide guidelines (Blake *et al.*, 2002) note that allowable displacement levels are established from engineering judgement and recommends criteria of 50 mm and 150 mm as an initial screen to help inform that process. NCHRP (2008) Report 611 recommends a general guideline of <100 mm (4 inches) displacement for a "stable" slope and >300 mm (12 inches) as unstable from a serviceability perspective.

The FHWA (2011) guidelines highlight the importance of considering issues in defining acceptable displacement but does not provide any guidance for establishing acceptable displacement criteria. This was extended by Antonopoulos (2018) for the geotechnical design of earthworks for the NCTIR recovery project to include numerical modelling of soil-foundation-structure interaction effects as well as slope displacement criteria.

The acceptability of the levels of displacements needs to be considered in relation to the expected deformation tolerance of the slope-forming materials and the assessed failure mechanisms. For fill materials, where the material properties can be well defined in the design and controlled and verified during construction, deformation will manifest as ductile displacement and cracking. Investigation and analysis of fill slope failures observed in the Kaikōura earthquake has shown where displacement of fill slopes exceeded the calculated Newmark displacement that resulted in excessive (metre-scale) displacement and deformation to the road carriageway requiring a lengthy outage to rebuild the damaged embankments. This may be the result of reduction of soil strength during displacements.

A resilience-based design approach of limiting slope displacements to a small amount through rigorous slope stability and displacement assessments, in combination with sound construction practices and ongoing proactive maintenance of drainage and other assets, provides a basis for resilient performance of fill embankments under earthquake loading.

The magnitude of deformation that natural geological materials can withstand before collapse, disaggregation and evacuative slope failure is highly variable (Glastonbury and Fell, 2002). Assessment of rock slope failures on the transport corridors in Kaikōura identified combinations of shallow-seated disaggregated rock mass failures and kinematic failure mechanisms including structurally-controlled slides and joint-controlled step-path slides where displacements led to rapid failure and collapse, due to the brittle nature of the rock mass. Strain thresholds (i.e. cumulative slope displacements normalised over the downslope length of the landslide) that led to evacuative failure have been assessed by Glastonbury and Fell (2002) and Singeisen *et al.* (Submitted); these range from <0.1% to over 10%, depending on the mechanism and scale of failure. However, the local strain in the loss of strength in brittle materials is likely to be more important than the overall strain over the failure mass. Also, further degradation of strength with time, especially with the degraded rock mass could lead to failures.

Therefore, considerable care needs to be taken in comparing the results of assessed displacements to slope performance thresholds based on displacement amounts for natural materials, particularly the highly fractured and tectonically deformed greywacke rock masses. In soil materials, the potential for reduction in strength to residual strength should be considered in adopting displacement limits. This will be particularly important for cohesive soils. For granular soils, large displacement parameters should be used in the analyses rather than peak strength values.

Even where displacements lead to cracking of the earthworks or natural slopes during an earthquake, could lead to infiltration of water in subsequent storm events or further displacement in aftershocks. Whilst cracking of embankments at the road surface can be observed and sealed to prevent water ingress, this is unlikely to be feasible in cracks above cut slopes. In Kaikōura cracks in slopes have led to failures in storms after the earthquake.

5.9.4 Incipient failures and post-earthquake hazards

Many areas of hillside and ridge cracking occurred during the earthquake without causing landslides. In other areas, landslides were initiated but the debris was retained on the hillslopes or in gullies above the transport corridor. The damage to the slopes along the transport corridor heightened the probability of subsequent landslide initiation during the post-earthquake recovery period. The types of failures that occurred included rock fall from head scarp regression, debris flows and rock falls from remobilised landslide debris perched on the hillslopes, new landslides or failure of incipient landslides (Justice *et al.*, 2018). These failures caused significant delays and additional costs for the earthquake recovery programme. The risks posed by incipient failures and the triggering or reactivation of landslides and debris flows in aftershocks or post-earthquake storms is therefore an important part of assessing the resilience of infrastructure.

Similarly, cascading hazards such as debris flows and erosion or deposition of landslide-derived sediment in the river systems are important to consider in the management of earthquake resilience of infrastructure, as these can have impacts on bridge clearances, abutment protection, stopbank capacity etc.

5.10 Potential impacts of slope failure

5.10.1 Introduction

In a resilience-based design approach, the impact of earthquake performance is important to consider, so that an appropriate level of performance in earthquakes can be chosen to give the required level of resilience.

The impacts of potential slope failure in the design event or other assessed events need to be assessed and related to the resilience expectations for the route. Impacts include direct damage (such as loss of service from inundation by landslide debris), outage impacts (including the outage time and costs for repair and remediation) and indirect impacts such as from cascading hazards.

Understanding the potential for slope failure impacts on infrastructure is necessary to help identify measures to enhance resilience, such as:

- Selecting locations or alignments that avoid the potential hazard,
- Strengthening key vulnerable assets or sections,
- Enhancing the redundancy in the network by strengthening alternative routes, and
- Selecting an appropriate form for earthworks that reduces the potential damage impacts and/or allows faster recovery.

The direct damage impacts of slope failures in the Kaikōura earthquake are described in Section 4 of this report.

5.10.2 Landslide runout

Other than rock fall analysis, the runout of debris from slope failures has not been commonly assessed as part of earthworks slope design. Given the damage and disruption to the coastal transport corridor through Kaikōura was primarily from inundation by landslide debris, the assessment of landslide susceptibility and runout is a critical part of a resilience-based approach to slope assessment and earthworks design.

Estimation of potential landslide debris runout distance and inundation area can be made by empirical or numerical (physics-based) methods. Empirically based methods rely on the debris inundation area of past landslides of a given type, to estimate the anticipated debris inundation area of future landslides of a similar type. Physics-based methods include a range of techniques that consider the site-specific topography, landslide volume, and equivalent rheological parameters of the failed landslide mass (Brideau *et al.*, 2021a).

Brideau *et al.* (2021b) collated NZ and international datasets of landslides with recorded volume and runout data, which provide the basis for empirical assessment of landslide runout for given volumes and probabilities of exceedance. The type and volume of the landslide and the characteristics of the slope in the runout path will affect the runout distance. In general, confined landslides (such as debris flows) travel further than landslides on unconfined open slopes, rainfall-triggered landslides with wet/saturated debris have travel further than earthquake-triggered landslides with dry debris of similar volumes, and larger volume landslides travel further than smaller volume landslides of the same type (Brideau *et al.*, 2021a). Figure 6 shows the relationships between runout (expressed as the ratio between the fall height (ΔH) and length (L)) and landslide volume for dry debris/rock avalanches.

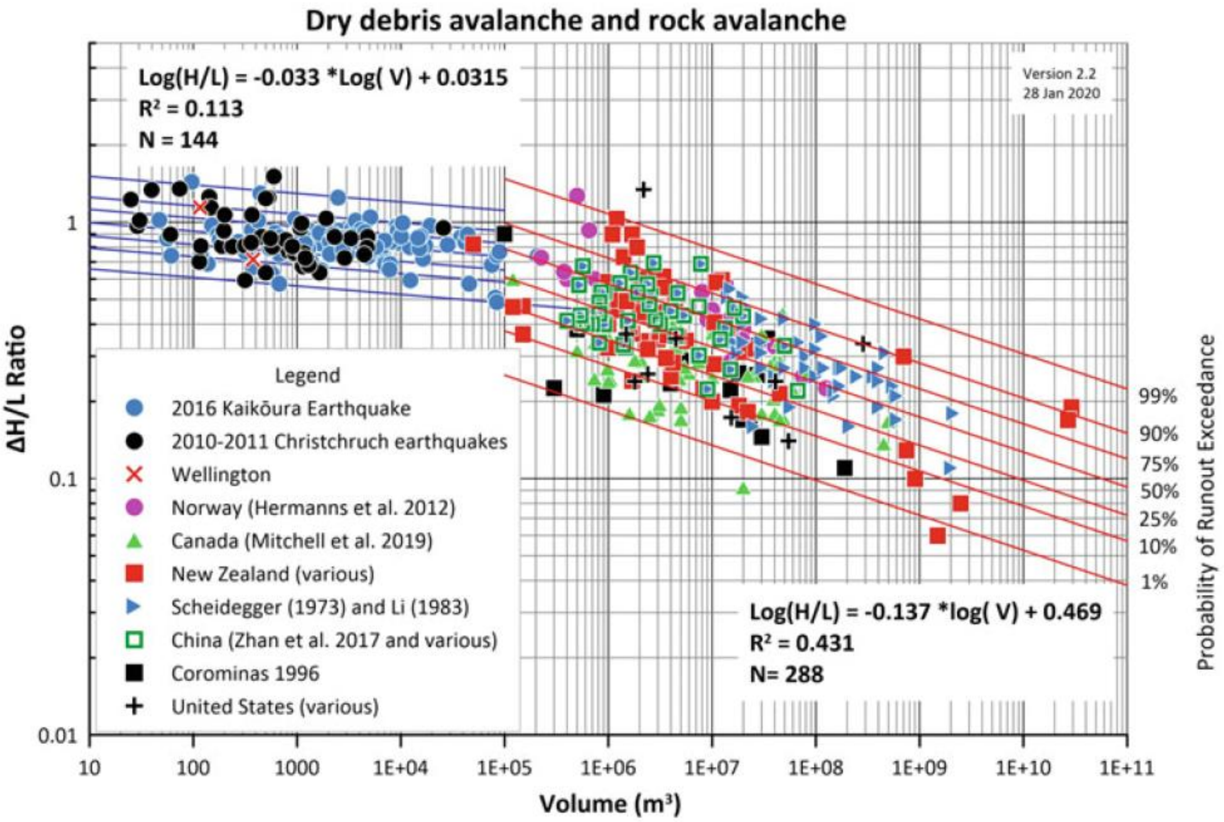

Figure 6: Empirical relationships between landslide volume and runout (ΔH/L ratio) for dry debris avalanches from Brideau et al. (2021b).

5.11 Ground investigations and models

The slope-forming materials, the feasible mechanisms of slope failure[1], and the strength parameters representing the failure mechanisms need to be characterised. A thorough site investigation will provide the basis for development of appropriately detailed engineering geological ground models that capture the materials and their variability.

The ground investigation could include desk study and engineering geology mapping to determine the geological materials and the precedent behaviour of slopes in the area in similar ground conditions, rock slope surveys supplemented by detailed logging of rock mass characteristics, drilling of cored boreholes and logging of ground conditions, surface and/or downhole geophysical surveys, and laboratory classification and strength testing. Static or piezocone cone penetration tests (CPT) would be valuable in soils, except in dense gravels where refusal can be reached quickly.

Three-dimensional ground models using software such as LeapFrog would be useful when assessing complex sites.

UAV or drone surveys are particularly useful to assess the slope geometry and conditions, as well as assess rock exposures using a 3-dimensional point cloud to identify rock defects that can control failure. An example of the use of such techniques to assess rock defects is illustrated in Figure 7.

[1] Discussion of slope failure mechanisms observed in the Kaikōura earthquake are given in Section 3 above.

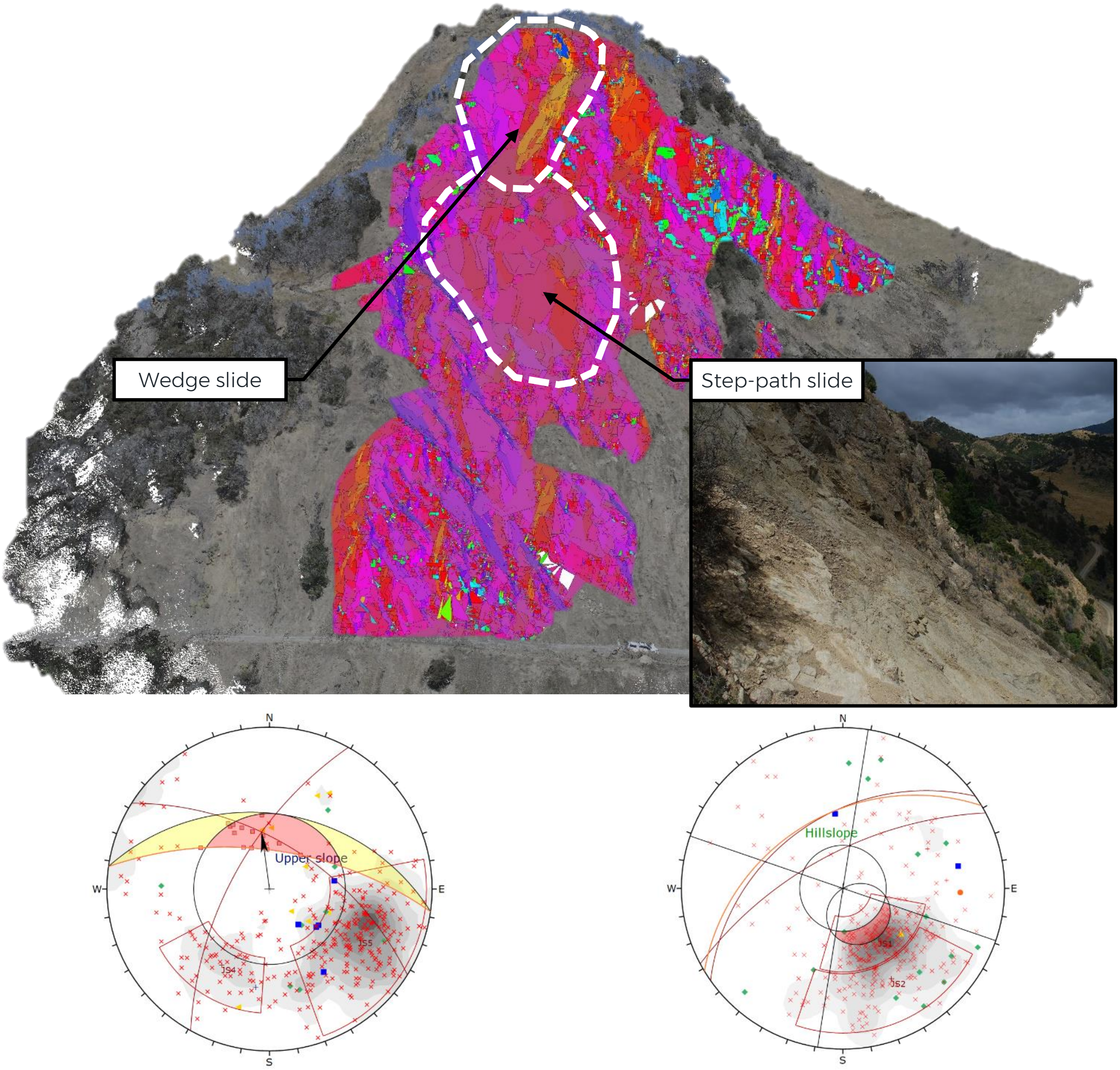

Figure 7: Point cloud and interpreted structural defects from UAV survey and stereograph analysis of wedge and step-path slides, Awatere Valley.

Downhole geophysical surveys such as acoustic or optical televiewer surveys are valuable to understand the rock defects and their orientation which are otherwise not exposed and would not be able to be assessed from boreholes alone. An example of the use of televiewer surveys to assess rock defects is illustrated in Figure 8.

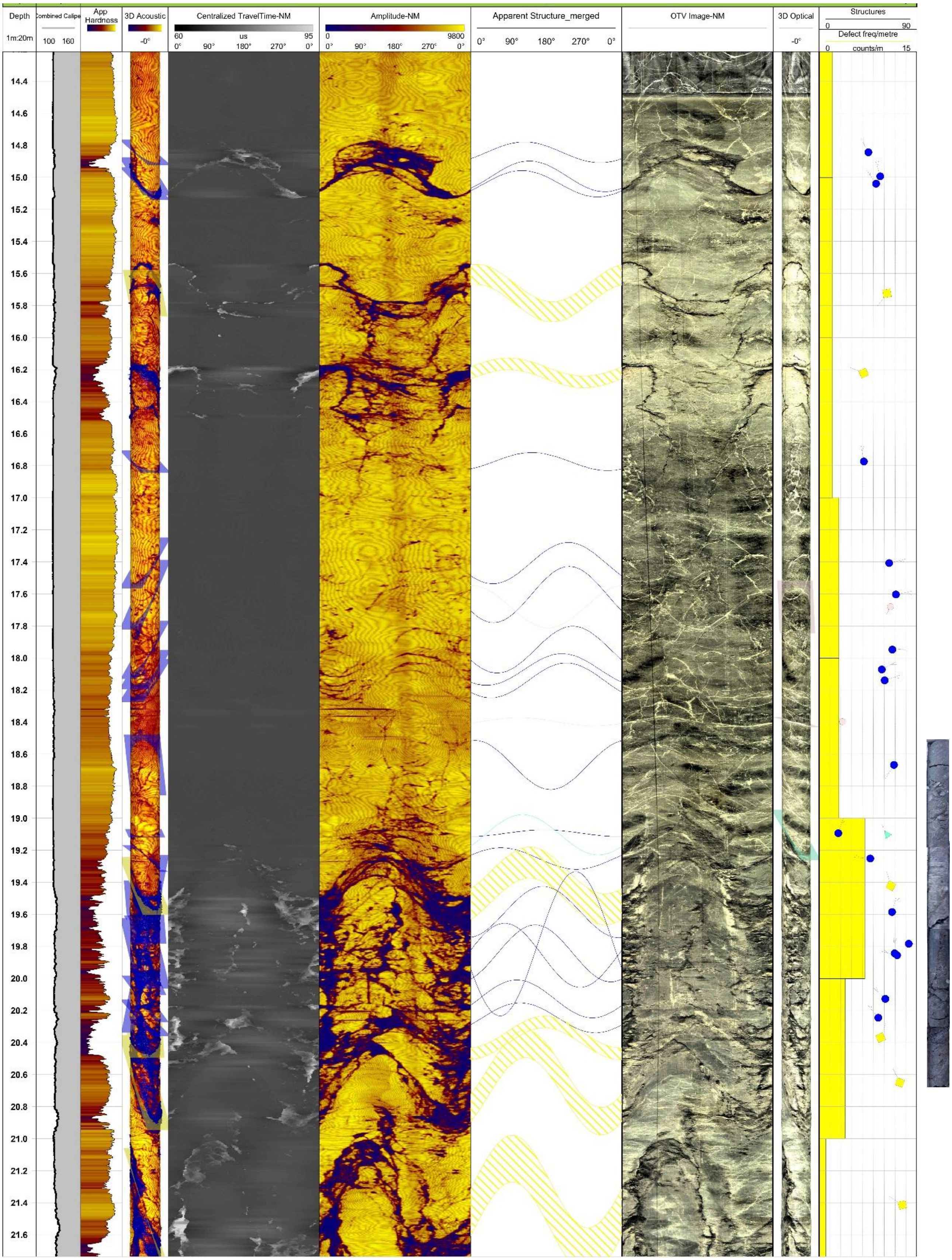

Figure 8: Interpreted structural defects from acoustic/optical televiewer survey and core samples from the Awatere Valley landslide

5.12 Discussion

5.12.1 Comparison with current design standards

The proposed design approach in this guidance is compared against existing standards or guidelines, to illustrate the changes in approach. This is summarised in Table 17, which illustrates how the proposed guidance, albeit preliminary, uses a novel resilience-based approach to cut slope design, and addresses significant gaps in current design guidance for cut slopes.

Table 17: Comparison of proposed guidance with existing design standards

Design feature	This guidance	Existing standards or guidelines			
		NZS 1170	NZGS-MBIE modules	Bridge Manual	Eurocode
Importance level	Uses NZS 1170 and Bridge Manual approach.	Provides importance level.	Not addressed.	Provides importance levels based on NZS 1170 and relevant to transport infrastructure.	-
Resilience	Resilience-based design approach, importance, earthquake motions and performance level.	Not considered beyond importance level.	Not addressed.	Not considered beyond importance level.	Not considered.
Earthquake motions	Consistent across all transport structures.	Only addresses buildings.	Provides PGA with higher levels in central New Zealand based on Cubrinovski *et al.* (2022).	Variable earthquake design level for different components of road. Low return period for cut slopes.	Provided.
Topographic effects	Topographic amplification factor. Reduction of TAF along slope based on literature or analysis.	Not provided for.	Not provided for, except for retaining walls in Module 6.	Not provided for.	Provided for buildings above slope.
Earthquake motions for pseudo-static slope design	Scaling of ground acceleration for deep-seated failures, based on international practice.	Slopes not provided for.	No specific guidance for slopes.	No specific guidance.	Provides for arbitrary reduction factor.

5.12.2 Application of design recommendations

Some of the recommendations provided have been trialled on projects since the development of the guidance as part of the NZ Transport Agency report RR613 (Brabhaharan *et al.*, 2018), and this research project. These are some observations from such applications:

(a) The application of the guidance results in levels of resilience appropriate to the proposed transport route and associated resilience expectations.

(b) UAV surveys and acoustic or optical televiewer surveys during site investigations, and interpretation of the rock defects using the results have been valuable to develop a better geological model and understand failure mechanisms.

(c) The upper parts of cut slopes result in being formed at a flatter slope angle, as a result of consideration of the amplified shaking and weaker soil / rock.

(d) Better understanding of failure mechanisms from more appropriate ground investigations and design for resilience results in cut slopes with lesser need for intrusive mitigation measures during construction, and therefore less disruption during construction and lower carbon footprints.

(e) Embankments designed for limited displacements can be quickly restored.

(f) Benches are identified as valuable features in cut slopes and berms in embankments to accommodate small rock falls, wedge failures and slumps.

(g) Rock fall barriers on benches and at the base of cut slopes being adopted to manage the life safety risk from rock fall and small failures.

(h) Location of transport routes away from slopes with the potential for large failures that are difficult to mitigate, such as adjacent to active faults and unstable hillsides.

(i) Transport routes that are resilient and able to be recovered quickly after hazard events.

(j) Consideration of the risk of debris inundation of facilities adjacent to slopes with potential for landslides.

6 Conclusions and Recommendations

Significant failures of earthworks and natural slopes occurred in the 2016 Kaikōura earthquake, which led to major impacts on the infrastructure in the North Canterbury and Marlborough regions and closure of rail and road corridors for many months to over a year. This led to poor resilience of the infrastructure, and considerable impact on the economy and sustainability.

There are limited national and international standards on the resilient seismic design of earthworks. Most international standards provide for pseudo-static design and provide for selection of peak ground accelerations, and some provide for use of reduced peak ground acceleration for design. Eurocode 8 does provide for some topographical amplification in slopes higher than 30 m.

Opus (now WSP) carried out a research programme for the NZ Transport Agency (now Waka Kotahi) including literature review and developed guidance for the seismic design of cut slopes (Brabhaharan *et al.*, 2018). This was followed by a multi-year research funded by MBIE into the performance of earthworks and landslides along the transportation corridors in the 2016 Kaikōura earthquake. This provided the opportunity to learn from the earthquake as well as build on the previous research by Brabhaharan *et al.* (2018). Recommendations for best practice design of earthworks to provide resilience have been compiled from this combined research.

A resilience-based design approach is recommended for the design of earthworks for infrastructure. This enables the development of economical and sustainable solutions that focus on the functionality of the infrastructure, in this case transport routes. This would provide for the design of earthworks to limit damage to provide continued functionality or quick recovery of access after earthquake events.

A design approach based the resilience importance of the earthworks structure is proposed, based on the resilience importance of the transport corridor and the potential impact of performance (based on scale of earthworks). The design approach is then selected based on these factors.

An important step in the design is carrying out ground investigations to characterise the ground conditions and develop a reliable ground model. Boreholes supplemented by Static or Piezocone penetration tests for soils, and acoustic / optical televiewer surveys for rocks would be prudent to provide the necessary soil and rock strength and defect parameters. Unmanned aerial vehicle (UAV) surveys are also valuable to provide topographic information and rock defect orientations and persistence from outcrops. A 3-dimensional ground model such as using LeapFrog may be useful for complex sites.

Peak ground accelerations for design are usually derived from the Bridge Manual for transportation projects. Given that a new National Seismic Hazard Model has been developed for New Zealand, and this has yet to be incorporated into the Bridge Manual, it would be prudent to consider this in the design. It is recommended that seismic design accelerations are chosen considering the importance and resilience expectations, and regardless of the type of structure. The current Bridge Manual proposes much lower seismic design parameters for cut slopes compared to embankments, retaining walls and bridges, and this is inappropriate.

The free field seismic shaking parameters should be modified to account for topographical amplification. Guidance is provided for selection of these factors.

Understanding and assessment of slope failure mechanisms is critical for the seismic design of earthworks. Slope failure mechanisms observed are presented to facilitate selection of the plausible mechanisms of failure for assessment.

Pseudo-static analyses can be used depending on the resilience importance and associated design approach. In carrying out pseudo-static analyses, it is important to select peak ground accelerations, depending on the scale of the failure in relation to the overall slope, because topographical amplification is dominant in the upper part of the slope. When considering full height failures in high slopes, the average peak ground acceleration may be lower, because of the incongruence of shaking at different levels and lower acceleration within the slope mass. Simple recommendations are provided to take this on board.

Decoupled slope stability and displacement assessment, resilience-based design and sound construction practices provide a basis for design to achieve resilience in the performance of fill embankments. In the Kaikōura earthquake, ductile displacement of the fill embankments generally meant displacement and cracking, rather than the brittle failure, disaggregation of rock masses and large runouts observed in the rock cut slopes.

Some earthworks can be designed to accommodate some deformation. A displacement-based design is generally suitable for embankments which comprise engineered slopes, and accommodating displacement is a good resilience-based approach where limited access can be maintained, and the deformations can be quickly repaired. However, as observed in the Kaikōura earthquake, excessive displacements can lead to reduction or loss of strength along failure surfaces which take a much longer time to recover.

Displacements can be assessed based on semi-empirical methods that use the Newmark (1965) sliding block approach, developed by statistical analyses of past slope displacements. Alternatively, numerical decoupled analyses can be adopted.

Cut slopes formed in natural materials, particularly rock, can fail in a brittle manner, where breakdown of the rock bridges can lead to en echelon failures and evacuative failure and collapse of the failed mass. Even where there are limited displacements and cracking, these can lead to water ingress and failure during subsequent aftershocks or storm events. Therefore, displacement should be avoided in such materials, or kept to a very low level.

Small failures in cut slopes (such as small wedge failures in rock in earthquakes) can be accepted and accommodated, while maintaining resilience, provided the debris can be contained in berms or can be quickly removed to restore access.

The runout mechanisms, particularly for cut slopes and natural slopes above transport corridors should be considered, as this would inform how much of the transport corridor might be engulfed, and, where necessary, help decide set back distances from high slopes. Guidance is provided on estimating runout distances form the wider EILD research programme.

The resilience of designs should be assessed using resilience metrics of availability and outage developed by Brabhaharan *et al.* (2006), and the design verified to assess whether it would meet resilience requirements for the corridor.

It is recommended that the resilience-based design approaches and seismic design methods be incorporated in NZGS slope stability guidance units as well as into the Bridge Manual, to ensure that future design of earthworks is engineered to provide good resilience. The investigation, assessment and resilience-based design principles will have applicability across a broad range of infrastructure and the built environment including the residential sector. The resilience parameters will need to be adapted to suit the built environment being considered for the NZGS guidance which would need to have wider applicability.

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