Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Risk assessment for Deans Head

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BIBLIOGRAPHIC REFERENCE

Massey, C. I.; Della Pasqua, F.; Taig, T.; Lukovic, B.; Ries, W.; Heron, D. 2014. Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Risk assessment for Deans Head. *GNS Science Consultancy Report* 2014/77. 122 p. + Appendices

REVIEW DETAILS

This report in draft form was independently reviewed by Dr L. Richards and Dr J. Wartman. Internal GNS Science reviews of drafts were provided by N. Litchfield, J. Carey, D. Mieler and R. Buxton.

Risk calculations were checked by R. Buxton (GNS Science).

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EXECUTIVE SUMMARY

ES 1 INTRODUCTION

This report combines recent field information collected from the Deans Head site with numerical slope-stability modelling to assess the risk to people in dwellings and users of Main Road from mass movements at the site.

Following the 22 February 2011 earthquakes, extensive cracking of the ground occurred in some areas of the Port Hills. In many areas, the cracks were thought to represent only localised relatively shallow ground deformation in response to earthquake shaking. In other areas however, the density and pattern of cracking and the amounts of displacement across cracks clearly indicated large mass movements.

Christchurch City Council contracted GNS Science to carry out further detailed investigations of these areas of systematic cracking, in order to assess the nature of the hazard, the frequency of the hazard occurring, and whether the hazard could pose a risk to life, a risk to existing dwellings and/or a risk to critical infrastructure. This work on what are termed mass movements is being undertaken in stages. Stage 1 is now complete (Massey et al., 2013) and stages 2 and 3 are detailed investigations of mass movements from highest to lowest priority.

The Stage 1 report identified 36 mass movements of concern in the Port Hills project area. Four of these were further subdivided based on failure type, giving a total of 46 mass movements including their sub areas. Fifteen of these were assessed as being in the Class I (highest) relative hazard-exposure category. Mass movements in the Class I category could cause loss of life, if the hazard were to occur, as well as severe damage to dwellings and/or critical infrastructure, which may lead to the loss of services for many people.

Deans Head mass movement area was assessed in the Stage 1 report (Massey et al., 2013) as being in the highest relative hazard exposure category (Class I, involving potential risk to life). Following the 22 February 2011 earthquakes, significant localised cracking was noted in the loess (soil) slope at the Deans Head mass movement, and the amount of slope displacement, coupled with the steep slope angles, suggested the slope could be susceptible to earth/debris flows.

This report, as part of the Stage 2 investigations, presents the risk assessment results for the Deans Head Class I mass movement.

ES 2 INVESTIGATION PROCESS AND FINDINGS

Detailed investigations of the site and its history were carried out by GNS Science. These investigations have identified several relict landslides (up to 10,000–15,000 m^3 in volume) at the site that appear to pre-date European settlement (about 1840 AD). Rockfalls are also apparent from the steep rock slope (called Shag Rock Reserve) in the 1946 and 1984 aerial photographs. Field mapping of the slope has identified areas of consistently bent mature trees and pre-2010/11 evidence of damage to dwellings. The evidence suggests that the slope has moved, albeit at low rates of movements, in the recent past. The areas of past movement coincide with the same areas that were cracked during the 2010/11 Canterbury earthquakes.

The slopes at Deans Head were significantly cracked during the 22 February 2011 earthquakes, and again during the 13 June 2011 earthquakes. Relatively little movement was reported in the other moderate sized earthquakes.

The absolute ground displacements at this site through the 2010/11 Canterbury earthquakes are constrained by survey markers installed to enable before-and-after measurements. Total recorded permanent displacements are estimated to be slightly less than one metre horizontally, with most of the movement being recorded in the lower and central part of the mass movement.

The bulk strength of the loess forming the slopes was weakened by cracking; and in particular, the presences of open surface cracks has made the slope more susceptible to the ingress of run-off water, which is expected to weaken the loess.

The main types of landslide hazard identified at the site originating from the loess slopes are earth/debris flows, which are a relatively fluid and rapid type of landslide. The risk to life of people in dwellings from debris avalanches and cliff top recession hazards associated with the steep rock slope of Shag Rock Reserve (collectively termed cliff collapse), has already been estimated and is reported by Massey et al. (2012).

By mapping cracks and relating these to the results of stability assessments, it has been possible to identify two potentially significant earth/debris flow source areas, from which landslides of variable volume could occur. The assessed source areas are not the only source areas for landslides on this slope; they are representative of the volumes of landslides that could occur from anywhere within the assessment area.

Numerical models have been used to assess the stability of the Deans Head slopes, in particular the two potential landslide sources. Analyses have considered both:

- static (without earthquake shaking); and
- dynamic (with earthquake shaking) conditions.

Earth/debris flows

The main triggering mechanism for the assessed source areas is considered to be rain, although earthquake shaking could trigger failure, especially if an earthquake occurs when the slope is wet. However, rainfall-induced failures are likely to be more mobile, and the return period of the triggering event more frequent, and these therefore pose the greatest risk.

The findings of the static analyses are that the loess and colluvium strengths appear sufficient to prevent slope collapse under relatively dry conditions. Based on published laboratory test results on loess, cohesion can reduce to near zero when the water content is increased. Should the water content of the loess/colluvium increase, then the loess/colluvium would become much weaker and the static stability analysis indicates that failure would be possible.

The water contents of the loess/colluvium at critical failure surfaces have not been measured to date, so the amount, duration and/or intensity of rainfall required to promote instability cannot be quantified at present. It is known however, that there have been numerous past Port Hills landslides triggered by rain, that the probability of triggering a given landslide increases with rainfall intensity and duration, and that the slopes in their present condition are particularly vulnerable to water ingress via the numerous open cracks in the ground surface.

For the two assessed source areas, the likely volume of material mobilised during a slope failure event, and the frequency of the slope failure triggering event, are both uncertain. Nonetheless, the slopes have remained stable during both earthquake aftershocks since the 22 February 2011 earthquake and significant rainfall without major failure.

Failure volumes and triggering frequencies

The volumes of material involved in, and the frequency of slope failures from the identified sources, are assessed. Three source-volume ranges (upper, middle and lower volumes per source area) and five event annual frequencies (representing return periods of 20, 50, 100 and 200 years) have been modelled. Both are uncertain and the frequency of the triggering events is particularly uncertain. Whilst the slopes survived substantial aftershocks and two notable rainfall episodes since the 22 February 2011 earthquake without major failure, the strength of the slope is weakened by cracking; and in particular the cracking has made the slope more susceptible to water ingress, which would be expected to weaken them further (possibly critically so) in a severe weather episode.

A risk assessment was carried out for each of the newly identified potential source areas, using a range of triggering frequencies and landslide volumes (upper, middle and lower source volume estimates) to reflect the associated uncertainties, and the overall annual individual fatality risk for a nearby resident or users of Main Road has been assessed.

The risk assessment for users of Main Road combines: 1) the earth/debris flows hazards at Deans Head; with 2) the cliff collapse hazards associated with the steep rock slope (Shag Rock Reserve), reported by Massey et al. (2012).

ES 3 CONCLUSIONS

ES3.1 Hazard

1. There is potential for volumes ranging from several hundreds to tens of thousands of cubic metres of earth/debris flows (source areas 1 and 2) of mixed loess and colluvium, which are in addition to the cliff-collapse failures previously assessed (Massey et al., 2012).
2. The most likely triggers for the assessed earth/debris flows sources are prolonged heavy rainfall and strong earthquake shaking (if ground conditions were wet).
3. The frequency of earth/debris flow events from these sources is difficult to estimate and could be anything from once every few tens to once every many hundreds of years.

ES3.2 Risk

ES3.2.1 Dwelling occupant

1. There are very few dwellings in the earth/debris flow runout zone, as most dwellings are located in the assessed source areas.
2. The main hazard affecting these dwellings is likely to be a combination of cracking and undercutting as the ground moves beneath the dwelling, potentially causing significant damage to the dwelling, as well as the impact from debris coming from further upslope.
3. It is difficult to assess what the levels of risk to the dwellings in the source areas are, given the uncertainties associated with the triggering event, source volume and area that could be affected. The risk associated with the assessed source areas is, therefore inferred to be the same as the risk in the runout zone immediately below the assessed source areas, which is shown as 10^{-4} or greater.
4. The numbers of dwellings affected by the upper source volume estimates are, as expected, larger than those few affected by the lower volume estimates, as the lower volume estimates are associated with smaller source areas.
5. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings from future earthquakes is still relatively high and similar to a Class II relative hazard exposure category. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.4 m. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the National Seismic Hazard Model.

ES3.2.2 Road user

1. Generally, the risk to road users of Main Road in the assessed section road below Deans Head is significantly higher than that at the other sites assessed to date (Wakefield Avenue and Quarry Road; Massey et al., 2014a,b).
2. The volumes of material reaching the road could be relatively high and could occur with relatively high (though uncertain) frequency.
3. There is limited means of escape for motor vehicle users from Main Road over the assessed section of road, other than by travelling forward or back along Main Road itself.
4. There are relatively high traffic densities for significant proportions of the time.
5. The road to the west of Deans Head lies next to relatively deep, fast moving water with only a wooden crash barrier to prevent road users inundated by rockfall or debris being washed into the sea.
6. There is potential for accident scenarios in which a queue of traffic is trapped on this section of the road at exactly the time that a significant (seismically triggered) slope failure occurs.

ES3.2.3 Risk management

1. A risk-management option of monitoring rainfall, soil moisture and pore-pressure in the source areas, may be of some value in providing warning of conditions approaching critical levels, but:
   a. Such early warning could not be assured, as experience in the Port Hills and elsewhere is that water levels in open tension cracks can rise very rapidly to critical values;
   b. There would be little time to evacuate potentially at-risk residents given the rapid nature of the hazard; and
   c. There is currently no precedent data for rates of change of groundwater or water content of loess to provide reliable alert criteria.
2. There appears to be reasonable scope for engineering measures to stabilise the slopes (e.g., by removal of loess/colluvium and installation of drainage measures). However, site access may be difficult due to the nature of the ground, and these works would need to be evaluated, designed and implemented by a suitably qualified engineering consultant.

ES 4 RECOMMENDATIONS

GNS Science recommends that based on the results of this study, Christchurch City Council:

ES4.1 Policy and Planning

1. Decide what levels of life risk to dwelling occupants and road users will be regarded as tolerable.
2. Decide how Council will manage risk on land and roads where life risk is assessed to be at the defined threshold of intolerable risk and where the level of risk is greater than the threshold.
3. Prepare policies and other planning provisions to address risk lesser than the intolerable threshold in the higher risk range of tolerable risk.

ES4.2 Short-term actions

ES4.2.1 Hazard monitoring strategy

1. Include the report findings in a slope-stability monitoring strategy with clearly stated aims and objectives, and list how these would be achieved, aligning with the procedures described by McSaveney et al. (2014). In the meantime, extend the current survey network further up the slope (particularly in source area 1 towards Kinsey Terrace), so as to maintain awareness of changes in the behaviour of the slope;
2. Ensure that the existing emergency management response plan for the area identifies the dwellings that could be affected by movement and runout and outlines a process to manage a response.

ES4.2.2 Risk monitoring strategy

Monitoring the slope for early warning of potentially dangerous trends in groundwater or slope movement as part of a hazard warning system, is currently not thought to be feasible. Monitoring alerts for slope deformation and groundwater changes cannot be relied upon to provide adequate early warning as experience from Port Hills and elsewhere shows that deformation and groundwater changes can occur rapidly, with little warning, and there is little site-specific information on which to build such a warning system.

ES4.2.3 Surface/subsurface water control

Reduce water ingress into the slopes, where safe and practicable to do so, by:

1. Identifying and relocating all water-reticulation services (water mains, sewer pipes and storm water) inside the identified mass-movement boundaries (at the slope crest) to locations outside the boundary, in order to control water seepage into the slope. In particular, a water main currently traverses the site between assessed source areas 1 and 2; and
2. Control surface water seepage by filling the accessible cracks on the slope and providing an impermeable surface cover to minimise water ingress.

ES4.3 LONG-TERM ACTIONS

ES4.3.1 Engineering measures

1. Assess the cost, technical feasibility and effectiveness of alternative longer term engineering and relocation solutions, for example (but not limited to):
   a. Removal/stabilisation of the slopes in the assessed source areas;
   b. Installation of drainage works;
   c. Relocation of houses to alternative locations within existing property boundaries; or
   d. Withdrawal and rezoning of the land for non-residential use.
2. Any proposed engineering works should require a detailed design and be carried out under the direction of a certified engineer, and should be independently verified in terms of their risk reduction effectiveness by appropriately qualified and experienced people.
3. For the section of Main Road within the risk zone, liaise with whoever is responsible for roading (within Christchurch City Council) to develop solutions, which both: 1) ensure that the key lifeline section of Main Road can continue to serve its purpose of connecting Sumner and the surrounding area to Christchurch; and 2) adequately safeguard road users from slope-collapse risk.

ES4.3.2 Reassessment

Reassess the risk and revise and update the findings of this report in a timely fashion, for example:

1. in the event of any significant changes in ground conditions; or
2. in anticipation of further development or significant land use decisions.

1.0 INTRODUCTION

This report brings together recent field information on the Deans Head site and uses numerical models of slope stability to assess the risk to people in dwellings and users of Main Road from mass movements at the site, over and above those assessed in an earlier cliff collapse study (Massey et al., 2012).

1.1 BACKGROUND

Following the 22 February 2011 earthquakes, members of the Port Hills Geotechnical Group (a consortium of geotechnical engineers contracted to Christchurch City Council to assess slope instability in the Port Hills) identified some areas in the Port Hills where extensive cracking of the ground had occurred. In many areas cracks were thought to represent only localised relatively shallow ground deformation in response to shaking. In other areas however, the density and pattern of cracking and the amounts of displacement across cracks clearly indicated that larger areas had moved systematically *en masse* as a mass movement.

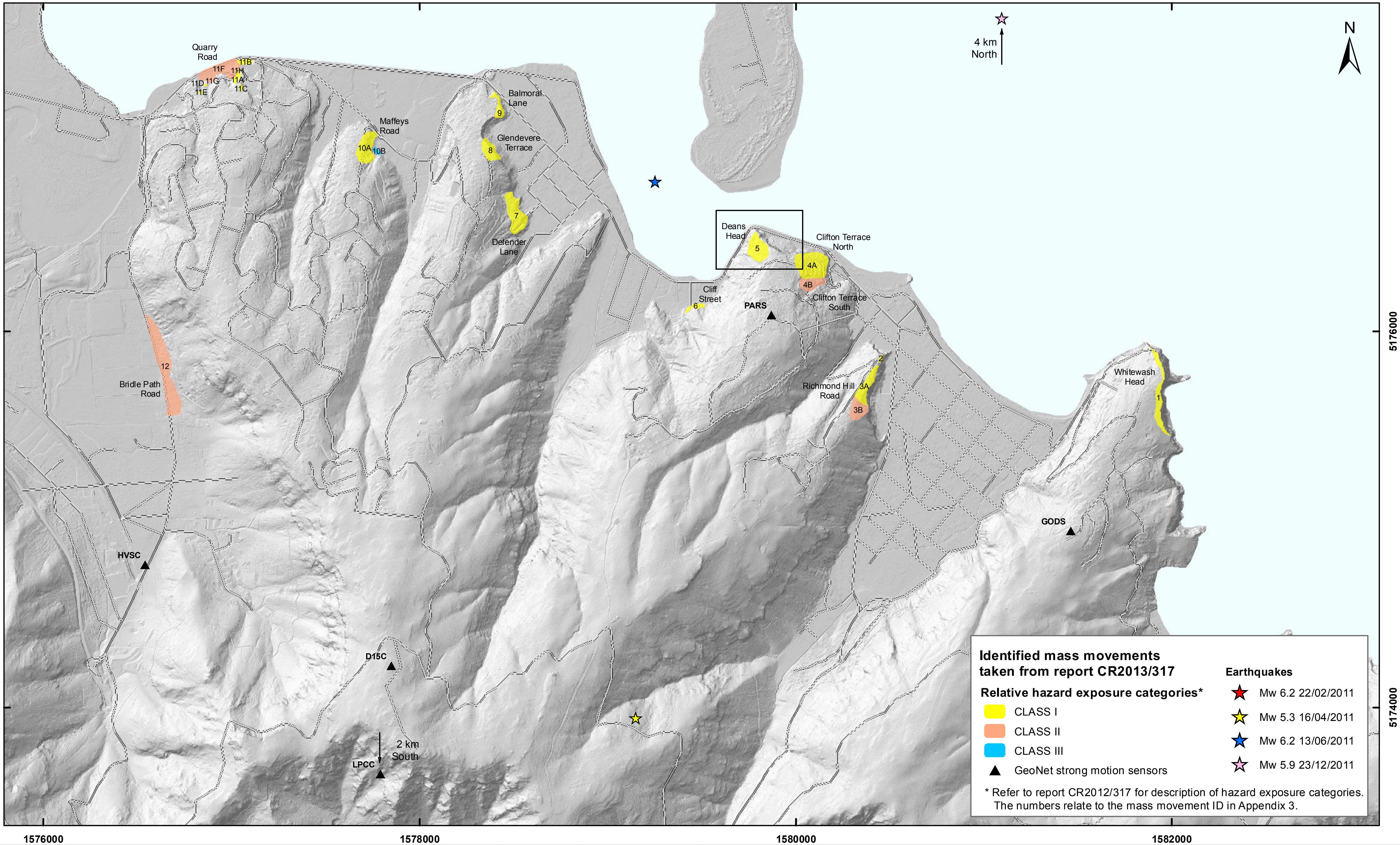
Christchurch City Council contracted GNS Science to carry out detailed investigations of the identified areas of mass movement in order to assess the nature of the hazard, the frequency of the hazard occurring, and whether the hazard could pose a risk to life, a risk to existing dwellings and/or a risk to critical infrastructure (defined as water mains, sewer mains, pump stations, electrical substations and transport routes). This work is carried out under Task 4 of contract No. 4600000886 (December 2011).

The main purpose of the Task 4 work is to provide information on slope-stability hazards in the Port Hills. This is to assist Christchurch City Council land-use and infrastructure planning and management in the area, as well as to establish procedures to manage on-going monitoring and investigation of the hazards.

The Task 4 work is being undertaken in stages. Stage 1 is now complete (Massey et al., 2013; hereafter referred to as the Stage 1 report) and comprised: 1) a list of the areas susceptible to significant mass movement; 2) the inferred boundaries of these areas (as understood at the time of reporting); and 3) an initial “hazard-exposure” assessment (Table 1) intended only to prioritise the areas with regards to future investigations.

The Stage 1 report identified 36 mass movements of concern in the Port Hills project area. Four of these were further subdivided based on failure type, giving a total of 46 mass movements including their sub areas (Figure 1). Fifteen of these were assessed as being in the Class I (highest) relative hazard-exposure category, and the results of their detailed investigation and assessment are presented in Stages 2 and 3, which includes this Stage 2 report on the Deans Head Class I mass movement. Mass movements assessed as being in the Class I category may cause fatalities, severe damage to dwellings and/or damage critical infrastructure leading to loss of services for many people if the hazard were to occur.

The Stage 1 report recommended that mass movements in the Class I relative hazard-exposure category be given high priority by Christchurch City Council for detailed investigations and assessment.



SCALE BAR: 0 0.5 1 km

EXPLANATION:

Refer to Appendices 2 and 3 of report CR2012/317 for maps and more details of each mass movement.

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM

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LOCATION MAP

Port Hills Christchurch

FIGURE 1

Map 1

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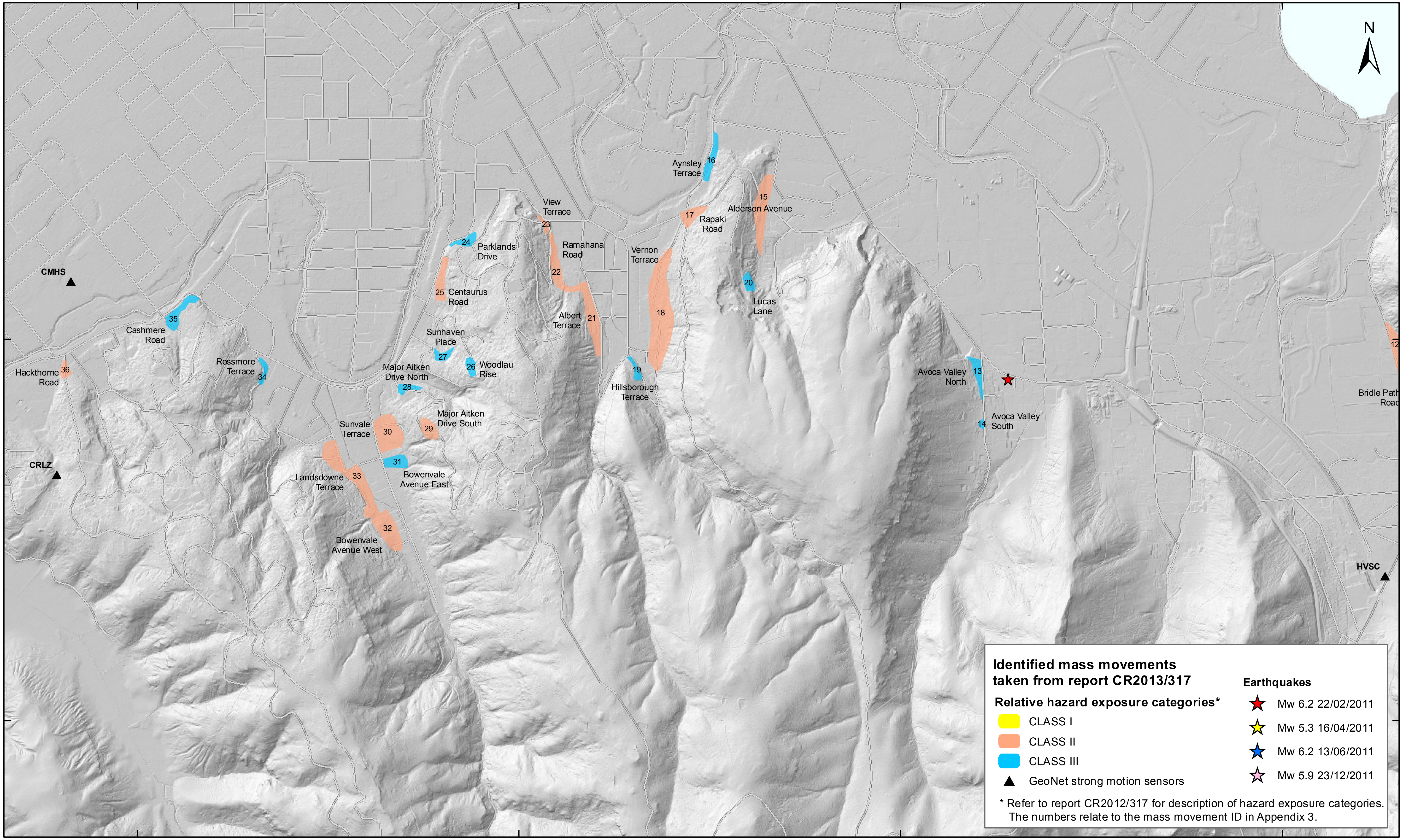
DATE: June 2014



SCALE BAR: 0 0.5 1 km

EXPLANATION:
Refer to Appendices 2 and 3 of report CR2012/317 for maps and more details of each mass movement.
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM

LOCATION MAP

Port Hills Christchurch

FIGURE 1

Map 2

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Table 1 Assessed mass movement relative hazard exposure matrix (from the Stage 1 report, Massey et al., 2013).

		Hazard Class		
		1. Displacement* greater than 0.3 m and debris runout	2. Displacement* greater than 0.3 m; no runout	3. Displacement* less than 0.3 m; no runout
Consequence Class	1. Life – potential to cause loss of life if the hazard occurs	**CLASS I**	**CLASS III**	**CLASS III**
	2. Critical infrastructure[1] – potential to disrupt critical infrastructure if the hazard occurs	**CLASS I**	**CLASS II[2]**	**CLASS II**
	3. Dwellings – potential to destroy dwellings if the hazard occurs	**CLASS I**	**CLASS II**	**CLASS III**

*Note: Displacements for each assessed mass movements are inferred by adding together the mapped crack apertures (openings) along cross-sections through the assessed mass movements. They are a lower bound estimate of the total displacement, as no account is given for plastic deformation of the mass and not every crack has been mapped.

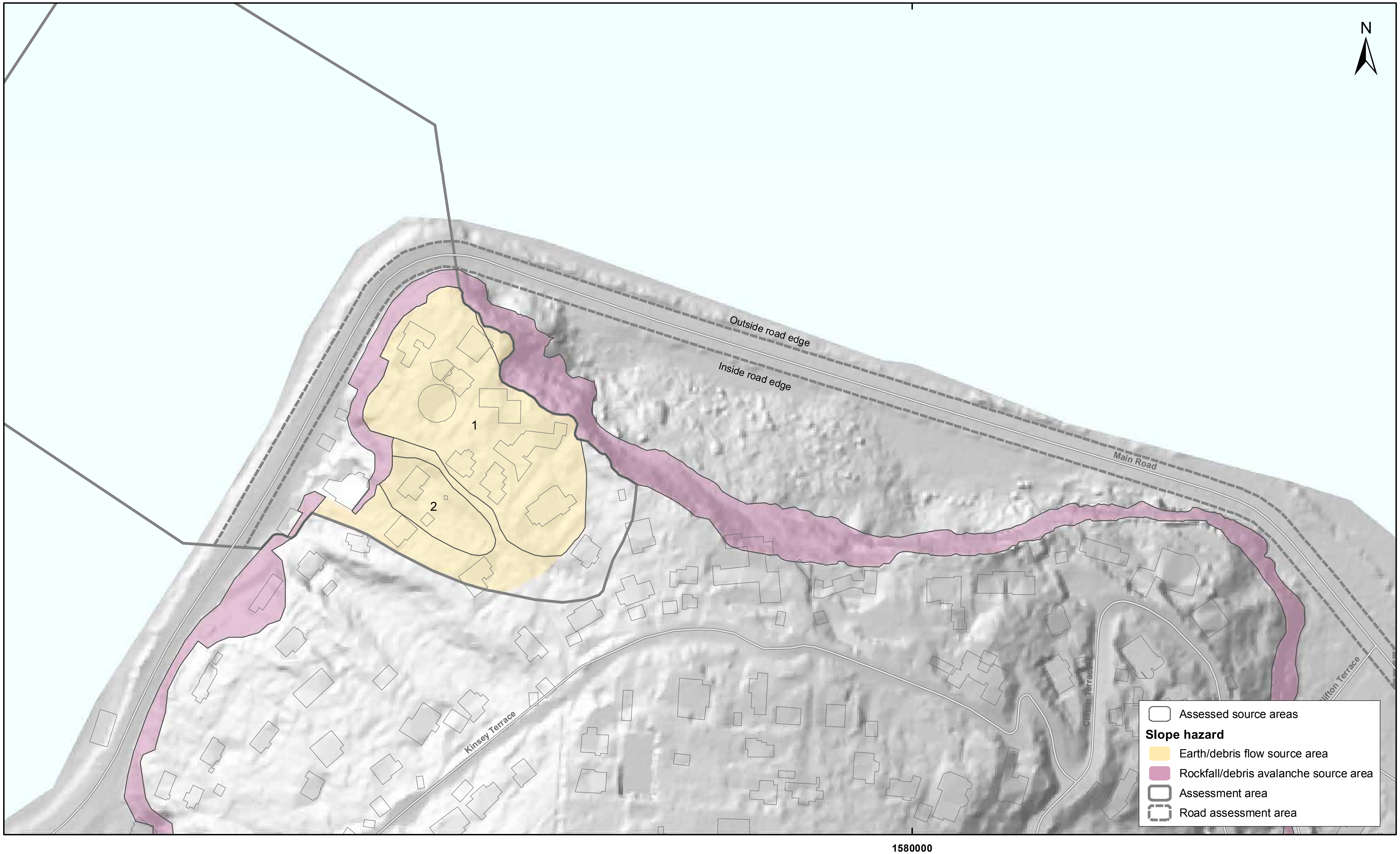
1 Critical infrastructure is defined, for the purpose of this report, as infrastructure vital to public health and safety. It includes transport routes (where there is only one route to a particular destination), telecommunication networks, all water related mains and power networks (where there is no redundancy in the network), and key medical and emergency service facilities. Networks include both linear features such as power lines or pipes and point features such as transformers and pump stations.

2 This relative hazard exposure category is based largely on an assumption that 'critical infrastructure' exists within these areas. Until further assessments are made on the nature of toe slumps and the existence of critical infrastructure in these areas, the relative hazard exposure category of these assessed mass movements has been appropriately assessed as "Class II". It is likely that many of the assessed mass movements in the Class II relative hazard exposure category (where the hazard class is 2 and the consequence class is 2) would be more appropriately classified as "Class III" following further assessments.

1.2 THE DEANS HEAD MASS MOVEMENTS

The Deans Head mass movement area is shown in Figure 1 and Figure 2. This mass movement area was assessed in the Stage 1 report (Massey et al., 2013) as being in the highest relative hazard exposure category (Class I).

This report presents the risk assessment results for the Deans Head Class I mass movement. The map in Figure 2 outlines the assessed potential landslide source areas (numbers 1 and 2), within the identified mass movement area, while Figure 3 and Figure 4 show the boundaries of the mass movement area in comparison with those of the cliff collapse area for which the dwelling risk has previously been assessed (Massey et al., 2012).



SCALE BAR: 0 50 m

EXPLANATION:

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and buildings footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

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MASS MOVEMENT LOCATION MAP

Deans Head
Christchurch

FIGURE 2

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1.3 PREVIOUS WORK AT THE DEANS HEAD SITE

Following the 22 February 2011 earthquakes, significant localised cracking was noted on the slope surface within the Deans Head mass movement (Figure 3 and Figure 4). Previous investigations of the site comprised:

1. The risk to life of people in dwellings at the cliff crest (but not to road users) from debris avalanches and cliff top recession hazards associated with the steep rock slope (collectively termed cliff collapse) were previously estimated by Massey et al. (2012);
2. Field mapping of the crack distributions was carried out by GNS Science, and the results are contained in the Stage 1 report (Massey et al., 2013);
3. Ground investigation of the site has involved drilling of two fully-cored drillholes, and inclinometer monitoring, carried out by Aurecon NZ Ltd, under contract to Christchurch City Council. The results of the drilling are contained in Codd and Revell (2013); and
4. Ground investigation and field mapping of the site was also carried out by Tonkin and Taylor Ltd, under contract to the Earthquake Commission (Tonkin and Taylor, 2012a). The ground investigations comprised the drilling of three drillholes (one cored, one open hole and one open barrel), three scala penetrometers and installation of one standpipe (piezometer) and one inclinometer, in selected drillholes.

Figure 3 Aerial view of the Deans Head mass movement is within the yellow dashed lines (approximate extent only – refer to Figure 2 for the mapped extent).

Figure 4 Cliff collapse area not included in this assessment (approximate extent only – refer to Figure 2 for mapped extent). The risk assessment for cliff collapse is described by Massey et al. (2012).

1.4 SCOPE OF THIS REPORT

The scope of this report as per Appendix A of contract No. 4600000886 (December 2011) is to:

1. Estimate the annual individual fatality risk for affected dwelling occupants from the failure of the assessed source areas 1 and 2, within the shown assessment area in Figure 2.
2. Estimate the fatality risk for users of Main Road from the failure of the assessed source areas (Figure 2) and from cliff collapse hazards as detailed in Massey et al. (2012), for the section of Main Road shown in Figure 2.
3. Provide recommendations to assist Christchurch City Council with considered options to mitigate life risks, associated with the assessed source areas.

For the purpose of this risk assessment, dwellings are defined as timber framed single-storey dwellings of building importance category 2a (AS/NZS 1170.0.2002). The consequences of the hazards discussed in this report on other building types, such as commercial buildings, have not been assessed.

The risk assessments contained in this report supersede the preliminary risk assessments contained in the Working Note 2013/03 (Della Pasqua et al., 2013).

1.5 REPORT STRUCTURE

- Section 1.6 of the report details the methodology.
- Section 2 details the data used in the assessments.
- Sections 3–5 contain the results from the engineering geological, hazard and risk assessments respectively.
- Section 6 discusses the results of the risk assessment and explores the uncertainties associated with the estimated risks.
- Section 7 summarises the assessment findings.
- Section 8 presents recommendations for Christchurch City Council to consider.

1.6 METHODS OF ASSESSMENT

The site assessment comprised three stages:

1. Engineering geology assessment;
2. Hazard assessment; and
3. Risk assessment.

The methodology adopted for each stage is described in detail in Appendix 1, and is summarised in the following sections.

1.6.1 Engineering geology assessment

The findings presented in this report are based on engineering geological models of the site developed by GNS Science. The engineering geological assessment comprised:

1. Interpretation of available aerial photographs covering the period 1940–2011, to determine the land use and development history of the site.
2. Surveying of cadastral survey marks within and around the mass movement to determine the magnitudes of slope displacement during the 2010/11 Canterbury earthquakes.
3. Assessment of the results from the surveying of monitoring marks installed on the site by Aurecon NZ Ltd. (under contract to Christchurch City Council), following the 22 February 2011 earthquake. This was undertaken to assess the amount of slope displacement relating to the 22 February, 16 April, 213 June and 23 December 2011 earthquakes.
4. Geological and geomorphological field mapping to identify the materials, processes and landforms that have been active within the assessment area.
5. Review of previous ground investigations carried out by Aurecon NZ Ltd. (Codd and Revell, 2013) and Tonkin and Taylor Ltd. (Tonkin and Taylor, 2012a).
6. Establish an engineering geological model of the site and construction of an engineering geological map and five cross-sections, based on the results from the aerial photograph interpretation, surveying, field mapping, and the site investigations. These were used as the basis for the hazard and risk assessments.

1.6.2 Hazard assessment

The hazard assessment method followed three main steps:

Step 1 comprises assessment of the static stability of the slope under non-earthquake (static) conditions, and an assessment of the dynamic (earthquake) stability of the slope, adopting selected cross-sections, to determine how likely landslides are to occur, and whether these can/cannot be triggered under static and/or dynamic conditions.

Step 2 uses the results from step 1 to define the likely failure geometries (source areas) of potential landslides, which are combined with the crack patterns and slope morphology and engineering geology mapping to estimate their likely volume. Three volumes are defined for each source area (upper, middle and lower volumes), which represent the range of potential source areas that could occur within the assessment area.

Step 3 involves the use of models to determine: 1) the distance the debris travels down the slope (runout); and 2) the volume of debris passing a given location, should the landslide occur. Modelling is done for each representative source area, and for the upper, middle and lower volume estimates.

The results from this characterisation are then used in the risk assessment.

1.6.3 Estimation of landslide volumes

The results of the engineering geological assessments and the slope stability modelling carried out by GNS Science have been used to define two potential landslide sources areas. These are located in areas where the bulk strength of the slope could have been degraded as a result of earthquake-induced cracking. The assessed source areas (shown in Figure 2) do not represent the only potential locations of the source areas that could occur in the assessment area. The assessed source areas are intended to represent the range of potential landslide locations and volumes that could occur in the assessment area:

- The most likely locations and volumes of potential failures were estimated based on the numerical analyses, current surveyed displacement magnitudes, material exposures, crack distributions and slope morphology. The purpose of this was to constrain the likely depth, width and length of any future failures. This was done by linking the main cracks and pertinent morphological features, in combination with the width, length and depth of the failure surfaces derived from the finite element and limit equilibrium modelling.
- Three failure volumes (upper, middle and lower) were estimated for each potential source area to represent a range of source volumes. The variation in failure volume reflected the uncertainty in the results from the modelling and mapping, e.g., the depth, width and length dimensions.

1.6.4 Risk assessment

The risk metric assessed is the annual individual fatality risk and this is assessed for dwelling occupants and road users from the landslides assessed in this report, mainly earth/debris flows. Cliff-collapse hazards (comprising debris avalanches and cliff-top recession) within the assessment area were previously assessed by Massey et al. (2012), and these results are combined with the results in this report, to present risk estimates relating to both landslide hazard types.

The quantitative risk assessment uses risk-estimation methods that follow appropriate parts of the Australian Geomechanics Society framework for landslide risk management (Australian Geomechanics Society, 2007). It provides risk estimates suitable for use under SA/SNZ ISO1000: 2009.

Using the Australian Geomechanics Society (2007) guidelines for landslide risk management, the annual fatality risk to an individual is calculated from:

$$R_{(LOL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times V_{(D:T)}$$ **Equation 1**

where:

$R_{(LoL)}$ is the risk (annual probability of loss of life (death) of a person) from debris/earth flows/avalanches;

$P_{(H)}$ is the annual probability of the initiating event;

$P_{(S:H)}$ is the probability that a person, if present, is in the path of the debris at a given location;

$P_{(T:S)}$ is the probability that a person is present at that location;

$V_{(D:T)}$ is the vulnerability, or probability that a person is killed if present and hit by debris.

The details relating to each of the above input parameters used in the risk assessments are discussed in Appendix 1.

Event annual frequencies

The frequency of occurrence of the events that could trigger the assessed earth/debris flow failure volumes is unknown.

- For non-earthquake triggers such as rainfall, a range of event annual frequencies (P(H)) of 0.05, 0.02, 0.01, and 0.005 corresponding to return periods of 20, 50, 100 and 200 years, were used for the assessment to represent the likely return period of the event that could trigger failure of the assessed source areas.
- For earthquake events, the annual frequency of a given magnitude of permanent displacement of the slope, in the assessment area has been estimated by using:
  a. The relationship between the yield acceleration (*Ky*) and the maximum average acceleration of the mass (K_{MAX}), derived from back-analysing the permanent displacement of the slope during the 2010/11 earthquakes; and
  b. The New Zealand probabilistic National Seismic Hazard Model (Stirling et al., 2012) to provide the annual frequencies (return periods) of free-field rock outcrop peak horizontal ground accelerations (A_{FF}) and therefore the annual frequencies of the equivalent maximum average acceleration of the mass (K_{MAX}).

The methods adopted are discussed in detail in Appendix 1.

2.0 DATA USED

The data and the sources of the data used in this report are listed in Table 2.

Table 2 Summary of the main data used in the analysis. LiDAR is Light Detecting and Ranging.

Data	Description	Data source	Date	Use in this report
Post-22 February 2011 earthquake digital aerial photographs	Aerial photographs were taken on 24 February 2011 by NZ Aerial Mapping and were orthorectified by GNS Science (0.1 m ground resolution).	NZ Aerial Mapping	Last updated 24 February 2011	Used for base maps and to map extents of landslides and deformation triggered by the 22 February 2011 earthquakes.
Post-13 June 2011 earthquake digital aerial photographs	Aerial photographs were taken between 18 July and 26 August 2011, and orthorectified by NZ Aerial Mapping (0.5 m ground resolution).	NZ Aerial Mapping	18 July–26 August 2011	Used to map extents of landslides and deformation triggered by the 13 June 2011 earthquakes.
Historical aerial photographs	Photographs taken in 1940, 1946, 1973, 1975 and 1984 by multiple sources and orthorectifed by NZ Aerial Mapping and GNS Science (at variable ground resolutions).	NZ Aerial mapping and GNS Science	1946, 1975, 1975 and 1984	Used to assess the site history before the 2010/11 Canterbury earthquakes.
LiDAR digital elevation model (2011c)	Digital Elevation Model derived from post-13 June 2011 earthquake LiDAR survey; re-sampled to 1 m ground resolution.	NZ Aerial Mapping	18 July–26 August 2011	Used to generate contours and shade models for the maps and cross-sections used in the report.
Christchurch building footprints	Footprints are derived from aerial photographs. The data originate from 2006 but have been updated at the site by CCC using the post-earthquake aerial photos.	Christchurch City Council	Unknown	Used to identify the locations of residential buildings in the site.
GNS Science landslide database	Approximate location, date, and probably trigger of newsworthy landslides	GNS Science	Updated monthly	Used to estimate the likely numbers and volumes of pre-earthquake landslides in the areas of interest.
Earthquake Commission claims database	Location, date and brief cause of claims made in the Port Hills of Christchurch since 1993.	Earthquake Commission	1993–August 2010	Used to estimate the likely numbers and volumes of pre-earthquake landslides in the areas of interest.

Data	Description	Data source	Date	Use in this report
Synthetic earthquake time/ accelerations	Earthquake time acceleration histories for the four main 2011 earthquakes: 22 February, 16 April, 13 June and 23 December.	GNS Science	February 2014	Used as inputs for the seismic site response analysis.
Rainfall records for Christchurch	Rainfall records for Christchurch from various sources, extending back to 1873.	NIWA	1873–present	Used to assess the return periods of past storms triggering landslides of known magnitudes in the Port Hills.
Drillhole logs	Results from the logging of three drillholes and three scala penetrometers carried out at the site.	Tonkin and Taylor Ltd. (Tonkin and Taylor, 2012a)	2012	Used to generate the engineering geological map and cross-sections.
Drillhole logs	Results from the logging of two drillholes carried out at the site	Aurecon NZ Ltd. (Codd and Revell, 2013)	January 2013	Used to generate the engineering geological map and cross-sections.
Downhole shear wave surveys	Downhole shear wave velocity surveys carried out in the URS Ltd. drillholes.	Southern Geophysical Ltd. (2013)	February 2014	Used to determine the dynamic properties of the materials in the slope for the seismic site response analysis.
Geotechnical laboratory data	Geotechnical strength parameters for selected soil and rocks in the Port Hills.	GNS Science (Carey et al., 2014)	February 2014	Used for static and dynamic slope stability analysis.
Field work	Field mapping of slope cracking and engineering geology and ground truthing of the risk analyses.	GNS Science and the Port Hills Geotechnical group	22 February 2011–present	Used in generating the engineering geological models of the site. Results from field checks used to update risk maps.
Traffic counts for Main Road (Causeway to the east of the assessment area) and the Ferrymead/Main Road junction (to the West)	Detailed motor vehicle counts at 2-year intervals, by hour of day and day of week, are available for the Causeway. Junction data is more sparse but provides a valuable breakdown into heavy and light vehicles	Christchurch City Council	2008, 2010 and 2012 surveys	Used to assess total numbers of road users, and to model likely average extent and frequency of delays (and hence extended average time at risk) on Main Road.

3.0 SITE ASSESSMENT (RESULTS)

The site assessment results and engineering geological conceptual models developed for the site by GNS Science are summarised below. Figure 5 shows the main features identified at the site from field mapping and the review of aerial photographs. Figure 6 presents an engineering geological map for the site. Figure 7 shows the locations of the various site investigations. Figure 8 presents six engineering geological cross-sections through the site.

3.1 SITE HISTORY

3.1.1 Aerial photograph interpretation

Aerial photographs of the site are available for various dates since 1940. Table 3 summarises the photograph details and main features noted.

Table 3 Summary of observations from aerial photographs used to assess the site history at Deans Head.

Date/scale of photo	Resolution	Comments
1940 1:10,000 (approx.)	Poor resolution	Several large arcuate features – possible relict landslide scars – are apparent at the site. These tend to be relatively narrow and linear features. No corresponding accumulations of debris are present, but any debris would have likely to have been washed away by the sea (Figure 5). One of these features, towards the south, is large, and much of the debris appears to have left the source area. A few dwellings are present on the slope, including the water reservoir. Main Road is already constructed around the toe of the site. There is no evidence of past quarrying at the site apparent in the aerial photographs.
30/05/1946 1:5,500 (approx.)	Good resolution	At the northern end of the site there appears to be several recent rockfalls apparent at the toe of the steep cliff in Shag Rock Reserve.
1973, 1:10,000 (approx.)	Poor resolution	No obvious change. A few dwellings have now been constructed on the slope.
1975, 1:10,000 (approx.)	Poor resolution	No obvious change. A few more dwellings have been constructed on the slope.
1984, 1:6,000 (approx.)	Good resolution	At the northern end of the site there appears to be several recent rockfalls apparent at the toe of the steep cliff in Shag Rock Reserve. A few more dwellings have now been constructed on the slope

3.1.1.1 Relict landslides

Review of aerial photographs and field mapping has identified several relict landslide scars on the slope within and adjacent to the Deans Head site (Figure 5). One of these is relatively large, with an estimated plan area of about 3,800 m^2, and depth of between 3 and 6 m, based on cross-section 6 in Figure 8. The shape of the landslide is long and narrow. Much of the landslide debris has vacated the scar, indicating the debris may have run out into the sea as an earth/debris flow (or series of flows). Rock was mapped in the upper part of the landslide scar underlying volcanic colluvium and loess, which was exposed in the flanks of the landslide. Seepage was noted at rockhead, within the upper part of the landslide scar. The shape of the landslide (cross-section 6) is consistent with a translational failure, with the failure surface mainly within the loess/volcanic colluvium, and is sub parallel to rockhead.

The estimated volume of the landslide is about 10,000–15,000 m^3, assuming a depth of between 3 and 6 m and a rounding factor of about 0.7. It is not known whether the scar represents one landslide event or multiple events over time.

3.1.2 Before the 2010/11 Canterbury earthquakes

- No large-scale slope deformation has been reported since European settlement (ca. 1840), i.e., from the “Paperspast” website.
- Geomorphological expressions of several landslide scars, within the assessment area and on the adjacent slopes, are apparent in the 1940 aerial photographs (the date of the earliest available aerial photographs) (Figure 5). These appear to be relatively old and probably pre-date European settlement (1840 AD). The scars tend to be sub-parallel to one another and are thought to represent topographic control (i.e., gullying) rather than some dominant structural geological control.
- One of these landslide scars is relatively large (estimated volume of the material evacuated from the scar is about 10,000–15,000 m^3).
- There is no evidence in the aerial photographs (1940, 1946, 1973, 1975, 1984 and 2011) of past quarrying at the site. The water reservoir (now disused) is apparent in the 1940 aerial photographs, along with several of the current dwellings.
- The cliff, referred to as Shag Rock Reserve, located on the eastern side of the site is a natural sea cliff, with no evidence of quarrying identified from the aerial photographs. Modification of the cliff may have occurred for the tram and road construction (in the 1900’s).
- Field mapping, carried out after the 2010/11 earthquakes, identified several areas where mature trees were consistently bent, and where pre-2010/11 repairs to dwellings had been made. Both are string indicators of slow ground movement before the 2010/11 earthquakes.
- No cracking was reported or observed following the 4 September 2010 earthquake.

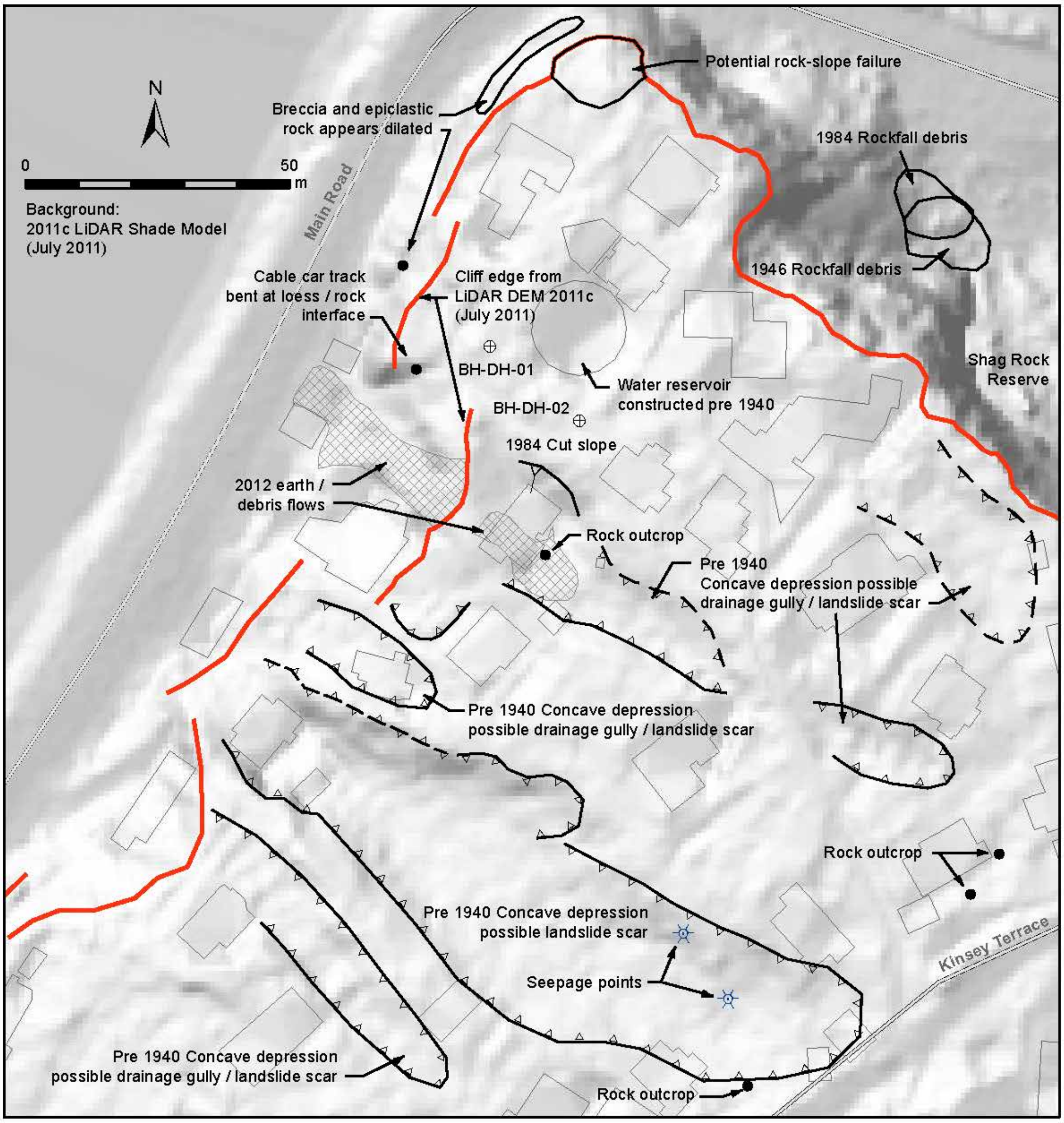


Figure 5 Main features identified at the site from field mapping and the interpretation of historical aerial photographs.

3.1.3 During the 2010/11 Canterbury earthquakes

- *22 February 2011 earthquakes* – the majority of cracks (shown on the maps in the Stage 1 report and displacements summarised in Table 3), were generated on 22 February 2011 by one or more earthquakes that occurred on this day. Permanent displacement of the area, inferred from the results of surveying of cadastral survey marks (by GNS Science, Table 4), was in the order of about 0.3–0.6 m. The cracks patterns suggest the mass moved as a coherent block, as most of the vertical displacements correspond to the identified "head scarp" of the area, with predominantly translational movement in the central and lower parts of the displaced mass. The largest displacements were recorded in the central and lower part of the slope.

- *16 April 2011 earthquake* – Permanent displacement of the area, inferred from the results of surveying of monitoring marks (by Aurecon New Zealand Ltd.), was in the order of about 0.01 m or less, with many of the monitoring marks showing displacements not exceeding the associated errors.
- *13 June 2011 earthquakes* – Permanent displacement of the area in response to the earthquakes on 13 June 2011, inferred from the results of surveying of monitoring marks (by Aurecon New Zealand Ltd.), was in the order of about 0.1 m.
- *23 December 2011 earthquake* – Permanent displacement of the area in response to the earthquake on 23 December 2011, inferred from the results of surveying of monitoring marks (by Aurecon New Zealand Ltd.), was in the order of about 0.01 m or less, with many of the monitoring marks showing displacements not exceeding the associated errors.

3.1.4 After the 2010/11 Canterbury earthquakes

- A small earth/debris flow (less than 50 m^3 in volume) occurred in August 2012 with wet slide debris spilling over the inside lane of Main Road and locally infilling the area behind several of the garages along Main Road with debris.
- A second smaller (less than 50 m^3) debris flow developed above the lower one during the same storm (Figure 5).
- A small (2.2 mm) displacement is recorded at 18.5 m depth in volcanic breccia in inclinometer tube installed in drillhole BH-DH-01 between 12 February and 4 April 2013; the bearing of displacement is towards 177° (slightly upslope to the dip of the slope) (Geotechnics Ltd., 2014). This may be related to deep-seated displacement of the slope through the rock mass.
- The inclinometer in drillhole BH-DH-02 was blocked at a depth of about 4.5–4.75 m below ground level. No baseline survey of this inclinometer was carried out. This may be related to displacement of the loess and volcanic colluvium above rockhead.
- Seepage of water and ponded water were identified in the head scarp of the pre-1940 relict landslide scar, near rockhead.
- The cable car track for 284A Main Road is bent at the interface between the loess/colluvium and underlying rock, suggesting movement of the loess overlying rock.

3.2 SITE INVESTIGATIONS

3.2.1 Geomorphological mapping

The results from field mapping of slope morphology, interpreted surface materials and their genesis, surface deformation mapping and other relevant information are shown in Figure 6.

The site consists of a symmetric north-tending spur, which ends abruptly at the sea in the north, where it forms a steep coastal cliff. The cliff is now abandoned by the sea following construction of Main Road, sometime around the turn of the 20th century. The area is called Shag Rock Reserve, which comprises reclaimed land between the former tram line (now Main Road) and the high cliffs. It is locally referred to as “Peacocks Gallop”. Peacock's father John Jenkins, when he rode by horse from Lyttelton to Sumner via Evans Pass, is said to have always been afraid of falling rocks, so he galloped along the base of the cliff.

Deans Head site is located on the northwest facing flank of the spur immediately adjacent to the step cliff. The cliff on the northern side is about 80 m high, 500 m long with a slope angle ranging from 60° to overhanging, in parts. The Deans Head site can be split into three main sections based on slope geometry and exposed materials: 1) a toe section, adjacent to Main Road, forming a steep (60° to vertical) slope in predominantly rock, which appears modified probably due to construction of Main Road and the water reservoir; 2) a central section, forming a less steep slope (25–30°) formed predominantly in soil, mainly loess; and 3) an upper section near Kinsey Terrace, forming an even gentler slope (10–20°) formed predominantly in rock.

3.2.2 Subsurface trenching and drilling

The ground investigation details are summarised in Table 4 and shown on Figure 6 and Figure 7. Geological logs and equipment installation details are contained in the reports by Aurecon NZ Ltd. (Codd and Revell, 2013; Tonkin and Taylor Ltd., 2012a).

Table 4 Summary of the ground investigations carried out at the site by Aurecon NZ Ltd. (Codd and Revell, 2013) and Tonkin and Taylor Ltd. (Tonkin and Taylor, 2012a).

ID	Data source	Type	Depth (m below ground level)	Instrumentation/ depth (m below ground level)
BH-DH-01	Aurecon NZ Ltd.	Cored hole	25.1	Inclinometer (25.1)
BH-DH-02	Aurecon NZ Ltd.	Cored hole	35.2	Inclinometer (35.2)
BH-KSY-5	Tonkin and Taylor Ltd.	Cored hole	15.0	Inclinometer
BH-KSY-5A	Tonkin and Taylor Ltd.	Open hole	15.0	Standpipe (tip at 3.85
BH-KSY-6	Tonkin and Taylor Ltd.	Open barrel	15.5	None
SC-KSY-4	Tonkin and Taylor Ltd.	Scala penetrometer	5.5	N/A
SC-KSY-5	Tonkin and Taylor Ltd.	Scala penetrometer	5.3	N/A
SC-KSY-12	Tonkin and Taylor Ltd.	Scala penetrometer	4.5	N/A

3.2.3 Surface movement

3.2.3.1 Surveyed slope displacements

The survey monitoring data are presented in Appendix 2, and summarised below. There are two data sets:

1. Cadastral survey marks (details held by Land Information New Zealand), i.e., property boundaries and roads footpaths etc.; and
2. Monitoring survey marks installed by Aurecon NZ Ltd., for Christchurch City Council, to monitor surface displacement.

Both datasets adopt reference control marks that are outside the area of movement, but still within the local area. Therefore, any regional offsets caused by tectonic displacements are removed from the data.

3.2.3.2 Cadastral marks (source: LINZ)

Available cadastral survey marks were measured by GNS Science to detect absolute ground movements spanning the earthquake period from before the 22 February 2011 earthquakes (the pre-earthquake survey dates for each cadastral mark vary) to 30 October 2012, and therefore include any displacement of the survey marks in response to the earthquakes within this time period.

The results of this survey are contained in Appendix 2. Vector displacements indicate permanent ground displacements, within the lower and middle part of the mass movement, that range from 420 to 740 mm, towards bearings 290–310° (west-northwest) (e.g., cadastral mark ID's 14, 15 and 16; Map 2 Appendix 2). In the upper part of the mass movement the magnitude of displacement is less, about 200–290 mm towards bearings 300–320°.

Calculated displacements from the cadastral survey marks are summarised in Table 5.

3.2.3.3 Monitoring marks (source: Aurecon NZ Ltd.)

The displacements calculated using the Aurecon survey data span the time period 19 March 2011–25 June 2013 and there are approximately 30 observations per mark. Note that the dates covered and the numbers of observations vary per survey mark. The marks are only installed on the lower part of the mass movement.

These data include any displacement of the survey marks in response to the earthquakes within the time period, mainly the 16 April, 13 June and 23 December 2011 earthquakes.

From the survey time series relating to each mark it has been possible to determine the magnitudes and bearings of any displacement caused by these earthquakes. The largest total displacements calculated from the monitoring marks (monitoring mark ID's 1, 4, 5, 6, 8 and 9, Maps 3, 4 and 5, Appendix 2), were about:

- 16 April 2011: 0.01–0.02 m towards bearings 290–300°;
- 13 June 2011: 0.2–0.5 m towards bearings 310–320°; and
- 23 December 2011: Movement not outside error.

The given magnitudes of displacements are outside the estimated survey error, which are shown as error ellipses on the maps in Appendix 2. Calculated displacements from the cadastral and monitoring survey marks are summarised in Table 5.

3.2.3.4 Estimating the 22 February 2011 displacements

No reliable monitoring data covers the period either side of the 22 February 2011 earthquakes, which caused virtually all the observed ground damage. However, it is possible to estimate the likely magnitude of the displacement of the main cracked area during the 22 February 2011 earthquake, by subtracting the combined inferred displacement (during the earthquakes on 16 April, 13 June and 23 December 2011) estimated using the monitoring marks (surveyed by Aurecon NZ Ltd.), from the total displacement estimated from the cadastral survey marks, as follows:

Total displacement of the cadastral survey mark 15 (Map 2, Appendix 2) in the middle of the mass movement (for all events) is about 740 mm. The combined total displacement of the nearest monitoring marks (7 and 8, Maps 3–5, Appendix 2) recorded during the 16 April, 13 June and 23 December 2011 earthquakes was 160–220 mm. Therefore the estimated 22 February 2011 displacement in this area would be 740 mm minus 160–220 mm, which is about 520–580 mm (taking into account survey error).

This suggests that the majority of the recorded displacement (derived from the cadastral survey) in the main part of the cracked area can be attributed to the 22 February 2011 earthquake, assuming that any displacement caused by any other events in this period was negligible.

Table 5 Summary of slope displacements inferred from crack apertures and the surveying of cadastral and monitoring marks installed on the slope. Only measurements outside survey error are shown.

Date	Survey type	Lower slope[1]	Middle slope	Upper slope
Pre-22 February 2011– 30 October 2012	Cadastral marks	742 mm (15) 436 mm (16) back tilted pavement	420 mm (14) 718 mm (17) loose peg in ground 698 mm (18)	293 mm (19) 258 mm (20) 200 mm (21)
22 February 2011	Inferred from survey of cadastral and monitoring marks	520–580 mm	No monitoring marks installed	No monitoring marks installed
16 April 2011	Monitoring marks	15 mm (5) 11 mm (6) 18 mm (7) Movement only marginally outside error.	No monitoring marks installed	No monitoring marks installed
13 June 2011	Monitoring marks	47 mm (4) 119 mm (5) 178 mm (6) 201 mm (7) 164 mm (8) 136 mm (9)	No monitoring marks installed	No monitoring marks installed
23 December 2011	Monitoring marks	No movement outside error	No monitoring marks installed	No monitoring marks installed

[1] The survey mark number is given in brackets; the locations are shown in Appendix 2.

3.2.3.5 Inferred slope displacement from crack apertures

Total cumulative displacement of the slope inferred from crack apertures along cross-sections 1–5, in response to the 2010/11 Canterbury earthquakes, is in the order of about 0.2–2.8 m (Table 6).

Table 6 Measured total cumulate crack apertures, which formed mainly during the 22 February, and less so during the 13 June, 2011 earthquakes, measured by GNS Science. Displacements are obtained from field mapping of tension crack apertures along survey lines. Errors are nominally estimated as being ±0.01 m.

Cross-section	Vertical component (mm)	Horizontal component (mm)	Resultant vector		Apparent dip of loess/rock interface from the horizontal (°)
			Magnitude (mm)	Dip (°)	
1 (source area 1)	770	2700 (1760)	2800 (1920)	16 (24)	20
2 (source area 1)	90	1020 (920)	1020 (920)	5 (6)	0
3 (source area 1)	30	240 (150)	240 (150)	6 (9)	13
4 (Cliff face)	230	240 (220)	330 (320)	44 (46)	14
5 (source area 2)	180	350 (350)	390 (390)	27 (27)	20

Values in brackets represent those displacements calculated using only those components with both horizontal and vertical measurements only.

The vectors of displacement (direction and angle of movement from the horizontal, inferred from crack apertures) for cross-sections 1, 3, 4 and 5 , are generally sub-parallel to the dip of the loess/colluvium and rock interface, suggesting displacement of the mass occurred along this interface.

The vectors of displacement for cross-section 1 indicate a steeper angle in the inferred headscarp of the main area of cracking and movement, and a lower angle in central part of the area, consistent with the dip of rockhead. The exception is cross-sections 2 and 4, where the dip of the resultant vectors further back from the cliff crest are close to that of the loess/colluvium and rock interface, but closer to the cliff edge the vectors become significantly steeper than the dip of the loess/colluvium and rock interface. It is thought that this movement relates to deeper-seated displacement of the rock mass.

The amount of total permanent slope displacement inferred from the survey marks is about half the amount inferred from crack apertures. This is thought to be because the amount of displacement inferred from crack apertures has been accumulated along the cross-sections, taking no account of any compression. Very few compression features were mapped in the area, as such features are very difficult to identify in the field, especially in vegetation. Therefore the displacements inferred from crack apertures are thought to be upper bound estimates.

3.2.4 Subsurface movement

Drillhole inclinometer tubes were used to monitor displacements at depth, assess whether movement was occurring along single or multiple slide-surfaces, and to independently verify the results of surface monitoring. Monitoring is undertaken manually by commercial contract (Geotechnics Ltd.).

Inclinometer tubes were installed in drillholes BH-DH-01, BH-DH-02 (Codd and Revell, 2013) and BH-KSY5 (Tonkin and Taylor, 2012a). The inclinometer displacements are monitored at 0.5 m intervals and the inclinometer accuracy is quoted as ±6 mm over 25 m of tubing (Slope Indicator, 2005). The measurement details are summarised in Table 7.

The inclinometer installed in drillhole BH-DH-01, has a deflection in the monitoring tube between the 18.25 and 18.75 m (below the collar elevation) intervals. The deflection is relatively small, about 2.2 mm towards bearing 177° in the tilt change plot, with a cumulative displacement of about 3 mm in the profile change plot (Geotechnics Ltd., 2014). The deflection has been recorded in multiple surveys, and is outside the error associated with the surveys. The deflection occurred between 12 February and 4 April 2013 inclinometer surveys (Geotechnics Ltd., 2014). Subsequent inclinometer readings on 10 July 2013 and 13 March 2014 however, show smaller tilt changes. These readings are only marginally in excess of the associated survey error. The bearing of displacement indicates movement is slightly upslope. Given the magnitude and bearing of displacement, it is not known whether this relates to displacement of the slope through rock, or to displacement of the inclinometer tube within the drillhole, unrelated to slope displacement. Further monitoring is required to resolve this issue.

The inclinometer in drillhole BH-DH-02 was blocked at a depth of about 4.5–4.75 m below ground level. No baseline survey of this inclinometer was carried out. The blockage occurred sometime between 18 January 2013 and 12 February 2013 and may be the result of shearing of the inclinometer tube in response to slope displacement. Survey results from the monitoring marks installed in this area however, show no displacement outside the associated error, for this period. It is therefore possible that the inclinometer may have been incorrectly installed. The depth of blockage corresponds to the depth of the logged volcanic colluvium layer.

The profile change plots from the inclinometer in drillhole BH-KSY-5 show no displacement outside of the error for the monitoring period (Tonkin and Taylor, 2012a). The profile change plots show a cumulative drift in the displacement from the bottom to the top of the inclinometer tube. This is thought to be the result of accumulating the errors between each surveyed interval from the bottom of the tube. No tilt change plots are reported by Tonkin and Taylor (2012a). No data for this inclinometer since 23 December 2011 have been supplied to GNS Science.

Table 7 Summary of drillhole inclinometer surveys.

Measuring date	Drillhole ID		
	BH-DH-01[1]	BH-DH-02[1]	BH-KSY-5
15/07/2011			Baseline
9/09/2011			No movement outside error
15/09/2011			No movement outside error
12/10/2011			No movement outside error
21/10/2011			No movement outside error
23/12/2011			No movement outside error
12/02/2013	Base reading	No base reading tube blocked at 4.25 m below ground level	No data
4/04/2013	Deflection in tube between 18.25 and 18.75 m below ground level	No data	No data
10/07/2013	Deflection in tube between 18.25 and 18.75 m below ground level	No data	No data
13/03/2014	Deflection in tube between 18.25 and 18.75 m below ground level	No data	No data

1 Geotechnics Ltd Report 720085.000/RPT (Geotechnics Ltd., 2014).

3.2.5 Groundwater

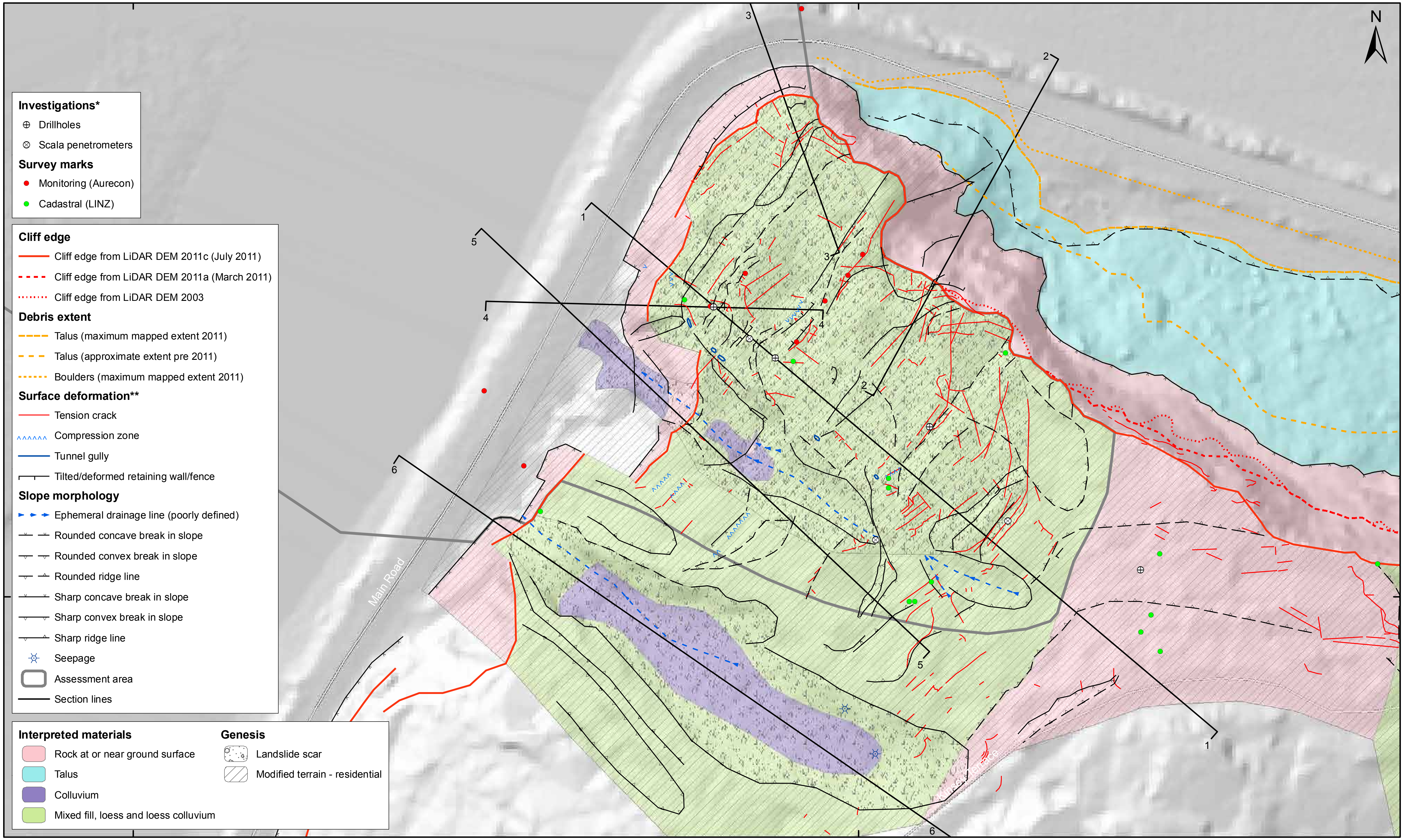
Drill water circulation conditions are not reported in drilling records (Codd and Revell, 2013). There is only one standpipe installed in the assessment area. This was installed by Tonkin and Taylor Ltd. in drillhole BH-KSY-5A. The tip of the standpipe is 3.0 m below ground level and is within loess. The response zone for the standpipe varies between two and five metres below ground level. However, given that the standpipe tip is at three metres below ground level, any groundwater levels below three metres would not be recorded.

Monitoring data from this standpipe comprised the manual measurement of water levels in the standpipe. Approximately 12 measurements or less were made over the reporting period 3 August 2011–17 July 2012 (Tonkin and Taylor, 2012a), indicating a poor temporal resolution. No more recent data have been provided to GNS Science. It is possible that groundwater is present in the piezometers, but that the poor temporal resolution and relatively high level of the standpipe tip (relative to the base of the loess and volcanic colluvium) in the drillhole have not allowed them to be resolved.

3.3 ENGINEERING GEOLOGICAL MODEL

An engineering geological map is presented in Figure 6, site investigation map in Figure 7 and cross-sections (1–5) in Figure 8. The map and cross-sections are based on the interpretation of features identified in aerial photographs, field mapping and ground investigation data, as summarised in Table 5.

Based on this work the main slope forming materials and groundwater conditions are summarised below.



SCALE BAR: 0 50 100 m

EXPLANATION:
* For details refer to site investigation map
** Taken from report 2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGY MAP

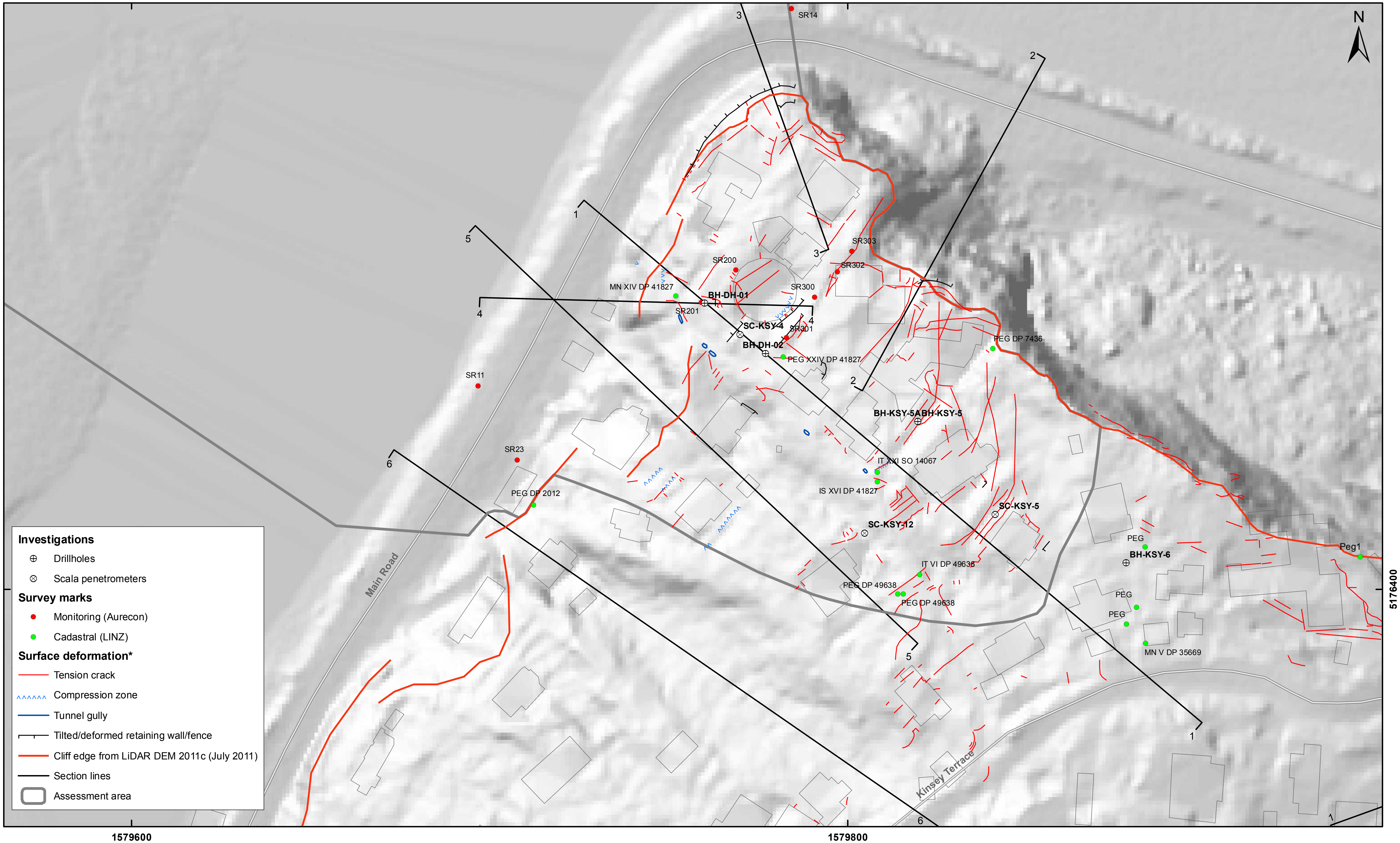
Deans Head Christchurch

FIGURE 6

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report 2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

SITE INVESTIGATION MAP

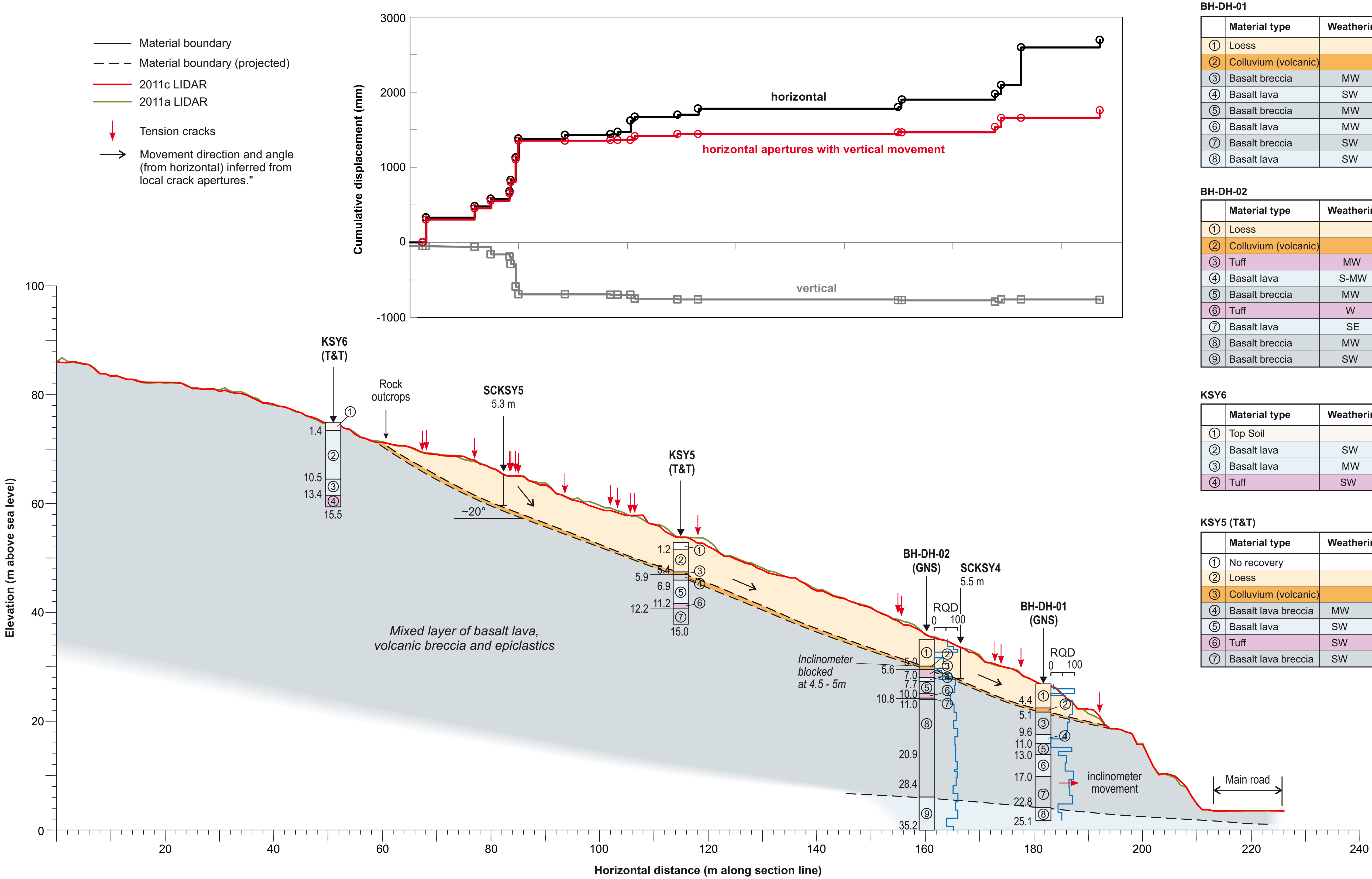
Deans Head Christchurch

FIGURE 7

FINAL

REPORT: CR2014/77

DATE: June 2014



BH-DH-01

	Material type	Weathering	Strength	RQD %
1	Loess			
2	Colluvium (volcanic)			
3	Basalt breccia	MW	W	70
4	Basalt lava	SW	MS	0
5	Basalt breccia	MW	W-MS	85
6	Basalt lava	MW	W-MS	60
7	Basalt breccia	SW	MW	80
8	Basalt lava	SW	S	

BH-DH-02

	Material type	Weathering	Strength	RQD %
1	Loess			
2	Colluvium (volcanic)			
3	Tuff	MW	W	30-72
4	Basalt lava	S-MW	S	20
5	Basalt breccia	MW	W-MS	86
6	Tuff	W	MS	100
7	Basalt lava	SE	S	
8	Basalt breccia	MW	W	80
9	Basalt breccia	SW	MW	85

KSY6

	Material type	Weathering	Strength	RQD %
1	Top Soil			
2	Basalt lava	SW	EW	
3	Basalt lava	MW	MS	
4	Tuff	SW	W	

KSY5 (T&T)

	Material type	Weathering	Strength	RQD %
1	No recovery			
2	Loess			
3	Colluvium (volcanic)			
4	Basalt lava breccia	MW	EW	
5	Basalt lava	SW	S	
6	Tuff	SW	W	
7	Basalt lava breccia	SW	S to MS	

Weathering (adopting NZGS (2005) terminology): CW completely weathered; HW highly weathered; MW moderately weathered; SW slightly weathered; UW unweathered.
Rock Strength (field strengths adopting NZGS (2005) terminology): EW extremely weak; VW very weak; W weak; MS moderately strong; S Strong; VS very strong; extremely strong.
Soil strength (field strengths adopting NZGS (2005) terminology): Coarse soils – VL very loose; L loose; MD medium dense; D dense, VD very dense. Cohesive soils – H hard; VSt very stiff; St stiff; F firm; So soft; VSo very soft.
RQD: Rock quality designation

DRW: PC

CHK: CM

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGY CROSS SECTION 1

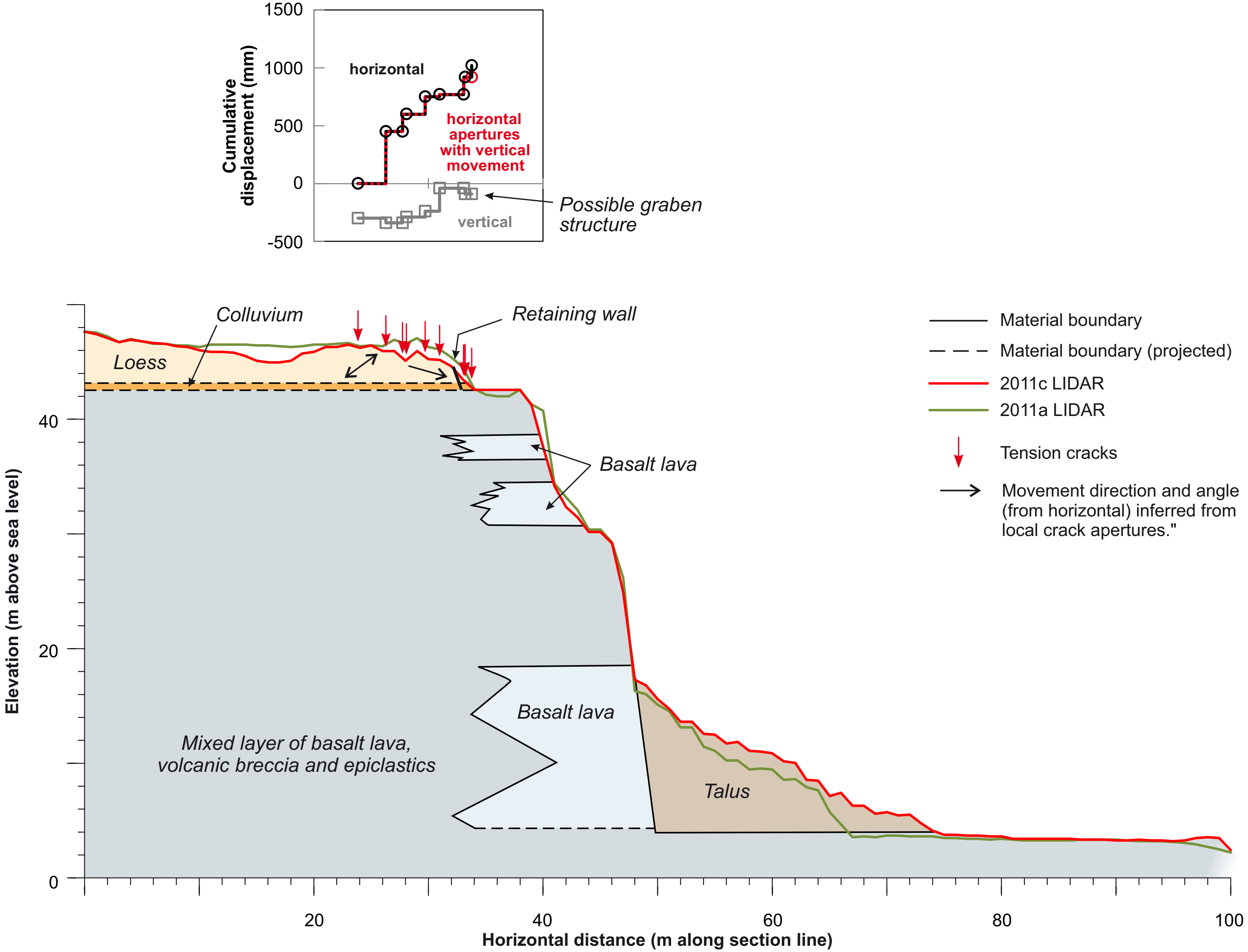
Deans Head
Christchurch

Figure 8

FINAL

REPORT: CR2014/77

DATE: June 2014



DRW: PC

CHK:

ENGINEERING GEOLOGY CROSS SECTION 2

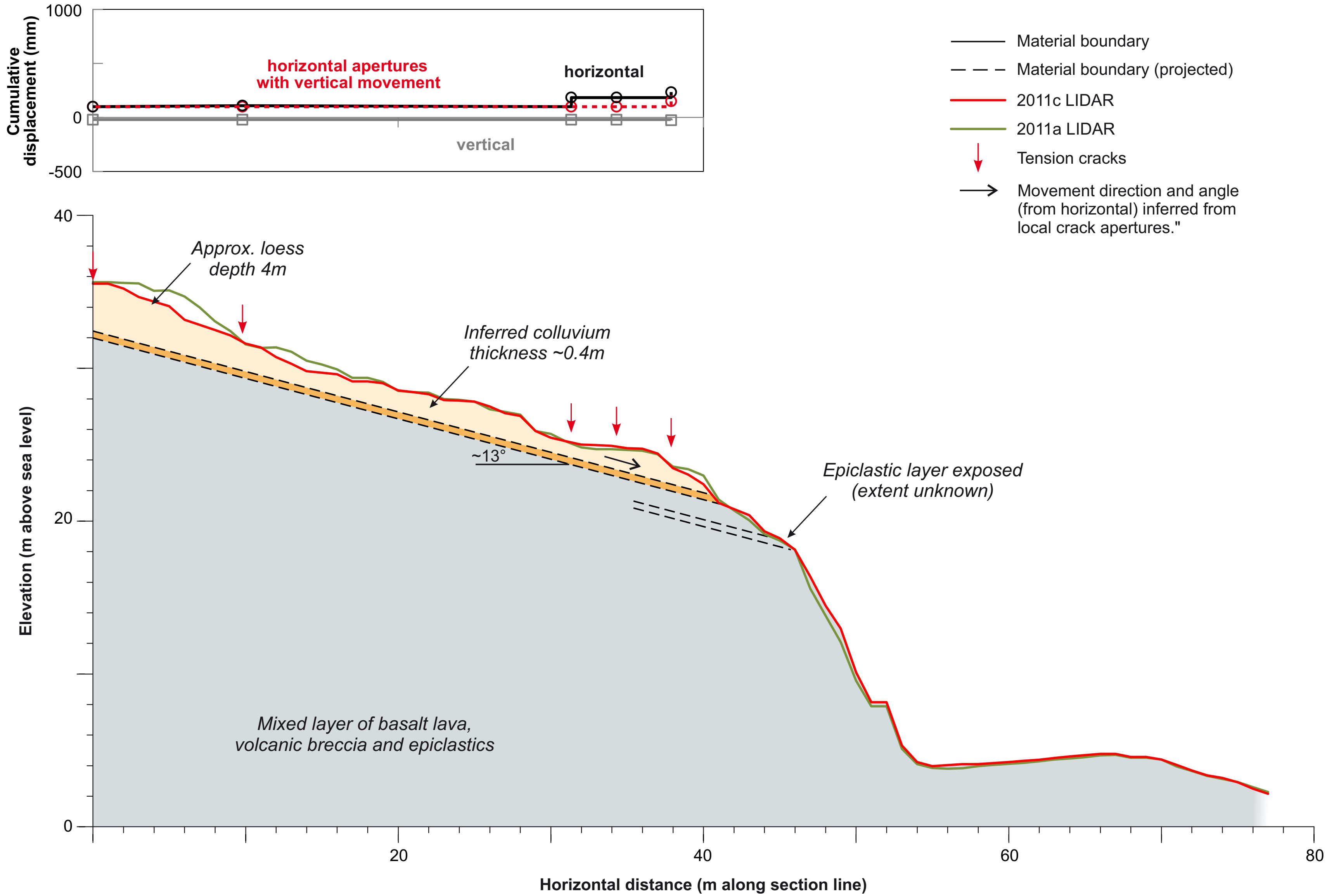
Deans Head Christchurch

FIGURE 8

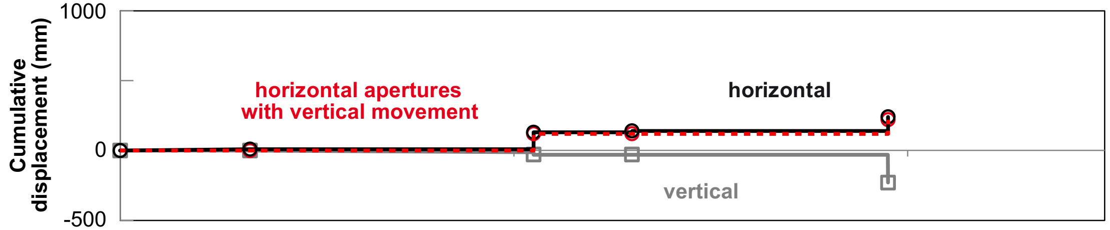
FINAL

REPORT: CR2014/77

DATE: June 2014

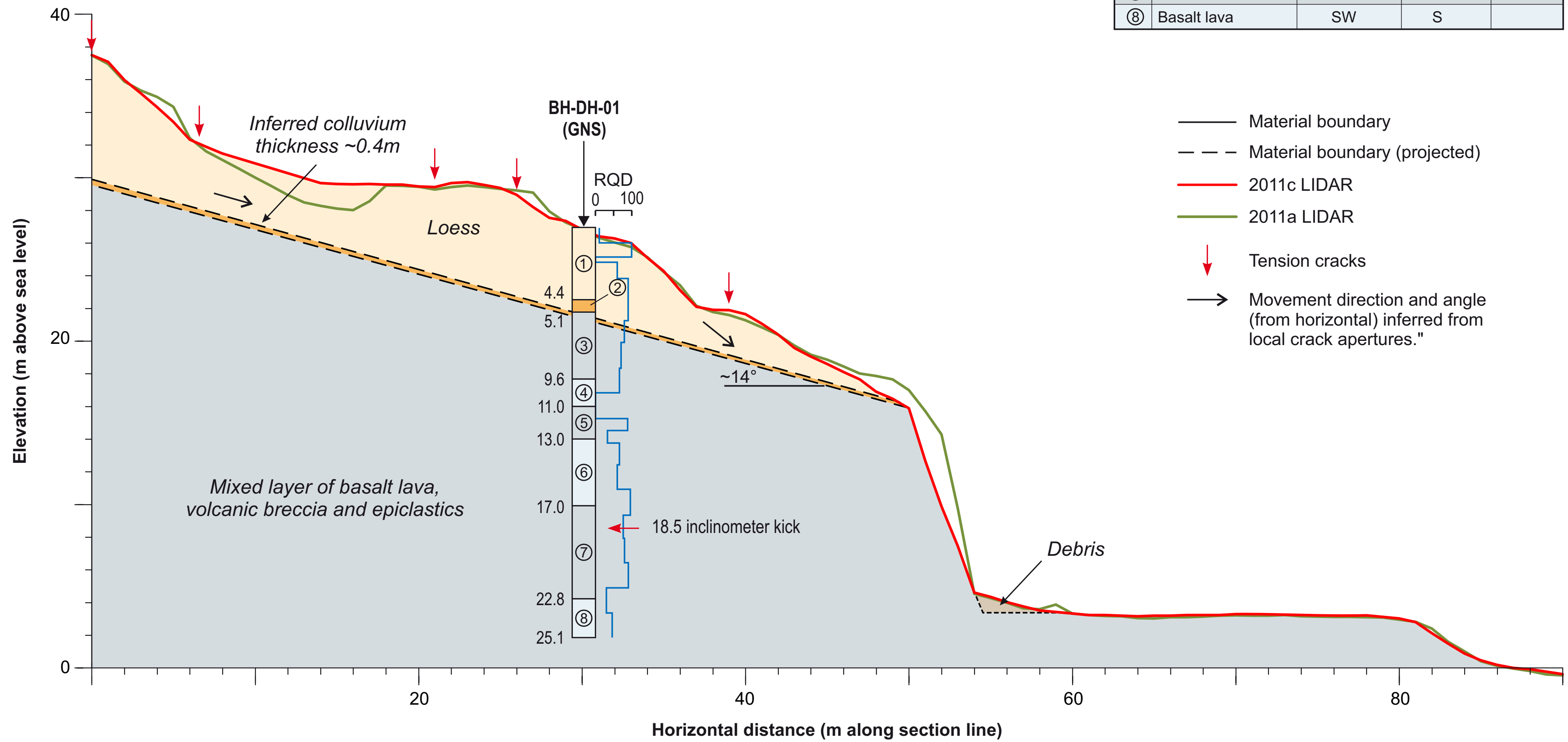


DRW: PC		**ENGINEERING GEOLOGY CROSS SECTION 3**	**FIGURE 8**	
CHK:	GNS SCIENCE TE PŪ AO	**Deans Head Christchurch**	**FINAL**	
			REPORT: CR2014/77	DATE: June 2014



BH-DH-01

	Material type	Weathering	Strength	RQD %
1	Loess			
2	Colluvium (volcanic)			
3	Basalt breccia	MW	W	70
4	Basalt lava	SW	MS	0
5	Basalt breccia	MW	W-MS	85
6	Basalt lava	MW	W-MS	60
7	Basalt breccia	SW	MW	80
8	Basalt lava	SW	S	



Weathering (adopting NZGS (2005) terminology): CW completely weathered; HW highly weathered; MW moderately weathered; SW slightly weathered; UW unweathered.
Rock Strength (field strengths adopting NZGS (2005) terminology): EW extremely weak; VW very weak; W weak; MS moderately strong; S Strong; VS very strong; extremely strong.
Soil strength (field strengths adopting NZGS (2005) terminology): Coarse soils – VL very loose; L loose; MD medium dense; D dense, VD very dense. Cohesive soils – H hard; VSt very stiff; St stiff; F firm; So soft; VSo very soft.
RQD: Rock quality designation

DRW: PC

CHK:

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGY CROSS SECTION 4

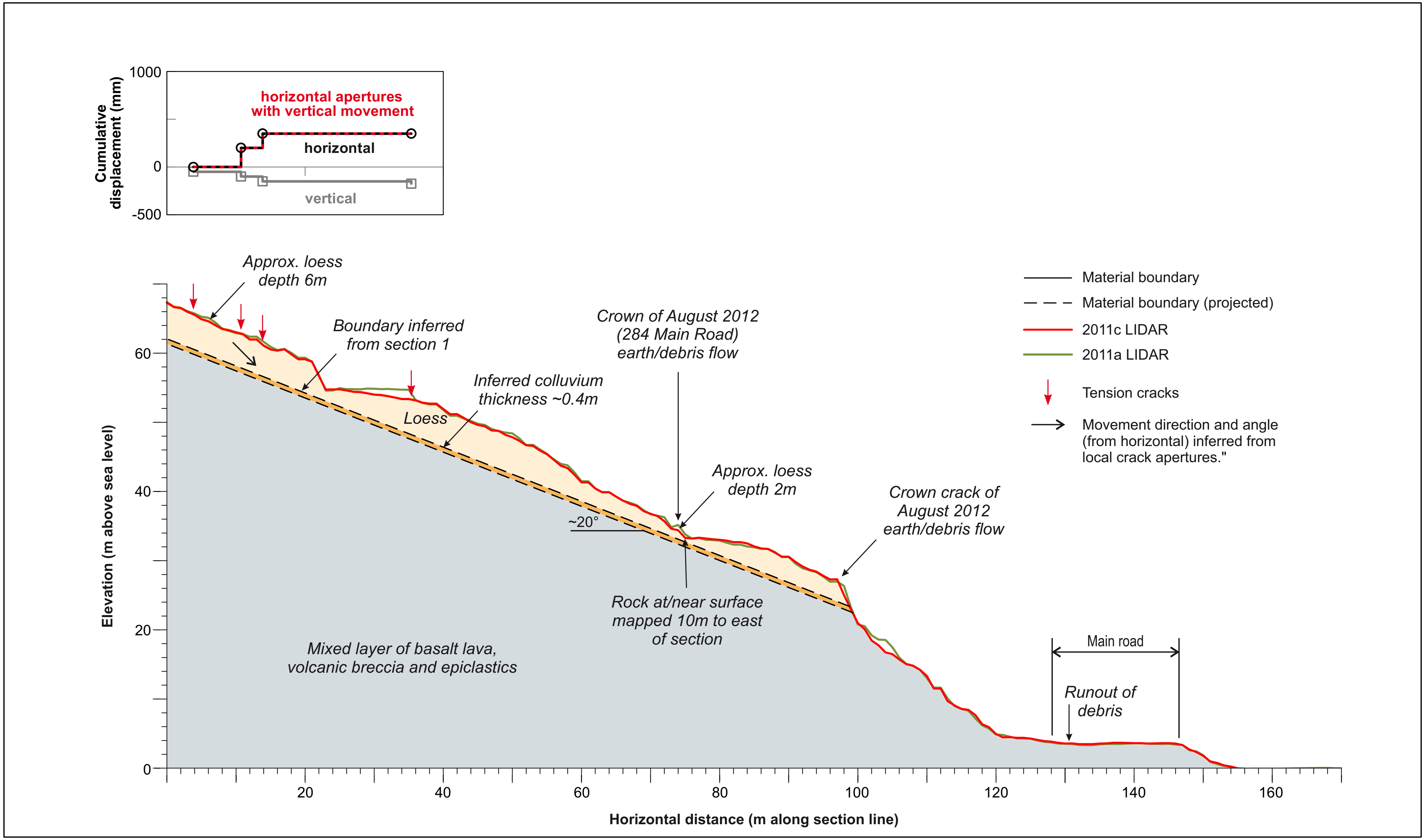
Deans Head Christchurch

FIGURE 8

FINAL

REPORT: CR2014/77

DATE: June 2014



DRW: PC

CHK:

ENGINEERING GEOLOGY CROSS SECTION 5

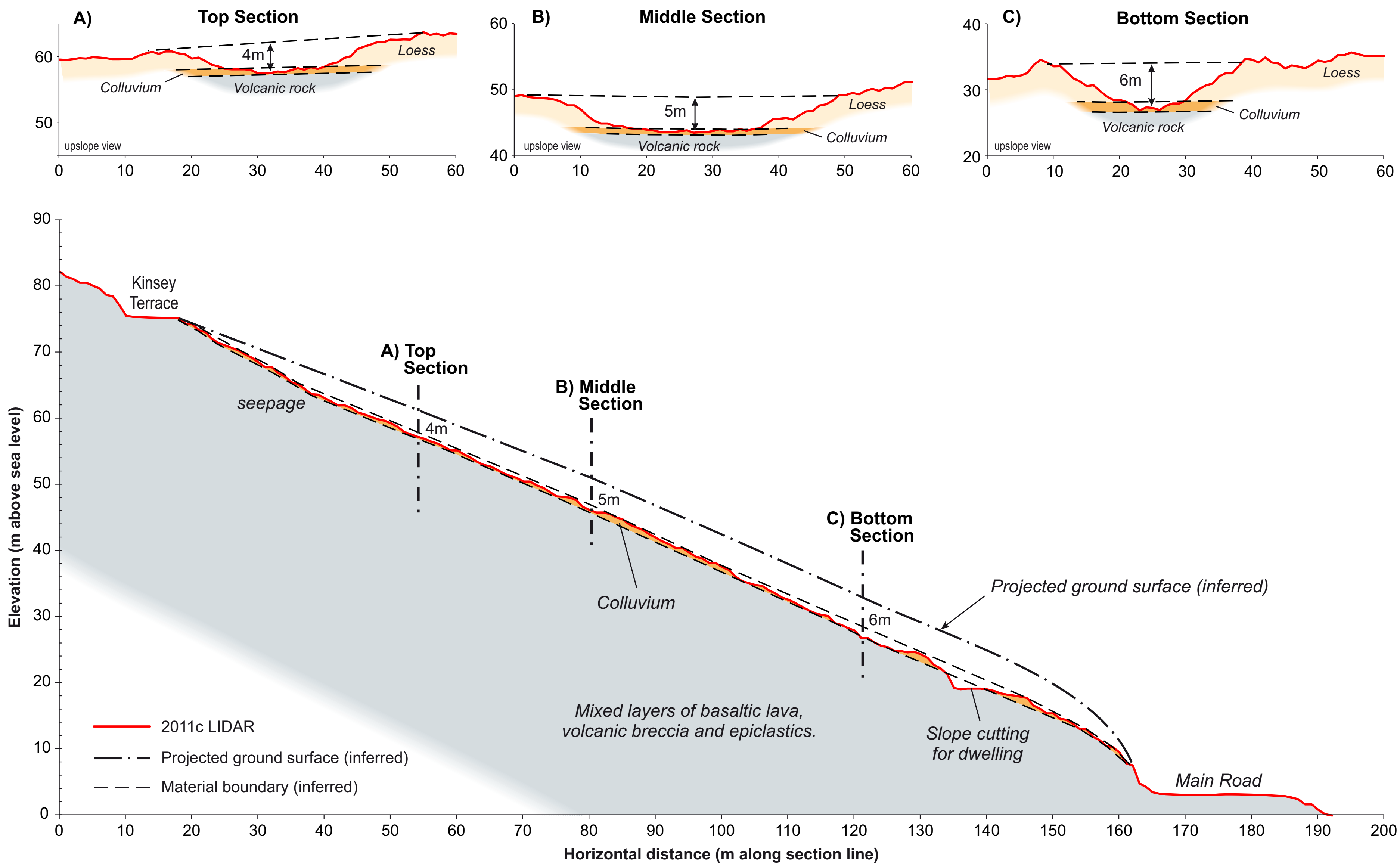
Deans Head Christchurch

FIGURE 8

FINAL

REPORT: CR2014/77

DATE: June 2014



1) Projected ground surface in inferred by projecting ground surface from section A), B) and C)

2) Thickness of material is inferred from field mapping.

DRW: PC

CHK: FDP/CM

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGICAL CROSS SECTION 6
Inferred Loess Paleo-surface Elevation

Deans Head Christchurch

FIGURE 8

FINAL

REPORT: CR2014/77

DATE: June 2014

3.3.1 Slope materials

3.3.1.1 Fill

Modified terrain with localised areas of fill relating to the construction of residential homes can be found over much of the site. The depth and extent of fill in these areas are unknown, although the inferred boundaries of the fill are shown on cross-sections in Figure 8 (cross-section 2). The fill, where encountered in drillholes, is described as soft and relatively weak silt with occasional clasts of basalt and concrete. The thickness of the fill is unknown, but it is estimated to be up to several metres in places.

3.3.1.2 Loess

The loess mantling the slope within the assessment area is similar to other areas of the Port Hills. It is a relatively cohesive silt-dominated soil with only minor clay mineral content. Its strength is largely controlled by the soil moisture content and this has been well studied, e.g., Bell et al. (1986), Bell and Trangmar (1987), McDowell (1989), Goldwater, (1990), Yetton (1992) and Carey et al. (2014). In some places, the loess appears to have been reworked by construction activities for the residential dwellings. At the toe of the slope, the loess forms recessive slopes of varying angle, above the underlying volcanic breccia. The loess is highly hydroscopic and when exposed to water (rain) it quickly disintegrates into muddy silt. The thickness of the loess, in drillholes and from field mapping of exposures, varies in thickness from a few metres to about 5 m in this area.

3.3.1.3 Volcanic colluvium

A layer, of sandy silt containing boulders and gravel with minor clay was logged in drillholes BH-DH-01, BH-DH-02 and BH-KSY-5. Codd and Revell (2013) describe this material as highly variable and dominated by either silts or gravel and cobbles. The thickness of the colluvium varies from about 1.0 m in the lower part of the site to less than 0.3 m in the upper part of the site.

Given that all drillholes encountered this material, it has been assumed that volcanic colluvium mantles rockhead and underlies the loess over most of the site. Where exposed in outcrop, the colluvium appears to have slightly higher clay content than those materials described in the drillhole logs. It is thought to represent the deposits of debris from past landslides and other erosion processes. The material derives mainly from weathered volcanic breccia and lava and remobilised loess. In drillholes and field exposures, the colluvium is highly variable. It ranges from gravel to boulder-sized clasts of volcanic basalt with a loess and clay matrix, to remoulded loess with occasional gravel and boulders.

3.3.1.4 Bedrock (volcanic basalt lava breccia and lava)

The Deans Head slopes are underlain by mixed layers of weak volcanic basalt breccia, stronger, but more jointed basalt lava flow sequences and occasional thin layers of epiclastic sediments (mainly sandstones and conglomerates) and tuffs. The lava flow sequences and epiclastics are laterally and vertically discontinuous. Drilling records from drillholes BH-DH-01 and BH-DH-02 (along cross-section 1) describe the underlying volcanic material as weak to moderately strong. Rock quality designation logs from drillhole BH-DH-02 range between 50 and 100% and are markedly better compared to those logs from BH-DH-01 situated further downslope and closer to the cliff edge, with rock quality designation values 30–90% and core loss intervals of up to 1.8 m in length (Codd and Revell, 2013).

The discontinuities described in the drillhole logs vary from clean to clay-infilled and from planar to rough. A clay weathered joint at 18.4 m above mean sea level (drillhole BH-DH-01) appears to coincide with a small deflection in the inclinometer tube of about 2 mm between 18.25 and 18.75 m below ground level.

Field mapping of the cliff face along Shag Rock Reserve was carried out by GNS Science and is reported by Massey et al. (2012). In general the materials exposed in the cliff face immediately to the east of the Deans Head site, are high discontinuous, with no obvious through going persistent material boundaries of discontinuities apparent, which would facilitate a structurally controlled failure of the slope. Cliff collapses from the rock slope occurred during the 2010/11 Canterbury earthquakes, but these tended to be randomly distributed failures of the predominantly basalt lava breccia. These are discussed in more detail in Massey et al. 2012).

3.3.1.5 Volcanic colluvium/loess and rock boundary

The surface boundary between the base of the volcanic colluvium and loess and the underlying volcanic rock (rockhead) was interpolated by GNS Science using: 1) drillhole intersections; 2) scala penetrometer test depths of refusal; and 3) field mapped rock outcrop exposures, as control points. There is not enough data to accurately represent the shape of the rockhead surface over much of the site, other than along cross-sections 1–5 (Figure 8). In general, rockhead shows an overall dip towards the west-northwest of about 20°, which is approximately coincident with the slope aspect. It should be noted that there is an apparent sub-parallel trend between the overall strike of cracks and the strike of rockhead.

3.3.2 Geotechnical properties

Material strength parameters have been assigned based on the results from in-house (GNS Science) laboratory tests and the published results of testing of similar materials from elsewhere in the Port Hills.

3.3.2.1 Loess

Material parameters adopted for the loess material in the assessment area are shown in Table 8. These are based on: 1) descriptions of the drillcore materials; 2) Port Hills soil strength test results reported by Carey et al. (2014) and other published data; and 3) numerical slope stability back-analysis.

***In situ* water contents**

A measure of the *in situ* water content (*in situ* meaning the water content of the sample as it was at the time of sampling, and before any testing was carried out) of loess in the slope was derived from *in situ* "block" samples collected from Maffeys Road, Lucas Lane and Vernon Terrace.

The *in situ* water content of the loess block samples varies mostly between 6 and 10%, with two samples in the 3–5% range (Carey et al., 2014). The samples used for testing were all taken from free-draining slopes exposed to the weather, and were sampled between January and February 2013, and January and February 2014, near the end of summer. The *in situ* water contents are therefore thought to represent the lower end of the range (Figure 9). The samples were taken from an east-facing slope formed in loess. Even if the samples were collected in winter, the water contents of the loess at this accessible site would still not be representative of the water content of the loess deeper in the slope, as the outside face of the slope is free draining.

The water contents of the loess in drillhole samples were all substantially higher than those for the block samples. The difference may reflect the sampling method, where drilling includes using water as a flush, and block sampling does not (Table 7).

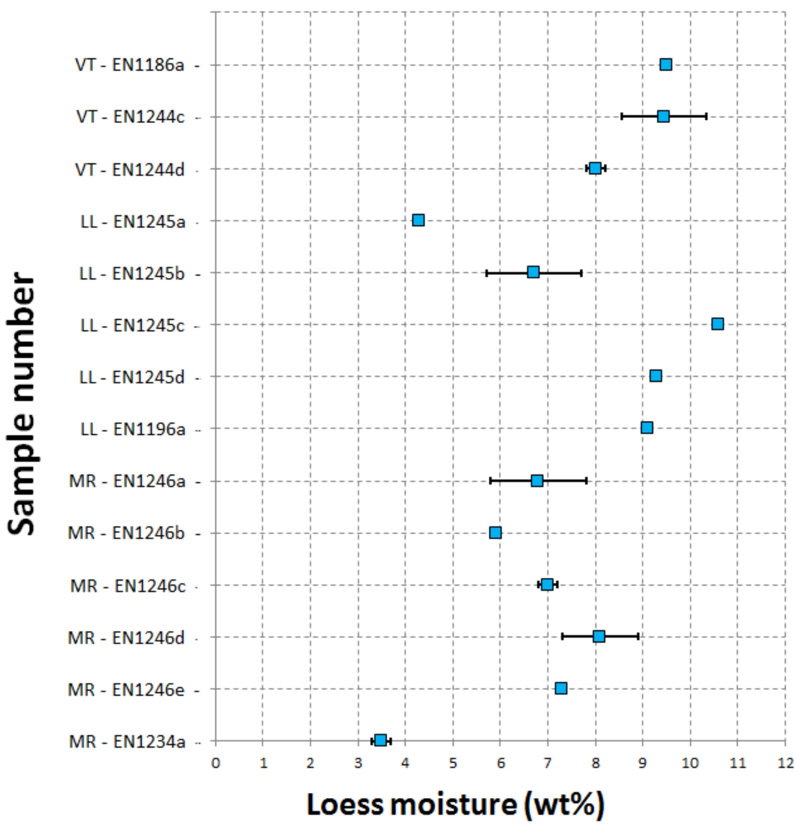


Figure 9 In-ground moisture (water, wt%) content of collected loess samples.

In-house shear strength tests

The shear strength of the loess was tested in-house at GNS Science using two types of ring shear equipment and one type of direct shear equipment (Carey et al., 2014). The results are summarised in Table 8 and Table 9 and plotted in Figure 10. The results show a wide variability in the tested friction and cohesion values. Where shear box tests indicated peak and residual strength characteristics, both the peak and residual friction and cohesion values have been plotted with "tie" lines joining the data points together.

With the exception of sample EN1243, all tests were carried out in saturated (water-added) conditions (at final post-test water contents of between 16 and 19%). As a consequence, these water contents are higher than those from the tested *in situ* samples. The water contents from the *in situ* samples are thought to better represent the bulk moisture content of the loess in the actual slope. Stability assessment results suggest that the slope would be susceptible to failure if shear strength values representing these water contents were adopted.

A shear box test on loess sample EN1243 was carried out without water added (i.e., non-standard testing procedure) at ~3.7% water content, to explore the effect of moisture content on shear strength. The test yielded residual value shear strength values of cohesion (c) = 42 kPa and friction (ϕ) = 48°, with peak shear strength values of c = 230 kPa and ϕ = 72°. This contrasts with the ring shear tests results undertaken for saturated soils, which yield residual shear strengths of c = 0–6 kPa and ϕ = 27–37°.

The shear strength results in Table 8 are considered to be more representative of the bulk residual strength parameters for the loess slope rather than peak strength parameters.

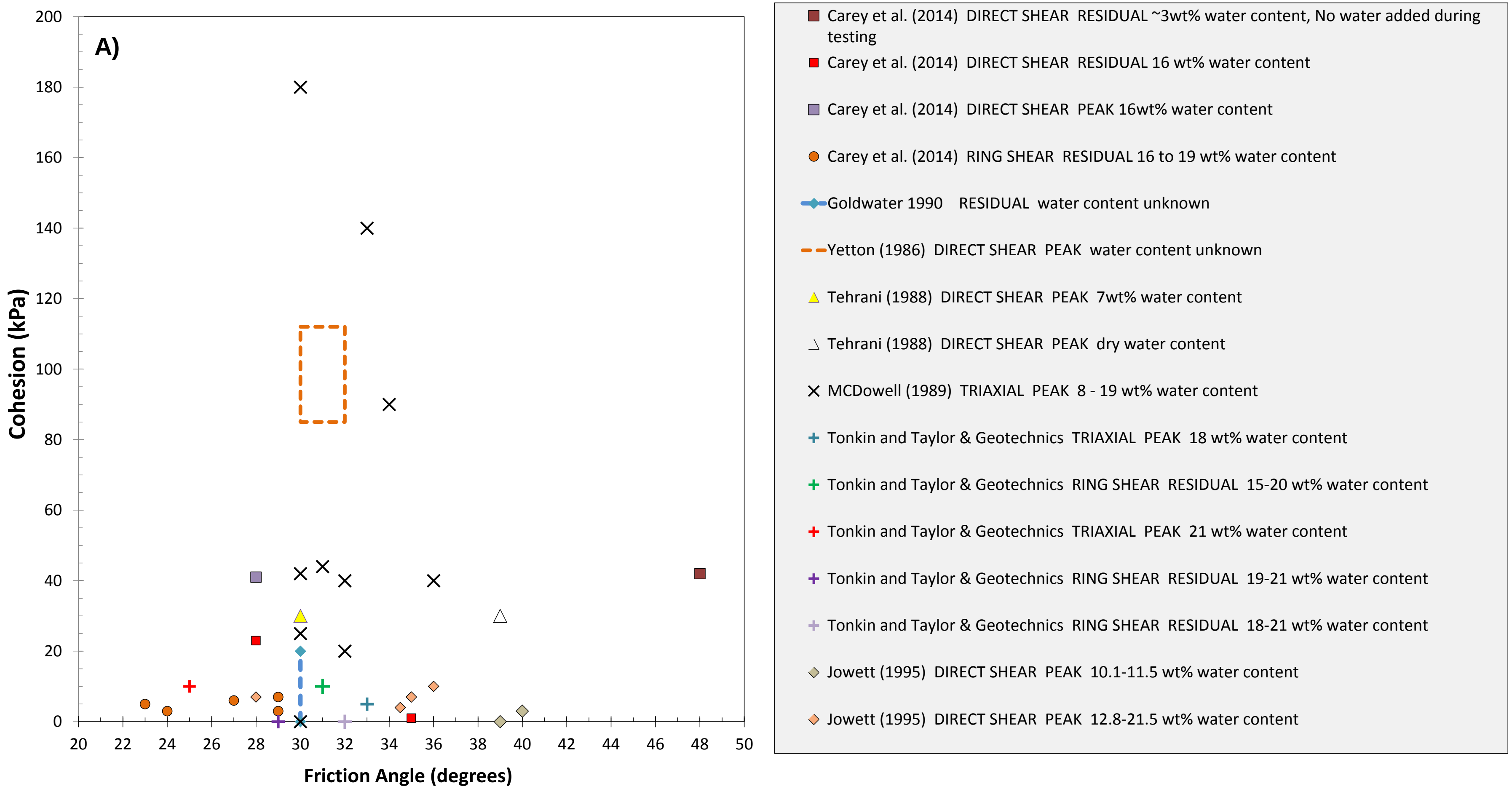
Effect of moisture content on loess shear strength

Comparison can be made with shear strength results from other published Port Hills investigations (Table 9) by plotting them alongside the results of the GNS Science testing (Figure 10).

The sensitivities of the friction angle (ϕ) and cohesion (c) to change in moisture content have been assessed using both GNS Science testing results (Carey et al., 2014) and results from tests by McDowell (1989), Tehrani (1988) and Tonkin and Taylor (2012a). The results show that, over the interval from 10 to 20 wt% moisture the loess friction angle is less sensitive than the cohesion to changes in water content.

For water contents between 10 and 20 wt%, the cohesion of the tested loess is very sensitive to changes in water content. These results illustrate a large variability in the strength parameters of the loess in the Port Hills, and that the complex effects of the water content may be critical to the loess strength. These results are consistent with the findings of others (e.g., McDowell, 1989; Goldwater, 1990).

The block samples of loess were all taken at the end of periods of dry weather (summer), where water contents were between 3.5 and 11 wt%, and therefore the shear strength of loess would likely be at the upper end of the range. During periods of prolonged wet weather it is feasible for water contents in the loess to increase leading to a reduction in the cohesion and increased susceptibility to failure. The data plotted in Figure 10 probably represent the range of strength parameters at the likely range of moisture contents that could be anticipated in the Port Hills loess.


A)
Cohesion (kPa)
Friction Angle (degrees)
0
20
40
60
80
100
120
140
160
180
200
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Carey et al. (2014) DIRECT SHEAR RESIDUAL ~3wt% water content, No water added during testing
Carey et al. (2014) DIRECT SHEAR RESIDUAL 16 wt% water content
Carey et al. (2014) DIRECT SHEAR PEAK 16wt% water content
Carey et al. (2014) RING SHEAR RESIDUAL 16 to 19 wt% water content
Goldwater 1990 RESIDUAL water content unknown
Yetton (1986) DIRECT SHEAR PEAK water content unknown
Tehrani (1988) DIRECT SHEAR PEAK 7wt% water content
Tehrani (1988) DIRECT SHEAR PEAK dry water content
MCDowell (1989) TRIAXIAL PEAK 8 - 19 wt% water content
Tonkin and Taylor & Geotechnics TRIAXIAL PEAK 18 wt% water content
Tonkin and Taylor & Geotechnics RING SHEAR RESIDUAL 15-20 wt% water content
Tonkin and Taylor & Geotechnics TRIAXIAL PEAK 21 wt% water content
Tonkin and Taylor & Geotechnics RING SHEAR RESIDUAL 19-21 wt% water content
Tonkin and Taylor & Geotechnics RING SHEAR RESIDUAL 18-21 wt% water content
Jowett (1995) DIRECT SHEAR PEAK 10.1-11.5 wt% water content
Jowett (1995) DIRECT SHEAR PEAK 12.8-21.5 wt% water content

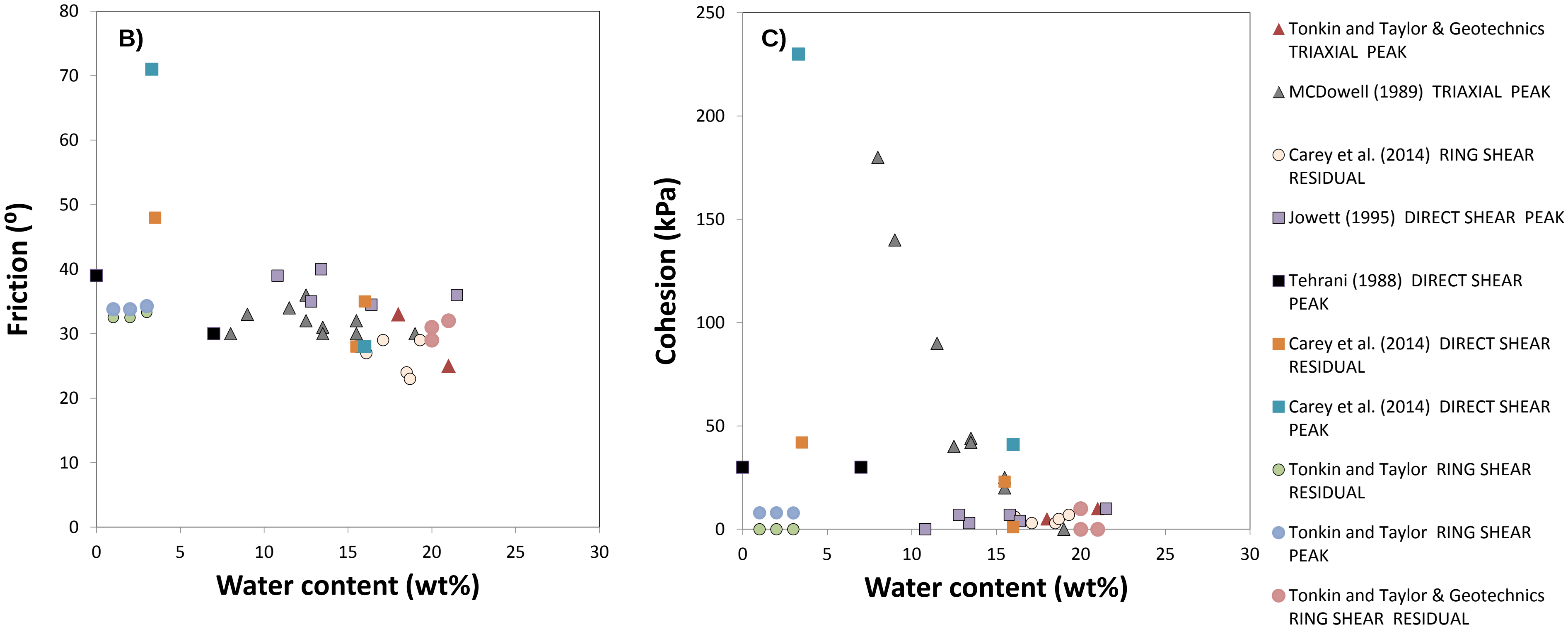


Figure 10 Loess residual shear strength results (from Table 8 and Table 9). A) Cohesion and friction laboratory results plotted for loess. B) Loess residual cohesion plotted against water content. C) Loess residual friction plotted against water content.

Table 8 Shear strength test results (from Carey et al., 2014).

Site	Sample	Sample *in situ* water content	Test type	Sampling method	Test starting water content[1] (%)	Test final water content (%)	Dry density	Peak cohesion c (kPa)	Peak friction ϕ	Residual cohesion C (kPa)	Residual friction ϕ	Lab test Number
Lucas Lane	EN1186	n/a	Ring Shear-C	Drillcore	19.8	18.7				3	24	EN1186b
			Ring Shear-C	Drillcore	19.8	18.7				5	23	EN1186d
			Shear Box	Drillcore	13.7	15.5	1.41	41	28	23	28	EN1186a
					13.7	13.7	1.45					EN1186c
Maffeys Road	EN1195	n/a	Ring Shear-C	Block Sample	?	16.1				6	27	EN1195b
			Ring Shear-G	Block Sample	?	17.9				0	37	EN1195c
Richmond Hill	EN1196	n/a	Ring Shear-C	Drillcore	18.1	17.1				3	29	EN1196b
			Ring Shear-C	Drillcore	17.18	19.3				7	29	EN1196f
			Ring Shear-G	Drillcore	18.1	18.6				6	31	EN1196c
			Ring Shear-G	Drillcore	17.1	16.6				15	35	EN1196e
			Shear Box	Drillcore	16.1	16	134	1	35	1	35	EN1196a
					16.1	13.9	1.32					EN1196d
Deans Head	EN1230	n/a	Ring Shear-G	Drillcore	17.1	17.9				20	35	EN1230b
Maffeys Road[2]	EN1243	n/a	Shear Box	Block Sample		3.3	1.37	230	71	42	48	EN1243a
						3.7	1.36					EN1243b

[1] This is unrelated to the original sample water content as it has had water added as part of the lab test procedure.

[2] This test was carried out under dry conditions with no added water, and therefore follows a non-standard testing procedure.

Table 9 Other published shear tests on loess in the Port Hills.

Area	Friction ϕ (°)	Cohesion c (kPa)	Water content (%wt)	Data source
Clifton Terrace (peak)	25–33	5–10	18–21	Tonkin and Taylor (2012a) for EQC
Clifton Terrace (residual)	31–32	0–5	15–20	
Vernon Terrace	29	0	19–21	Tonkin and Taylor (2012c) for EQC
Maffeys Road (peak)	34	8	No data	Tonkin and Taylor (2012d) for EQC
Maffeys Road (residual)	33	0	No data	
Defender Lane (peak)	34	8	No data	Tonkin and Taylor (2012b) for EQC
Defender Lane (residual)	33	0	No data	
Glendevere Terrace(peak)	34	8	No data	Tonkin and Taylor (2012e) for EQC
Glendevere Terrace (residual)	33	0	No data	
Port Hills	30–35	85–112	No data	Yetton (1992)
Not known	30–39	30	No data	Tehrani (1988)
Port Hills	29–34	0–80	8–19	McDowell (1989)
Port Hills	30	0–20	No data	Goldwater (1990)

3.3.2.2 Volcanic colluvium

Material parameters adopted for the volcanic colluvium layer underlying the loess material in the assessment area are based on: 1) descriptions of the drillcore materials; 2) Port Hills soil strength test results reported by Carey et al. (2014), and others; and 3) numerical slope stability back-analysis.

Figure 11 shows the results from the numerical slope stability back-analysis of the colluvium, for cross-section 1, representing source area 1, overlain with the results from the laboratory testing of loess and volcanic colluvium (Figure 10).

A reasonable lower estimate of the shear strength of the colluvium, needed to derive a static factor of safety for the slope of 1.0, was friction (ϕ) of 20° and cohesion (c) of 5 kPa, or any other combination of friction and cohesion on the factor of safety = 1 line. A similar result was obtained for the colluvium from numerical slope stability back-analysis of cross-section 5, representing source area 2.

The static factor of safety of these slopes is likely to be higher than 1.0 for at least the part of the year when the slopes are dry. The range of shear strengths derived for the colluvium from the numerical slope stability back-analysis – assuming a factor of safety for the slope of around 1 – are below the lower end of those derived from laboratory testing of volcanic colluvium and loess, the main materials from which the colluvium derives.

The results from ring-shear testing of the matrix material from a drillcore sample of highly weathered volcanic breccia, taken from drillhole BH-CH-03 (located at Clifton Terrace, about 200 m east of the site), could be representative of the strength parameters of the matrix material forming the more clast-dominated volcanic colluvium, as it derives from the same material. The results reported by Carey et al. (2014) indicate a residual friction angle (ϕ) of 21° and cohesion (c) of 15 kPa.

The main material forming the matrix of the colluvium, as described in the drillhole logs, is reworked loess. The shear strength results for the loess, discussed in the previous section, are thought to be representative of the reworked loess forming the colluvium.

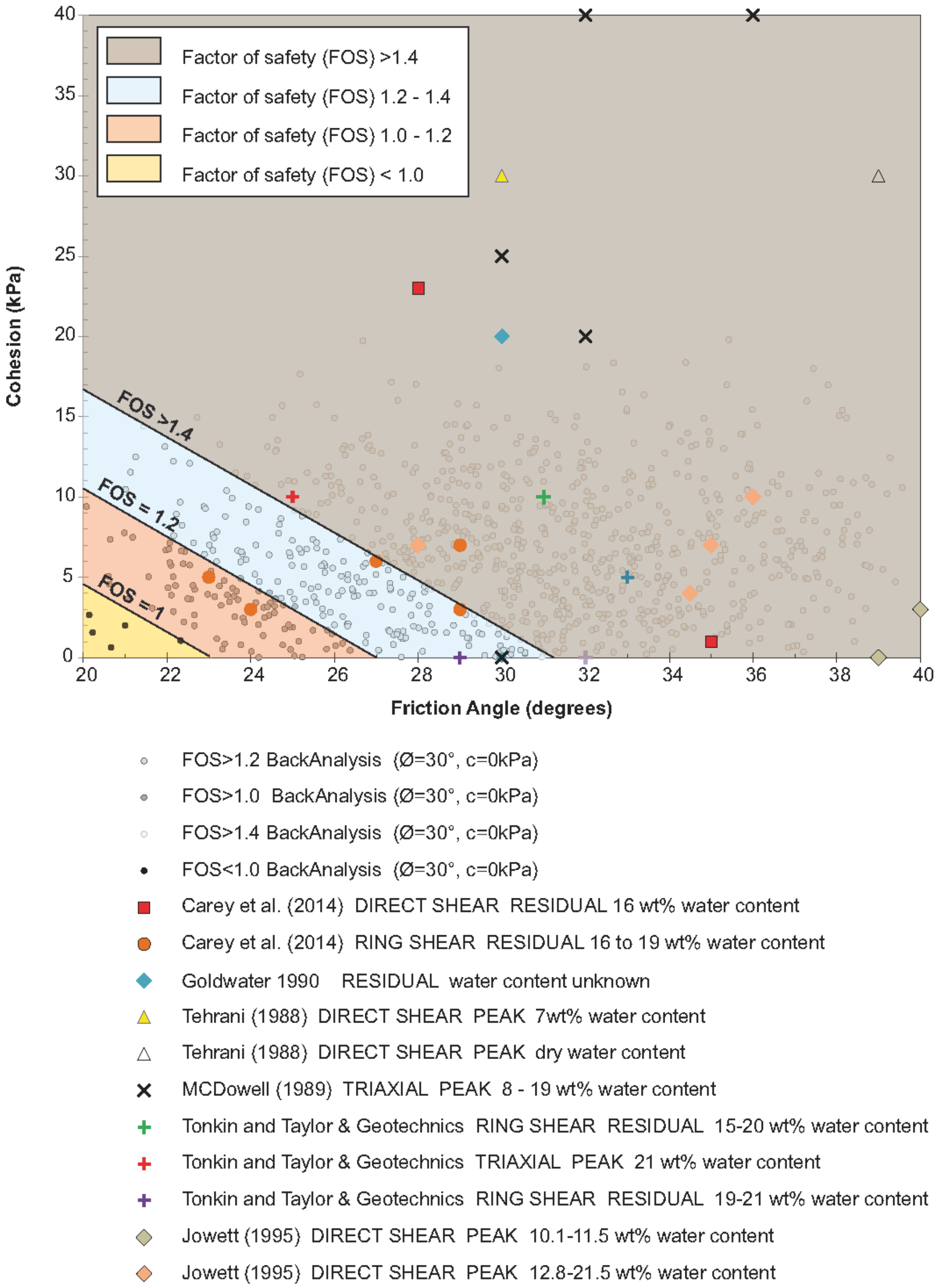


Figure 11 Numerical slope stability back-analysis of the colluvium material for cross-section 1, representing source area 1. Note: each data point represents a modelled slide surface at a given combination of cohesion and friction adopted for the fill. Those slide surfaces (adopting the path-search function), shown as squares, represent those combinations of cohesion and friction that would yield a static factor of safety of less than 1. The results from the laboratory testing (Figure 10) are also shown for comparison purposes.

Results from slope stability back-analysis show that the factor of safety is sensitive to relatively small changes in cohesion and friction of the weak colluvium layer. Such changes in cohesion can be caused by increases or decreases in soil suction caused by fluctuations in the bulk water content of the colluvium. Bulk water content is therefore likely to play a critical role in the overall static stability of the slope.

Shear modulus

No shear wave velocity surveys were carried out in the assessment area. The in situ shear modulus of the loess and volcanic colluvium were derived from:

1. Results from the downhole shear-wave velocity surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) based on the survey results from drillholes BH-CH-02 and BH-CH-03 carried out by Aurecon NZ Limited at Clifton Terrace (about 200 m east of the site); and
2. Results from the dynamic probing carried out by Tonkin and Taylor for the Earthquake Commission at Clifton Terrace (Tonkin and Taylor, 2012a) (Table 9).

The results from the dynamic probing are summarised in Figure 12. The mean shear wave velocity is 306 m/s (±93 m/s at one standard deviation) and the mode is 222 m/s. Based on these results reported in Tonkin and Taylor (2012a), there is no measurable difference between the shear wave velocities of the loess and the colluvium, and the data are plotted together (Figure 12).

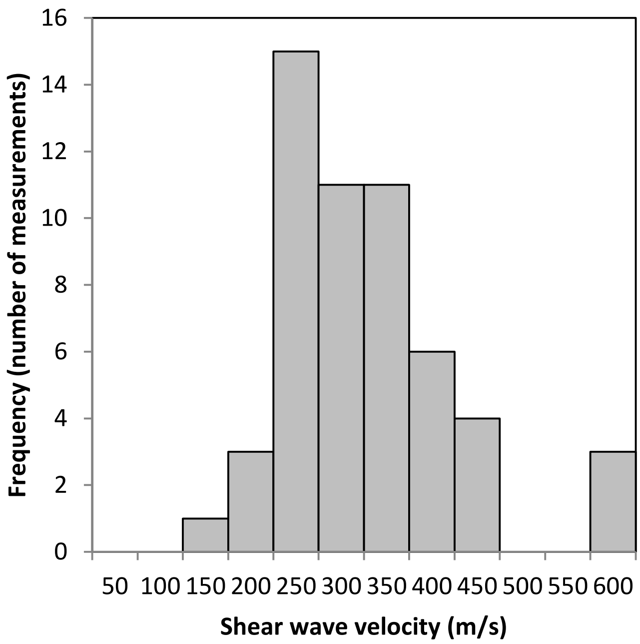


Figure 12 Loess and colluvium shear wave velocity results from dynamic probing reported by Tonkin and Taylor (2012a) for the loess at Clifton Terrace.

These values are also consistent with shear wave velocity trends defined by Rinaldi et al. (2001) for Argentinean loess as a function of normal stress and moisture content (Table 10) where in the 2–14 m depth range (corresponding to 30–240 kPa range of overburden pressure) the range of loess shear wave velocity was 280–300 m/s, at a water contents of ~16%, and 300–320 m/s for a water contents of 6.4 wt%.

Applying the relationship for shear wave velocity:

$$G = \rho \, . \, V_s^2 \qquad \textbf{Equation 2}$$

Where ρ is the density of the loess 1700 kg/m^3 and V_S is the shear wave velocity (mean = 306 m/s, and mean plus one standard deviation = 399), yields a bulk shear modulus value of about 160–280 MPa when adopting the mean and the mean plus one standard deviation shear wave velocities from the dynamic probing.

Table 10 Shear wave velocity profiles from Port Hills and other loess.

Material	Shear wave velocity V_S (m/s)	Data source
Port Hill loess (drillholes BH-CH-02 and BH-CH-03)	295–390	Southern Geophysical Ltd. (2013)
Port Hills loess from Clifton Terrace dynamic probing	126–582	Tonkin and Taylor (2012a)
Loess Moisture content ~16wt%	280–300	Rinaldi et al. (2001)
Loess Moisture content 6.4wt%	300–320	Rinaldi et al. (2001)

3.3.2.3 Volcanic bedrock

The exposed rock mass in the steep cliffs of Shag Rock Reserve (locally referred to as Peacocks Gallop) is highly variable, with the material units being highly discontinuous and there is a lack of persistent defects in the rock mass. Given the anisotropic nature of the materials, especially with regards to the lateral and vertical extent of the lava sequences within the predominant volcanic breccia, and the generally low shear wave velocities derived from the down-hole surveys, it was decided to treat the volcanic materials as one unit, comprised mainly of breccia.

In order to derive rock mass strength parameters for the volcanic breccia and lava that take into account the nature of the discontinuities as well as the intact strength of the breccia and lava, the geological strength index (Hoek, 1999) was adopted using Rocscience RocLab software.

The geological strength index values adopted for the breccia are shown in Figure 13. Strength tests of Deans Head rock samples from drillholes BH-DH-01 and BH-DH-02 are shown in Table 12, and are taken from Carey et al. (2014). Mohr-Coulomb parameters (cohesion and friction) were derived from Rocscience RocLab software by line fitting over the appropriate stress range of the slope.

Shear Modulus

The shear moduli for mixed volcanic breccia and lava were derived from the downhole geophysical surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) in drillholes BH-CH-02 and BH-CH-03, located on Clifton Terrace, about 200 m east of the site.

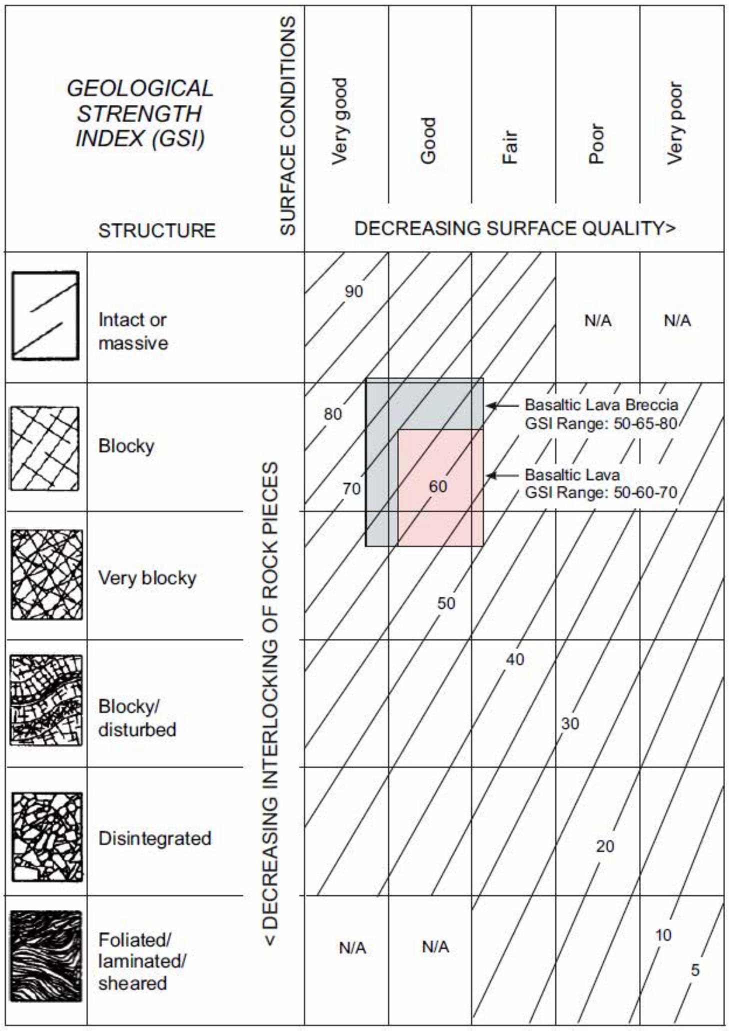


Figure 13 Geological strength index plot for volcanic breccia at Deans Head (modified after Hoek, 1999).

3.3.2.4 Adopted parameters for numerical models

For the purpose of stability assessment, material strength parameters were selected as shown in Table 11 and Table 12.

Table 11 Range of bulk geotechnical material parameters adopted for Deans Head soils.

Soil Unit	Unit weight (kN/m^3)	Intact Young's modulus E_i (MPa)[1]	Poisson's ratio[1]	Cohesion c (KPa)	Friction ϕ (°)	Tensile strength (KPa)	Shear wave velocity (m/sec)	Shear modulus[2] G_S (MPa)
Colluvium	17	30	0.3	0–15	21–30	0	200	68
Loess	17	30	0.3	10	30	10	200	68

1 Derived from published test results.

2 Shear Modulus G_s (MPa) derived from dynamic testing of loess and colluvium down-hole shear wave velocity survey of drillholes BH-CH-02 and BH-CH-03. Where $G_s = \rho * Vs^2$. Where ρ = density (Kg/m^3) and V_s = shear wave velocity (m/s).

Table 12 Range of adopted rock strength parameters.

Unit		Rock mass properties								Rock mass properties					
Unit		Lab UCS (MPa)	Bulk unit weight (kN/m^3)	Tensile (MPa)	Intact modulus E_i (MPa)	Poisson's ratio	Slope height (m)	GSI	m_i[2]	Cohesion[3] c (KPa)	Friction[3] ϕ (°)	Tensile strength (KPa)	Rock mass modulus E_M (MPa)	QR4 Shear wave velocity (m/sec)	Shear modulus G_S[4] (MPa)
Basalt lava breccia	MIN[1]	1.3	16	0.3	820	0.01	0–20	50	4	50	25	7	251	570	520
	AVG	2.6	18	0.4	1,478	0.05		65	8	100	42	23	930	890	1,426
	MAX	3.7	19	0.4	1,900	0.11		80	11	220	51	74	1,670	1,200	2,736
Basalt lava	MIN[1]	146	28	9.7	39,000	0.29	0–20	50	14	930	67	250	16,800	540	780
	MAX	243	27	16.7	54,700	0.22		70	21	4,500	67	1200	28,600	1,100	3,400

[1] MIN, AVG and MAX represent the range (minimum, average, maximum) of test results and field measurements.

[2] The m_i values shown, represent the range in the ratio of unconfined compressive strength to tensile strength, derived from tested samples of basalt lavas and basalt lava breccias (Carey et al., 2014), and not the ratio of unconfined compressive strength to tensile values shown in the table.

[3] Mohr-coulomb parameters (cohesion and friction) were derived from RocLab by line fitting over the appropriate stress range of the slope.

[4] Shear Modulus (G_s) is derived from down-hole shear wave velocity survey of drillholes BH-CH-02 and BH-CH-03, where $G_s = \rho * Vs^2$ and ρ=density (Kg/m^3) and V_s = shear wave velocity (m/s).

3.3.3 Rainfall and groundwater response

In general, there are two main effects that groundwater has on the stability of slopes that need to be considered: 1) rising groundwater within the slope leading to an increase in pore pressures and a reduction in the effective stress of the materials; and 2) infiltration from prolonged rainfall, leading to the deepening of the wetting band accompanied by a decrease in matric suction (e.g., Kim et al., 2004) and loss of cohesion. Owing to the lack of monitoring data, it is not known which mechanism could be the main contributor to rainfall-induced slope failures in the Port Hills. Loss of cohesion during long duration rainfall is a known cause of instability in fine grained, non-cohesive soils and therefore is likely to be a significant contributory factor to landslides in loess and loess derived materials.

The relationship between rainfall and landslides in the Port Hills has been summarised by McSaveney et al. (2014). Heavy rain and long-duration rainfall have been recognised as potential landslide triggers on the Port Hills for many years. Loess earth/debris flows were noted frequently, even before the era of wider urban development in the Port Hills. A long historical landslide record has been gathered by searching “Paperspast” (http//paperspast.natlib.govt.nz). This electronically searchable record of daily and weekly newspapers in New Zealand has been searched over the period 1860–1926, but its landslide information is very incomplete, being only what newspapers of those times considered to be “newsworthy”. A summary of past landslides in the Port Hills and Banks Peninsular is contained in Appendix 3.

McSaveney et al. (2014) examined a list of Earthquake Commission claims for landslide damage for the period 1997–2010 and a Geotechnical Consulting Ltd. landslide investigations list, which covers the period 1992–2009. Any duplicate records for the period 1997–2009 contained in the data sets were removed. These records, though incomplete with respect to all of the landslides that occurred over those intervals, may be approximately complete with respect to the episodes of rain associated with landslide occurrences that damaged homes and urban properties (Figure 14).

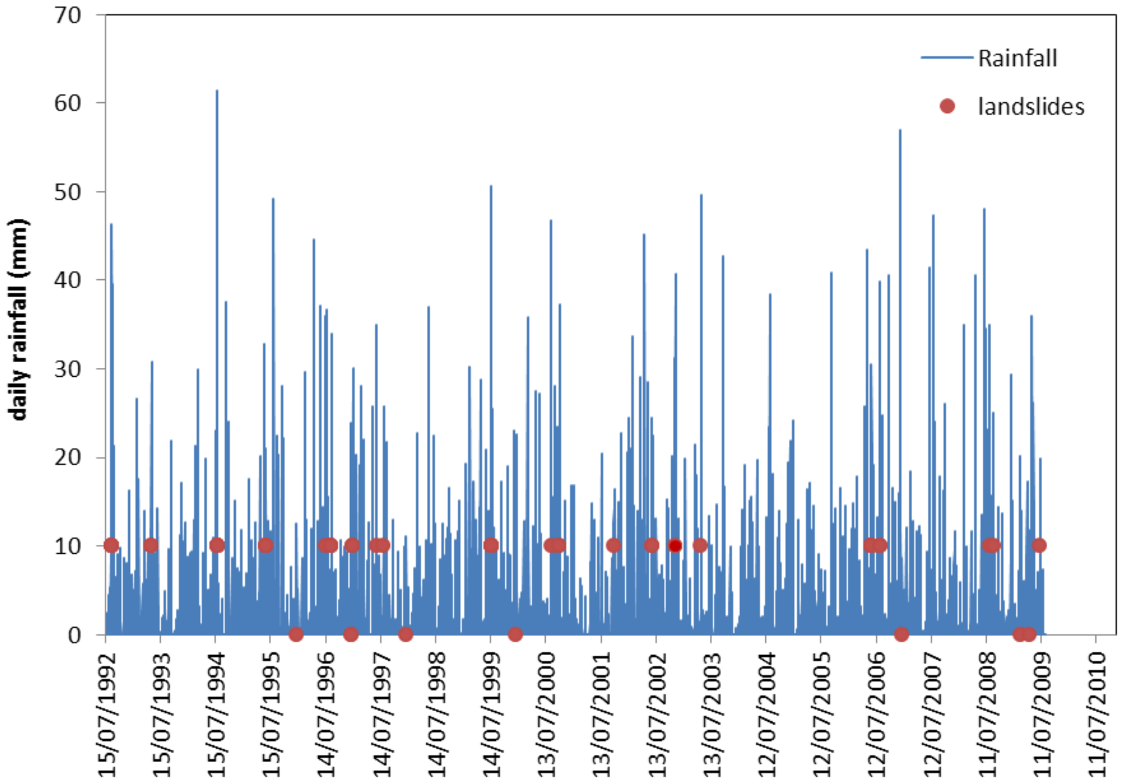


Figure 14 Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills. Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills investigated by Geotechnical Consulting Ltd, or listed by the Earthquake Commission as causing damage to homes. Landslides without rain are plotted at 0 mm, all others are plotted at 10 mm of rain (the minimum rainfall for triggered landslides).

McSaveney et al. (2014) conclude that: comparison of the record of damaging landslides and daily rainfall for the period 1992–2010 shows that:

1. Landslides can occur without rain, but the probability of landslides occurring increases with increasing intensity of rainfall;
2. Landslides occurred much more frequently on days with rain, but there were many rainy days when no landslides were recorded; and
3. As the amount of daily rainfall increased, a higher proportion of the rainy days had recorded landslides.

Following the 2010/11 Canterbury earthquakes there have been two notable rainfall events (Table 13):

- 11–17 August 2012: occurred at the end of winter following a long period of wet weather. During this period a total of 92 mm of rainfall was recorded at the Christchurch Botanic Gardens. The maximum daily rainfall (24 hourly rainfall recorded 9 am–9 am) during this period occurred on 13 August 2012 and totalled 61 mm.
- 3–5 March 2014: occurred at the end of a period of dry weather. During these three days, a total of 118 mm of rain was recorded at the GNS Science rain gauge installed at Clifton Terrace in the Port Hills (approximately 2 km west of Deans Head). The maximum daily rainfall (24 hourly rainfall recorded 9 am–9 am) during this period occurred on 5 March 2014 and totalled 85.4 mm.

The frequency of high-intensity rainfalls in Christchurch has been well studied (e.g., Griffiths et al., 2009, Figure 15, and McSaveney et al., 2014). Griffiths et al. (2009) use rainfall records for the period 1917–2008 from gauges all over Christchurch. McSaveney et al. (2014) use a composite rainfall record, for the period 1873–2013, mainly from the Christchurch Gardens gauge, but substituting averages for other nearby stations where gaps in the Christchurch Gardens data exist.

The annual frequencies for four rain events, including the two notable events are given in Table 13. Rainfall depth-duration-return period relations for Christchurch Gardens and Van Asch St, Sumner are taken from Griffiths et al. (2009) and for Christchurch Gardens from McSaveney et al. (2014).

Table 13 Annual frequencies of given rainfall in the Christchurch for four main events following the 2010/11 Canterbury earthquakes (rainfalls are calculated daily from 09:00 to 09:00 NZST).

Date	Total rainfall (mm)	Station	Max daily rainfall/date	Annual frequency Christchurch Gardens Griffiths et al. (2009)	Annual frequency Christchurch Gardens McSaveney et al. (2014)	Annual frequency Van Asch, Sumner Griffiths et al. (2009)
11–17 August 2012	92	Christchurch Gardens (CCC/NIWA)	61 mm 13 August 2011	92 mm = no data available 61 mm = 0.5 (once every 2 years)	92 mm = 0.4 (once every 2.7 years) 61 mm = 5 (5 times per year)	N/A
3–5 March 2014	118	Clifton Terrace (GNS Science)	89 mm 5 March 2014	N/A	N/A	118 mm = 0.1 (once every 10 years) 89 mm = 0.1 (once every 10 years)
3–5 March 2014	141	Christchurch Gardens (NIWA)	130 mm 5 March 2014	141 mm = 0.05–0.02 (once every 20–50 years) 130 mm = 0.02–0.01 (once every 50–100 years)	141 mm = 0.05 (once every 20 years) 130 mm = (>0.01) less than once every 100 years	N/A
18 April 2014	68	Lyttelton (NIWA)	68 mm	N/A	N/A	68 mm = 0.5 (once every 2 years)
29 April 2014	20	Clifton Terrace (GNS Science)	20 mm	N/A	N/A	Greater than 0.5 (occurs frequently every year)

The small (less than 50 m^3) earth/debris flow at the lower part of the site was triggered by the 11–17 August 2012 rain. No further landslides or movement of the slopes within the assessment area have been reported to GNS Science.

Regardless of the dataset used, both suggest that the heavy rainfalls recorded in the Port Hills following the 2010/11 Canterbury earthquakes are unexceptional. Although the three-day rainfall of 118 mm had an annual frequency of 0.1–0.05 (once every 10–20 years), it occurred at the end of summer when the ground would have had a seasonally low water content.

These observations suggest that antecedent water conditions are also important as an indicator of slope instability. For example, large daily rainfalls occurring during periods of wet weather are more likely to trigger movement and landslides than very high daily rainfalls during long periods of dry weather.

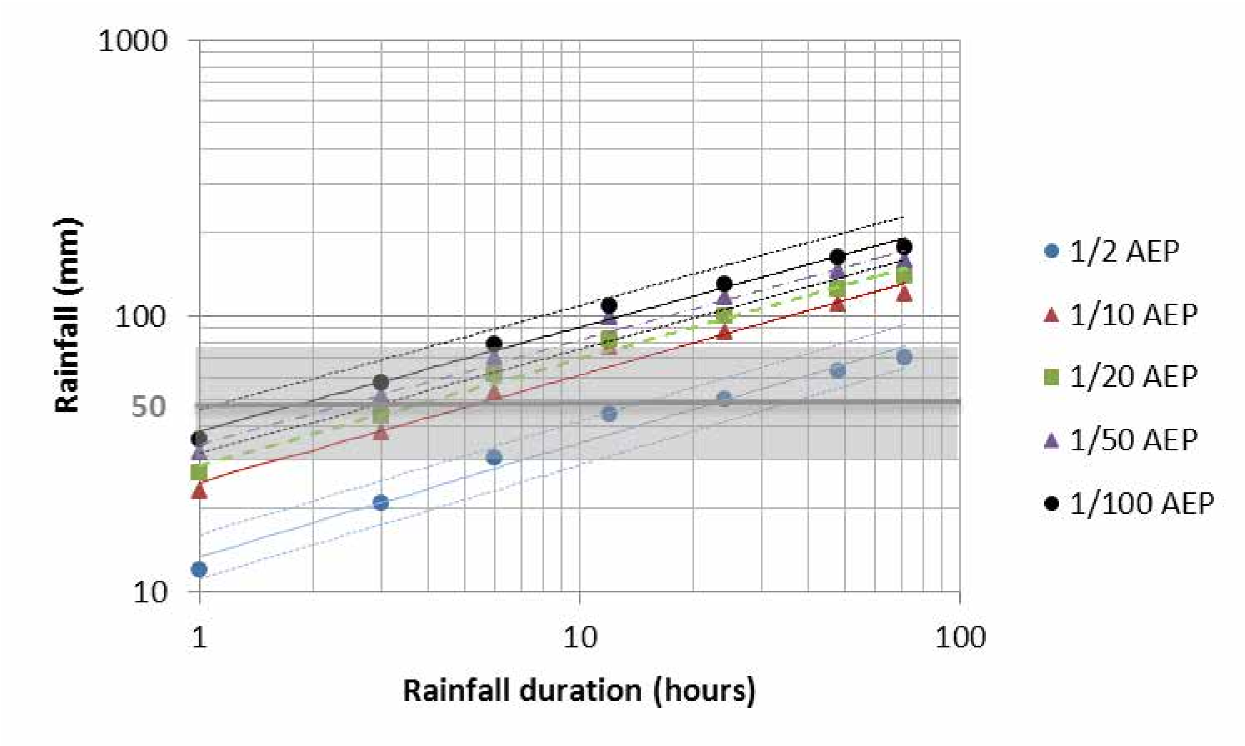


Figure 15 Rainfall depth-duration-return period relations estimated for Christchurch Gardens by Griffiths et al. (2009) using recorded rainfall data. Error limits of 20% are shown by dotted lines for the 1/2 and 1/100 AEP curves. Shaded area covers the range of 30–75 mm of rainfall over which the expected number of soil landslides in the Port Hills rises from very few to many. Rockfalls can occur without rain, but the probability of rockfalls occurring increases with increasing intensity of rainfall.

There is significant variation in rainfall across Christchurch in individual storms. The return period of the 89 mm of rain recorded at the GNS Science rain gauge at Clifton Terrace on the 5 March 2014 was about 10 years (using the data from Griffiths et al. (2009) for Van Asch Street in Sumner). The return period of the 130 mm of rain recorded at Christchurch Gardens for the same storm on the same day, was between 50 and 100 year (using the data from Griffiths et al. (2009) for the Christchurch Gardens).

At Lyttelton, about 135 mm of rain was recorded on the 5 March 2014, which is considerably higher than the 89 mm recorded at Clifton Terrace, which is only about 5 km north of Lyttelton.

3.4 SLOPE FAILURE MODELS

3.4.1 Landslide types affecting the site

Based on the results from the mapping, monitoring of survey marks and ground investigations, the pattern of slope deformation recorded in the assessment area in response to the 2010/11 Canterbury earthquakes is thought to be consistent with:

- Earthquake-induced coherent "soil block slide" (adopting the scheme of Keefer, 1984), where the basal slide surface appears to be within the volcanic colluvium layer overlying bedrock. There are no continuous and persistent boundaries or defects exposed in the adjacent cliff face of Shag Rock Reserve that would allow such a structurally controlled failure, within the underlying rock mass to occur. It is likely that any slide surfaces anastomose between the volcanic colluvium and completely/highly weathered underlying breccia. In general, the vector displacements inferred from survey marks and crack apertures are consistent with displacement of the mass sub-parallel to the dip of rock head.
- Earthquake-induced insipient coherent rock slide/slumps through the underlying rock mass, located at the crest of the steeper rock slopes towards the north and east of the site. Vector displacements inferred from survey marks and crack apertures suggest that parts of the cliff edges have displaced during the 2010/11 earthquakes, and that the vector of displacement (from the horizontal) are significantly steeper than the rock head boundary, suggesting failure thought the underlying rock mass, with "break out" of the failure possibly near the bottom of the cliff where the rock mass appears dilated (Figure 5).

Based on these results, it is possible that coherent soil block slide, within the assessment area, could develop into more mobile earth/debris flows. This is because:

1. The slope is currently cracked allowing surface water to infiltrate the slope more readily;
2. The shear strength of the loess and colluvium will reduce with increasing water contents and the slope will be subjected to increased pore-water pressures within the slope mass and in open tension cracks; and
3. There is a large relict landslide scar (10,000–15,000m^3), adjacent to the assessment area, suggesting large slope failures have occurred in the area.

3.4.2 Failures mechanisms adopted for modelling

The main identified slope-failure mechanisms in the assessment area that have been adopted for numerical modelling are:

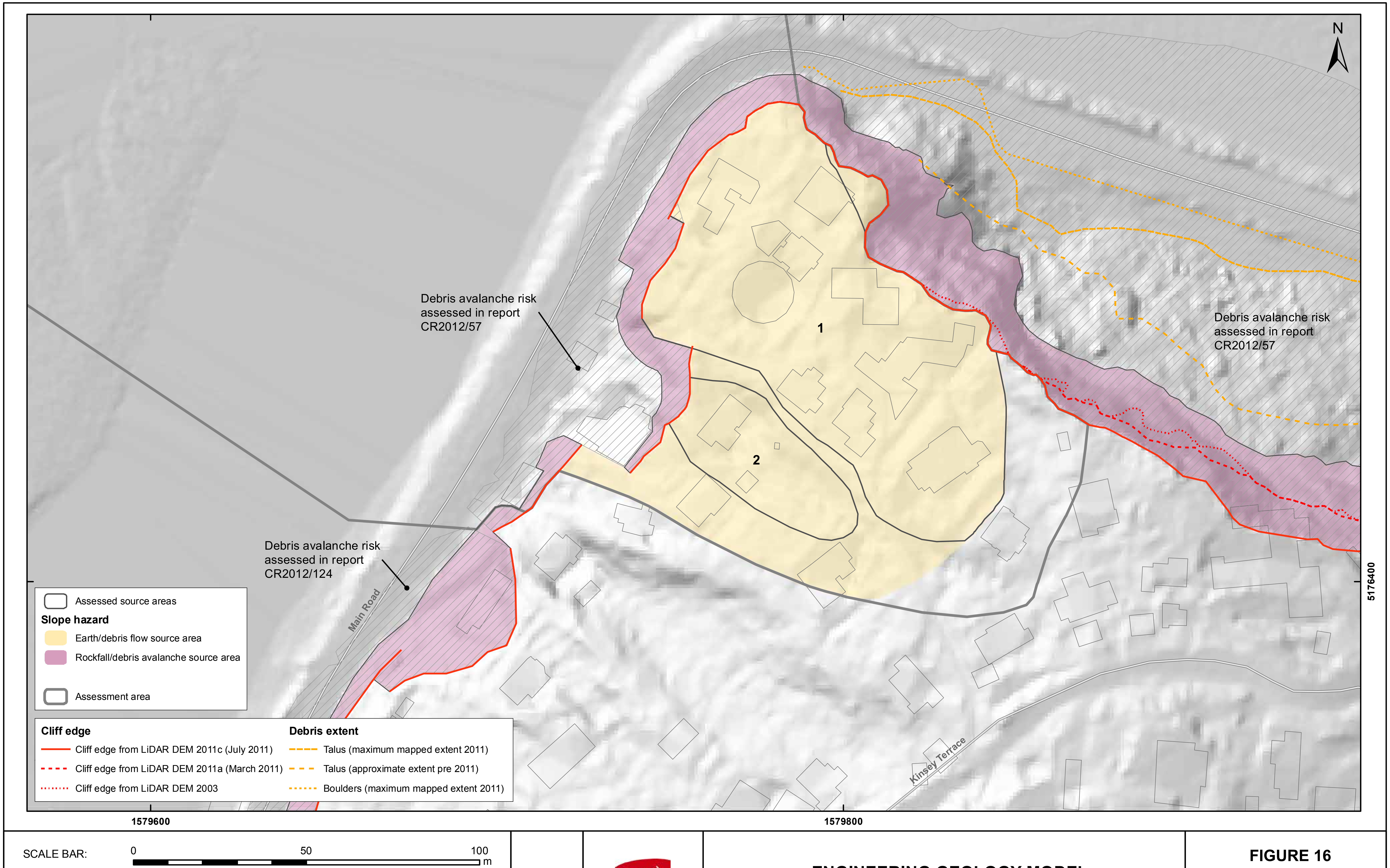
- *Mechanism 1:* Shallow failure of loess, at the lower part of the slope, forming several discrete landslide source areas. These represent the lower volume estimates for source areas 1 and 2.
- *Mechanism 2:* Deeper block-slide failure of the loess through the underlying volcanic colluvium layer, forming a coherent slide. A colluvium layer was intercepted in all drillholes within the main area of cracking. For the purpose of the model, the colluvium layer is considered to extend beneath the loess, above the volcanic lava sequences, over most of the site. These represent the middle and upper volume estimates for source areas 1 and 2, where the failure mechanism is thought to be translational, with the failure surface in the loess/volcanic colluvium being sub parallel to rockhead. This

failure mechanism appears to be consistent with that of the relict landslide scar, to the south, which is inferred to be a relatively long, shallow translational failure of loess and volcanic colluvium, with the inferred failure surface being sub parallel to rockhead.

- *Mechanism 3*: Deep-seated failure through the volcanic breccia, of the outside edge of the rock slope, resulting in cliff collapse (debris avalanches and cliff-top recession). For cross-section 1, such a failure may correspond to the deflection measured in the inclinometer tube installed in drillhole BH-DH-02. The main area of concern with regards to this mechanism is area around cross-section 3, where the rock mass (basalt breccia) exposed in the cliff face above the cliff toe, appears dilated and disturbed. The risk associated with these types of failure has already been assessed by Massey et al. (2012).

Cliff collapse (mechanism 3) is a credible failure mechanism, but the risk from such failures has already been addressed in a previous report (Massey et al., 2012) and so the risk from cliff collapse has not been reassessed in this report.

The main additional hazard affecting the site (additional to cliff-collapse hazards) is from earth/debris flows. The results from the site investigations have been used to define two main source areas within the assessment area, these are referred to as source areas 1 and 2 (Figure 16). These source areas are thought to represent the shape and extent of the likely earth/debris flows that could occur in the assessment area.



EXPLANATION:

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGY MODEL

Deans Head Christchurch

FIGURE 16

FINAL

REPORT: CR2014/77

DATE: June 2014

4.0 HAZARD ASSESSMENT RESULTS

4.1 SLOPE STABILITY – STATIC CONDITIONS

For source areas 1 and 2, the engineering geological cross-sections in Figure 8 were used as the basis of the numerical slope stability modelling. Geotechnical material strength parameters used in the modelling are from Table 11 and Table 12, and models using variable shear strength parameters for the key materials were used to assess the sensitivity of the slope – along a given cross-section – to failure.

Graphic examples of stability assessment outputs are shown in Figure 17–Figure 19 for cross-section 1. Cross-section 1 was chosen as it best represents the site conditions where the angle of rockhead was steepest, and it is the cross-section closest to where the permanent slope displacements were measured from survey marks. Graphic examples of stability assessment outputs are shown in Figure 20 for cross-section 3. Cross-section 3 was modelled to assess the stability of the rockslope.

Table 14 shows the results from the assessment of the three failure mechanisms. In summary the analysis shows:

- *Mechanism 1 – Failure through the loess (Figure 17, cross-section 1):* The results from the assessment, adopting loess shear strength parameters of friction angle (ϕ) of 30° and cohesion (c) of 10 kPa, suggest that the slope factor of safety is about 1.7 for the range of material parameters adopted. The factors of safety of the modelled slide surfaces are very sensitive to the adopted cohesion value.
- *Mechanism 2 – failure through the volcanic colluvium layer (Figure 18, cross-section 1):* As for mechanism 1, the presence of a weak layer results in a low factor of safety. When a weak volcanic colluvium layer is taken into account, the factor of safety is about 1.2–1.3 for the range of material parameters adopted. These results suggest that the slope is marginally stable under static conditions if a weak layer of volcanic colluvium, above rockhead, is included in the assessment. The sensitivity of the factor of safety to changes in friction angle of the colluvium is shown in Figure 11. The range of friction-angle values in Figure 11 are within the range of residual strength values obtained from ring- and direct-shear tests of the loess and colluvium, and they are likely to represent a lower bound range of the bulk strength conditions of the loess and volcanic colluvium. Nonetheless, the results highlight the sensitivity of the slope to the presence of a weak layer.
- *Mechanism 3 – Failure through the rock (Figure 19 and Figure 20, cross-sections 1 and 3):* The factor of safety for the modelled conditions ranges from 1.6 to 5.3 (for cross-section 1) and 1.3–2.7 (for cross-section 3), adopting the range of material parameters in Table 14. The variability in the results is due to the range of bulk strength parameters adopted for the volcanic rock mass.

Table 14 Example results from slope stability assessment of source area 1 (cross-section 1).

Simulated failure mechanism	LOESS	Colluvium	Rock	Water level	FoS[1] SLIDE	Search surface	SRF[2] PHASE2
	Cohesion (kPa) / Friction (°)	Cohesion (kPa) / Friction (°)	Cohesion (kPa) / Friction (°)				
Mechanism 1, failure through loess	10 / 30	Not simulated	100 / 42	Drained	1.7	CIRC	1.7
Mechanism 2, failure through colluvium	10 / 30	0 / 28	100 / 42	Drained	1.2	BLOCK	1.3
	10 / 30	15 / 21	100 / 42	Drained	1.3	BLOCK	1.4
	10 / 30	0 / 30	100 / 42	Drained	1.3	BLOCK	1.4
Mechanism 3, failure through rock	10 / 30	10 / 30	50 / 25	Drained	1.6	PATH SEARCH	
	10 / 30	10 / 30	100 / 42	Drained	3.1	PATH SEARCH	2.9
	10 / 30	10 / 30	220 / 51	Drained	5.3	PATH SEARCH	

[1] FoS is the factor of safety derived using the general limit equilibrium method of Morgenstern and Price (1965).

[2] The finite element model was also used for comparison. Where the slope has been assessed using the finite element model, the stability of the slope is assessed in terms of the Stress Reduction Factor.

Note: the shear strength reduction method is used to determine the stress reduction factor (SRF) or factor of safety value that brings a slope to the verge of failure (Dawson et al., 1999).

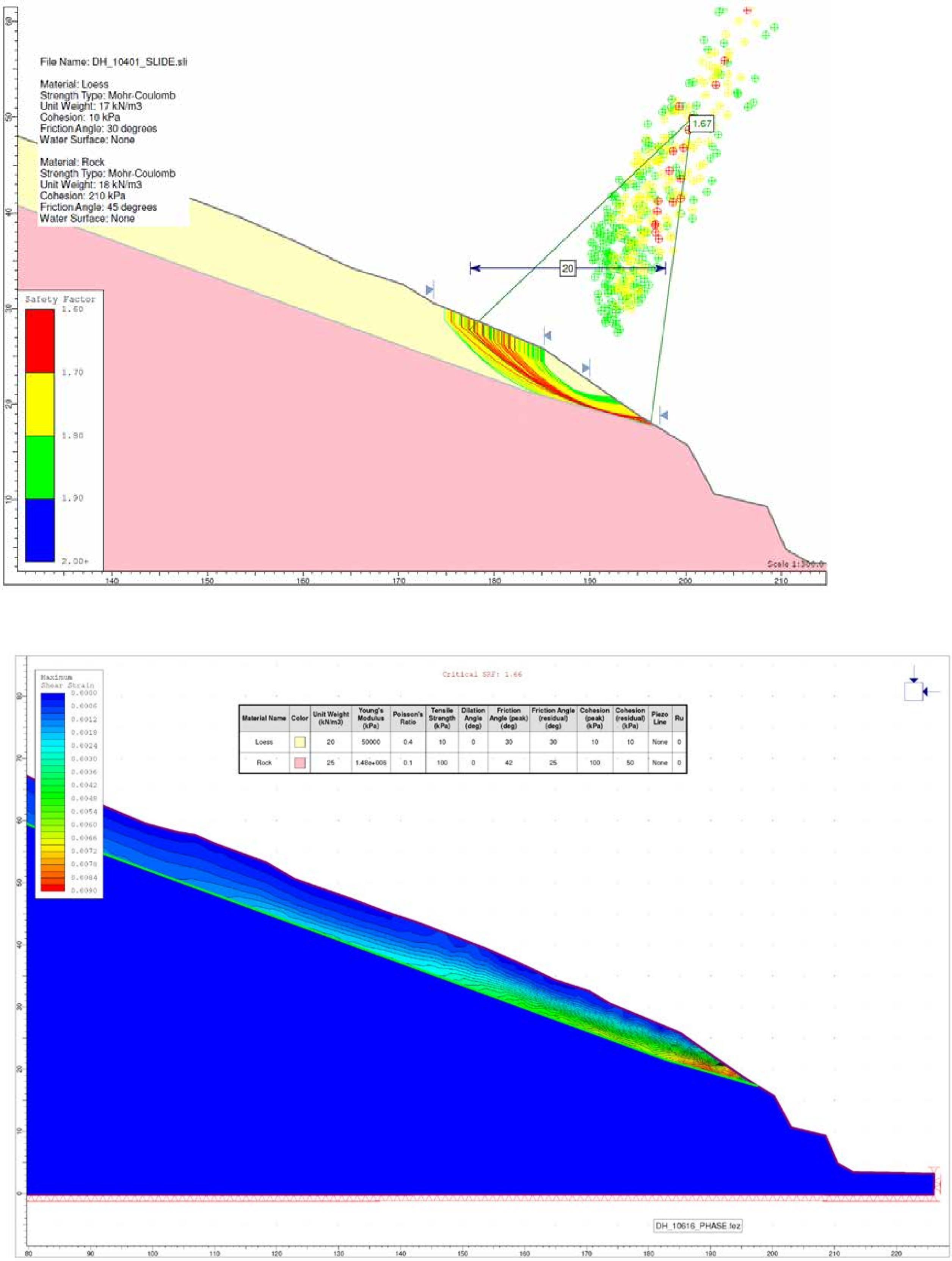
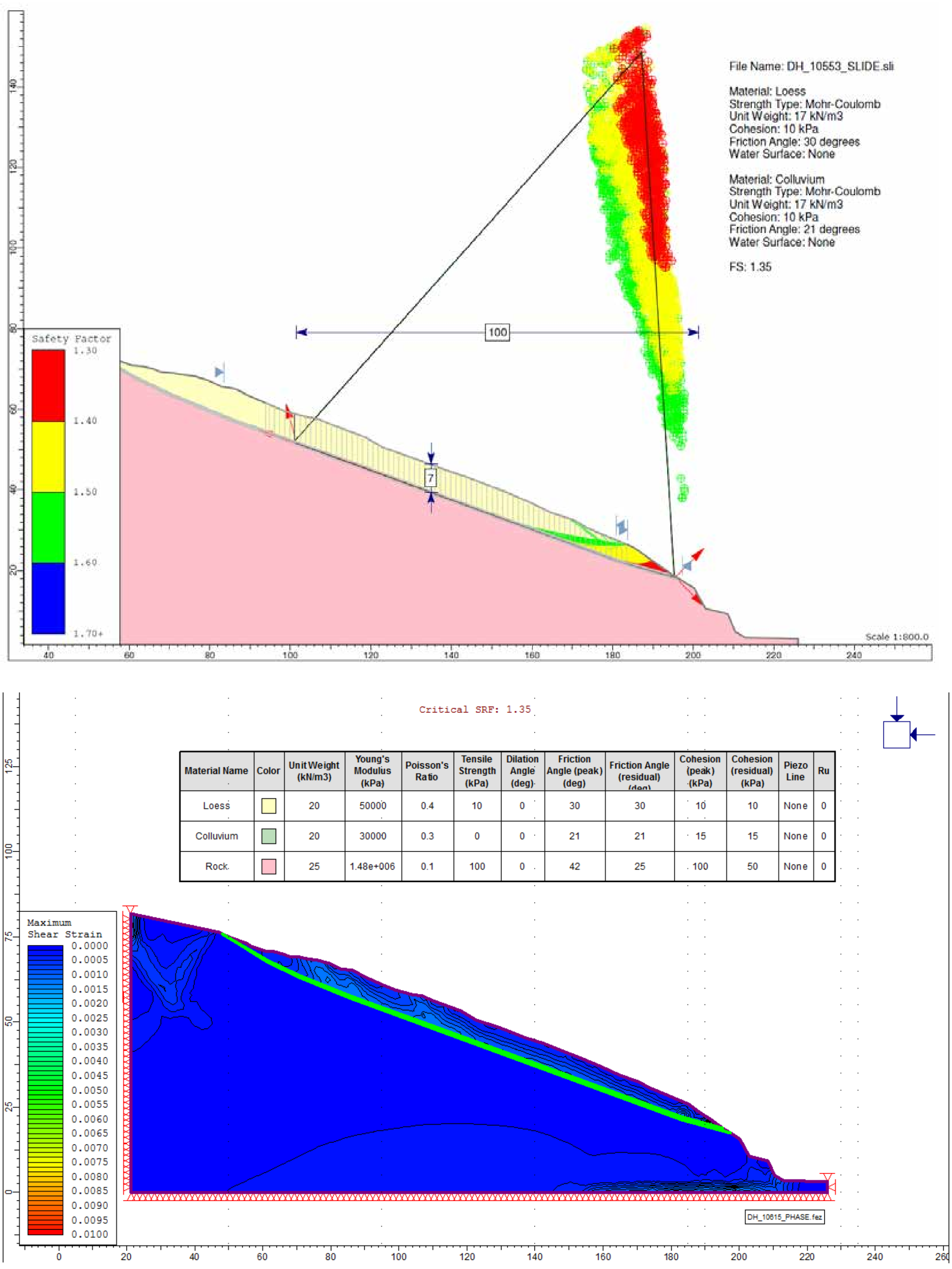


Figure 17 Failure mechanism 1. Example of limit equilibrium and finite element modelling results for cross-section 1, representing failure of loess in source area 1.



Material Name	Color	Unit Weight (kN/m3)	Young's Modulus (kPa)	Poisson's Ratio	Tensile Strength (kPa)	Dilation Angle (deg)	Friction Angle (peak) (deg)	Friction Angle (residual) (deg)	Cohesion (peak) (kPa)	Cohesion (residual) (kPa)	Piezo Line	Ru
Loess		20	50000	0.4	10	0	30	30	10	10	None	0
Colluvium		20	30000	0.3	0	0	21	21	15	15	None	0
Rock		25	1.48e+006	0.1	100	0	42	25	100	50	None	0

Figure 18 Failure mechanism 2. Example of limit equilibrium and finite element modelling results for cross-section 1, representing source area 1, for failure through the colluvium layer.

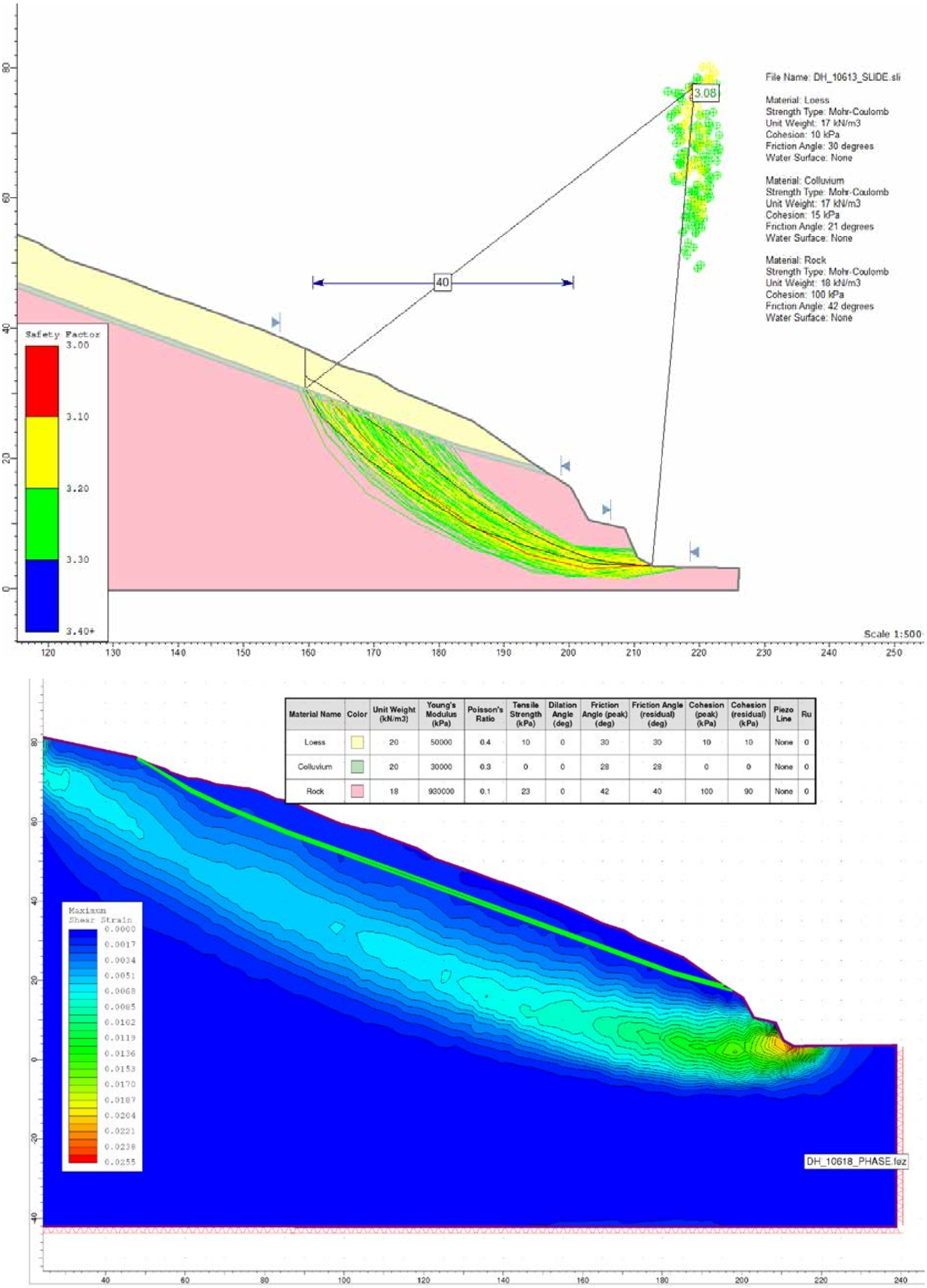
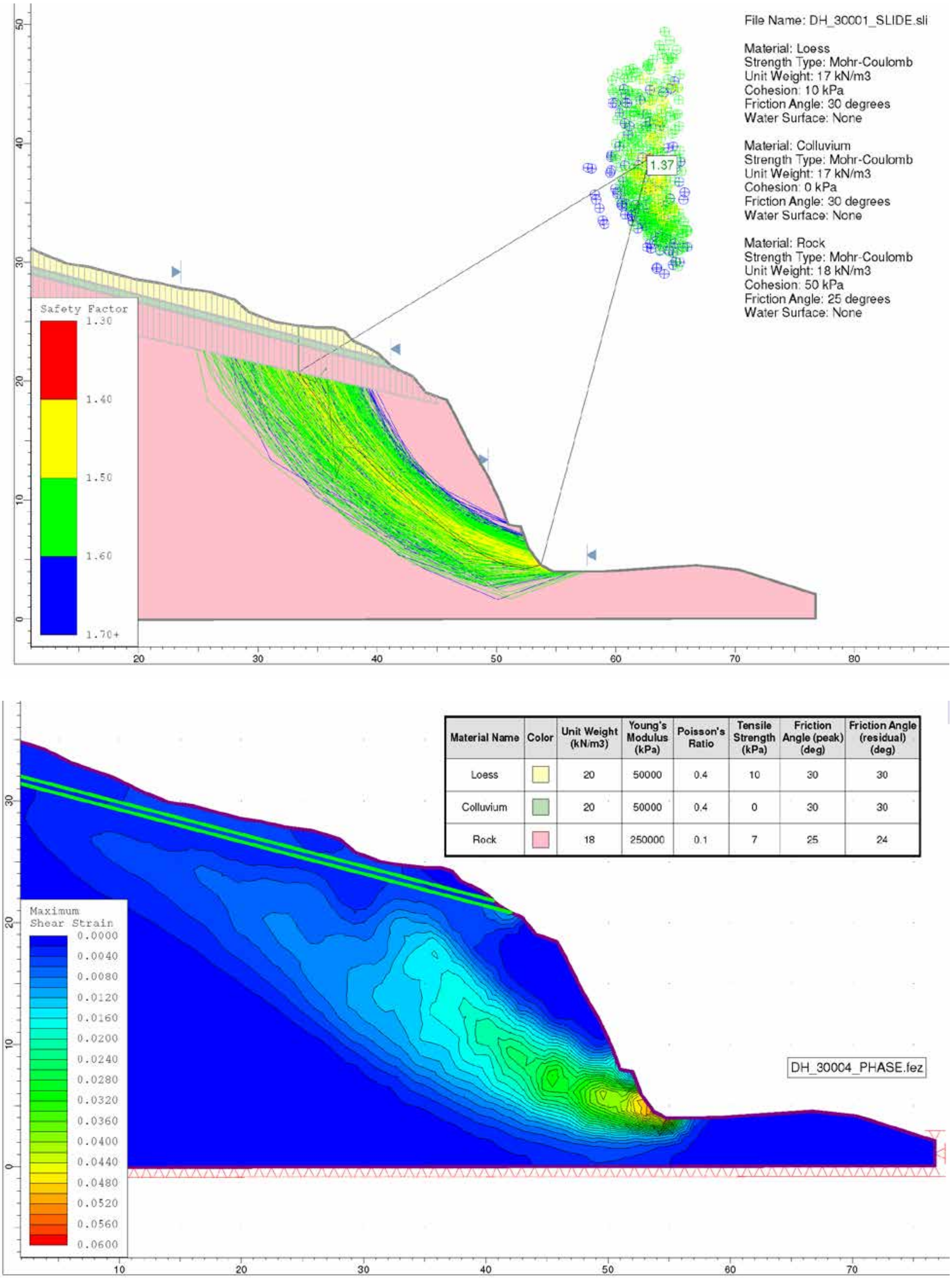


Figure 19 Failure mechanism 3. Example of limit equilibrium and finite element modelling results for cross-section 1, representing source area 1, for failure mechanism through the rock.



Material Name	Color	Unit Weight (kN/m3)	Young's Modulus (kPa)	Poisson's Ratio	Tensile Strength (kPa)	Friction Angle (peak) (deg)	Friction Angle (residual) (deg)
Loess		20	50000	0.4	10	30	30
Colluvium		20	50000	0.4	0	30	30
Rock		18	250000	0.1	7	25	24

Figure 20 Failure mechanism 3. Example of limit equilibrium and finite element modelling results for cross-section 3, for failure mechanism through the rock.

4.1.1 Model sensitivity to groundwater

The effects of changing water content of the volcanic colluvium on the slope factor of safety were modelled using variable cohesion values. The results from this sensitivity assessment are shown in Table 14.

The sensitivity of the slope factor of safety to changes in transient ground water (pore pressure) for mechanisms 1 and 2, has been simulated by modelling: 1) an initial piezometric line at rockhead and by increasing the piezometric head levels from the initial starting level, at given increments; and 2) pore pressures acting within tension cracks, where the tension cracks are assumed to extend from the surface to rockhead. Results are shown in Figure 20 and Figure 21.

The results show that an increase in piezometric head levels of about two metres above rockhead reduces the factor of safety to about one, adopting the lower estimates of the colluvium shear strength. Approximately four metres of piezometric head are required to reduce the factor of safety to about onw, for colluvium shear strengths of cohesion (c) of 0–15 kPa and friction (ϕ) 21–30°. These results indicate the slope is relatively sensitive to increases in piezometric head levels.

The inclusion of water filled tension cracks within the model, reduces the piezometric head levels needed to yield a factor of safety of one, from 4 m to about 3.7 m, when adopting colluvium shear strengths of cohesion (c) of 0–15 kPa and friction (ϕ) 21–30°. It should be noted that the stability model (Slide) used for modelling can only model one water-filled tension crack. In reality there would be many water-filled tensions cracks and so these results do not fully reflect the impact of water filled tension cracks on slope stability.

In reality, it is probably more reasonable to assume that failure of the slope could occur through a combination of reducing shear-strength parameters of the loess and colluvium in response to increasing water content linked to rainfall, water infilling tension cracks and the development of a continuous pore pressure surface within the slope leading to a reduction in the effective stress within the saturated loess. However, this is conjecture, as there is little groundwater or soil moisture monitoring data for the site. However, the assessment area comprises a relatively large catchment area for rainfall, which would be directed into the now cracked area of the slope.

There are currently no groundwater records of use for the site. Therefore it is not known whether a piezometric head level rise of between two and four metres above rockhead, is feasible, or under what weather conditions such an increase might occur. Groundwater levels recorded by GeoNet in standpipe piezometers, in loess, within drillholes in the Kinsey Terrace area have recorded 2–4 m rises in groundwater levels.

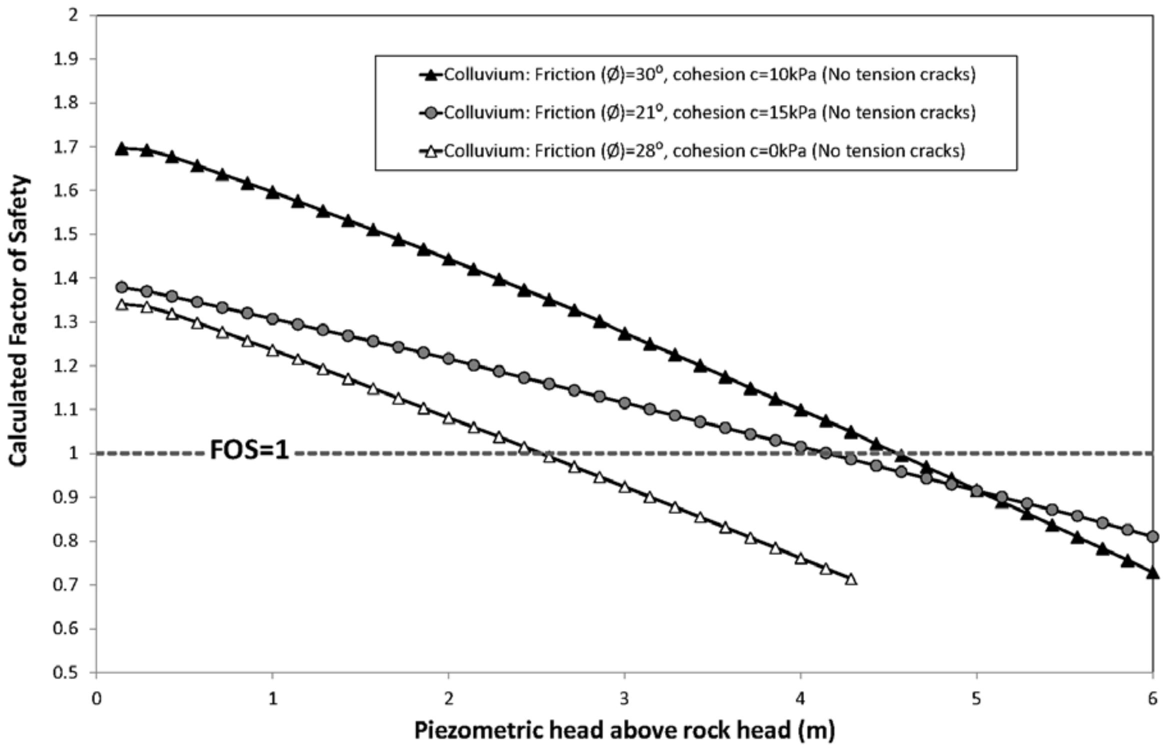


Figure 21 Sensitivity assessment of the slope factor of safety (source area 1) in response to changing piezometric head levels above rock head for a range of colluvium strength parameters.

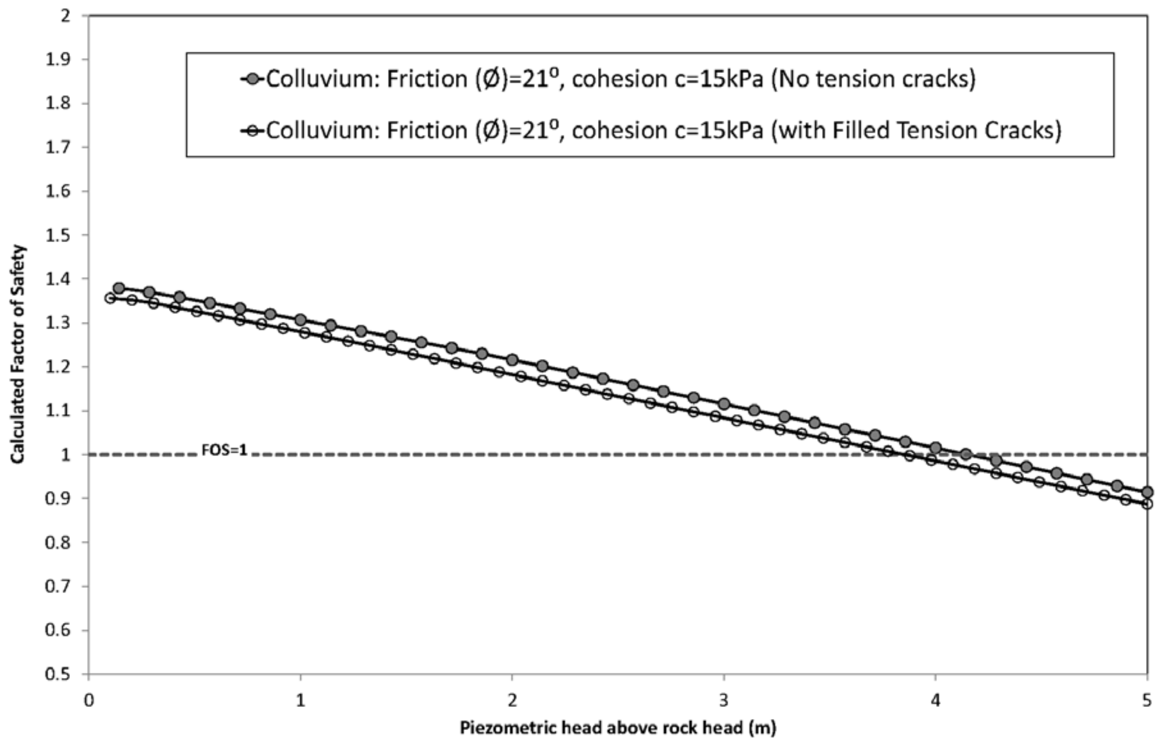


Figure 22 Sensitivity assessment of the slope factor of safety (source area 1) in response to including filled tension cracks in the model. Tension cracks were assumed to be 100% filled.

4.2 SLOPE STABILITY – DYNAMIC CONDITIONS

Dynamic stability assessment comprised: 1) back-analysing the performance of the slope during the 2010/11 Canterbury earthquakes to calibrate the models and verify that the calculated displacements are consistent with those recorded during the earthquakes; and 2) using the calibrated models to forecast the likely magnitudes of future displacements under potential future peak ground acceleration scenarios. Cross-section 1 has been assessed under dynamic conditions, assuming a drained slope.

4.2.1 Amplification of ground shaking

The first stage of the assessment was to calculate the maximum acceleration at the slope crest (A_{MAX}) to quantify any amplification effects caused by topography and or contrasting materials between the peak ground acceleration of the free field rock input motion and the peak acceleration at the slope crest (A_{MAX}). The slope crest is defined as the convex break in slope between the lower steeper slope and the upper less steep slope. Results from the dynamic site response assessment are contained in Appendix 4.

Results from this assessment suggest that modelled peak acceleration at the slope crest (A_{MAX}) varies approximately linearly with the peak ground acceleration of the free-field input motion. Over the range of modelled peak horizontal accelerations, the peak ground acceleration amplification factor (S_T) with respect to the free-field peak accelerations (A_{FF}) for cross-section 1, is about 2.6 (±0.3) for horizontal motions and 2.4 (±0.1) for vertical motions. The input peak accelerations are those derived from the out-of-phase synthetic free-field rock outcrop earthquake time acceleration histories described by Holden et al. (2014).

The results from cross-section 1, showing the response of the slope during the 22 February 2011 earthquake (Figure A4.2 in Appendix 4), suggest that the impedance contrasts between the materials contribute most to the amplification of shaking, but that the peak horizontal accelerations (for all modelled earthquakes) concentrate around the convex break in slope, defined as A_{MAX}.

Given the increased amplification of shaking within the loess and colluvium, coupled with the coseismic landslide displacement inferred from surveying, it is likely that the basal slide surface is coincident with the boundary between the colluvium and the underlying rock. In experimental data, as the slope displaces during an earthquake, the slide surface can “base isolate” the mass above, resulting in lower levels of shaking and displacement. Therefore, the reported amplification factors are near the upper bound of published topographic amplification factors. Assessment of this is outside the scope of this report.

4.2.2 Back-analysis of permanent slope deformation

Earthquake-induced permanent displacements were calculated using the decoupled method (Makdisi and Seed, 1978) and the Slope/W software. The two failure mechanisms assessed were: 1) failure through the volcanic colluvium underlying the loess (mechanism 2); and 2) failure of the rock mass coincident with the deflection recorded in the inclinometer tube in drillhole (BH-DH-01) (mechanism 3). Mechanism 1 was not back-analysed as this mechanism could not account for the observed slope displacement in the assessment area during the 2010/11 Canterbury earthquakes. Also, the shear strength parameters of the colluvium adopted for the assessment are consistent with those of the loess.

For each failure mechanism, a range of slide surfaces were assessed adopting the “block search” function. Permanent displacements were estimated along each slide surface, where the displacing mass was treated as rigid-plastic body and no internal plastic deformation of the mass was accounted for, and the mass accrued no displacement at accelerations below the yield acceleration.

The out-of-phase synthetic rock outcrop earthquake time acceleration histories from the 22 February and 13 June 2011 earthquakes were used as inputs for the modelling, as permanent coseismic displacement of the Deans Head slopes were recorded during these events. The synthetic rock outcrop earthquake time acceleration histories from the 16 April and 23 December 2011 earthquakes were also modelled to ensure that either no modelled movement or very minor (undetectable) movement of the slopes occurred. Variable material strength parameters were used for the critical materials present, and the different parameters used in the modelling are listed in Table 15.

For these assessments the displacements inferred from the cadastral and monitoring surveys are assumed to represent the coseismic permanent displacement of the slope, along cross-section 1, during the 22 February, 13 June and 23 December 2011 earthquakes. The results from each modelled scenario were then compared to the recorded coseismic permanent slope displacements for each earthquake.

Table 15 Material strength parameters used for modelling permanent coseismic displacements for cross-section 1. Coseismic displacements are inferred from survey records and field mapping of cracks.

Mechanism	Earthquake	Material	Cohesion (c) (kPa)	Friction (ϕ) (degrees)	Total recorded coseismic displacement (m)
2	22 February, 13 June, 16 April, and 23 December 2011	Loess Volcanic colluvium Volcanic colluvium Volcanic colluvium Mixed basalt lava and breccia	10 15 0 10 100	30 21 30 30 42	0.3–0.6 (22 Feb) 0.1 (13 Jun) <0.01 (16 Apr) <0.01 (23 Dec)
3	22 February 2011	Loess Volcanic colluvium Mixed basalt lava and breccia	10 15 50 100	30 21 25 42	0.3–0.6 (22 Feb) 0.1 (13 Jun) <0.01 (16 Apr) <0.01 (23 Dec)

The results from the modelling of the 22 February and 13 June 2011 earthquakes, adopting the parameters listed in Table 14, are summarised in Table 16. Figure 23–Figure 26 show the results for the different failure mechanisms:

- Mechanism 2 – failure through the volcanic colluvium above rockhead; and
- Mechanism 3 – failure through the basalt lava breccia to coincide with the deflection recorded in the inclinometer tube (BH-DH-01)

Table 16 Results from the dynamic modelling of cross-section 1. Total inferred coseismic displacements are from measurements of survey marks. Yield accelerations and permanent displacements are calculated from the decoupled assessment and represent the modelled slide surface with the lowest yield acceleration for the given material parameters and failure mechanism. Those rows highlighted in grey represent the material parameters that give the best correlation between the modelled and recorded permanent displacements, for a given earthquake and failure mechanism. Modelled displacements are rounded to the nearest 0.1 m.

Failure mechanism	Earthquake	Failure method and critical material	Cohesion c (kPa)	Friction ϕ (º)	Lowest yield acceleration (g)	Modelled coseismic displacement (m)	Total inferred coseismic displacement (m)
2	22 February 2011	Block, colluvium	15	21	0.17	0.7	0.3–0.6
2	22 February 2011	Block, colluvium	0	30	0.19	0.7	0.3–0.6
2	22 February 2011	Block, colluvium	10	30	0.28	0.4	0.3–0.6
2	13 June 2011	Block, colluvium	15	21	0.17	0.1	0.1
2	13 June 2011	Block, colluvium	0	30	0.19	0.1	0.1
2	13 June 2011	Block, colluvium	10	30	0.28	0	0.1
3	22 February 2011	Block, rock breccia	50	25	0.29	0.1	0.3–0.6
3	22 February 2011	Block, rock breccia	100	42	0.95	0	0.3–0.6

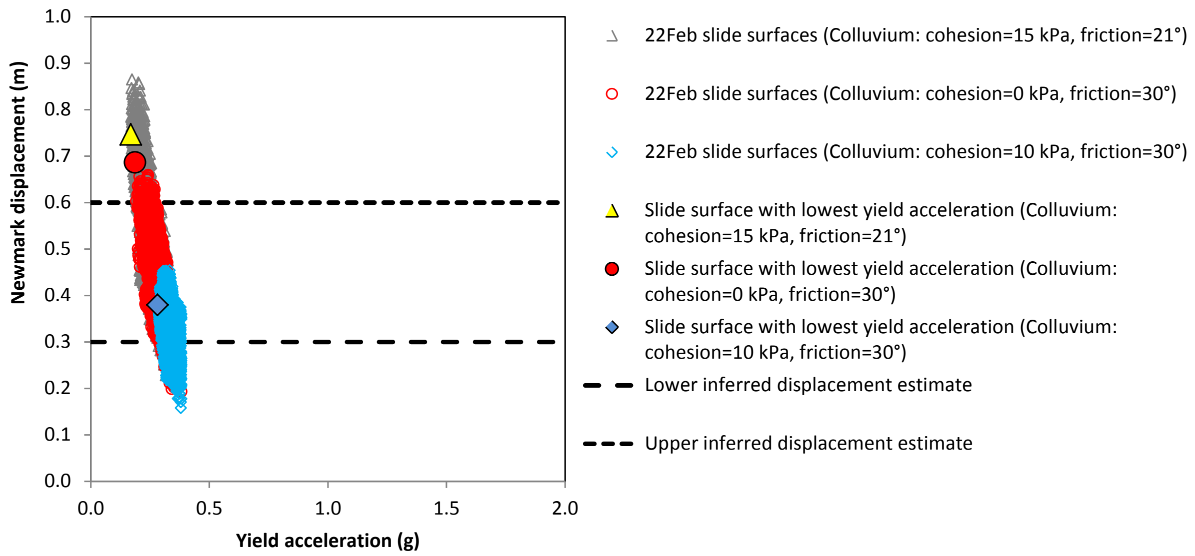


Figure 23 Failure mechanism 2. Modelled Slope/W decoupled displacements of cross-section 1 for the 22 February 2011 earthquake and adopting variable estimates of the material strength of the volcanic colluvium. Each datapoint represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic permanent displacement of the slope along the cross-section during the given earthquake.

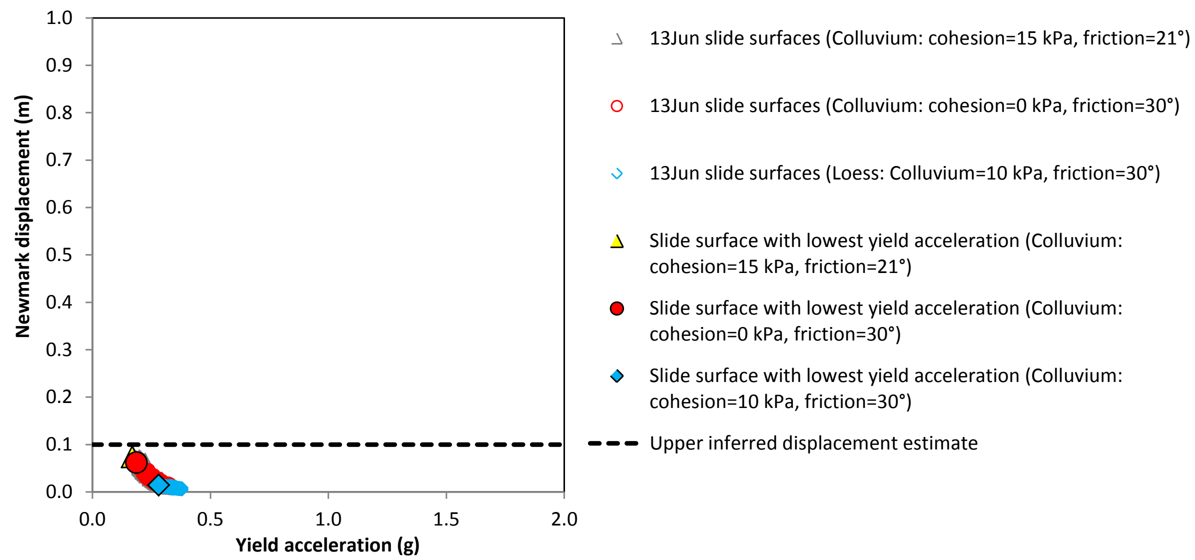


Figure 24 Failure mechanism 2.13 June 2011 earthquake, modelled Slope/W decoupled displacements for cross-section 1, and adopting variable estimates of the material strength of the volcanic colluvium. Each datapoint represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 13 June 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed line represents the inferred coseismic permanent displacement of the slope along the cross-section during the given earthquake.

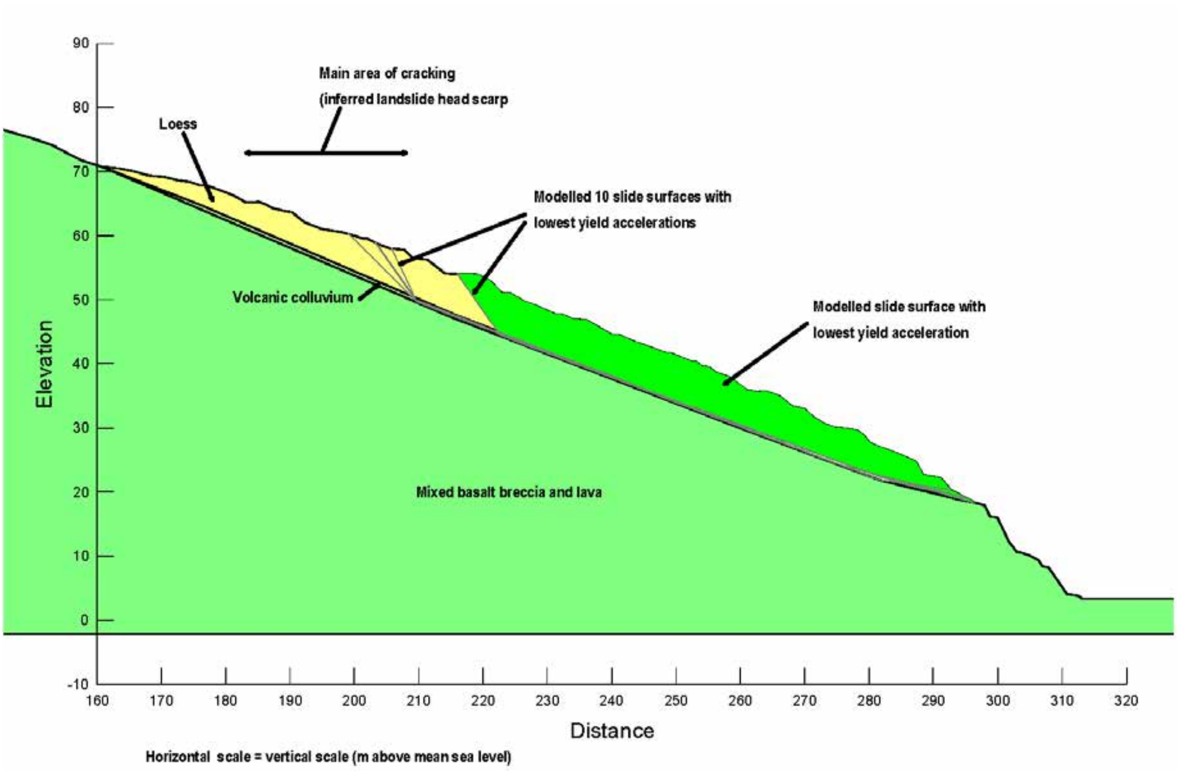


Figure 25 Results from the seismic slope stability assessment for failure mechanism 2, cross-section 1, for the 22 February 2011 earthquake.

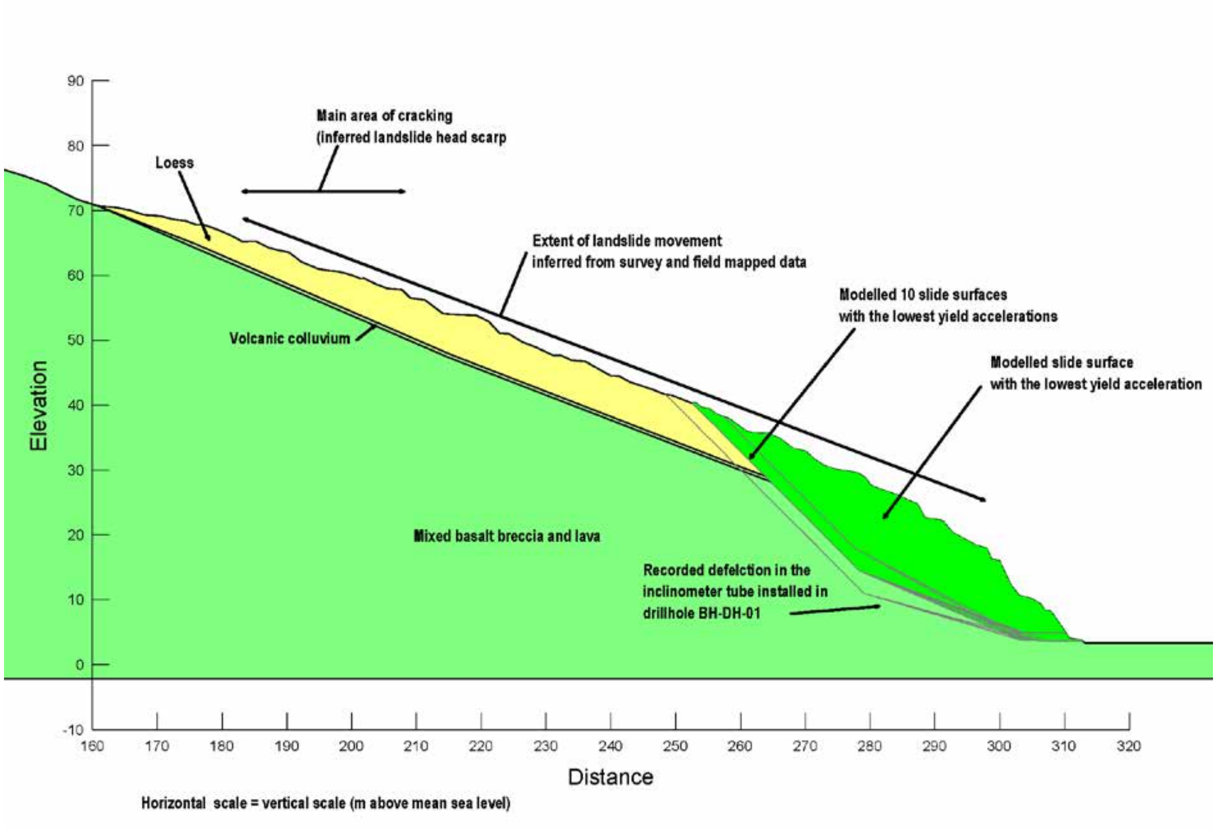


Figure 26 Results from the seismic slope stability assessment for failure mechanism 3, cross-section 1, for the 22 February 2011 earthquake.

The results show that:

4.2.2.1 Failure mechanism 2

- A good correlation between the recorded permanent coseismic displacements and modelled displacements of the slope for the 22 February and 13 June 2011 earthquakes was obtained for modelled slide surfaces adopting shear strength parameters for the volcanic colluvium of cohesion (c) of 1–15 kPa and friction (ϕ) of 21–30°;
- The lowest yield accelerations for the modelled slide-surface geometries were 0.17–0.19 g (adopting strength parameters for the volcanic colluvium of cohesion 0–15 kPa and friction (ϕ) of 21–30°);
- Modelled permanent displacements of the slope adopting the 16 April and 23 December 2011 earthquakes were less than 0.01 m, adopting shear strength parameters for the volcanic colluvium of cohesion (c) 0–15 kPa and friction (ϕ) of 21–30°;
- There is a good correlation between the locations and shape of the slide surfaces derived from the limit equilibrium and finite element static stability modelling, and those from the dynamic modelling; and
- The largest permanent slope displacements are for slide surfaces in the lower and middle part of the inferred landslide. These locations are consistent with those survey marks showing the largest recorded permanent slope displacements.

4.2.2.2 Mechanism 3

- The estimated displacements for mechanism 3, on its own, could not account for the total inferred displacement of the slope, even by adopting the lower shear strength parameters for the breccia (cohesion (c) of 50 kPa and friction (ϕ) of 25°), which are at the lower end of the range thought to be credible;
- The depth of the modelled slide surfaces is consistent with the deflection recorded in the inclinometer tube installed in drillhole BH-DH-01;
- However, it is still possible that movement through the rock did occur, but that any such displacements would be minor in comparison with those estimated for mechanism 2;
- Modelled permanent displacements in response to the modelled 16 April, 13 June and 23 December 2011 earthquakes were zero;
- The lateral persistence and strength of such a failure is uncertain and cannot be constrained by the current field mapping and drillholes alone; and
- The lowest yield acceleration for the modelled slide-surface geometries is about 0.29 g, which is higher than the yield accelerations for mechanism 2 adopting the lower shear strength parameters for the volcanic colluvium.

4.2.3 Forecast modelling of permanent slope displacement

4.2.3.1 Cross-section 1

Permanent displacements, from the decoupled assessment results from the 22 February and 13 June 2011 modelled earthquakes, were calculated for a range of slide-surface geometries with different ratios of yield acceleration (Ky) to the maximum average acceleration of the failure mass (K_{MAX}). The maximum average acceleration (K_{MAX}) was calculated for each selected slide surface by taking the maximum value of the average acceleration time history from the response to the synthetic earthquake. About 5–10 slide surfaces were chosen to represent the results from each earthquake input motion, adopting different estimates of the shear strength of the key materials (listed in Table 14).

The results from the assessment, adopting failure mechanism 2, are shown in Figure 27, for those slide surface shown in Figure 25. The results show that between Ky/K_{MAX} values of 0.1 and 0.6, and Ky/A_{FF} values of 0.2 and 0.6, the data are well fitted to a straight line (exponential trend line) in semi-log space. The coefficient of determination (R^2) is 0.95 for Ky/K_{MAX} and 0.84 for Ky/A_{FF}, and includes all of the plotted data (N = 30). The poorer coefficient of determination for ratios of Ky/A_{FF} is not unusual as Newmark (1965) displacements are highly sensitive to the high frequency components of the input motions, which can vary from event to event. By comparison, K_{MAX} “filters” the higher frequency components, and thus is less sensitive to the input motion characteristics.

The peak ground acceleration of the input motion (A_{FF}) does not take into account amplification effects caused by the slope geometry, and at this site, the material contrasts within the slope, between the loess/volcanic colluvium and the underlying rock (Appendix 4). From the data in Figure 27, the mean ratio of K_{MAX} to A_{FF} for cross-section 1 is 1.4 (±0.3 at one standard deviation), meaning that K_{MAX} is 1.7 times greater than the peak horizontal ground acceleration of the input motion, if assuming the mean plus one standard deviation of the mean.

For ratios of Ky/A_{FF} in Figure 27, the estimated magnitudes of displacement are consistent with those reported by Jibson (2007), where these data plot between the ranges for earthquakes of M6.5–7.5 as reported by Makdisi and Seed (1978) and plotted by Jibson (2007).

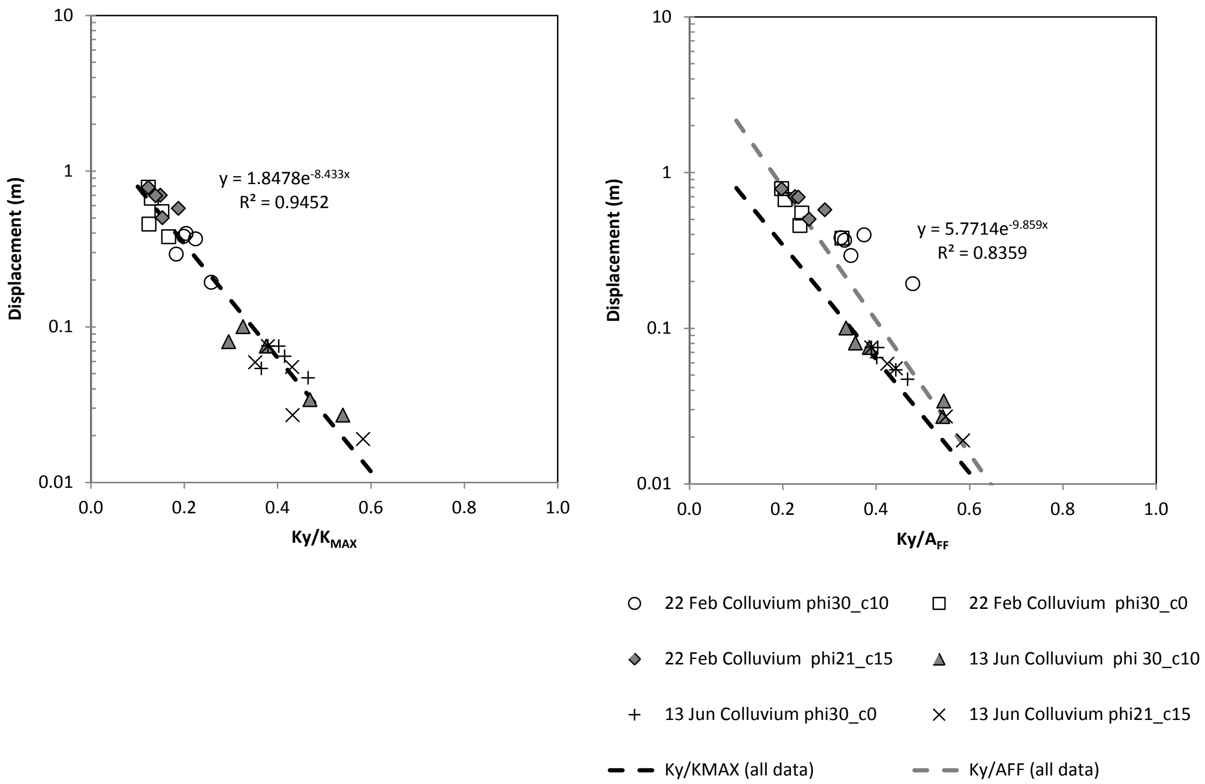


Figure 27 Cross-section 1, failure mechanism 2. Decoupled Slope/W displacements calculated for different ratios of yield acceleration to maximum average acceleration of the mass (Ky/K_{MAX}), and maximum acceleration of the mass (Ky/A_{MAX}), for selected slide-surface geometries, and given material shear strength parameters. A_{FF} is the peak acceleration of the input earthquake time acceleration history. Synthetic rock outcrop time acceleration histories for the 22 February and 13 June 2011 earthquakes were used as inputs for the assessment (N = 30). The dashed lines are exponential trend lines fitted to the semi–log data. The formula and the coefficient of determination (R^2) for the trend lines are shown.

The results from the decoupled assessment show that the magnitude of permanent slope displacement during an earthquake will vary in response to the:

1. shear strength of the loess and volcanic colluvium at the time of the earthquake;
2. failure mechanism;
3. pore pressures within the slope at the time of the earthquake; and
4. duration and amplitude of the earthquake shaking.

The relationship between the yield acceleration and the maximum average acceleration (from Figure 27) has been used to determine the likely range of displacements of a given failure mass with an adopted yield acceleration (*Ky*) at given levels of peak free field horizontal ground accelerations (A_{FF}) and the equivalent maximum average ground acceleration (K_{MAX}). The results are shown in Table 17. Conservative yield accelerations have been adopted to take into account the possibility that the current shear strength of the materials has degraded as a result of the past movement.

Displacement of the slide mass will not occur at maximum average accelerations (K_{MAX}) less than the critical yield acceleration. However, the critical yield acceleration depends upon the strength of the slide surface and any pore pressures present at the time of the earthquake.

Table 17 Forecast modelling results from the dynamic slope stability assessment for cross-section 1. Estimated displacements are rounded to the nearest 0.1 m.

Cross-section	1			
Adopted yield acceleration (*Ky*) (g)	0.17			
Free field peak ground acceleration (A_{FF}) (g)	0.2	0.5	0.7	1.0
Adopted K_{MAX} to A_{FF}[1] ratio	1.7 (mean + 1 standard deviation)			
Equivalent K_{MAX}	0.3	0.9	1.2	1.7
Estimated displacements (m)	<0.1	0.4	0.6	0.8

1 A_{FF} represents the peak horizontal ground acceleration of the free field input motion.

4.2.4 Cross-section 3

A simple assessment of the stability of the slope under dynamic conditions was carried out adopting the pseudostatic method of assessment to determine the critical yield acceleration of the slope. The critical yield acceleration of a given slide mass is the minimum pseudostatic acceleration required to produce instability of the mass (Kramer, 1996).

The results are shown in Figure 28 for various fill shear strength parameters. The critical yield acceleration of the slope varies between about 0.25 and 0.8 for the range of parameters assessed (Table 16).

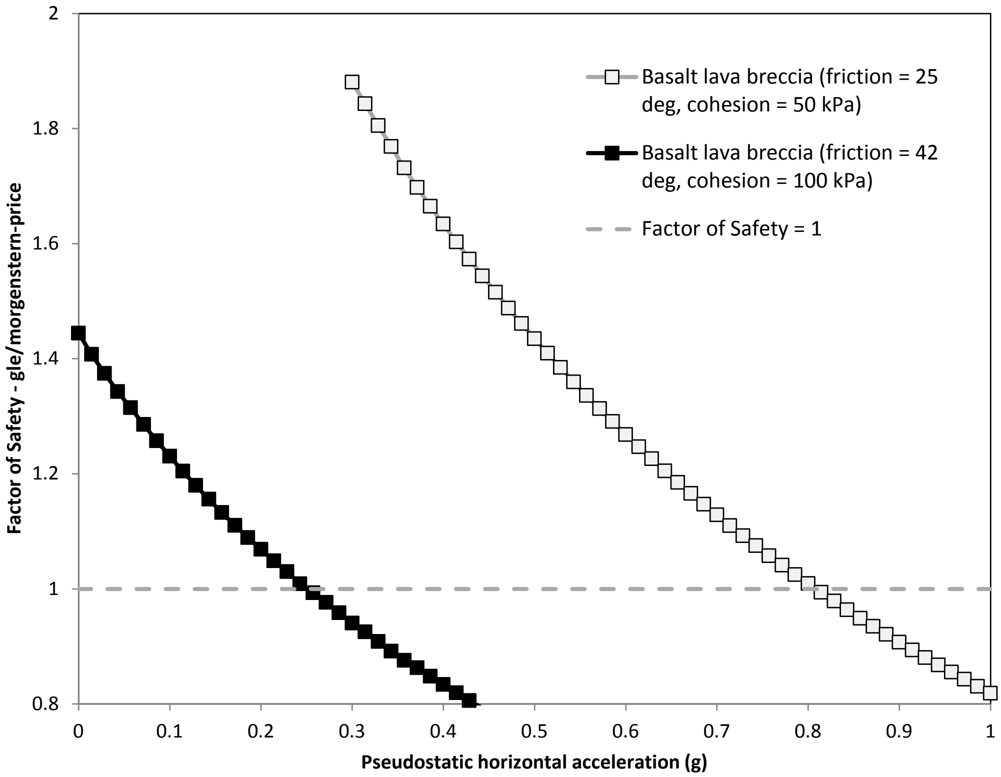


Figure 28 Yield acceleration of the slope based on variable fill parameters and loess parameters shown in Table 11. Yield acceleration calculated adopting the pseudostatic method (Kramer, 1996).

4.3 SLOPE STABILITY – SUMMARY OF RESULTS

4.3.1 Cross-section 1

Under current conditions, it is possible for failure of the trial slide surfaces to occur under either static or dynamic conditions. However, it should be noted that material strengths – and therefore the slope factors of safety – may vary through time (weathering), water content, and further movement of the slope under either static or dynamic conditions. The main findings from the assessments are:

1. The modelled slide surfaces through the volcanic colluvium (mechanism 2), underlying the loess, best simulated the locations of cracks and compression features mapped in the field.
2. Based on the dynamic back-analysis of slope stability the minimum values of friction (ϕ) and cohesion (c) of the colluvium (and loess) are about 21–30° and 0–15 kPa, to achieve the recorded displacements of the slope during the 2010/11 Canterbury earthquakes inferred from survey results. These are likely to be at the upper end of the range considered to be reasonable as they represent summer water content conditions, i.e., dry. The static factor of safety of the assessed slope, under dry conditions, is therefore thought to range between 1.2 and 1.3 for the failure mechanisms and range of material parameters assessed.
3. Given the relatively low static factors of safety, an increase in pore water pressure (piezometric head levels of about two to four metres above rockhead) within the colluvium and loess and or within open tension cracks, could lead to instability of the slope under static conditions (i.e., short duration high intensity rain, and or longer periods of wet weather). Changes in the water content of the materials could also lead to a reduction in the cohesion and therefore the static factor of safety.
4. Given the relatively low yield acceleration of the slope (estimated to be in the range of about 0.1–0.3 g) it is likely that future earthquakes could reactivate the slope, leading to permanent displacements that could be quite large. The magnitude of any coseismic permanent displacements will depend upon:
    a. The shear strength of the materials at the time of the earthquake;
    b. The pore pressure/water content conditions within the slope at the time of the earthquake as affected by antecedent rainfall; and
    c. The duration and amplitude of the earthquake shaking at the site.
5. Rainfall-induced failures are likely to be more mobile however, and the return period of the triggering event more frequent, and therefore pose a greater risk than earthquake induced failures.

4.3.2 Cross-section 3

1. The modelled slide surfaces through the mixed basalt lava breccia (mechanism 3), underlying the loess and colluvium, simulated best the displacement directions and angles inferred from crack apertures, and damage recorded at the toe of the slope.
2. Given the relatively low static factors of safety (estimated to be in the range of 1.3–2.7), a small increase in pore water pressures (piezometric head levels) within open tension and cracks and joints in the rock mass, could lead to instability of the slope under static conditions (i.e., short duration high intensity rain, and or longer periods of wet weather.

3. Given the relatively low yield acceleration of the slope (estimated to be in the range of 0.25–0.8 g based on the pseudostatic method) it is likely that future earthquakes could also cause the slope to fail.

Cliff collapses are anticipated to occur from anywhere along the steep cliff. The risk from cliff-collapse hazards has already been addressed for this site by Massey et al. (2012). No re-assessment of the risk from these hazards has been carried out in this report.

4.4 POTENTIAL SOURCE VOLUME ESTIMATION

The likely locations and volumes of potential source areas (1 and 2), adopting failure mechanism 3, have been estimated based on:

1. Numerical stability analyses results;
2. Mapped crack distributions relating to the 2010/11 Canterbury earthquakes; and
3. Engineering geology and morphology of the slope.

Three possible failure volume estimates – lower, middle and upper range estimates – have been calculated for each potential source area. The variation in failure volumes reflects the uncertainty in the source shape (depth, width and length dimensions) estimated from site conditions and the modelling.

Volumes were calculated by estimating the shape of any future failures by calculating their surface areas and multiplying this by the average depth to rockhead. A rounding factor of 0.7 was then applied to the volume to take into account variations in the source area depth around the margins. Estimated volumes are shown in Table 18.

Table 18 Example of estimated source volumes (the first digit in the number is significant) and fahrboeschung angles.

	Source volume estimation					Fahrboeschung angle	
Source area	Source volume	Source surface area (m)	Average depth (m)	Rounding factor applied	Volume (m^3)	Mean	Mean – 1 STD
1	LOWER	90	5	0.7	320	23.2	15.2
	MID	580	6	1	3,500	18.5	11.9
	UPPER[1]	6,860	6	1	40,000	14.5	9.1
2	LOWER	62	4	0.7	180	24.6	16.2
	MID	503	3	1	1,500	20.1	13.0
	UPPER	1491	3	1	4,500	18.0	11.5

[1] The upper volume estimate is based on the entire cracked area (source area 1, Figure 2)

For source areas 1 and 2 the credibility of the earth/debris flow potential source volumes have been evaluated by comparing them against estimated volumes of individual landslides in loess and loess derivative materials, such as colluvium in the Port Hills, mapped by Townsend and Rosser (2012). The distribution of the 124 landslides is shown in Figure 29, and the data are well modelled by a log-normal distribution, adopting the area depth relationships of Larsen et al. (2010).

The range of estimated volumes in Table 18 is well within the range of this datasets.

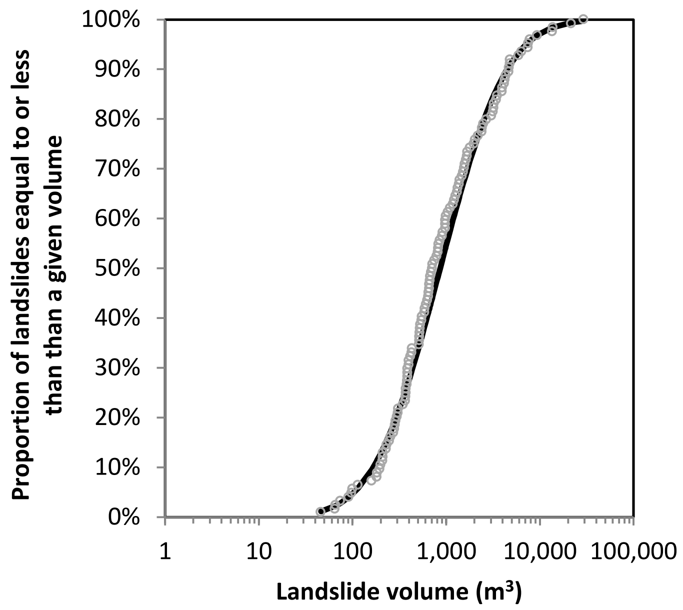


Figure 29 Estimation of landslide volumes in the Port Hills loess from Townsend and Rosser (2012), adopting the area depth relationships of Larsen et al. (2010).

4.5 RUNOUT DISTANCE

The debris runout distance from the identified potential source areas was assessed both empirically and numerically.

4.5.1 Empirical method

The procedure followed for estimating the empirical run out distance, in terms of the fahrboeschung angle, is detailed in Appendix 1.

Figure 30 shows the estimated mean and mean minus one standard deviation debris flow fahrboeschung angles derived for assessed source areas 1 and 2. For each source area a range in fahrboeschung angles is estimated based on the range in volumes (lower, middle and upper) as shown in Table 18.

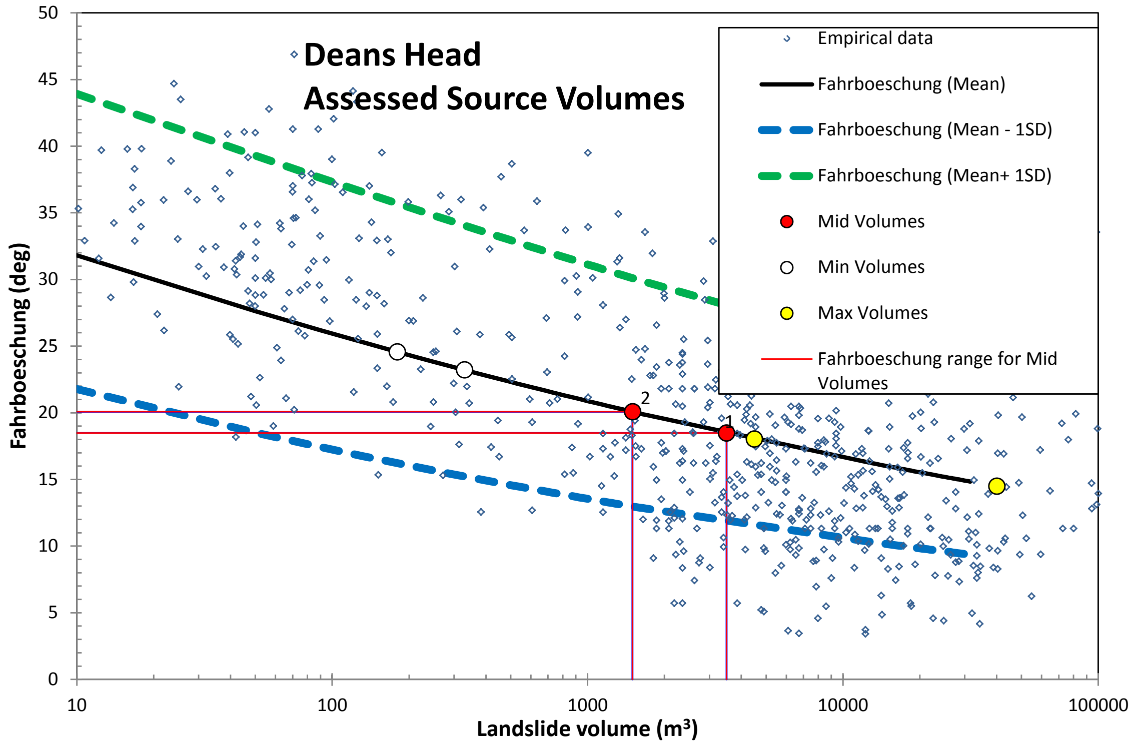


Figure 30 Estimation of debris flow fahrboeschung angles based on empirical runout data presented in Massey and Carey (2012).

4.5.2 Runout distance (Numerical method)

The physical model of RAMMS Debris Flow uses the Voellmy friction law. This model divides the frictional resistance into two parts: 1) a dry-Coulomb type friction (coefficient μ) that scales with the normal stress; and 2) a velocity-squared drag or viscous-turbulent friction (coefficient xi). RAMMS software takes into account the slope geometry of the site when modelling debris runout.

The RAMMS model parameters were calculated from the back-analysis of four Port Hills debris flows. The modelled parameters μ and x_i were optimised to obtain a good correlation between the modelled versus actual runout and deposited debris heights. The model, with calibrated input parameters (μ = 0.06 (7°) and xi = 200 m/s^2), were used to estimate the likely velocity and depth of the debris at given locations down the slope for the given failure volumes. The μ and xi values are comparable to results from other assessments compiled by Andres (2010) for debris flows (Figure 31).

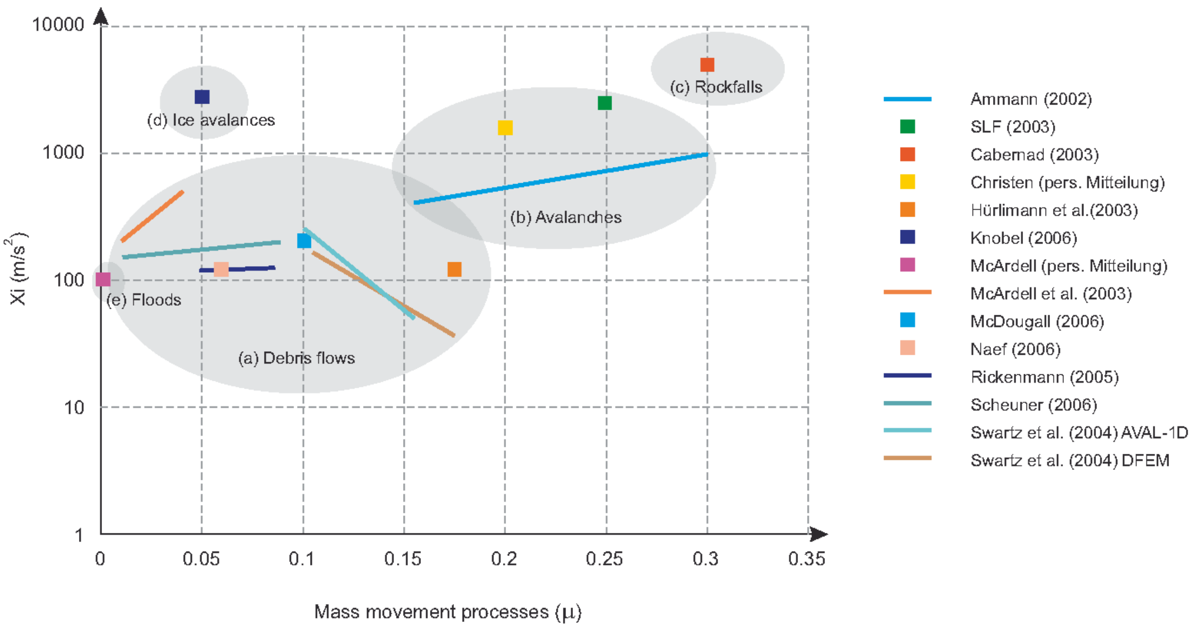
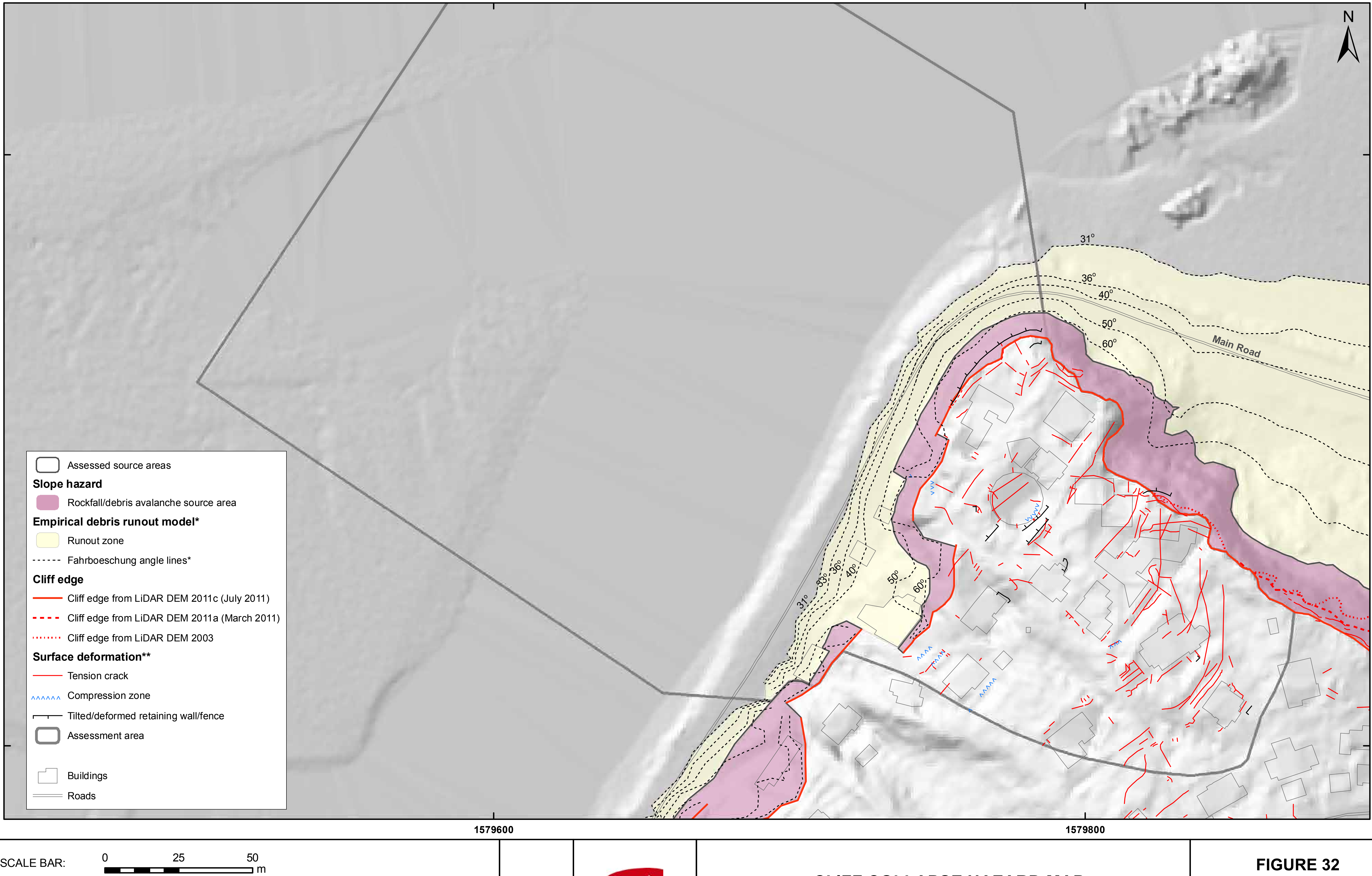


Figure 31 Range of parameters for different assessed source areas processes: **a**) debris flows, **b**) snow avalanches, **c**) rockfalls, **d**) ice avalanches, **e**) debris floods. Modified from Andres (2010).

4.5.3 Forecast runout modelling

A hazard map (Figure 32) presents the empirical and numerical runout limits from the modelling. The mean and mean minus one standard deviation fahrboeschung angles for each source area assuming the upper volume estimates, are shown. The estimated runout distances from RAMMS are shown in Appendix 5 (debris height) and Appendix 6 (debris velocity), for source areas 1 and 2 (upper, middle and lower source volume estimates), along with the corresponding mean and mean minus one standard deviation fahrboeschung angles.



EXPLANATION:

* Modified from reports CR2012/57 and CR2012/124
** Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

CLIFF COLLAPSE HAZARD MAP

Deans Head Christchurch

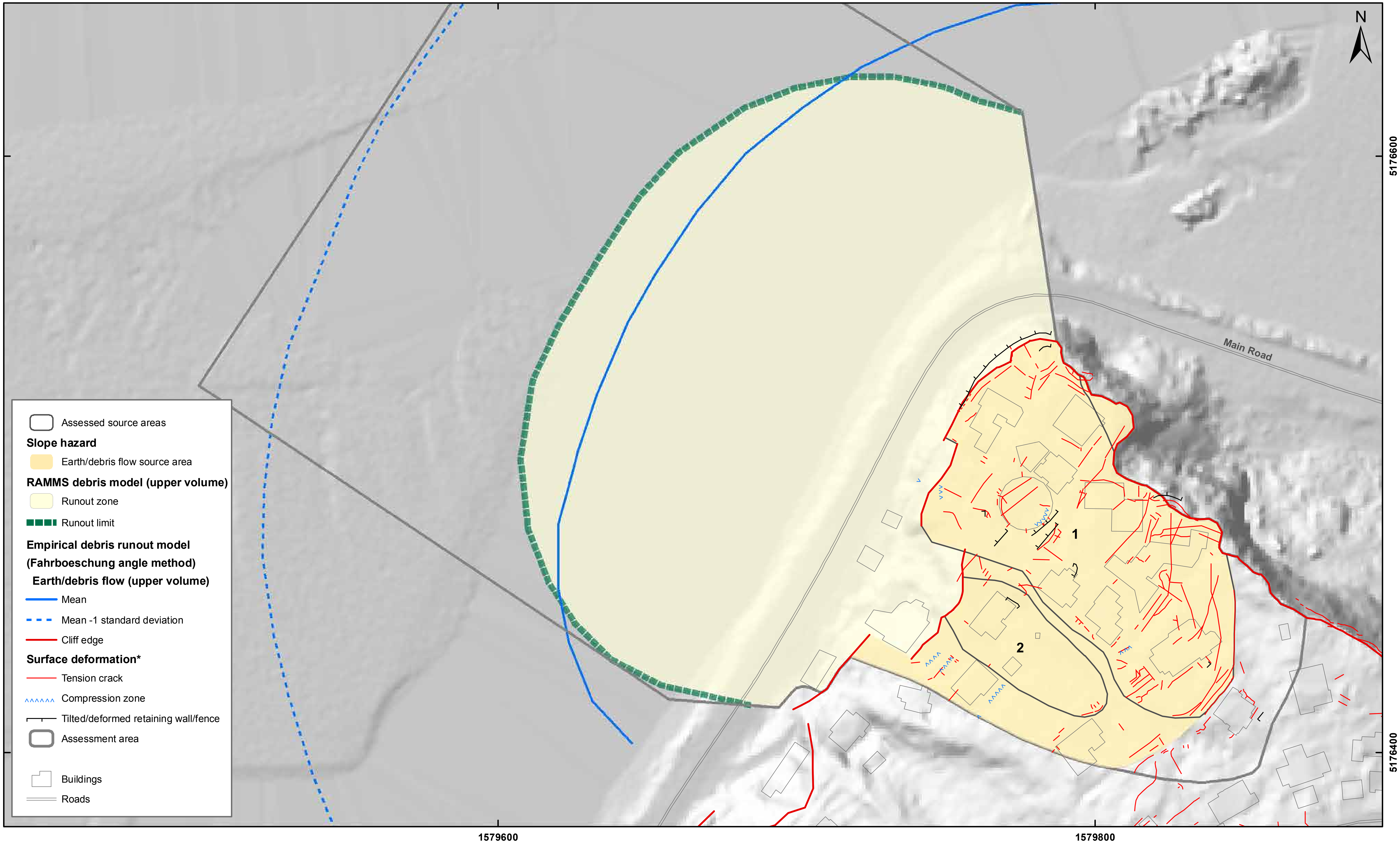
FIGURE 32

Map 1

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 25 50 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, FDP

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EARTH/DEBRIS FLOW HAZARD MAP

Deans Head Christchurch

FIGURE 32

Map 2

FINAL

REPORT: CR2014/77

DATE: June 2014

5.0 RISK ASSESSMENT RESULTS

5.1 TRIGGERING EVENT FREQUENCIES

Failure of the assessed sources could be triggered by earthquakes (dynamic conditions) or by water ingress (static conditions).

5.1.1 Frequency of earthquake triggers

For earthquake triggers, the frequency of a given free-field peak ground acceleration occurring is obtained from the National Seismic Hazard Model for New Zealand (Table 19) (Stirling et al., 2012). The increased level of seismicity in the Christchurch region is incorporated in a modified form of the 2010 version of the National Seismic Hazard Model (Gerstenberger et al., 2011).

5.1.1.1 Peak ground acceleration and permanent slope displacement

For these assessments, peak ground acceleration is used to represent earthquake shaking intensity, as peak ground acceleration is the ground-motion parameter directly related to coseismic landslide initiation (Wartman et al., 2013).

The estimated magnitude of permanent slope displacement of the assessed sources in a future earthquake was based on the decoupled assessment results. The permanent displacement of each source at a given level of free-field peak ground acceleration (A_{FF}) was estimated from the relationship between the yield acceleration (Ky) and the maximum average acceleration of the mass (K_{MAX}) (Figure 27). Different levels of peak ground acceleration were adopted, and each multiplied by the site-specific ratio of K_{MAX} to A_{FF} (assuming the mean plus one standard deviation) to estimate the equivalent maximum average acceleration of the mass (K_{MAX}) for the given value of A_{FF}. For example, an A_{FF} of 0.2 g would have an equivalent K_{MAX} of 0.3 g, assuming a ratio of 1.7.

Table 19 The annual frequency of a given peak ground acceleration occurring on rock (site class B) for different years adopting the 2012 National Seismic Hazard Model (NSHM) for Christchurch (Gerstenberger et al., 2011), and the associated estimated permanent displacement of cross-section 1. Note: these are free-field rock outcrop peak ground accelerations (equivalent to A_{FF}).

Free field peak horizontal ground accelerations (A_{FF})[1] (g)	**0.2**	**0.5**	**0.7**	**1.0**
Year 2016 annual frequency of event (from NSHM)	0.090	0.0157	0.0059	0.00164
Year 2016 return period (years)	11	64	169	610
Next 50-year average annual frequency of event (from the NSHM)	0.042	0.0072	0.0027	0.00076
Next 50-year average return period (years)	24	139	370	1316
Adopted K_{MAX}[2] to A_{FF} ratio	1.7 (mean + 1 STD)			
Equivalent K_{MAX} for the given A_{FF}	0.3	0.9	1.2	1.7
Estimated displacements (m) for Yield acceleration (*Ky*) of 0.1 (g)	**0.2**	**0.7**	**0.9**	**1.1**
Estimated displacements (m) for Yield acceleration (*Ky*) of 0.2 (g)	**<0.1**	**0.3**	**0.5**	**0.7**
Estimated displacements (m) for Yield acceleration (*Ky*) of 0.3 (g)	**0**	**0.1**	**0.2**	**0.4**

1 A_{FF} represents the peak horizontal ground acceleration of the free field synthetic input motion.

2 K_{MAX} represents the maximum average acceleration of the failure mass taken from the relationship in Figure 26.

5.1.1.2 Permanent slope displacement and likelihood of catastrophic slope failure

It is difficult to estimate the probability of triggering failure leading to catastrophic slope collapse where the debris runs out down slope forming an earth/debris flow. It is possible that permanent slope displacements could cause catastrophic damage to dwellings located on the assessed source areas, even if the debris does not leave the source. The predicted levels of displacement that have been used to differentiate between safe and unsafe behaviour (Abramson et al., 2002) range from 0.05 m to 0.3 m. Some examples are:

a. Hynes-Griffin and Franklin (1984) suggest that up to 0.1 m displacements may be acceptable for well-constructed earth dams.

b. Wieczorek et al. (1985) used 0.05 m as the critical parameter for a landslide hazard map of San Mateo County, California.

c. Keefer and Wilson (1989) used 0.1 m for coherent slides in southern California.

d. Jibson and Keefer (1993) used a 0.05–0.1 m range for landslides in the Mississippi Valley.

e. The State of California (1997) finds slopes acceptable if the Newmark displacement is less than 0.15 m. A slope with a Newmark displacement greater than 0.3 m is considered unsafe. For displacements in the “grey” area between 0.15 and 0.3 m, engineering judgement is required for assessment.

Permanent slope displacements during the 2010/11 Canterbury earthquakes estimated from survey marks were about 0.5–0.7 m, and the slope did not fail catastrophically (i.e., with the debris running out as an earth/debris flow). This did not mean the slope was “safe” however, as the dwellings located in the assessed source areas still suffered significant damage.

The relationship in Figure 27 is based on past performance of the slope. However, the slope moved about 0.7 m during the 2010/11 Canterbury earthquakes. It is now possible that catastrophic failure of the slope could occur at further permanent slope displacements of less than 0.7 m. At free-field peak ground accelerations of greater than 1.0 g the amount of permanent displacement could lead to catastrophic failure and runout of debris, as performance of the slope at these levels of ground acceleration is unknown. At these ratios of Ky/K_{MAX} (less than 0.1) the magnitude of displacement tends to increase rapidly with only relatively small changes in the Ky/K_{MAX} ratio.

The annual frequency of such a peak ground acceleration (A_{FF} of 1.0 g) occurring is 0.00164 (once in every 610 years) adopting the year 2016 National Seismic Hazard Model results, and 0.00076 (once in every 1,320 years) adopting the 50-year average National Seismic Hazard Model results.

It should be noted that the displacements at different ratios of Ky/K_{MAX} were calculated using the synthetic earthquake acceleration time histories for the 22 February and 13 June 2011 earthquakes, which were both near-field earthquakes of short duration but high amplitude. The calculated displacements in Figure 27 represent displacements in response to these earthquakes (adopting failure mechanism 2). Earthquakes of longer duration may affect the site in different ways. For example, the response of the loess and volcanic colluvium (at higher water contents representative of winter conditions) may be non-linear and could lead to larger permanent displacements. Conversely, the peak amplitudes relating to longer duration earthquakes from more distant sources are likely to be lower and may not trigger displacement of the slope.

It should also be noted that parts of the slope have already moved more than 0.5 m during the 2010/11 Canterbury earthquakes and it is not known how much more movement the slope can take before failing catastrophically.

5.1.1.3 Deaggregation of the National Seismic Hazard Model

The seismic performance of the slope in future earthquakes was inferred from assessing its performance in past earthquakes, mainly the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, using the relationship established between peak ground acceleration and the amount of permanent slope displacement. These earthquakes varied in magnitude between M5.2 and M6.3 and were "near-field", i.e., their epicentres were very close, within 5 km, of the Deans Head site.

The annual frequencies of a given level of peak ground acceleration occurring in the area are given by the National Seismic Hazard Model (Stirling et al., 2012). The National Seismic Hazard Model combines all of the various earthquake sources that could contribute to the seismic hazard at a given location. The National Seismic Hazard Model estimates for the Port Hills are based on a combination of different earthquake sources: 1) subduction interfaces; 2) mapped active faults; and 3) unknown or "background" earthquakes. For the risk assessment it is important to deaggregate the national seismic hazard model to assess which earthquake sources contribute the most to it.

R. Buxton and G. McVerry (personal communications 2014) suggest that it is magnitude M5.3–6.3 earthquakes on unknown active faults within 20 km of the site that contribute most to the seismic hazard. These earthquakes are similar to the 22 February, 16 April 13 June and 23 December 2011 earthquakes.

5.1.2 Frequency of rainfall triggers

The return period of the rainfall that could initiate failure is unknown because:

- There is evidence of historical and prehistoric earth/debris flows at the site;
- The 5 March 2014 rainstorm in Lyttelton (130 mm) triggered several large earth/debris flows. The return period of the rainfall at Lyttelton was about 100 years, but the lower amount of rainfall at the site (89 mm) had a return period of only about 10–20 years;
- It is likely that the slope could fail if the water content of the loess increases, but the likelihood of this happening is not known; and
- Even though there was no recorded movement of the slope during the 10–20 year return period event, the water content of the loess and colluvium at the time of this event was likely to have been seasonally low as the storm occurred at the end of a dry summer.

It is therefore difficult to estimate the annual frequency of the event that could initiate catastrophic failure of the assessed source areas.

5.1.3 Overall triggering event frequency

Given the previous results, rainfall-induced earth/debris flows are likely to be more mobile and the return period of the triggering event more frequent than earthquake-induced failures, and therefore pose the greatest risk.

For rainfall (static) triggers:

- For the risk assessment, various return periods of 20, 50, 100 and 200 years for the triggering event were assumed, and the sensitivity of the risk estimates to these return periods assessed.
- Failures of the slope could occur from anywhere within the identified source area and could vary greatly in volume. The assessed source areas represent the geometries and volumes of the sources that could potentially fail forming earth/debris flows.

It should be noted that under dynamic conditions (earthquakes) permanent displacement (slumping and cracking) of the currently cracked area could also occur, which could still pose a risk to any dwellings located in this area.

5.2 DWELLING OCCUPANT RISK

The results from the risk assessment are shown in Figure 33 (Maps 1–3) as the annual individual fatality risk. Map 1 shows the original annual individual fatality risk estimated for cliff collapses (debris avalanches and cliff top recession), modified from Massey et al. (2012). Map 2 shows the estimated annual individual fatality risk from the assessed earth/debris flows. Map 3 shows the annual individual fatality risk from combining the results shown in Maps 1 and 2, to produce a map showing the total risk from the combined different hazards present at the site.

5.2.1 Earth/debris flows

The risk from earth/debris flows from source areas 1 and 2, adopting the estimated lower, middle and upper source volumes, and a return period of 50 years for the triggering event, are shown in Figure 33, Map 1.

A 10-m wide strip is added at the crest of these source areas, to allow for the future retrogression of the sources in an up-slope direction, beyond the currently assessed extent. This has been termed an “earth/debris flow source 10 m enlargement area”. The risk in this area has not been assessed.

Three annual individual fatality risk lines representing the position of the 10^{-4} risk contour are shown on the map for the upper, middle and lower volumes, assuming a 50-year return period. The area shown as the “greater than 10^{-4} (all volumes)” represents the area of slope where the risk could be greater than 10^{-4} for all assessed failure volumes.

The area shown as the “10^{-4} uncertainty zone” represents the area of slope where the risk could be greater than 10^{-4} for the upper source volume, but less than 10^{-4} for the lower source volume and equal to or greater than 10^{-4} for the middle source volume.

The area of slope beyond (further away from the assessed source areas) the 10^{-4} uncertainty zone but within the assessed extent of debris runout represents the area of slope, within the runout zone, where the annual individual fatality risk has been assessed as being less than 10^{-4} for all source volumes.

At Deans Head, much of the debris from the assessed source areas extends across Main Road and into the sea. Therefore, differences in the position of the risk lines caused by changes in the source volume and event annual frequency have little impact on the area at risk.

Nearly all of the dwellings potentially at risk are located in the assessed source areas. The risk associated with the assessed source areas is inferred to be the same as the risk in the runout zone immediately below the assessed source areas, which is shown as 10^{-4} or greater.

5.2.1.1 Other variables adopted for the risk assessment

Other variables used in the risk assessment were discussed at a workshop with Christchurch City Council on 18 March 2014. Based on the results from the workshop the risk estimates presented in Figure 33 adopt the following main variables:

- $P_{(H)}$ annual frequency of the triggering event of 0.02 (once every 50 years).
- $P_{(S:H)}$ the probability that a person, if present, is in the path of the debris is based on variable (lower, middle and upper) estimates of the debris volume and height, that could be triggered in an event.
- $P_{(T:S)}$ the probability that a person is present at a particular location, as the debris moves thought it, of 67%. Assuming an "average" person spends 16 hours a day at home. For this assessment, GNS Science has assumed the same "average" occupancy rate value adopted by the Canterbury Earthquake Recovery Authority.
- $V_{(D:T)}$ the vulnerability of a person, if present and inundated by debris, is a function of the debris velocity. A variable vulnerability of between 0 and 100% has been adopted.

5.2.2 Combined risk

The total risk from the combined hazards is shown in Map 3 of Figure 33. The results combine the risk presented by Massey et al. (2012) from cliff collapse of the slope with the annual individual fatality risk estimated adopting the middle source volume estimates for the earth/debris flow hazards (assessed source areas 1 and 2).

The original risk assessment by Massey et al. (2012) was updated to make it consistent with the input parameters used in the risk assessments contained in this report and other Stage 2 and 3 reports; these comprised:

1. Annual frequency of an earthquake triggering event: For this assessment GNS Science has adopted the year 2016 National Seismic Hazard Model annual frequencies for earthquake peak ground accelerations.
2. Probability of a person being present: For this assessment, GNS Science has assumed the same "average" occupancy rate value adopted by the Canterbury Earthquake Recovery Authority, i.e., that an average person spends on average 16 hours a day at home (16/24 = 0.67 or 67%).
3. Vulnerability of a person if present and hit by debris: For this assessment GNS Science has adopted a constant vulnerability factor of 70% as it was the factor adopted by the Canterbury Earthquake Recovery Authority for the previous risk assessments.

Annual individual fatality risk bands (e.g. 10^{-3} to 10^{-4}) – The risk of being killed in any one year is expressed as a number such as 10^{-4} ("ten to the minus four"). 10^{-4} can also be expressed as one chance in 10,000 of being killed in any one year.

Cliff collapse - Includes debris avalanche and cliff recession hazards.

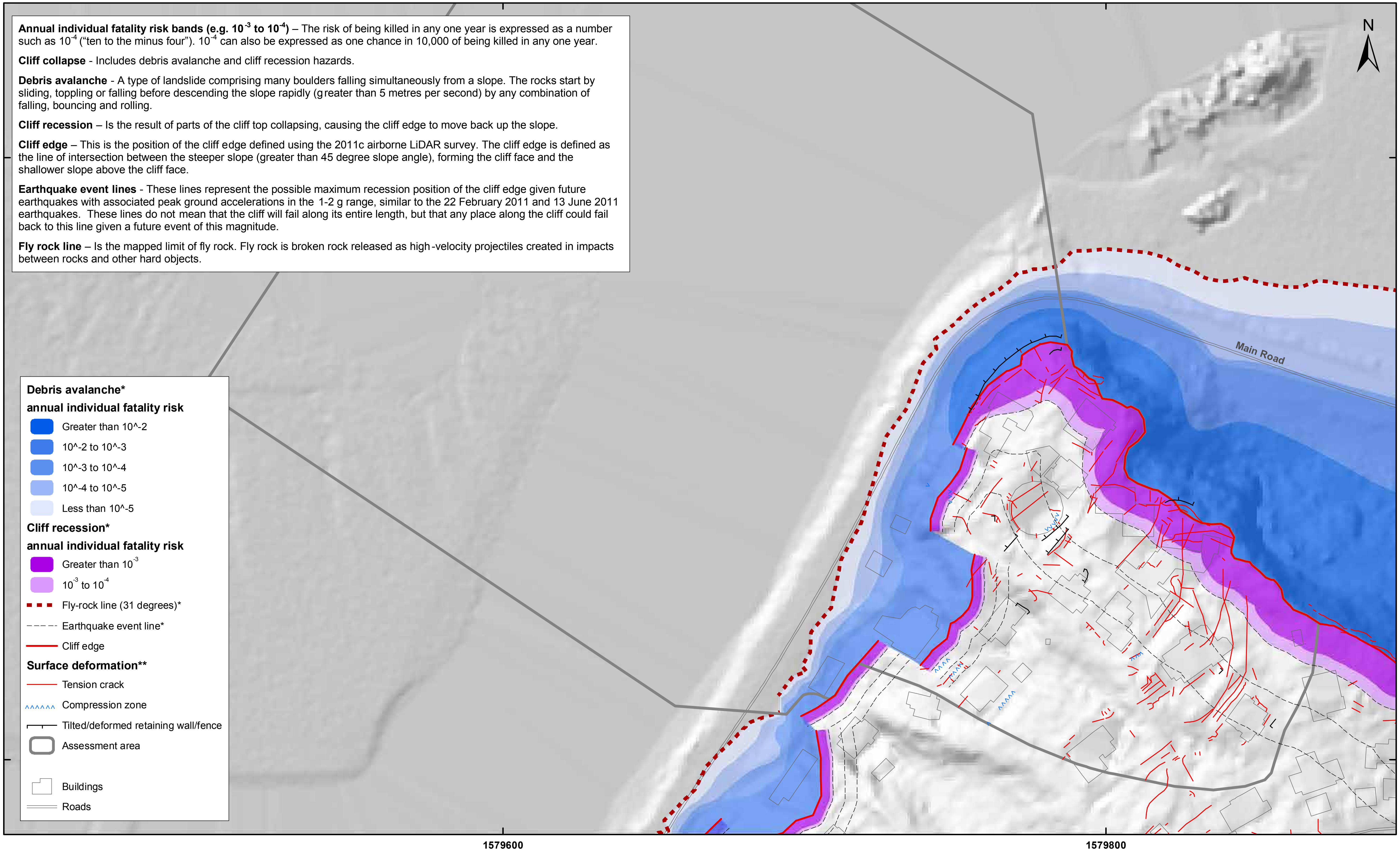
Debris avalanche - A type of landslide comprising many boulders falling simultaneously from a slope. The rocks start by sliding, toppling or falling before descending the slope rapidly (greater than 5 metres per second) by any combination of falling, bouncing and rolling.

Cliff recession – Is the result of parts of the cliff top collapsing, causing the cliff edge to move back up the slope.

Cliff edge – This is the position of the cliff edge defined using the 2011c airborne LiDAR survey. The cliff edge is defined as the line of intersection between the steeper slope (greater than 45 degree slope angle), forming the cliff face and the shallower slope above the cliff face.

Earthquake event lines - These lines represent the possible maximum recession position of the cliff edge given future earthquakes with associated peak ground accelerations in the 1-2 g range, similar to the 22 February 2011 and 13 June 2011 earthquakes. These lines do not mean that the cliff will fail along its entire length, but that any place along the cliff could fail back to this line given a future event of this magnitude.

Fly rock line – Is the mapped limit of fly rock. Fly rock is broken rock released as high-velocity projectiles created in impacts between rocks and other hard objects.



SCALE BAR: 0 50 100 m

EXPLANATION:

* Modified from reports CR2012/57 and CR2012/124
** Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: FDP, CM

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CLIFF COLLAPSE
ANNUAL INDIVIDUAL FATALITY RISK
(From GNS science reports CR2012/57 & CR2012/124)

Deans Head
Christchurch

FIGURE 33

Map 1

FINAL

REPORT: CR2014/77

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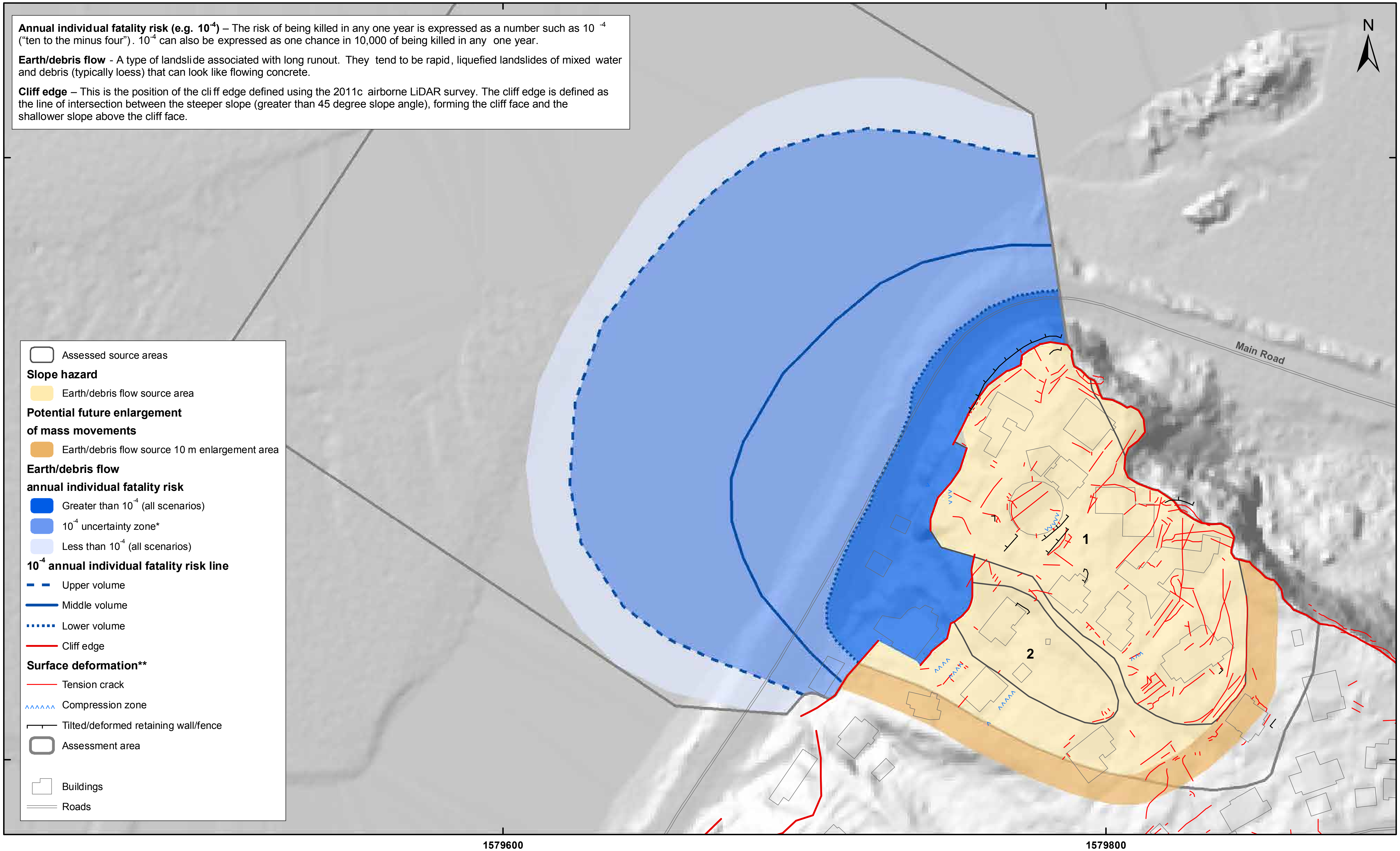

SCALE BAR: 0 50 100 m

EXPLANATION:

* Greater than 10^{-4} for upper volume, greater or less than 10^{-4} for the middle volume but below 10^{-4} for the lower volume
** Taken from report CR2012/317
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL
CHK: FDP, CM

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EARTH/DEBRIS FLOW ANNUAL INDIVIDUAL FATALITY RISK

Deans Head Christchurch

FIGURE 33

Map 2

FINAL

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Annual individual fatality risk bands (e.g. 10^{-3} to 10^{-4}) – The risk of being killed in any one year is expressed as a number such as 10^{-4} ("ten to the minus four"). 10^{-4} can also be expressed as one chance in 10,000 of being killed in any one year.

Earth/debris flow - A type of landslide associated with long runout. They tend to be rapid, liquefied landslides of mixed water and debris (typically loess) that can look like flowing concrete.

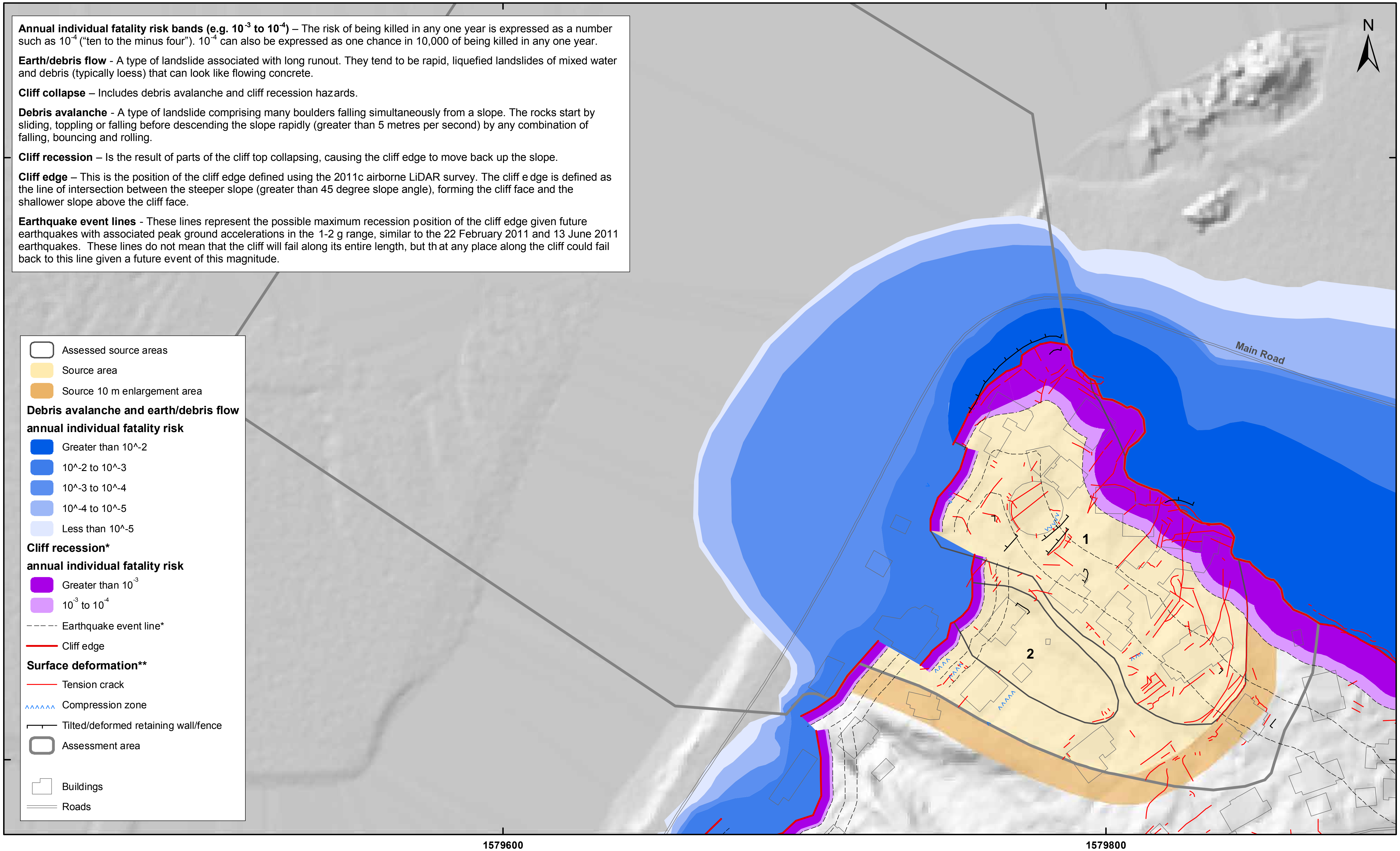
Cliff collapse – Includes debris avalanche and cliff recession hazards.

Debris avalanche - A type of landslide comprising many boulders falling simultaneously from a slope. The rocks start by sliding, toppling or falling before descending the slope rapidly (greater than 5 metres per second) by any combination of falling, bouncing and rolling.

Cliff recession – Is the result of parts of the cliff top collapsing, causing the cliff edge to move back up the slope.

Cliff edge – This is the position of the cliff edge defined using the 2011c airborne LiDAR survey. The cliff e dge is defined as the line of intersection between the steeper slope (greater than 45 degree slope angle), forming the cliff face and the shallower slope above the cliff face.

Earthquake event lines - These lines represent the possible maximum recession position of the cliff edge given future earthquakes with associated peak ground accelerations in the 1-2 g range, similar to the 22 February 2011 and 13 June 2011 earthquakes. These lines do not mean that the cliff will fail along its entire length, but th at any place along the cliff could fail back to this line given a future event of this magnitude.



SCALE BAR: 0 50 100 m

EXPLANATION:
* Modified from reports CR2012/57 and CR2012/124
** Taken from report CR2012/317
The results combine the annual individual fatality risk modified from reports CR2012/57 and CR2012/124 with the annual individual fatality risk from source areas 1 and 2, adopting Scenario B
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: FDP, CM

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COMBINED CLIFF COLLAPSE AND EARTH/DEBRIS FLOW ANNUAL INDIVIDUAL FATALITY RISK

Deans Head
Christchurch

FIGURE 33

Map 3

FINAL

REPORT: CR2014/77

DATE: June 2014

5.2.3 Sensitivity to the annual frequency of the triggering event

The return period of the event that could initiate failure is unknown. The sensitivity of the risk estimates for the assessed source areas to different event return periods has been assessed. This was done by plotting the location of the 10^{-4} annual individual fatality risk contour, for the upper source volume estimates only, adopting return periods of 20, 50, 100 and 200 years for the triggering event. The results are plotted for in Figure 34.

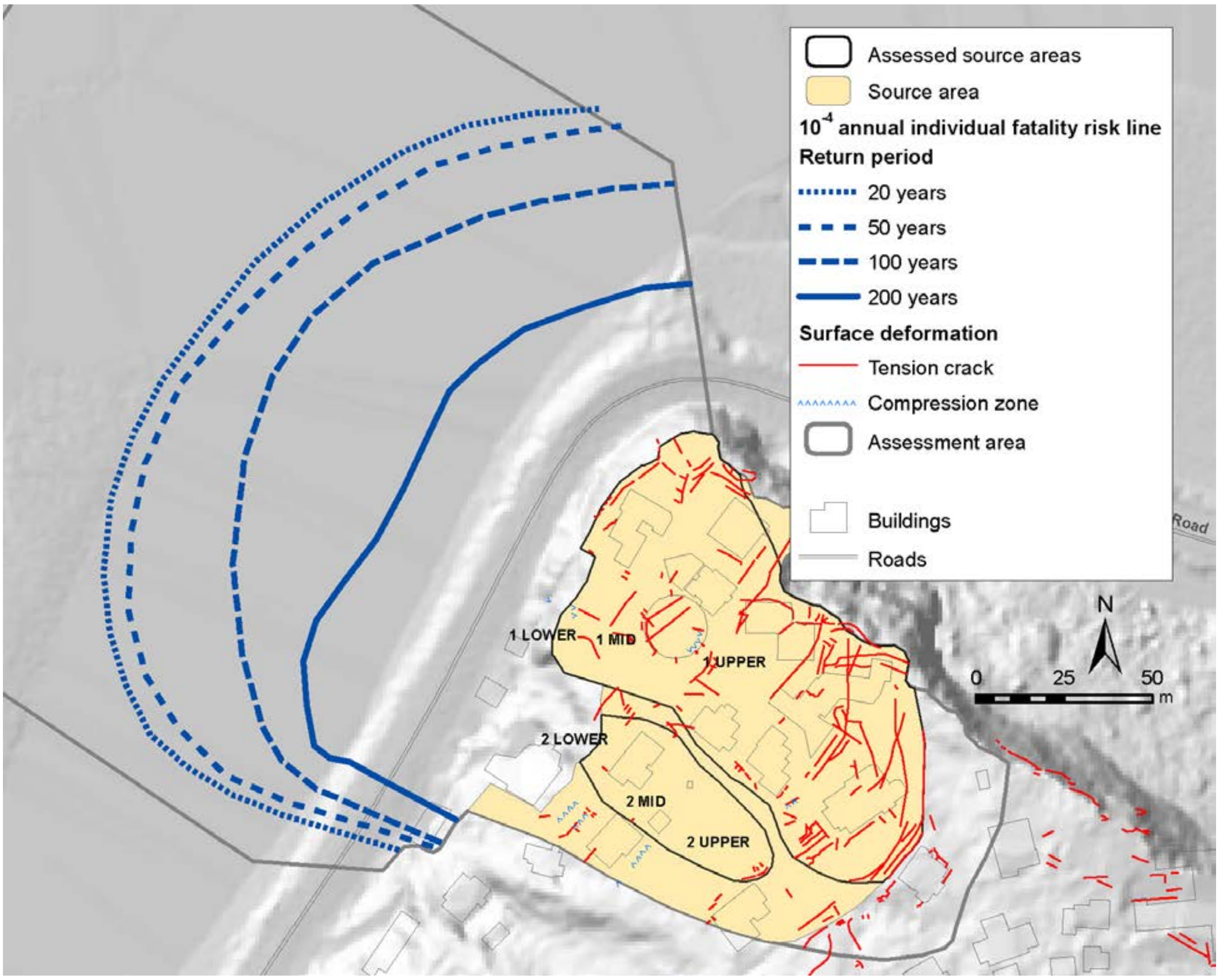


Figure 34 Sensitivity of the risk estimates, upper volume estimates, for triggering event return periods of 20, 50, 100 and 200 years.

The results show that area within the 10^{-4} risk contour reduces between the 20 and 100 year return periods. This is because the volume of the failure, and therefore runout distance of the debris, remains fixed, but the return period of the event increases, leading to a reduction in the risk at the longer return periods.

For the assessed source areas, for the 20–200 year return periods, the changing risk has little impact on the numbers of dwellings within the 10^{-4} annual individual fatality risk contour. Therefore the 50-year return period adopted for the risk estimates shown in Figure 33 is considered reasonable.

5.3 ROAD USER RISK

The section of Main Road assessed for this report (Figure 2) has a number of important differences from the road sections (Wakefield Avenue and Main Road below Quarry Road) previously assessed (Massey et al., 2014a,b), in particular:

a. to the West of Deans Head the road runs immediately next to relatively deep, fast-moving estuarine water with only a wooden crash barrier between traffic and the edge of the water;

b. the only way in and out of the road section concerned is along Main Road – there are no side roads between Clifton Terrace at the eastern end of the section and Cliff Street which is further to the West and outside the assessed section of road;

c. the sources modelled in previous reports (Massey et al., 2012) were for cliff-collapse hazards only, and the risk were estimated for dwelling occupants. For the section of road past Deans Head, earth/debris flows also need to be considered (from assessed source areas 1 and 2). These are in addition to the cliff-collapse hazards (Massey et al., 2012); and

d. several buses run daily, including on schooldays several between Redcliffs Primary School and Sumner, through the assessed section of road.

Point b) leads to the possibility of traffic becoming trapped in parts of the assessed section of road by any sort of accident. Of particular concern is the possibility of multiple earthquakes within minutes of each other, e.g., as per the earthquakes on 22 February and 23 December 2011, in which an initial earthquake leads to debris blocking the road, and trapping vehicles on the road, which is then followed a few minutes later by another earthquake, exposing any trapped road users to further cliff collapse.

It is unlikely (but not impossible) that cliff collapse, triggered mainly by earthquakes, would occur at the same time as an earth/debris flow, triggered mainly by rain. For this reason the risk results are presented for: 1) cliff collapse hazards only; and 2) combined cliff collapse and earth/debris flows, to provide a "worst case" scenario for consideration.

The assessed section of Main Road represents a particular risk for road users, both individually and collectively. The risk for the assessed section of road has been assessed in terms of:

a. fatality risk per single journey and per year, for six modes of transport: car, bus and truck occupants, and for motorcyclists, pedal cyclists and pedestrians;

b. aggregate risk in terms of expected fatalities per year for road users by transport mode and collectively; and

c. "Societal Risk" the frequency of multiple fatality accidents involving different numbers of deaths for car, truck and bus users.

Individual road user risks per journey and per year are assessed using the same cellular grid as that used for the dwelling occupant risk assessment. The following hazards considered in each cell are, for a road user:

1. being directly impacted or inundated by debris (Hazard 1); and
2. driving into or swerving to avoid debris on the road, and in the process (Hazard 2)
    a. driving into debris on the road;
    b. driving into objects on or on the side of the road; or
    c. driving off the road into the sea (if driving in the seaward direction).

Societal risk is assessed using a simple event-based model. For each assessed earth/debris flow source area or cliff collapse trigger scenario, a length of road is calculated within which it is considered very likely that road users will be either crushed or swept into the sea, based on the volume of debris passing through the section of road for a given event. The number of vehicles within that stretch is then estimated as a probability distribution, based on the frequency of trips, using the latest relevant traffic count data for Main Road (collected by Christchurch City Council).

Different vulnerabilities were applied to the assessed road section using Deans Head as the boundary between the eastern and western assessed sections of road. This was done because the eastern part of the assessed section of road runs along the edge of the sea, with little room between the road and the sea, and where the sea can be particularly fast flowing during tidal changes. Therefore there is a high probability that road users (excluding pedestrians), if swerving, could end up in the sea. In contrast, the western part of the assessed section of road is adjacent to Sumner Beach, and/or shallow water).

The frequency with which different outcomes (in terms of numbers of fatalities) occur can be calculated, by combining:

a. the frequency of the source/trigger scenario;
b. the percentage of the time for which different numbers of vehicles are exposed;
c. the number of people in those vehicles; and
d. the proportion killed – as a result of hazards 1 and 2 (vulnerability).

Figure 35 shows the results in terms of risk per journey for cliff-collapse hazards only (excluding debris from assessed source areas 1 and 2), in comparison with the risk of motor vehicle crashes for an average New Zealand stretch of urban road of the same length as the assessed section of road. Figure 36 shows the same results as those in Figure 35, but with the additional earth/debris flow hazards (assessed source areas 1 and 2) now introduced, adopting an event annual frequency of 0.05 events per year (return period of 20 years) and the upper source volume estimates.

The range of risk for each mode of transport (Figure 35) is similar to those ranges estimated for road users of Wakefield Avenue and Quarry Road (Massey et al., 2014a,b), where:

- the risk closer to the cliff is greater than that further from the cliff, for all road users;
- motor vehicle risk (cliff collapse hazards) is comparable with road crash risk (somewhat smaller for cars, somewhat larger for buses with trucks in-between); and
- the cycle/pedestrian road user risk(cliff collapse hazards) is comparable with or less than the corresponding motor vehicle crash risk.

Figure 35, shows the risk results from combining both the cliff collapse and the earth/debris flow hazards. These results differ to those in Figure 35, as the:

a. risk is now higher than motor vehicle crash risk for all road users except motorcyclists; and

b. cliff-collapse risk on the far side (seaward) side of the road from the cliff is comparable with or greater than that on the landward (cliff) side.

The first point (a) is a result of the large volumes of debris associated with the failure of assessed source area 1. The second point (b) is a result of the assumed higher probability of road users swerving into the sea when on the seaward side of the road outweighing the lower debris volumes reaching the section of road.

Figure 37, Map 1 shows graphically the contribution to risk per journey for a car occupant, from individual cells along the outer and inner edges of Main Road, from cliff-collapse hazards only. Figure 37, Map 2 shows the risk per trip for a car occupant along the outer and inner edges of Main Road from cliff collapse and earth/debris flow hazards combined, for the earth/debris flow upper volume estimates, adopting the event annual frequency of 0.05 (return period of 20 years).

Figure 38–Figure 40 show the same results as Figure 35, but with:

a. the lower source volume estimates for assessed source areas 1 and 2 (Figure 38);

b. lower assumed event annual frequencies for assessed source areas 1 and 2 (Figure 39); and

c. lower source volume estimates (a) and event annual frequencies (b) combined (Figure 40).

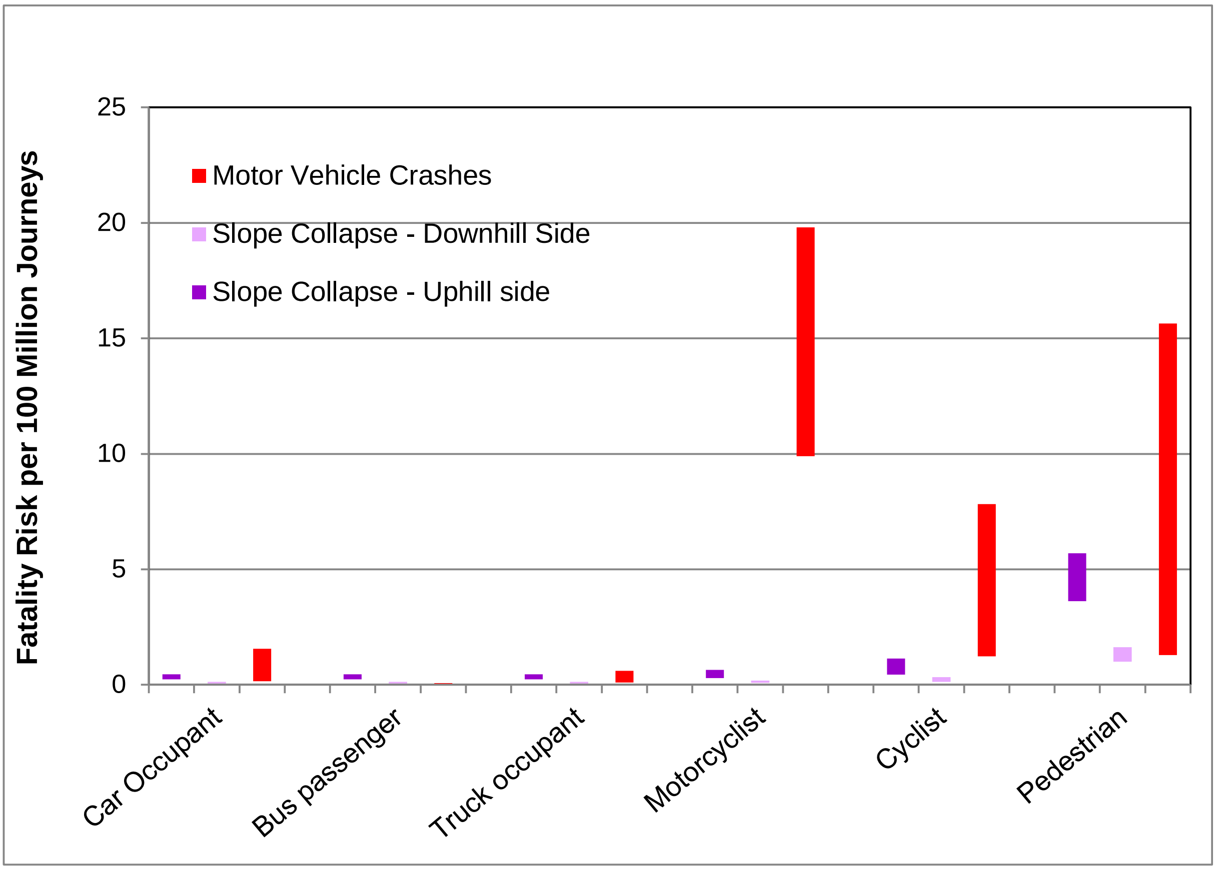


Figure 35 Risk per journey, cliff-collapse hazard only.

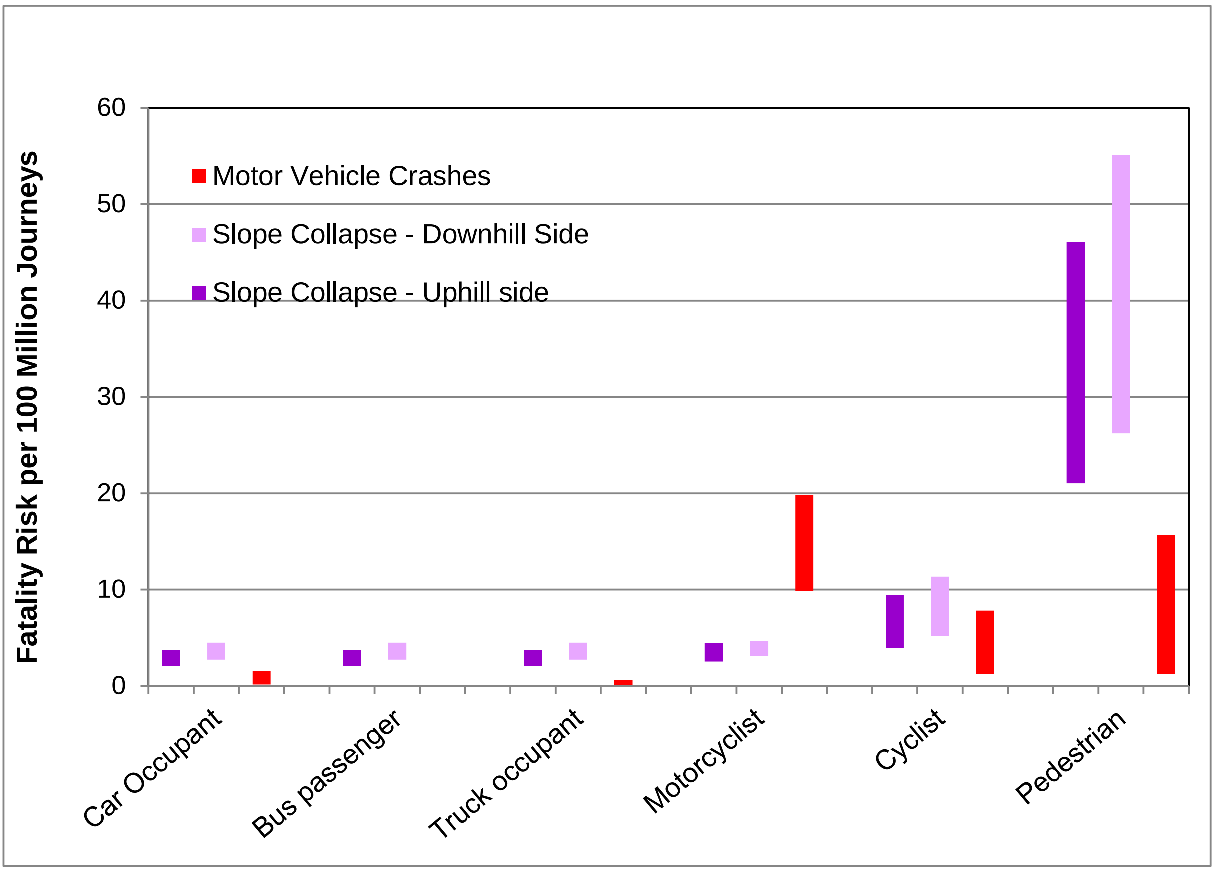
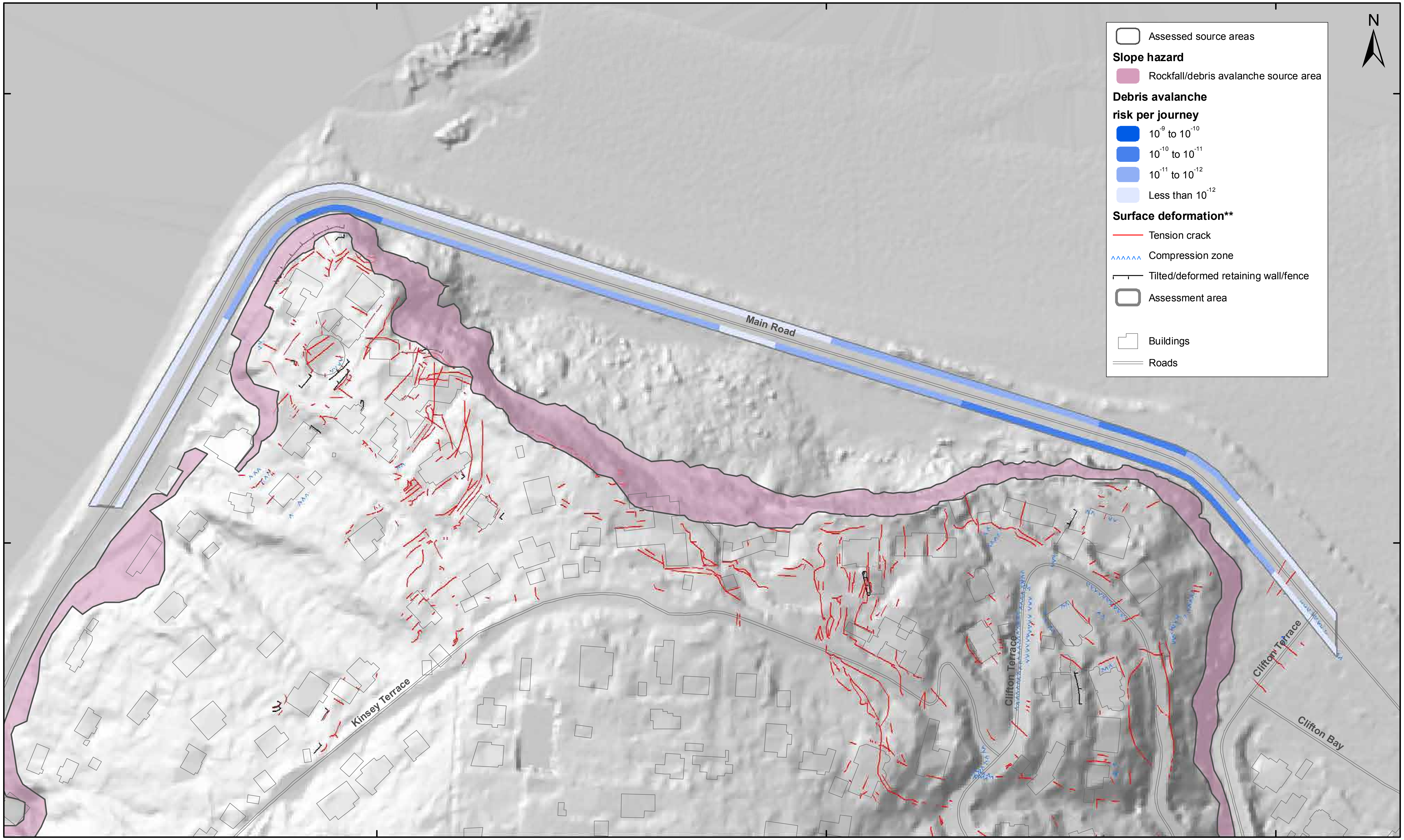


Figure 36 Risk per journey, cliff collapse and earth/debris flow hazards (source areas 1 and 2, adopting an event annual frequency of 0.05 events/yr (return period of 20 years) and the upper source volume estimates.



SCALE BAR: 0 50 100 m

EXPLANATION:

* Map shows road user risk contributions from individual model cells for car occupants.
** Taken from report CR2012/317
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: DH, BL

CHK: CM, FDP

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ROAD USER RISK
(from debris avalanche only)
Higher Estimate - Car Occupant

Deans Head
Christchurch

FIGURE 37

Map 1

FINAL

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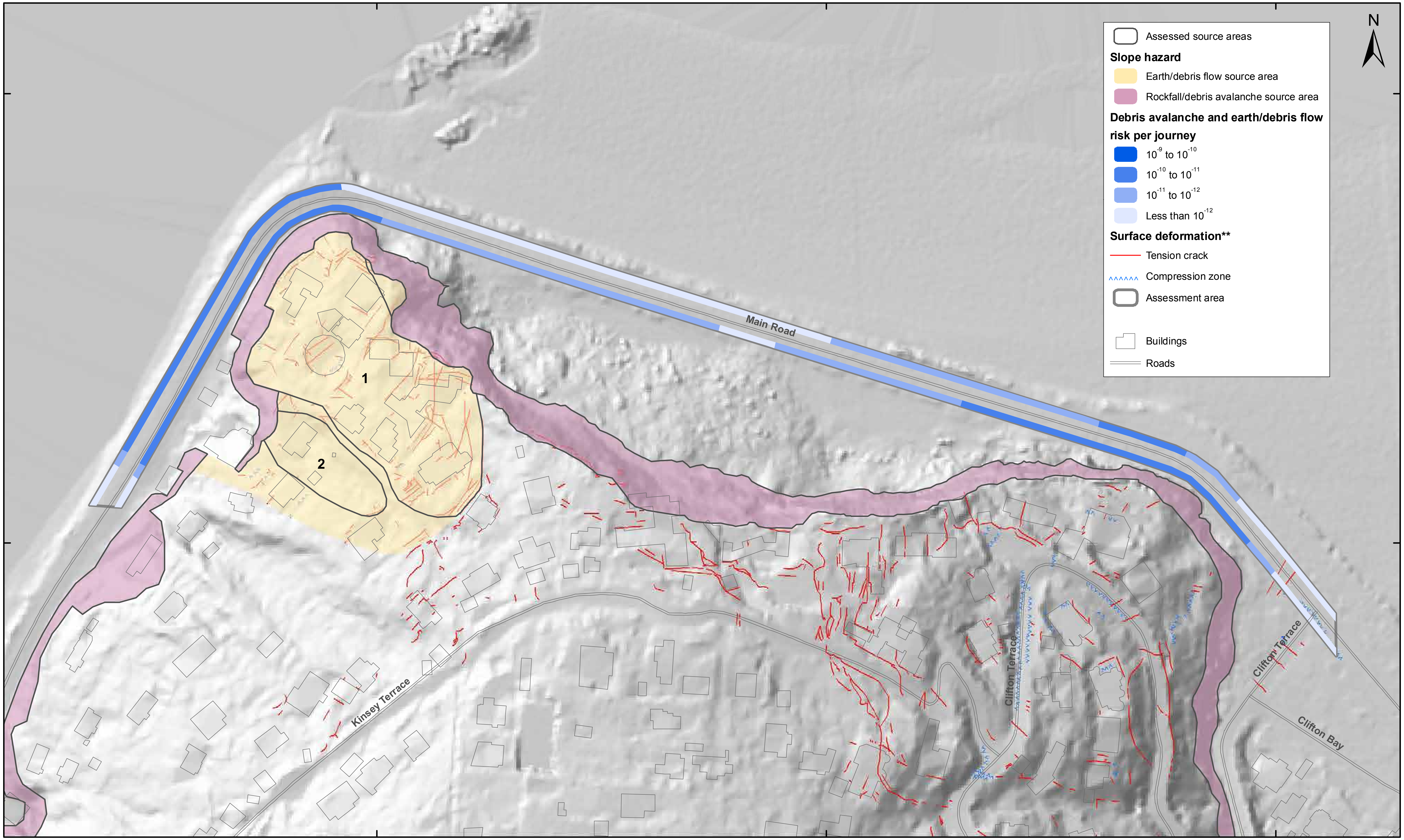

SCALE BAR: 0 50 100 m

EXPLANATION:

* Map shows road user risk contributions from individual model cells for car occupants.
** Taken from report CR2012/317
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: DH, BL

CHK: CM, FDP

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ROAD USER RISK
(from debris avalanche and earth/debris flow)
Higher Estimate - Car Occupant

Deans Head
Christchurch

FIGURE 37

Map 2

FINAL

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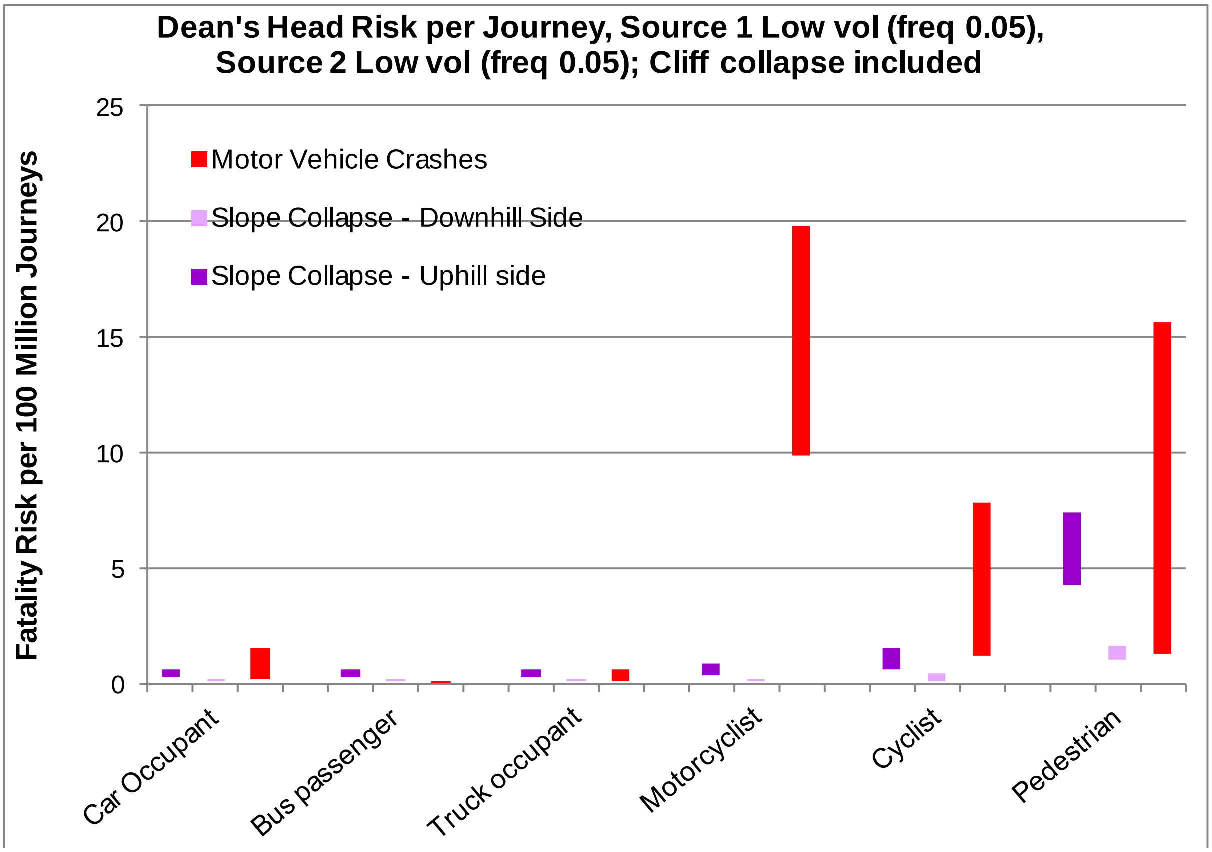

Figure 38 The effect of reducing the source volumes of the assessed source areas 1 and 2, on the risk results.

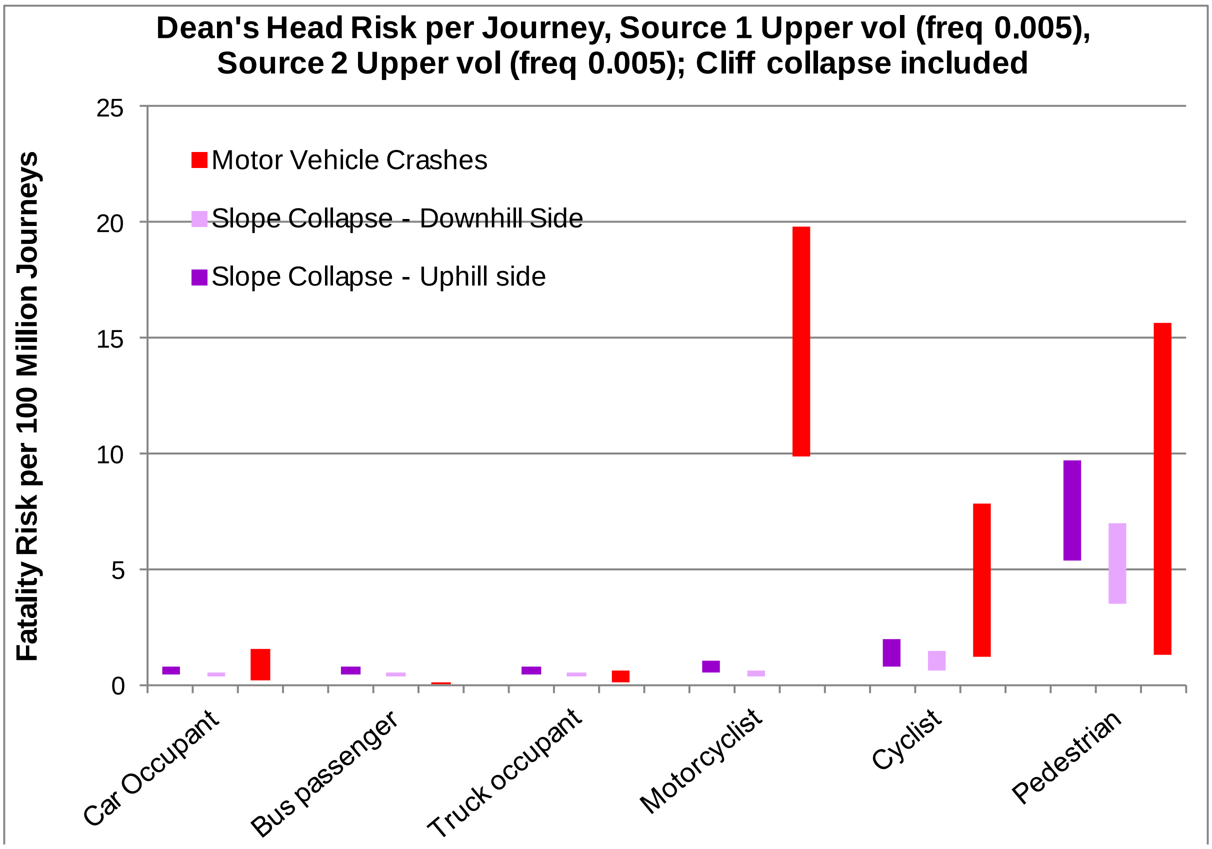

Figure 39 The effect of reducing the event annual frequencies of the assessed source areas 1 and 2, on the risk results (annual frequency of 0.005, return period of 200 years).

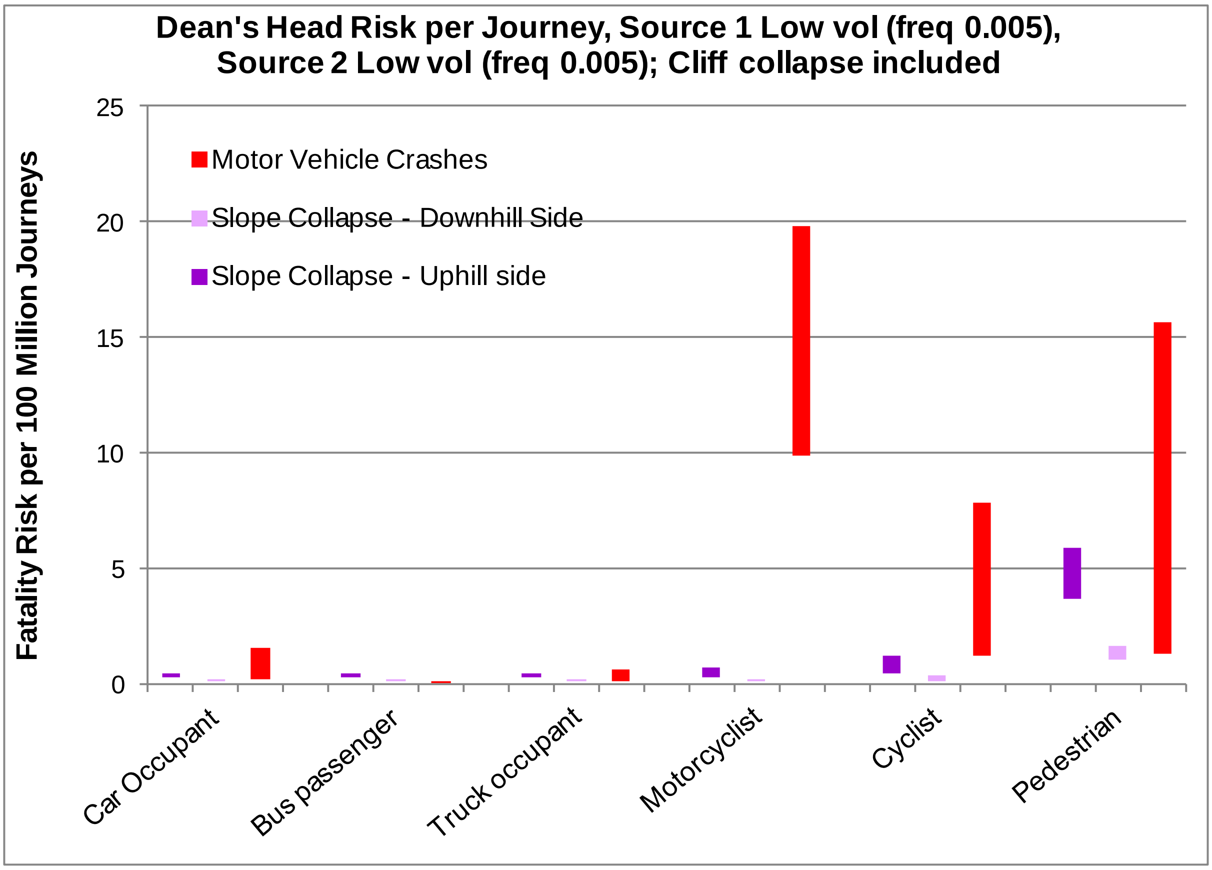

Figure 40 The effect of reducing the source volumes and event annual frequencies (0.005, or return period of 200 years) of the assessed source areas 1 and 2, on the risk.

The results are highly sensitive to the volume and annual frequency of the assessed source areas 1 and 2. With lower volume and frequency assumptions, the relativity of road accident risk and slope collapse risk is significantly altered, whereby the road accident risk is notably higher than the risk from cliff collapse and earth/debris flows.

Figure 41 shows the complementary cumulative distribution function or "f/N curve" relating the frequency of events killing N (the number) or more people, to the number N that are killed (assuming upper source volumes and maximum event annual frequencies for assessed source areas 1 and 2). Note that this assessment considers motor vehicle users only, so does not include motorcyclists, pedal cyclists or pedestrians. Important points of note include:

a. A fatal accident (N≥1) is anticipated about every 15 years;

b. An accident killing 10 or more people is expected every few hundred years;

c. An accident killing 20 or more people is expected about every 1000 years; and

d. A substantial proportion of the major accident risk involves buses (of which about 75% are buses full of schoolchildren).

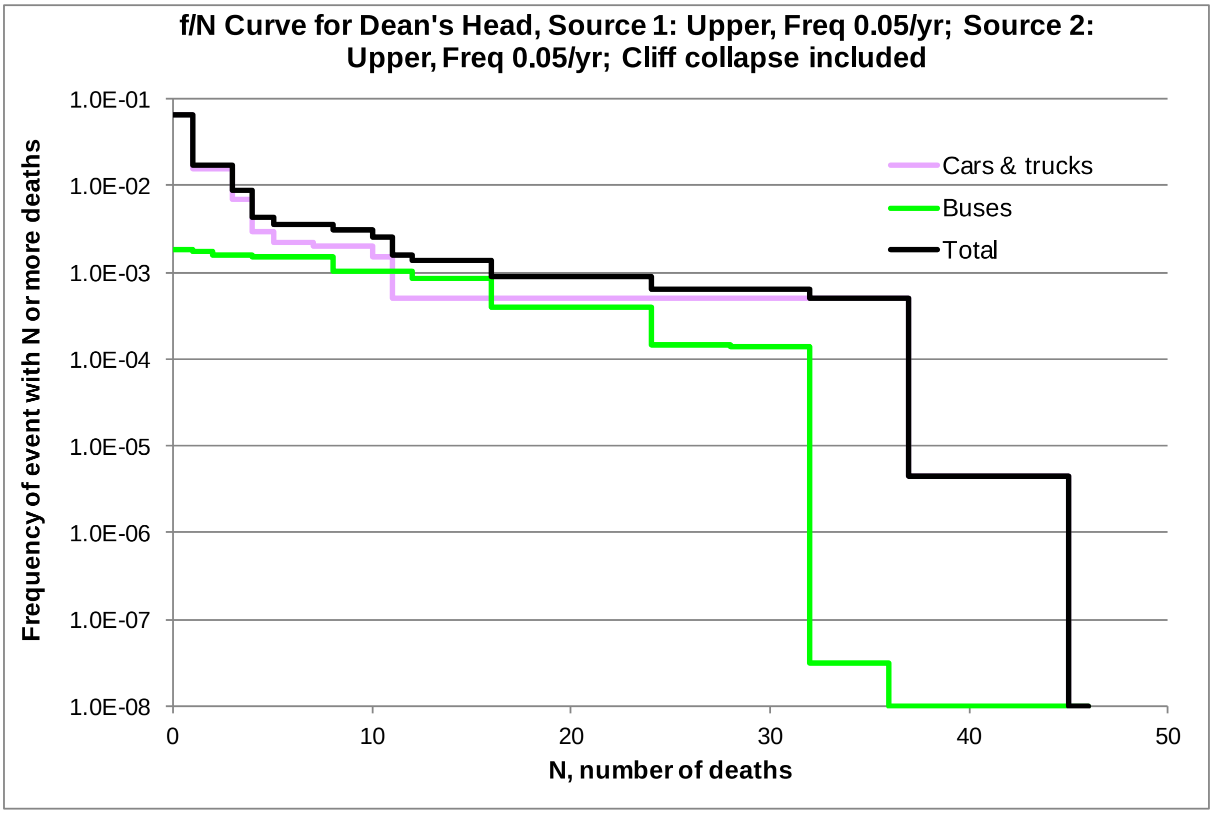

Figure 41 f/N Curve for combined cliff collapse and earth/debris flow hazards to motor vehicle occupants at Deans Head.

The main point of note in Figure 41 is that the assessed cliff collapse and earth/debris flow events have much greater potential to affect (and kill) people in greater numbers than do "ordinary" road accidents (a large majority of which involve just one or two vehicles). This is because:

a. they involve inundating substantial lengths of Main Road with rocks (in the case of cliff collapse) or loess debris (in the case of assessed source areas 1 and 2);

b. there is a significant proportion of the time when traffic is closely packed along this section of road, in the morning and evenings; and

c. there is the possibility that an earthquake-triggered cliff collapse is preceded by a smaller but significant earthquake, leading to blockage of the road and a queue of traffic being present when a second collapse occurs.

6.0 DISCUSSION

6.1 DWELLING OCCUPANT RISK

Important points of note from the results of the hazard and risk assessment undertaken in this study include:

Earth/debris flow hazards (failure mechanism 2):

1. The annual frequencies of the events that could trigger the earth/debris flow are unknown. Indeed, whether such events could be triggered is uncertain. However, a nominal 50-year return period (annual frequency of 0.02) was assumed after it was determined that the area at risk changed little with the value of annual frequency.
2. There are no known dwellings located within the "greater than 10^{-4}" annual individual fatality risk zone, corresponding to the debris runout zone; and
3. Nearly all the affected dwellings are located within the assessed source areas. The risk associated with the assessed source areas is inferred to be the same as the risk in the runout zone immediately below the assessed source areas, which is shown as 10^{-4} or greater.

Cliff-collapse hazards:

1. Cliff collapses are a credible hazard affecting the site (failure mechanism 3), and could occur from anywhere along the cliff.
2. The risk associated with cliff collapse hazards were previously assessed by Massey et al., 2012) and have not been reassessed in this report.

6.1.1 Annual frequency of the event

The frequency of occurrence of the events that could trigger the assessed failure volumes is unknown. Future earth/debris flows at these sites could be more frequent, i.e., occurring at lower triggering thresholds (e.g., rainfall magnitudes).

The area has already undergone more than 0.7 m of permanent slope displacement during the 2010/11 Canterbury earthquakes and this displacement may have reduced the shear strength of critical materials in the slope, making the slope more susceptible to future earthquakes. In addition, there may be an unknown amount of further displacement that the slopes may be able to undergo before failing catastrophically (i.e., where the magnitude of displacement causes the failure mass to break down to become a mobile failure). At the current time there is no practical means for estimating the numerical value of the "degraded strength", of the slope.

No permeability or loess/volcanic colluvium water content measurements have been made at the site and groundwater records from measurements of the standpipe within the loess have poor temporal resolution. It is therefore not possible to directly assess whether the earthquake-induced cracks have increased the susceptibility of these sites to future failures. However:

- Loess shear strength is critically dependent on water content, and the volcanic colluvium present at the site appears to be, in part, reworked loess.

- At high water contents the range of shear strengths, derived from ring-shear testing, could feasibly represent the strength of the loess in the slope.
- Under such conditions, results from the numerical slope stability back-analysis indicate that failure of the slope is likely.
- It is more likely that failure would occur through specific zones within the loess, e.g., through more permeable zones where water contents are likely to increase more readily, or above permeability boundaries such as soil fragipans, or the volcanic colluvium layer.
- Pore pressure above rockhead and within the colluvium and loess, as well as pore water pressures within the open cracks would also reduce the slope factor of safety.
- Given the now-cracked nature of the slopes, it is feasible that water contents of the loess and colluvium could increase in response to rainfall, as water can more readily enter the slope via the cracks and broken services.
- There is a large relict landslide scar (estimated volume of about 10,000–15,000 m^3) located towards the southern boundary of the assessment area, indicating past failures of similar volumes, similar to those assessed in this report, have occurred on these slopes. Very little of the debris is still present in this source area.

6.1.2 Risk assessment sensitivity to uncertainties

In this section, the sensitivity of the risk models for the assessed source areas 1 and 2 to key uncertainties and reliability of the assessments are discussed.

The sensitivity of the estimated risk has been assessed to the following risk-contributing factors:

1. The volumes of earth/debris flows that could be triggered in future events. This was done by comparing the three volume ranges which account for variation in the likely source volumes. The three volume ranges also take into account variability in the debris runout velocities and inundation heights, as larger volumes of debris tend to travel further down slope at higher velocities.
    a. There are quite large differences between the positions of the 10^{-4} annual individual fatality risk contours between the modelled lower and upper source volumes. However, this has little effect on the numbers of dwellings affected in the debris runout zones (Figure 34), as most dwellings are in the assessed source areas.
    b. The risk to the dwellings in the assessed source areas is uncertain. The numbers of dwellings affected by the upper source volume estimates are, as expected, larger than those few affected by the lower volume estimates, as the lower volume estimates are associated with smaller source areas.
    c. The main hazard affecting these dwellings is likely to be a combination of cracking and undercutting as the ground moves beneath the dwelling, as well as the impact from debris coming from further upslope.
    d. Source areas 1 and 2 and the lower, middle and upper volume estimates assessed in this report, represent the range of failure volumes that could occur within the assessment area. The locations of the assessed source areas are based on an interpretation of the key geological information relating to the site.

However, failures could occur from anywhere within the assessment area, and be any volume within the range assessed.

e. It is therefore difficult to assess what the levels of risk to the dwellings in the source areas are, given the uncertainties associated with the triggering event, source volume and area that could be affected.

f. The failure mechanisms affecting the site are also uncertain. Whilst the main failure mechanisms have been inferred from site assessment results – mainly the performance of the slope during the 2010/11 Canterbury earthquakes, it is possible that different failures mechanisms may occur at the site in the future.

2. Changes to the annual frequency of the event that could trigger failure of source areas. Risk models were run adopting event annual frequencies of 0.05, 0.02, 0.01 and 0.005, corresponding to return periods of 20, 50, 100 and 200 years respectively. Results from the assessment show that there is little change between the risk results adopting the 20-year and 200-year return periods, because most of the affected dwellings are located in the assessed source areas.

3. Vulnerability: the probability of death is a function of debris height and velocity. The risk assessment may not adequately take into account the sheltering effect of buildings. Variable vulnerabilities have been adopted linked to debris velocity. However, the vulnerability of a person in a dwelling is related to the nature of the structure, for which there are no New Zealand specific data available for use in the risk assessment. It is possible that the risk could reduce by an order of magnitude or more if the dwelling could withstand inundation by debris without collapsing. This could have a large effect on the risk especially in the distal run out zones where the debris is travelling at lower velocity.

The results (Figure 34) show that the largest impact on the risk is from the volumes of material that could be generated in an event from the assessed source areas.

For source areas 1 and 2 the uncertainties combine to give an order of magnitude uncertainty, in either direction, on the risk estimates.

Most of the dwellings at risk are located in the assessed source areas. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings from future earthquakes is still relatively high. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.4 m. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about one in every 140 years, adopting the results from the national seismic hazard model.

6.1.3 How reliable are the results?

Potentially significant uncertainties noted and their likely implications for risk are summarised in Table 20.

Table 20 Uncertainties and their implications for risk.

Issue	Direction and scale of uncertainty	Implications for risk
Earth/debris flows		
a. Choice of whether to use different event annual frequencies other than 0.02 (50-year return period)	Moderate uncertainty between the use of the 50-year and 100-year, return periods. But larger uncertainty between the 50-year and 15-year plus return periods, and 50-year and 200-year return periods. However, has little impact on the number of dwellings affected.	Longer term risk is potentially 2–4 times lower, but shorter term risk could be 2–3 times higher.
b. Volume of debris produced by a source area, and the location of the source area.	Largest uncertainty between upper volume and the lower volume, and then the lower volume and middle volume.	About a factor of 5 between the upper and lower volume estimates. But a factor of 3 between the lower and middle estimates, and a factor of 2 between the middle and upper estimates.
c. Changing the assumed debris height where probability of inundation = 0 from 0.3 m to 0.5 m and 0.1 m	Small uncertainty in either direction	Would change modestly by a factor of about 1.2 in either direction.
d. Occupancy (proportion of time people are at home)	Assumption of 100% occupancy instead of 67% would modestly increase the estimated risk.	Would increase modestly by a factor of about 2.
e. Vulnerability (probability of being killed if inundated by debris)	Variable vulnerabilities have been adopted linked to debris velocity. However, the vulnerability of a person in a dwelling is related to the nature of the structure, for which there is no data available for use in the risk assessment. Potentially large uncertainty in either direction but very difficult to quantify.	Could be relatively large depending on the nature of the dwelling construction and age/ability of the person to get out of the way of the debris. Possibly over an order of magnitude uncertainty in either direction

6.2 ROAD USER RISK

The risk to road users from cliff collapse and earth/debris flows onto Main Road around Deans Head is critically dependent on both the volume and the frequency with which failures of source areas 1 and 2 occur.

Both parameters (volume and frequency of failure) are themselves highly uncertain, meaning that the risk to road users could lie anywhere in a wide range from *"comparable with or smaller than the risk of 'ordinary' road accidents"* on the one hand, to *"substantially larger than the risk of 'ordinary' road accidents for most road users"* on the other.

Generally, the risk to road users on Main Road below Deans Head is significantly higher than that at the other sites assessed to date (Wakefield Avenue and Quarry Road, Massey et al., 2014a,b) because:

a. The volumes of material reaching the road could be relatively high, and could occur with relatively high frequency (although uncertain);

b. There is no means of escape for motor vehicle users from Main Road over the assessed section of road other than by travelling forward or back along Main Road itself;

c. There are relatively high traffic densities for significant proportions of the time;

d. The road to the west of Deans Head lies next to relatively deep, fast moving water with only a wooden crash barrier to prevent road users inundated by rockfall or debris being washed into the sea; and

e. There is potential for accident scenarios in which a queue of traffic is trapped on this section of the road at exactly the time that a significant (seismically-triggered) slope failure occurs.

There has to date been no substantive discussion with Christchurch City Council on the levels of fatality risk considered tolerable or acceptable for road users, and no quantitative risk criteria have been established. In the absence of such criteria though, Deans Head stands out as a case of exceptional slope-collapse risk to road users because of the juxtaposition of the factors (a)–(e) above and the potential for substantial slope failures.

7.0 CONCLUSIONS

With reference to source area boundaries as show in Figure 2, the conclusions of this report are:

7.1 HAZARD

1. There is potential for volumes ranging from several hundreds to tens of thousands of cubic metres of earth/debris flows (source areas 1 and 2) of mixed loess and colluvium, which are in addition to the cliff-collapse failures previously assessed (Massey et al., 2012).
2. The most likely triggers for the assessed earth/debris flows sources are prolonged heavy rainfall and strong earthquake shaking (if ground conditions were wet).
3. The frequency of earth/debris flow events from these sources is difficult to estimate and could be anything from once every few tens to once every many hundreds of years.

7.2 RISK

7.2.1 Dwelling occupant

1. There are very few dwellings in the earth/debris flow runout zone, as most dwellings are located in the assessed source areas.
2. The main hazard affecting these dwellings is likely to be a combination of cracking and undercutting as the ground moves beneath the dwelling, as well as the impact from debris coming from further upslope.
3. It is difficult to assess what the levels of risk to the dwellings in the source areas are, given the uncertainties associated with the triggering event, source volume and area that could be affected. The risk associated with the assessed source areas is inferred to be the same as the risk in the runout zone immediately below the assessed source areas, which is shown as 10^{-4} or greater.
4. The numbers of dwellings affected by the upper source volume estimates are, as expected, larger than those few affected by the lower volume estimates, as the lower volume estimates are associated with smaller source areas.
5. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings from future earthquakes is still relatively high and similar to a Class II relative hazard exposure category. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.4 m. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the national seismic hazard model.

7.2.2 Road user

1. Generally, the risk to road users of Main Road in the assessed section of road below Deans Head is significantly higher than that at the other sites assessed to date (Wakefield Avenue and Quarry Road, Massey et al., 2014a,b).
2. The volumes of material reaching the road could be relatively high, and could occur with relatively high (though uncertain) frequency.
3. There is limited means of escape for motor vehicle users from Main Road over the assessed section of road, other than by travelling forward or back along Main Road itself.
4. There are relatively high traffic densities for significant proportions of the time.
5. The road to the west of Deans Head lies next to relatively deep, fast moving water with only a wooden crash barrier to prevent road users inundated by rockfall or debris being washed into the sea.
6. There is potential for accident scenarios in which a queue of traffic is trapped on this section of the road at exactly the time that a significant (seismically triggered) slope failure occurs.

7.2.3 Risk management

1. A risk-management option of monitoring rainfall, soil moisture and pore-pressure in the source areas, may be of some value in providing warning of conditions approaching critical levels, but:
   a. Such early warning could not be assured, as experience in the Port Hills and elsewhere is that water levels in open tension cracks can rise very rapidly to critical values;
   b. There would be little time to evacuate potentially at-risk residents given the rapid nature of the hazard; and
   c. There is currently no precedent data for rates of change of groundwater or water content of loess to provide reliable alert criteria.
2. There appears to be reasonable scope for engineering measures to stabilise the slopes (e.g., by removal of loess/colluvium and installation of drainage measures), however, site access may be prohibitive due to the nature of the ground, and these works would need to be evaluated, designed and implemented by a suitably qualified engineering consultant.

8.0 RECOMMENDATIONS

GNS Science recommends that based on the results of this study, Christchurch City Council:

8.1 POLICY AND PLANNING

1. Decide what levels of life risk to dwelling occupants and road users will be regarded as tolerable.
2. Decide how Council will manage risk on land and roads where life risk is assessed to be at the defined threshold of intolerable risk and where the level of risk is greater than the threshold.
3. Prepare policies and other planning provisions to address risk lesser than the intolerable threshold in the higher risk range of tolerable risk.

8.2 SHORT-TERM ACTIONS

8.2.1 Hazard monitoring strategy

1. Include the report findings in a slope-stability monitoring strategy with clearly stated aims and objectives, and list how these would be achieved, aligning with the procedures described by McSaveney et al. (2014). In the meantime, extend the current survey network further up the slope (particularly in source area 1 towards Kinsey Terrace), so as to maintain awareness of changes in the behaviour of the slope;
2. Ensure that the existing emergency management response plan for the area identifies the dwellings that could be affected by movement and runout, and outlines a process to manage a response.

8.2.2 Risk monitoring strategy

Monitoring the slope for early warning of potentially dangerous trends in groundwater or slope movement as part of a hazard warning system, is currently not thought to be feasible. Monitoring alerts for slope deformation and groundwater changes cannot be relied upon to provide adequate early warning as experience from Port Hills and elsewhere shows that deformation and groundwater changes can occur rapidly, with little warning, and there is little site-specific information on which to build such a warning system.

8.2.3 Surface/subsurface water control

1. Reduce water ingress into the slopes, where safe and practicable to do so, by:
2. Identifying and relocating all water-reticulation services (water mains, sewer pipes and storm water) inside the identified mass-movement boundaries (at the slope crest) to locations outside the boundary, in order to control water seepage into the slope. In particular, a water main currently traverses the site between assessed source areas 1 and 2; and
3. Control surface water seepage by filling the accessible cracks on the slope and providing an impermeable surface cover to minimise water ingress.

8.3 LONG-TERM ACTIONS

8.3.1 Engineering measures

1. Assess the cost, technical feasibility and effectiveness of alternative longer term engineering and relocation solutions, for example (but not limited to):
    a. Removal/stabilisation of the slopes in the assessed source areas;
    b. Installation of drainage works;
    c. Relocation of houses to alternative locations within existing property boundaries; or
    d. Withdrawal and rezoning of the land for non-residential use.
2. Any proposed engineering works would require a detailed design and be carried out under the direction of a certified engineer, and should be independently verified in terms of their risk reduction effectiveness by appropriately qualified and experienced people.
3. For the section of Main Road within the risk zone, liaise with whoever is responsible for roading (within Christchurch City Council) to develop solutions, which both: 1) ensure that the key lifeline section of Main Road can continue to serve its purpose of connecting Sumner and the surrounding area to Christchurch; and 2) adequately safeguard road users from slope-collapse risk.

8.3.2 Reassessment

Reassess the risk and revise and update the findings of this report in a timely fashion, for example:

a. in the event of any significant changes in ground conditions; or

b. in anticipation of further development or significant land use decisions.

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APPENDICES

A1 APPENDIX 1: METHODS OF ASSESSMENT

A1.1 METHODS OF ASSESSMENT

A1.1.1 Engineering geology assessment methodology

The findings presented in this report are based on engineering geological models of the site developed by GNS Science.

The scope of the investigation works comprised: 1) engineering geological and geomorphological mapping of the site; 2) construction of four cross-sections through the site area; 3) interpretation of aerial photographs ranging in date from 1940–2011; and 4) assessment of available LiDAR data for the site and the construction of a digital terrain model.

A1.1.2 Hazard assessment methodology

A1.1.2.1 Slope stability modelling

The key output from the static stability assessment is a factor of safety of the given volume, while the key output from the dynamic assessment is the magnitude of permanent slope displacement expected at given levels of earthquake-induced ground acceleration. These two assessments are then used to determine the likely volumes of material that could be generated under the different conditions.

A1.1.2.2 Static slope stability

If a slope has a static factor of safety of one or less, the slope is assessed as being unstable. Slopes with structures designed for civil engineering purposes are typically designed to achieve a long-term factor of safety of at least 1.5 under drained conditions, as set out in the New Zealand Transport Agency (NZTA) 3rd edition of the bridge manual (NZTA, 2013).

Static assessment of the slope was carried out by limit equilibrium method using the Rocscience SLIDE® software and the general limit equilibrium method (Morgenstern and Price, 1965). The failure surfaces were defined using the path search feature in the SLIDE® software, and a zone of tension cracks was modelled corresponding to mapped crack locations on the surface and in exposures. For the assessment, tension cracks were assumed to extend to the rockhead.

Models were run based on geological cross-sections 1–5. The critical slide surface was determined based on the lowest calculated factor of safety. Sensitivity analyses were run assuming a range of geotechnical material strength parameters based on the estimates of their strength to test model sensitivity. These were derived from in-house laboratory testing on samples of materials taken from the site, and samples of similar materials taken from other sites in the Port Hills and published information on similar materials. Strength parameters were also assessed by back-analysis in the limit equilibrium and dynamic analyses.

The finite element modelling adopts the shear strength reduction technique for determining the stress reduction factor or slope factor of safety (e.g., Dawson et al., 1999). Finite element modelling was undertaken on the same cross-sections adopted for the limit equilibrium modelling assessment, using the Rocscience Phase^2 finite element modelling software. This was done to check the outputs from the limit equilibrium modelling, because the finite element models do not need to have the slide-surface geometries defined.

A1.1.2.3 Dynamic stability assessment (decoupled method)

In civil engineering, the serviceability state of a slope is that beyond which unacceptably large permanent displacements of the ground mass take place (Eurocode 8, EN-1998-5, 2004). Since the serviceability of a slope after an earthquake is controlled by the permanent deformation of the slope; analyses that predict coseismic slope displacements (permanent slope displacements under earthquake loading) provide a more useful indication of seismic slope performance than static stability assessment alone (Kramer, 1996).

The dynamic (earthquake) stability of the slope was assessed with reference to procedures outlined in Eurocode 8 (EN-1998-5, 2004) Part 5. For the Deans Head assessed source areas the magnitude of earthquake-induced permanent displacements was assessed for selected cross-sections adopting the decoupled method and using different synthetic earthquake time-acceleration histories as inputs.

The decoupled seismic slope deformation method (Makdisi and Seed, 1978) is a modified version of the classic Newmark (1965) sliding block method that accounts for the dynamic response of the sliding mass. The “decoupled” assessment is conducted in two steps:

1. A dynamic response assessment to compute the “average” accelerations experienced at the base by the slide mass (Chopra, 1966); and
2. A displacement assessment using the Newmark (1965) double-integration procedure using the average acceleration time history as the input motion.

The average acceleration time history is sometimes expressed as the horizontal equivalent acceleration time history (e.g., Bray and Rathje, 1998), but they are both the same thing. The average acceleration time history represents the shear stress at the base of the potential sliding mass, as it captures the cumulative effect of the non-uniform acceleration profile in the potential sliding mass. The method assumes that the displacing mass is a rigid-plastic body, and no internal plastic deformation of the mass is accounted for.

The two steps above are described below in more detail.

1. Dynamic response assessment:
    a. Two-dimensional dynamic site response assessment using Quake/W was carried out adopting synthetic time acceleration histories for the four main earthquakes known to have triggered debris avalanches, cliff-top deformation and cracking in the Port Hills. The modelled versus actual displacements inferred from survey results and crack apertures were compared to calibrate the models.
    b. Synthetic out-of-phase free-field rock-outcrop time acceleration histories for the site – at 0.02 second intervals for the 22 February, 16 April, 13 June and 23 December 2011 earthquakes – were used as inputs for the assessment (refer to Holden et al. (2014) for details).
    c. The equivalent linear soil behaviour model was used for the assessment, using drained conditions. Strain-dependent shear-modulus reduction and damping functions for the rock materials were based on data from Schanbel et al. (1972) and Choi (2008). At present, GNS Science do not have dynamic test data for the loess– dynamic testing is currently being carried out by GNS Science as part of a

research project. Therefore for loess shear modulus and damping ratio functions from Ishibashi and Zhang (1993) were adopted assuming a plasticity index of five (Carey et al., 2014) and variable confining (overburden) stress, based on the overburden thickness of the loess at each cross-section assessed.

d. Shear wave velocity surveys were carried out by Southern Geophysical Ltd. for GNS Science (Southern Geophysical Ltd., 2013). These works comprised the surveying of a surface-generated shear wave signal at 2 m intervals between the surface and the maximum reachable depth inside nearby drillholes at Clifton Terrace.

2. Displacement assessment steps:

   a. The dynamic stress response computed with Quake/W – from each input synthetic earthquake time history – were assessed using Slope/W Newmark function to examine the stability and permanent deformation of the slope subjected to earthquake shaking using a procedure similar to the Newmark (1965) method (detailed by Slope/W, 2012).

   b. For the Slope/W assessment, a range of material strength parameters was adopted for the rock, colluvium and loess (based on the results from laboratory strength testing, published information and static back-analysis of slope stability), to assess the sensitivity of the modelled permanent deformation to changing material strength.

   c. For each trial slide surface, Slope/W uses: 1) the initial lithostatic stress condition to establish the static strength of the slope (i.e., the static factor of safety); and 2) the dynamic stress (from Quake/W) at each time step to compute the dynamic shear stress of the slope and the factor of safety at each time step during the modelled earthquake. Slope/W determines the total mobilised shear arising from the dynamic inertial forces. This dynamically driven mobilised shear force is divided by the total slide mass to obtain an average acceleration for a given slide surface at a given time step. This average acceleration response for the entire potential sliding mass represents one acceleration value that affects the stability at a given time step during the modelled earthquake.

   d. For a given trial slide surface Slope/W:

      i. Computes the average acceleration corresponding to a factor of safety of one. This is referred to as the yield acceleration. The critical yield acceleration of a given slide mass is the minimum acceleration required to produce movement of the block along a given slide surface (Kramer, 1996). The average acceleration of the given slide mass, at each time step, is then calculated along the slide surface (base of the slide mass).

      ii. Integrates the area of the average acceleration (of the trial slide mass) versus time graph when the average acceleration is at or above the yield acceleration. From this it then calculates the velocity of the slide mass at each time interval during the modelled earthquake.

      iii. Estimates the permanent displacement, by integrating the area under the velocity versus time graph when there is a positive velocity.

   e. To calibrate the results, the permanent displacement of the slide mass for a given trial slide surface geometry (for a given cross-section) was compared with crack apertures and survey mark displacements, and also with the geometry and inferred mechanisms of failure that occurred during the 2010/11 Canterbury earthquakes. Those soil strength parameters that resulted in modelled displacements of similar magnitude to the recorded or inferred slope displacements were then used for forecasting future permanent slope displacements under similar earthquakes.

A1.1.2.4 Forecasting permanent slope displacements

To forecast likely slope displacements in future earthquakes, the relationship between the yield acceleration (Ky) and the maximum (peak) acceleration (K_{MAX}) of the average acceleration of a given slide mass, was used. Using the results from the decoupled (Slope/W) assessment, the maximum average acceleration (K_{MAX}) was calculated for each selected slide surface (failure mass), from the average acceleration versus time plot – where the average acceleration versus time plot is the response of the given slide mass to the input acceleration history. The decoupled assessment uses the 22 February and 13 June 2011 synthetic earthquake acceleration histories as inputs (Holden at al., 2014), and the calibrated material strength parameters derived from back-analysis (bullet 2. e. above).

The Ky/K_{MAX} relationship was used to determine the likely magnitude of permanent displacement of a given failure mass – with an associated yield acceleration (Ky) – at a given level of average acceleration within the failure mass (K_{MAX}).

Permanent coseismic displacements were estimated for a range of selected trial slide surfaces from each cross-section. These results were then used in the risk assessment to assess the probability of failure of a given range of slide surfaces.

A1.1.2.5 Estimation of slope failure volumes

The most likely locations and volumes of potential failures were estimated based on the numerical analyses, current surveyed displacement magnitudes, material exposures, crack distributions and slope morphology.

Three failure volumes (upper, middle and lower) were estimated for each potential source area to represent a range of source volumes. The credibility of these potential failure volumes was evaluated by comparing them against: 1) the volumes of relict failures recognised in the geomorphology near the site and elsewhere in the Port Hills; 2) historically recorded failures; and 3) the volumes of material lost from the Deans Head assessed source area slope and other similar slopes, during the 2010/11 Canterbury earthquakes.

There are four main sources of information on historical non-seismic failures for the Port Hills:

1. archived newspaper reports from between 1870 and 1945 (a selection of which is presented in Appendix 2);
2. the GNS Science landslide database, which is “complete” only since 1996;
3. insurance claims made to the Earthquake Commission for landslips which are “complete” only since 1996; and
4. information from local consultants (M. Yetton, Geotechnical Consulting Ltd. and D. Bell, University of Canterbury) which incompletely covers the period from 1968 to present (McSaveney et al., 2014).

A1.1.2.6 Debris runout modelling

The potential runout of debris from the slope was assessed empirically by the fahrboeschung method and also by numerical modelling.

1. Empirical fahrboeschung method:

    a. The fahrboeschung model is based on a relationship between topographical factors and the measured lengths of runout of debris (Corominas, 1996). The fahrboeschung[1] (often referred to as the "travel angle") method (Keylock and Domaas, 1999) uses the slope of a straight line between the top of the source area (the crown) and the furthest point of travel of the debris. The analysis adopts the slope crest as the crown of each potential source area.

    b. The volume of earth/debris passing a given location within the assessment area is based on an empirical relationship established from a compilation of runout distances from published international and local (in the Port Hills) earth/debris flows. For earth/debris flows, which tend to be very fluid ("soupy" to "porridge-like" in consistency), the empirical relationship is based on a data set of over 700 earth/debris flows from New Zealand (including the Port Hills and Banks Peninsular) and overseas, compiled by Massey and Carey (2012).

2. Numerical methods:

    a. Numerical modelling of landslide runout was carried out using the RAMMS® debris-flow software. This software, developed by the Snow and Avalanche Research Institute based in Davos, Switzerland, simulates the runout of debris flows and snow and rock avalanches across complex terrain. The module is used worldwide for landslide runout analysis and uses a two-parameter Voellmy rheological model to describe the frictional behaviour of the debris (RAMMS, 2011). The physical model of RAMMS Debris Flow uses the Voellmy friction law. This model divides the frictional resistance into two parts: a dry-Coulomb type friction (coefficient μ) that scales with the normal stress and a velocity-squared drag or viscous-turbulent friction (coefficient xi). However, to the best of our knowledge there is no direct physical means of deriving these parameters from field measurements, other than back-analysis of past earth/debris flows in similar materials and terrain.

    b. RAMMS software takes into account the slope geometry of the site when modelling debris runout. The RAMMS model parameters were calculated from the back-analysis of 23 debris avalanches (ranging in volume from 200 to 30,000 m^3) that fell from the slopes at Richmond Hill Road, Shag Rock Reserve and Redcliffs during the 22 February and 13 June 2011 earthquakes.

    c. The model was calibrated by "back-analysing" the runout of five Port Hills and Banks Peninsula earth/debris flows and the modelled parameters optimised to obtain a good correlation between the modelled versus actual runout.

    d. The modelling results give likely debris runout, area affected, volume, velocity and the maximum and final height of debris in a given location at any moment in the runout.

[1] Fahrboeschung is a German word meaning "travel angle" adopted in 1884 by a pioneer in landslide runout studies, Albert Heim. It is still used in its original definition.

e. The RAMMS modelling uses a “bare earth” topographic model, and so the runout impedance of buildings and larger trees was not considered.

A1.1.3 Risk assessment

The risk metric assessed is the annual individual fatality risk and this is assessed for dwelling occupants and users of Main Road from the assessed earth/debris flows in initiating from each source area. The quantitative risk assessment uses risk-estimation methods that follow appropriate parts of the Australian Geomechanics Society framework for landslide risk management (Australian Geomechanics Society, 2007). It provides risk estimates suitable for use under SA/SNZ ISO1000: 2009.

A1.1.3.1 Fatality risk for dwelling occupants

The risk assessment is based on the following method and assumptions:

1. Divide the entire assessment area into a series of 1 m by 1 m grid cells.
2. Consider the possible range of triggering events from non-earthquake triggers (mainly rain). The annual frequency of the event (rainfall) that could trigger failure of any of the identified source areas is difficult to estimate given the lack of precedence in the Port Hills. The variation of risk across the slope has, therefore been assessed using a range of event frequencies and earth/debris flow source volumes:
   - It has been assumed that the return period of the event (mainly rainfall) that could trigger failure of the assessed source area is unlikely to be less than 10–20 years (event annual frequency of 0.1–0.05), as the rainfall recorded in the Port Hills 3–5 March 2014 (which did not cause substantial failures), was equivalent to a 10–20 year return period rain event.
   - Event annual frequencies ($P_{(H)}$) of 0.05, 0.02, 0.01, and 0.005 corresponding to return periods of 20, 50, 100 and 200 years, were used for the assessment.
   - The main source area was characterised based on the geological evidence and assessment collected to date, from which estimates of the likely failure volumes were made.
   - Three scenarios were considered based on: 1) lower; 2) middle; and 3) upper estimates of the source volume.
   - Each source volume scenario was assessed as having an equal probability of failure in a given event, of a given annual frequency.
3. For each representative event, and for each scenario, estimate:
   a. The frequency of the event and the volume of debris, for a given source scenario, produced in that event ($P_{(H)}$).
   b. The height of the debris reaching/passing a given grid cell and the probability of a person at that location being inundated (buried) by the debris ($P_{(S:H)}$). This is discussed in a later section.
   c. The probability that a person is present at a given location in their dwelling as the debris moves through it ($P_{(T:S)}$).

d. The probability that a person is killed if present and inundated by debris ($V_{(D:T)}$). In some risk assessments the vulnerability has been linked to landslide intensity, which is a combination of the landslide velocity and the volume of debris (e.g., Du et al., 2013). For this assessment a variable vulnerability has been adopted based on the velocity of the debris.

3. Combine 3(a)–(d) for each source area scenario to estimate the annual individual fatality risk at different locations below the slope at different event annual frequencies.

4. These values were then modelled using ArcGIS®. ArcGIS is used to interpolate between the risk calculated at given grid cells so as to produce contours of equal risk. A single contour was presented for each scenario (lower, middle and upper source volumes) for each event annual frequency, representing the estimated risk of 10^{-4} (ten to the minus four, or 1 chance in 10,000 of dying per year).

5. The annual individual fatality risk value of 10^{-4} was chosen as this has been used previously by Christchurch City Council and the Canterbury Earthquake Recovery Authority to delineate existing dwellings that are exposed to potentially unacceptable levels of risk from rockfalls.

Probability of inundation

$P_{(S:H)}$ is the probability of a person at a given location being inundated (buried) by the debris, should the person be present in that location as the debris moves through it. The height of debris passing a given location was estimated using the RAMMS model outputs. The maximum height of the debris reaching/passing a given grid cell at any time step during the modelled earth/debris flow was used. These were combined with simple models of probability (of inundation) as a function of the height of debris reaching/passing a given grid cell, where:

1. Probability of inundation $P_{(INUN)}$ = 0 if the maximum height of the debris reaching/passing the grid cell is ≤ 0.3 m.
2. Probability of inundation $P_{(INUN)}$ = 1 if the maximum height of the debris reaching/passing a given grid cell is ≥ 1 m.
3. Probability of inundation $P_{(INUN)}$ is between 0 and 1 for debris heights greater than 0.3 m but less than 1.0 m, adopting a linear interpolation.

The inundation height probabilities adopted for the assessment reflect the dominant movement mechanism and nature of the debris associated with the earth/debris flows. An earth/debris flow in loess (a fine grained material) with a flow height 0.3 m or less is unlikely to bury a person, as the debris is very fluid and would likely flow around a person, regardless of the debris velocity, as the debris has significantly less mass than a debris flow/avalanche comprising larger cobble and boulder-sized clasts.

Probability of a person being present

$P_{(T:S)}$ is the probability an individual is present in the portion of the slope when the debris moves through it. It is a function of the proportion of time spent by a person at a particular location each day and can range from 0% if the person is not present, to 100% if the person is present all of the time.

For planning and regulatory purposes it is established practice to consider individual risk to a "critical group" of more highly-exposed-to-risk people. For example, there are clearly identifiable groups of people (with significant numbers in the groups) who do spend the vast majority of their time in their homes – the very old, the very young, the disabled and the sick.

The assumption used in the previous risk assessment (Massey et al., 2012) for judging whether risk controls should be applied to individual homes was thus that most-exposed individuals at risk would be those who spend 100% of their time at home.

In other international rockfall risk assessments (e.g., Corominas et al., 2005), values ranging from 58% (for a person spending 14 hours a day at home) to 83% (for a person spending 20 hours a day at home), have been used to represent the "average" person and the "most exposed" person, respectively. However, in reality the most exposed person is still likely to be present 100% of their time.

For the land zoning assessments carried out by the Canterbury Earthquake Recovery Authority – with regards to rockfall and debris avalanche risk – their policy adopted an "average" occupancy rate, to assess the average annual individual fatality risk from rockfall across the exposed population in order to estimate the risk to the average person.

For this assessment, GNS Science has assumed the same "average" occupancy rate value adopted by the Canterbury Earthquake Recovery Authority, i.e., that an average person spends on average 16 hours a day at home (16/24 = 0.67 or 67%).

When a person is at home they tend to spend more time in their home than in their garden. Whilst in their home they cannot occupy every part of it at the same time. To proportion the person's time across their home, GNS Science has assumed that Port Hills homes have a footprint area (assuming a single story dwelling) of A_F = 100 m^2. The probability that a person will be occupying a given area within their home at any one time can be expressed as:

$$P_{(T;S)} = \frac{(0.67)}{(A_F / P_A)} \qquad \textbf{Equation 3}$$

Where 0.67 (67%) is the proportion of time a person spends in their home and P_A is the area of home occupied by a person at any one time. For this assessment, GNS Science has adopted a 2 m by 2 m (4 m^2) area to represent P_A. Therefore the probability of a person being present in a given 4 m^2 area within their home is 0.03 (3%) for the average person. No distinction is made between single versus multiple storey dwellings.

Probability of the person being killed if inundated by debris

This is the probability of a person being killed if present and inundated (buried) by debris. Vulnerability (*V*) depends on the landslide intensity, the characteristics of the elements at risk, and the impact of the landslide (Du et al., 2013).

This probability is expressed as vulnerability, the term used to describe the amount of damage that results from a particular degree of hazard. Vulnerability ranges between 0 and 1 and for fatality risk represents the likelihood of an injury sustained by the individual being fatal (1) and the possibility of getting out of the way to avoid being struck. For earth/debris flows people tend to be killed because they are inundated (buried) by debris, and if the velocity of the debris is rapid, it is possible that a person could be knocked off their feet and buried.

Studies from Hong Kong (e.g., Finlay et al., 1999) summarised the vulnerability ranges and recommended likelihood of death “if buried by debris”. The vulnerability of an individual in open space if buried by debris is given as 0.8–1.0 but if only hit by debris (and not buried) the vulnerability is 0.1–0.5, with recommended values of 1 and 0.3 respectively, assuming that it may be possible to get out of the way. For people in homes, it would be unlikely that a person would be able to take evasive action as they would not see the debris coming. However, this argument is counterbalanced by the level of protection a house may provide by stopping debris from entering it.

There is scant data on the performance of New Zealand homes when inundated by debris. However, in one such recent case of a home being impacted by earth/debris flow, the building offered little protection and the person was killed (Page, 2013). Finlay et al. (1999) recommend using a vulnerability factor of 0.9–1.0 if a person is in a building and if the building is hit by debris and collapses, but ranging to 0.0–0.1 if the debris strikes the building only.

However, Du et al. (2013) recommend that vulnerability and landslide intensity are also a function of the velocity of the debris when it impacts a person or building. Given that debris flows are triggered by rain it is most likely that people would be inside homes when debris flows trigger and therefore some protection is likely.

For loess earth/debris flows where the debris tends to be very fluid, it is likely that homes (even wooden ones) would provide some protection from the debris. In this risk assessment the probability of being inundated has been calculated separately as $P_{(S:H)}$. Therefore it would be appropriate to apply different vulnerabilities to different parts of the debris trail based on debris velocity.

For the risk assessment, the velocity ranges given in Australian Geomechanics Society (2007) were used, and these were linked to the vulnerabilities reported by Finlay et al. (1999) and Du et al. (2013), as no specific information on how Zealand buildings perform when impacted by debris was available (Table A1). The RAMMS model outputs were used to calculate debris velocity at different locations along the earth/debris flow trail, using the ranges given in Table A1.

Table A1 Vulnerability factors for different debris velocities used in the risk assessment.

Velocity (m/s)	Description	Vulnerability
>5	Building collapse or building inundated with debris, death almost certain.	1
0.5– 5	Inundated building with debris, but person not buried.	0.6
0.05–0.5	Building is hit but the person not buried and escape possible.	0.2
<0.05	Debris strikes building only.	0

A1.2 ROAD-USER RISK ASSESSMENT

This section builds on the method described by Taig and Massey (2014) for Wakefield Avenue, with a number of developments to take into account the more detailed slope collapse modelling undertaken in this assessment and the nature of the road section involved. This appendix describes:

- The background and context in terms of the road, its users and the slope-collapse hazards they face;

- The general modelling approach adopted;
- Main Road traffic parameters for this road section, including the effect of the road being blocked at the time of a slope collapse event;
- The estimation of individual road user risk per journey due to impact or inundation by slope collapse debris;
- The estimation of individual road user risk per journey due to driving into, or swerving to avoid, slope collapse debris on the road ahead;
- Calculation of aggregate risk per journey and other risk metrics derived from it (A1.1.6); and
- Calculation of “societal risk” for motor vehicle users.

It should be emphasised from the outset that the risk estimates for road users throughout this report use simple models which in many cases cannot be, and have not been, directly validated against hard evidence. There is a good deal of rough estimation, informed by the authors’ knowledge of the area and of transport accidents more generally. Risk estimates per journey are presented as approximate ranges of possible values; presenting “point values” might provide a spurious sense of the accuracy of the assessment results.

A1.2.1 Background and Context

The section of Main Road modelled here extends from the Clifton Terrace junction to the east, westward around Deans Head to a point just below 274A Main Road, on the eastward side of Monck’s Bay. There are no side roads along this section and limited property entrances in which vehicles could turn. To the north of the road along the section west of Deans Head is a flimsy fence and relatively deep water as shown in Figure A1.1 below (a screenshot taken from Google StreetView).

Figure A1.1 View northeast along westward section of road modelled (Google image).

To the east of Deans Head the road is bordered to the north by Sumner Beach, with Shag Rock approximately opposite Deans Head itself. Figure A1.2 shows a similar Google StreetView facing northwest along this section. As can be seen in Figures A1.1 and A1.2 there are currently "No Stopping" warning signs in place about rockfall, and containers placed below the cliff around Deans Head to protect road users from flyrock.

Figure A1.2 View northwest below 300 Main Road, showing Sumner Beach and Shag Rock (Google image).

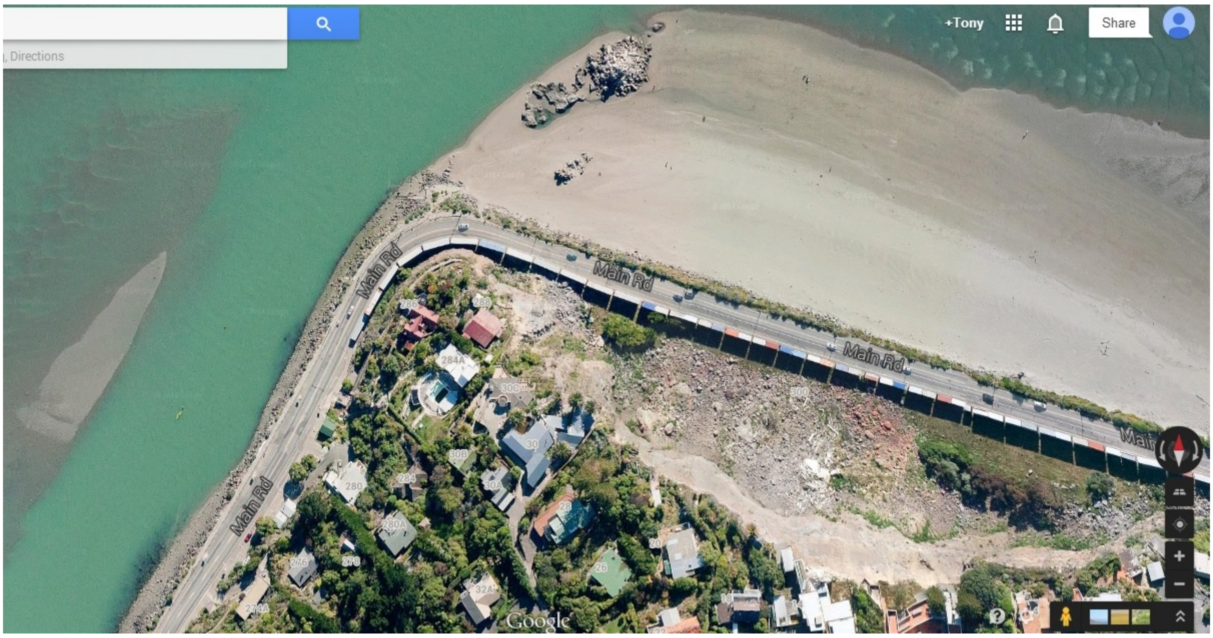


Figure A1.3 Google Earth view showing the contrast between the deep water immediately next to the road to the west of Deans Head, with the relatively shallow water/beach immediately next to the road to the east.

There is clear potential for rockfall or loess slip events to force road users into, or cause them to swerve into, the sea. This would be particularly hazardous to the western side of Deans Point where there is deep water close to the road. While it would certainly be very unpleasant to be forced off the road to the east of Deans Head it is less likely that people in vehicles would be subject to an immediate trapping/drowning hazard.

Two main hazards are assessed for road users on this road section:

1. Direct impact of rockfall or loess inundation onto them or their vehicles (including the possibility of being swept off the road into the sea or onto the beach); and
2. Driving into slope collapse debris on the road, or swerving to avoid debris on the road (with the possibility of either colliding with another vehicle or roadside object, or of leaving the road and driving into the sea or onto the beach).

Road user risk is assessed for:

a. Car occupants;

b. Bus occupants;

c. Truck occupants;

d. Motorcyclists;

e. Pedal Cyclists; and

f. Pedestrians.

The modelling approach is explained in Section A1.2.2.

A1.2.2 Risk Modelling Approach

Risk is assessed in terms of:

a. the risk per journey to road users as listed above; and

b. the risk per slope collapse event in terms of the probability of different numbers of people being killed.

Risk per journey is calculated cell by cell using the same grid developed and used in the dwelling risk assessment. To streamline the calculation, risk is calculated for cells running along the near (landward) and far (seaward) sides of the road from the cliff, rather than for all cells within the road area. The basic equation used to estimate risk per journey via Hazards 1 and 2 is (with dimensions of each term in brackets):

Risk (probability of death per journey) = Slope collapse event frequency (events/yr)

X Probability of death per event, if present (P_{death}/event)

X Time present per journey (years per journey).

Equation 4

Risk contributions are calculated for each cell, each road user and each slope collapse scenario then summed to provide overall estimates of risk per journey for each side of the road.

The risk per journey outputs are then used to estimate risk per year to heavy users of this section of road, and to estimate the average expected total annual fatalities due to slope collapse. The risks per journey are compared with the background motor vehicle crash risk that would be expected for this length of an average New Zealand urban road.

There is clear potential for multiple vehicles/road users to be involved in a single slope collapse event at this site, and a rough and ready calculation has been made of how often different numbers of people might be killed. This calculation starts from consideration of each slope collapse scenario, and uses a cruder and more simplistic approach to estimate the length of road in each case that would be subject to a “Major rockfall or inundation” hazard.

Both the individual and multiple road user risk calculations rely on being able to estimate how many road users travel over the road section in question, how fast they travel, and how many are present in the “at risk” areas for each slope collapse hazard for what proportion of the time. These issues are addressed in Section A1.2.3.

A1.2.3 Traffic Parameters on Main Road around Deans Head

For an individual road user’s trip, their travel speed determines the time they are at risk. Traffic does generally keep moving along this stretch of road, but at peak times becomes congested meaning vehicles are closer together (hence more are at risk) and travelling somewhat more slowly (hence at risk for longer periods) than at other times.

Average speeds and traffic densities (in terms of spacing between vehicles) taking into account periods of slow or static traffic are worked out using the traffic count data collected by Christchurch City Council on an hour by hour basis at Sumner West Surf Lifesaving Club. This is the nearest point for which traffic counts are available, and is a good proxy for this section of road as it lies on Main Road just to the East of Clifton Terrace, which would not be expected to take or add more than a very small percentage to traffic passing the Surf Club before/after passing Deans Head.

Table A1.1 provides the most recent available traffic counts for each hour of the week at the Surf Club both westward (towards Christchurch, Table A1.1a) and eastward (towards Sumner, Table A1.1b). These are counts of motor vehicle traffic; “vulnerable road users” (motorcyclists, pedal cyclists and pedestrians) are not included. While there is considerable use of this road section by pedal cyclists and a moderate level of motorcycle traffic, there is relatively light pedestrian usage as: 1) there is nowhere particular to walk from/to around Deans Head; and 2) many leisure walkers and dog walkers walk on Sumner Beach rather than on the road. More comprehensive counts of different road users are available for Main Road considerably further to the west (at the junction with Ferrymead Terrace) and have been used to inform rough estimates of the split of motor vehicles between cars and trucks. Rough estimates based on the authors’ own observations are made of cyclist, motorcyclist and pedestrian numbers of road users. Buses are considered separately (see below).

Table A1.1(a) Westbound traffic at Sumner Surf Club, September 2012.

Period	Mon	Tues	Wed	Thur	Fri	Sat	Sun	Averages	
Ending								4Day	7Day
01:00	4	2	3	5	7	13	19	4	8
02:00	2	3	2	2	5	12	14	2	6
03:00	3	2	3	13	2	17	14	5	8
04:00	5	5	4	12	9	12	11	7	8
05:00	14	8	10	26	24	12	9	15	15
06:00	48	50	40	50	55	29	21	47	42
07:00	220	203	216	216	222	85	51	214	174
08:00	847	837	793	759	733	176	119	809	609
09:00	776	814	758	898	803	311	277	811	662
10:00	562	567	530	613	617	511	483	568	555
11:00	504	465	422	493	517	660	639	471	529
12:00	462	468	391	472	539	704	725	448	537
13:00	384	380	338	394	458	522	644	374	446
14:00	361	353	303	329	428	545	628	336	421
15:00	373	334	326	416	451	466	609	362	425
16:00	403	376	368	410	465	439	585	390	435
17:00	383	377	378	381	418	417	414	380	395
18:00	302	294	315	301	323	229	219	303	283
19:00	283	264	296	310	334	218	211	288	274
20:00	205	167	193	218	238	220	123	196	195
21:00	86	87	115	101	108	69	77	97	92
22:00	58	70	66	69	79	63	41	66	64
23:00	37	34	39	32	51	52	23	36	38
00:00	8	7	9	15	30	35	8	10	16

Table A1.1(b) Eastbound traffic at Sumner Surf Club, September 2012.

Period	Mon	Tues	Wed	Thur	Fri	Sat	Sun	Averages	
Ending								4Day	7Day
01:00	20	13	20	27	42	74	107	20	43
02:00	8	9	5	6	16	36	44	7	17
03:00	5	5	5	25	4	33	27	10	15
04:00	5	5	3	12	9	11	11	6	8
05:00	4	2	3	7	6	3	2	4	4
06:00	13	14	11	13	15	8	6	13	11
07:00	59	54	58	58	60	23	14	57	46
08:00	211	208	197	189	183	44	30	201	152
09:00	276	289	269	320	285	110	99	289	236
10:00	214	216	202	233	235	194	184	216	211
11:00	242	222	202	236	248	316	306	226	253
12:00	271	274	229	277	315	413	424	263	315
13:00	393	389	346	403	470	534	659	383	456
14:00	441	432	371	402	523	667	769	412	515
15:00	502	449	438	559	608	628	819	487	572
16:00	583	545	532	593	672	634	845	563	629
17:00	632	621	622	629	688	688	681	626	652
18:00	738	720	771	735	790	560	536	741	693
19:00	461	431	483	505	543	356	344	470	446
20:00	267	218	251	284	309	285	160	255	253
21:00	168	169	224	198	210	134	152	190	179
22:00	152	184	174	180	207	164	109	172	167
23:00	134	124	140	113	183	184	81	127	137
00:00	38	36	42	75	147	174	38	48	79

There is a clear inverse correlation between traffic density and speed. Table A1.2 has been developed by the authors to provide a rough representation of the way in which vehicles speeds vary with traffic levels; it has been tailored so that, when coupled with the traffic counts here and in our Quarry Road report (Massey et al., 2014), the predicted average traffic speeds at different times of day are broadly consistent with our own (considerable) experience of using this road over the past two years. The average separations shown are those resulting from uniform distribution of the average number of vehicles in each category, assuming all travel exactly at the average speed.

Table A1.2 Correlation between traffic levels and average speeds/separations.

1-way vehicles/hr	Speed range (kph)		Average separation (m)	
	lower speed	upper speed	lower speed	upper speed
<400	40	50	>95	>120
400-600	38	48	95	120
600-800	36	45	60	75
800-900	32	40	40	50
900-1000	22	30	24	33
1000-1100	15	20	15	20
>1100	10	15	9	14

This table can now be used in combination with the traffic levels in Table A1.1 to provide estimates of the average traffic speeds for each hour of the day and day of the week, in both directions along the road. Average traffic speeds for the purpose of estimating average times at risk from slope collapse hazards are then estimated simply by averaging over 24 x 7 hours, to produce the following estimates:

- Average speed (both directions, lower) = 37.8 km/hr
- Average speed (both directions, upper) = 47.4 km/hr.

Note that the lower speed corresponds to higher risk estimates, as it results in longer dwell times in the at-risk areas. School buses are assumed to travel at lower speeds corresponding to the peak times at which they run. A summary of assumed numbers of road users, average speeds, and numbers of journeys per day for heavy road users (used as the basis for estimating annualised individual fatality risk for heavy road users) is provided in Table A1.3.

Table 1.3 Summary of road user numbers and average speeds (cars/trucks split as per Main Rd/Ferrymead Rd junction; cycles/pedestrians estimated by authors).

Road user	Trips/day, heavy user		Trips/year		Average speed		% of buses that are school buses
	lower	upper	vehicles	people	lower risk	higher risk	
Cars	1	2	4351954	6907761	47.4	37.8	
Buses	1	2	40444	646660	47.4	37.8	4.0%
Heavy goods	1	2	411	652	47.4	37.8	
Motorcycles	1	2	109575	109575	47.4	37.8	
Cyclists	1	2	365250	365250	25	15	
Pedestrians	1	2	36525	36525	5	3	

Within the average speeds applying for each hour, there is considerable variation in traffic density and speeds. Of particular importance for estimating multiple fatalities is the proportion of time for which vehicles are close together. A further estimate has been made as shown in Table A1.4(a) of the proportion of time for which vehicles might be either "nose to tail" (assumed 6 m apart) or "heavily congested but keeping moving" (assumed 20 m apart).

Table 1.4(a) Derivation of assumed proportions of time traffic is close-spaced.

	Lower Speed		Upper Speed		Nose to Tail (a)		Heavy but Moving (b)		Rest of Week (not a or b)	
Traffic Density (vehicles per hour)	Ave Speed	Count hrs/wk	Ave Speed	Count hrs/wk	Assumed % hour spent nose to tail	% of week with nose to tail traffic	Assumed % hour spent 'v heavy but moving'	% of week with heavy but moving traffic	Rem % of week	Ave vehicle separation
<400	40	243	50	243	0%	0.00%	0.5%	0.36%	71.97%	225.0
400-600	38	53	48	53	1%	0.16%	2.0%	0.32%	15.29%	86.0
600-800	36	33	45	33	5%	0.49%	10.0%	0.98%	8.30%	57.9
800-900	32	7	40	7	10%	0.21%	20.0%	0.42%	1.44%	42.4
900-1000	22	0	30	0	15%	0.00%	30.0%	0.00%	0.00%	27.4
1000-1100	15	0	20	0	20%	0.00%	40.0%	0.00%	0.00%	16.7
>1100	10	0	15	0	25%	0.00%	50.0%	0.00%	0.00%	11.4
						0.86%		**2.08%**		
	Assumptions for Societal Risk estimate:					**1.0%**		**2.0%**	**97.00%**	
	Assumed vehicle separation: every					6	(metres)	20		

Table A1.4(b) provides a summary of the assumptions used as to the proportions of time traffic is differently spaced for this assessment. The right hand column incorporates modified (increased) proportions of time with traffic very closely spaced which are applied for seismically triggered slope collapse scenarios, as explained below.

Table A1.4(b) Summary of assumed proportions of time versus traffic spacing.

Vehicle Separation (m)	% time (non-seismic)	% time (seismic triggers)
6	1.0%	10.0%
20	2.0%	1.8%
42	1.4%	1.3%
58	8.3%	7.5%
86	15.3%	13.9%
225	72.0%	65.4%

The further circumstance which is considered at this site, and leads to the modified proportions of time for which traffic is close-spaced for seismically triggered slope collapses, is the possibility that something happens directly associated with a slope collapse event which causes the road to block and traffic to back up in a queue in the at-risk road section. For scenarios where slope failure is primarily caused by severe rainfall/storm conditions, traffic would generally be expected to be lighter than average. For seismically-triggered cliff collapse though, the picture could be very different. Figure A1.4 shows the pattern of earthquakes of magnitude 4 or greater around the four largest quakes in the 2010/11 Canterbury earthquakes (based on GeoNET data).

Figure A1.4 shows that several earthquakes of high magnitude were experienced within a few tens of minutes of each of these large earthquakes – in the case of all but the December 2011 earthquake within a few minutes. It is known that at least one of the victims of rockfall in the 22 February 2011 earthquake was killed in an aftershock some hours after the main earthquake.

There is considered to be a significant possibility that a major seismically-triggered slope collapse might be preceded by a smaller but still significant shake causing the road around Deans Head to be blocked. If this occurred within a few minutes or tens of minutes of a larger earthquake, and if traffic had not been able to be diverted and evacuated from the road, there would then be expected to be a queue of nose to tail traffic in the at risk area on either side of the blockage.

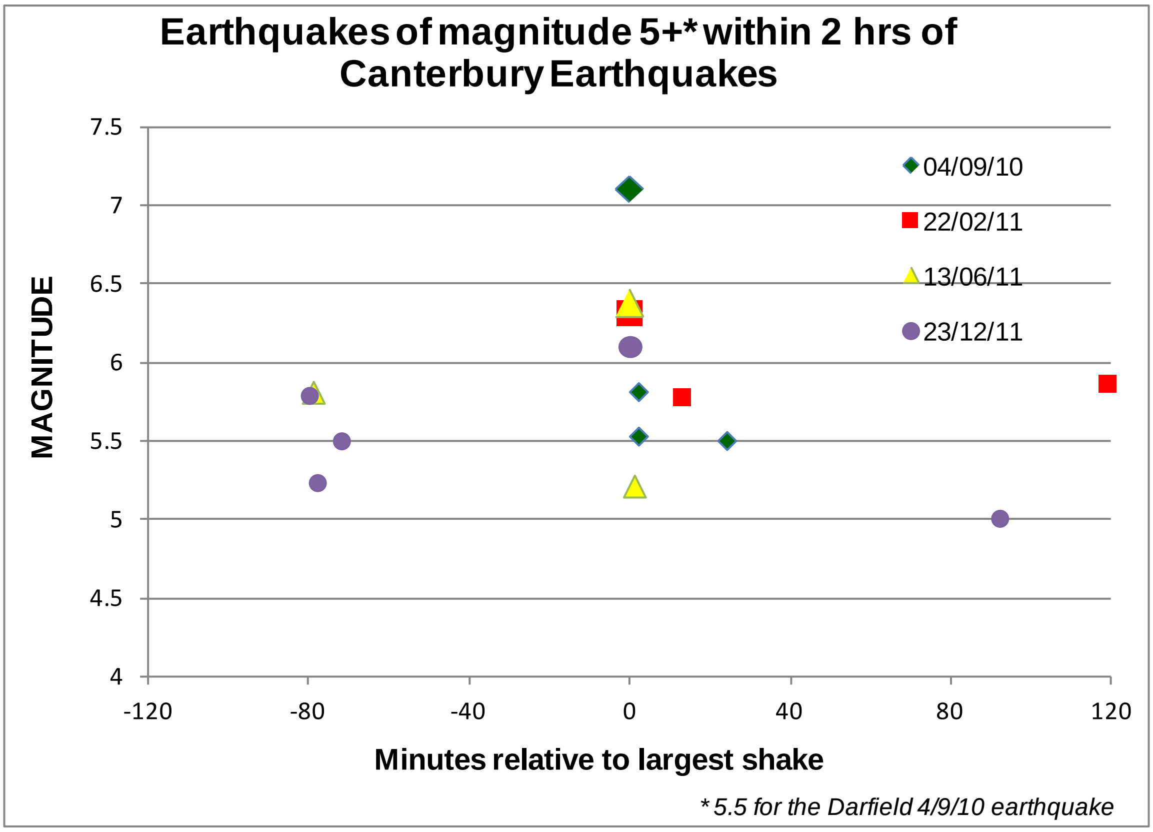


Figure A1.4 Large earthquakes within two hours of Canterbury Earthquakes.

To take this into account assumptions are included in this assessment as to:

a. the probability of a seismically generated substantial cliff collapse being preceded by a shake sufficient to cause the road at Deans Head to be blocked (assumed to be a few tens of % based on the possibility of significant preceding shakes as per Figure A1.4); and

b. the probability, if so, that traffic would be queued at the tightest spacing shown in Table A1.4(b) (i.e., 6 m apart on average) on either side of the blockage (also assumed to be a few tens of % based on the time likely to be involved in stopping traffic entering the at-risk area and evacuating that already there).

The percentages of the time for which other traffic densities apply are then reduced *pro rata* to the additional “nose to tail” time applied for seismically-triggered cliff collapse.

These traffic volumes and densities are used in the calculation of societal risk (frequency of accidents killing N or more people) and in the derivation of further risk estimates from risks per journey as described in Section A1.2.4.

A1.2.4 Individual risk per journey – Hazard 1 (impacted/inundated by debris)

In reviewing our model for the impact of rockfall on road users for loess debris we have taken the opportunity to refine and update the "boulder impact" model in order to improve the calculation of the probability that a random boulder passing through a cell will strike a road user whose centre is also within that cell. Vulnerabilities (probabilities of death if in the path of a boulder) have then been reviewed to take into account the different circumstances at Deans Head, in particular the presence of deep water next to the road west of Deans Head.

The impact of loess inundation is then assessed by analogy with the model used for dwelling occupants, but with reduced vulnerabilities to take into account that road users, in contrast with people in dwellings, are all outdoors and facing their direction of travel at all times.

A1.2.4.1 Rockfall modelling

A road user located within a 1 m by 1 m cell could be hit by a boulder passing through that cell or through the cells either side, as illustrated in Figure A1.5 for cell width W, boulder diameter d and person diameter D.

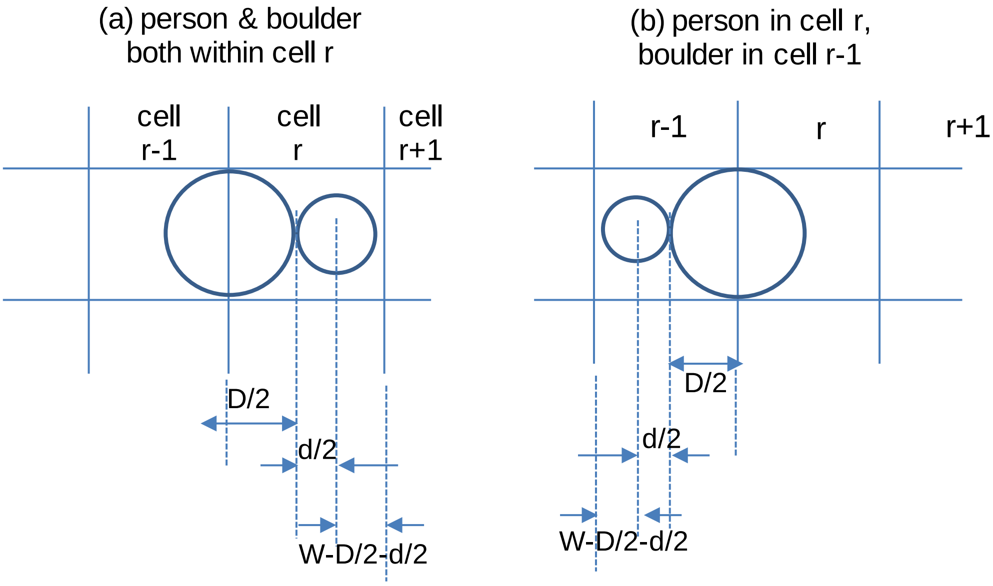


Figure A1.5 Possible boulder/road user collision configurations

In the first situation, if (D+d)/2 > W then collision is inevitable. But in this assessment that is not the case; we have D = 1 m, d = 0.5 m and W = 1 m. So with the person located with their centre on the left edge of the cell as in Figure A1.5(a), there is a gap of width W – D/2 – d/2 within which the centre of the boulder can pass without striking the person. As the person shifts to the right this gap decreases, reaching zero when the person's centre is D/2 + d/2 from the right hand edge of the cell (W – D/2 – d/2 from the left edge). There is thus an average gap of width 0.5(W – D/2 – d/2) pertaining over a distance (W – D/2 – d/2) from the left hand edge of the cell, and the same again on the right. The proportion of the cell within which the boulder can pass without striking the person is therefore:

2 (right & left side) x (0.5/W).(W-D/2-d/2) average gap as proportion of cell width

x (2/W).(W-D/2-d/2) proportion of cell width over which gap present.

$= (W-D/2-d/2)^2 / W^2$

The probability P1,r of the person in cell r being struck by a boulder passing randomly through cell r is thus $1 – (W\text{-}D/2\text{-}d/2)^2 / W^2$. **Equation 5**

We now consider a boulder passing randomly through cell r-1 to the left of the cell containing the person situated on the extreme left edge of cell r. If the boulder centre is within (D/2 + d/2) of the right edge of cell r-1 then it will strike the person. The width of the space within cell r-1 within which the boulder must pass to strike the person in cell r decreases linearly as the person shifts to the right, reaching 0 when the person centre is D/2 + d/2 from the left edge of the cell. There is thus an average width of:

0.5 (D/2+d/2)/W as a proportion of the width of the cell, applying over a distance

(D/2+d/2)/W proportion of cell r from the left edge of the cell,

for which the boulder will strike the person. The same probability of the person in cell r being struck applies to a boulder passing randomly through cell r+1 to the right of cell r. Denoting these probabilities P1,r-1 and P1,r+1 respectively we then have:

$P_{1,r-1} = P_{1,r+1} = 0.5\ (D/2+d/2) / W^2$ **Equation 6**

For a single boulder passing randomly through each of these three cells, the probability P_1 of the person being in the path of 1 or more boulders is given by:

$P_1 = 1 – (1 – P_{1,r-1}) \times (1 – P_{1,r}) \times (1 – P_{1,r+1})$ **Equation 7**

The probability of death for road user j per single boulder passing through each of these cells is now calculated as:

$P_{death,1,j} = P_{1,j} \times V_{1,j,rockfall}$ **Equation 8**

A significant complication now is that the number of boulders passing through each cell may be different. This might be possible to model if the cells formed a continuous straight line along an axis of the model grid, but in this case they do not. We therefore introduce the approximation for the purposes of calculating the probability of being killed by N boulders passing through the cell that THE SAME number of boulders passes through the cells either side. The probability of death for N boulders passing through the cell is then:

$P_{death,N,j} = 1 – (1 - P_{death,1,j})^N$ **Equation 9**

This is then multiplied by the proportion of a year for which the user is present in the cell (based on the average travel speeds in Table A1.3 above) and the frequency of the triggering event which gave rise to the N boulders per cell (as per equation 4 above) to calculate the contribution of this cell and this slope collapse scenario to the road user's individual risk per journey.

The values of the parameters used in this assessment are as follows:

No. of boulders passing through cell – taken directly from dwelling model output

Years present in cell per journey – as shown in Table A1.5 (based on average road user speeds as in Table A1.3 above).

Table A1.5 Road user speeds and times per journey within 1 m cell.

	Ave Speed (kph)		Time in cell (years)	
Road user	lower risk	higher risk	lower risk	higher risk
Car Occupant	47.4	37.8	2.4E-09	3.0E-09
Bus Occupant	47.4	37.8	2.4E-09	3.0E-09
Truck Occupant	47.4	37.8	2.4E-09	3.0E-09
Motorcyclist	47.4	37.8	2.4E-09	3.0E-09
Pedal Cyclist	25.0	15.0	4.6E-09	7.6E-09
Pedestrian	5.0	3.0	2.3E-08	3.8E-08

Vulnerabilities – Values of 0.4 (lower) and 0.7 (higher) are used for motorcyclists, and of 0.3 (lower) and 0.5 (higher) for all other road users. We gave serious consideration to modifying the vulnerability for road sections adjacent to deep water (to the west of Deans Head), but decided that for rockfall the primary hazard was of being crushed by boulders. Note that these are probabilities of death if in the path of a single boulder; each successive boulder confers the same probability of death again. This contrasts with some of our earlier assessments in which we applied the vulnerability to the "Probability of being in the path of 1 or more boulders". This approach (treating vulnerability as independent of number of boulders) was based on the primary contribution to survival being the ability of the individual to get out of the way of boulders. With the lack of any safe place of escape in the event of rockfall at Deans Head we consider it more appropriate here to assume that getting out of the way is unlikely. We recognise that motor vehicles will provide some modest protection against boulders relative to the vulnerable road users (cyclists and pedestrians), but consider that for pedestrians and pedal cyclists this is offset by their greater ability to hear what is going on off the road and to take evasive action before boulders fall. Motorcyclists are considered to have the worst of both worlds (vulnerability if struck, and inability to hear environmental noises), hence their higher assumed vulnerability.

Example calculation: Hazard 1, Rockfall

For all road users we assume D = 1 m, d = 0.5 m, W = 1 m.

So from equation 5, $P_{1,r} = 1 – (W\text{-}D/2\text{-}d/2)^2 / W^2 = 1 – 0.25^2 =$ **0.9375**

And from equation 6, $P_{1,r-1} = P_{1,r+1} = 0.5\ (D/2+d/2)^2 / W^2 = 0.5 \times 0.752 =$ **0.4395.**

So $P_1 = 1 – (1\text{-}0.9375) . (1\text{-}0.4395) . (1\text{-}0.4395) =$ **0.9804**

And $P_{1,death} = 0.9804 . Vj$ for road user j. So for a pedestrian, with vulnerability in the range 0.3 to 0.5, we calculate $P_{1,death}$ in the range **0.29–0.49**.

Note that this is the same for any rockfall scenario. So let us now consider the Band 3 seismic trigger scenario. This has an estimated frequency of occurrence of 0.00159 events per year, and is estimated to generate 3.71 boulders passing through cell 37345.

The contribution to pedestrian risk per journey from cell 37345 is now given by:

Risk/journey	=	$(1 - (1-P_{1,death})^N)$	(probability of death if present)
	x	T_j	(years present in cell per journey)
	x	f	(frequency of slope collapse scenario)
which	=	(0.73–0.92)	(probability of death if present)
	x	(2.3×10^{-8} to 3.8×10^{-8})	(years in cell per journey, Table A1.3)
	x	0.00159	(frequency of slope collapse)
	=	**3.6×10^{-11} to 6.0×10^{-11}**	risk/journey contribution

A1.2.4.2 Loess inundation modelling

The threat to road users from loess failures is more to do with inundation and being carried along by moving loess than with being crushed by boulders. As such we consider, as for dwelling occupants, that the depth and the velocity of the loess flow are critically important for road users.

As for dwelling occupants, the debris flow parameters (height and velocity) were calculated using the RAMMS model and used as input to our road user risk assessment. Road user speeds and times per journey within a cell are the same as for rockfall (see Table A1.5). As for dwelling occupants, a probability of inundation was calculated for each road cell modelled on the basis that:

P(inundation) = 0 for h (debris height) < 0.3 m;

P(inundation) = 1 for h > 1.0 m; and

P(inundation) = linearly interpolated in-between , i.e., $P_h = (h-0.3)/(1-0.3)$.

The probability of death if present is then taken as the product of P(inundation) with a vulnerability factor which in this case is estimated separately for road cells adjacent to deep water. A number of significant considerations lie behind the selection of appropriate vulnerability values for road users in comparison with those assumed for dwelling occupants in other parts of this report. In particular:

1. Dwelling occupants are asleep for a good part of the time and indoors, with some degree of isolation from the outside environment, for more of it. In contrast, road users are awake, and are generally in a state of continuous vigilance for hazards on the road ahead of them.
2. Motor vehicles would be expected to provide a reasonable degree of protection against being pushed along on a flat surface or gentle slope in a stream of mud and loess. Any such protection would quickly become a liability, though, if the vehicle were pushed into deep water, which would be a likely outcome in a major debris flow to the west of Deans Head.

3. There is an inverse correlation between the protection provided by a vehicle and the extra awareness and manoeuverability available to a cyclist or pedestrian. These factors are considered to offset each other and the same vulnerabilities are applied to all road users except motorcyclists, who (as for rockfall) are considered to have the worst of both worlds in terms of vulnerability if impacted by debris and lack of prior awareness of a developing environmental incident.

With these considerations in mind it was considered appropriate to use somewhat higher thresholds of debris velocity to correlate with probabilities of death for road users than for dwelling occupants. The thresholds adopted are as shown in Table A1.6.

Table A1.6 Debris velocity thresholds for road user vulnerability.

Debris Speed	Associated Road User Assumptions
<0.5 m/s	Road users are assumed able to escape/avoid debris
0.5–2 m/s	Road users are assumed to have sufficient power to make headway against debris; P(death) small
	Note: 2 m/s of debris 1m deep with density 1.5 te/m^3 corresponds to 3 kW per m length of road user
2–5 m/s	Road users' own power may not be sufficient to overcome debris; generally medium P(death)
>5 m/s	Road users not able to resist debris inundation; higher P(death)

The resulting vulnerabilities used in the assessment based on the principles discussed above are shown in Table A1.7.

Table A1.7 Road user vulnerabilities for loess inundation. Note: these are all conditional on inundation of the road – they are multiplied by P(inundation to calculate probability of death.

(a) Other than next to deep water areas to west of Dean's Head				
	Motorcyclists		**All other road users**	
Debris speed	lower risk	higher risk	lower risk	higher risk
V > 5m/s	0.4	0.7	0.3	0.5
5 > V > 2m/s	0.25	0.4	0.15	0.3
2 > V >0.5 m/s	0.1	0.2	0.05	0.1
V < 0.5m/s	0	0	0	0

(b) On FAR side of road (i.e. nearer to sea) next to deep water areas				
	Motorcyclists		**All other road users**	
Debris speed	lower risk	higher risk	lower risk	higher risk
V > 5m/s	0.9	1	0.8	1
5 > V > 2m/s	0.5	0.8	0.4	0.6
2 > V >0.5 m/s	0.2	0.5	0.1	0.2
V < 0.5m/s	0	0.1	0	0.05

(c) On NEAR side of road (i.e. further from sea) next to deep water areas				
These are taken as average of (a) and (b)				
	Motorcyclists		**All other road users**	
Debris speed	lower risk	higher risk	lower risk	higher risk
V > 5m/s	0.65	0.85	0.55	0.75
5 > V > 2m/s	0.375	0.6	0.275	0.45
2 > V >0.5 m/s	0.15	0.35	0.075	0.15
V < 0.5m/s	0	0.05	0	0.025

Using equation 4 the contribution to road risk per journey from each source failure in a given cell is then given by:

Risk per journey		P(inundation) x Vulnerability	(Prob of death if present)
	X	T	(Time at risk, years/journey)
	X	f	(Source failure frequency).

The time at risk per journey is exactly the same as that for rockfall (Table A1.3). The source failure frequencies for loess are not known, but as for the dwelling assessment we have considered a range of possibilities from 0.05 per year (once in 20 years to 0.002 per year (once in 500 years). The choice of frequency for each of source areas 1 and 2 is a user input to the spreadsheet used for road user risk assessment.

Example Calculation: Hazard 1, Loess Inundation

If source area 1 fails, generating the middle of the volume estimates considered in this assessment, the estimated debris height and velocity at cell 72909 (on the landward side of the road in a section to the west of Deans Head next to deep water) are 0.82 m and 6.3 m/s respectively.

Thus P(inundation) = (0.82 – 0.3) / (1 – 0.3) = **0.75**

Now consider a motorcyclist present in this cell. The vulnerability (Table A1.7c) is in the range 0.65–0.85 because the debris speed is greater than 5 m/s. If the source failure frequency is 0.01 (1 in 100 years) then the risk per journey contribution from cell 72909 is given by:

Risk	=	0.75 x (0.65 to 0.85)	(Probability of death if present)
	X	(2.4×10^{-9} to 3.0×10^{-9})	(Time/journey in cell, Table A1.3)
	X	0.01	(Source failure frequency)
	=	**1.2×10^{-11} to 1.9×10^{-11}**	

A1.2.4.3 Hazard 2: Driving into or swerving to avoid debris on the road

There are a number of possibilities here, modelled using the event tree shown in Figure A1.6.

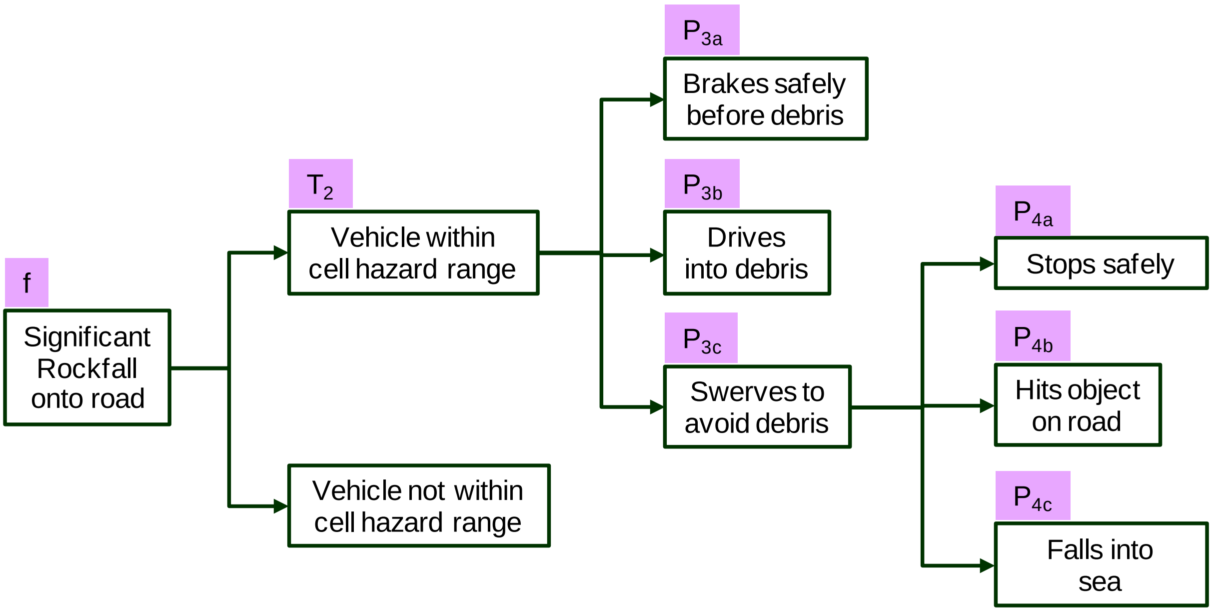


Figure A1.6 Event tree – driving into/swerving to avoid debris on road.

Assuming the road user is "within the hazard range for the cell" (see discussion for T_2 below), the first set of options relates to whether the road user stops safely, drives into the debris or swerves to try and avoid it. If the road user swerves then they may stop safely, or hit another object on the road, or go off the road (assuming this is in the direction away from the slope collapse this would inevitably mean "towards the sea" for this stretch of road). This hazard is not applied to pedestrians, who can stop virtually instantly and in any case would be unlikely to hurt themselves very much by walking into some debris on the road.

The risk contribution per journey is given by (from equation 4):

Risk/journey	=	$\{P_{3b}.V_{3b} + P_{3c}.(P_{4b}.V_{4b} + P_{4c}.V_{4c})\}$	(Probability of death if present)
	X	T_2	(Time at risk per journey)
	X	f	(Frequency of source failure)

Equation 10

The parameters in this equation, and appropriate values for Deans Head, are discussed below, working from left to right in Figure A1.6.

The **time at risk** in our earlier work on Wakefield Avenue (Taig and Massey, 2014), which considered longer sections of road each of several tens of metres long, was considered to be the braking distance/time ahead of debris on the road. This was originally adopted here, but the resulting values of risk from this hazard appeared anomalously high. This was because the braking distance is long relative to the cell size. To illustrate the point, suppose the braking distance is 20 m and the cell width (as in this assessment) 1 m. There are then 20 x 1 m cells upstream of a particular cell at risk from this hazard for a given cell. But each of these 20 cells is also associated with 19 other cells either side of the cell of interest. To calculate precise contributions from each possible upstream cell to each "target" cell would be extremely complex. For the current assessment we have made the simplifying approximation that each cell has associated with it the risk associated with 1m of travel across it. Thus the time at risk is the time to travel 1 m, which is that shown in Table A1.3, though the hazard itself is still one associated with being close to debris when it falls on the road in front of the road user.

Before applying the relatively complex formula involving steps 3 and 4 in Figure A1.6, we apply a simple criterion to decide whether or not a given cell has sufficient debris on the road to trigger this hazard at all. The criteria used are:

For rockfall – 0.5 boulders passing per cell

For loess – 0.2 m depth of debris in cell

These parameters are included as user-adjustable inputs in the assessment spreadsheet prepared in parallel with this report.

Given that the criteria as appropriate (for rockfall and loess) are met, the probabilities applied at levels 3 and 4 in Figure A1.6 are shown in Tables A1.8 and A1.9 respectively.

Starting with the options at level 3, and bearing in mind the nature of this hazard and its association with being “close to but not in” the cell where the debris falls, the first option is that the road user is able to brake safely. Most modern vehicles have good brakes and most drivers are alert for hazards on the road ahead of them, so there is assumed to be a good chance of this outcome (30–50%).

Table A1.8 Probabilities of different actions on approach to slope collapse debris on Main Road.

Level 3 Probabilities and Vulnerability									
					P3a Values				
				Road user	lower risk	higher risk			
P_{3a}	Probability of braking safely before rockfall IF within hazard range of debris when it hits road	Simple estimation - brake times known to be conservative	Dimensionless - P(brake OK|event & in BD)	Car Occupant	0.50	0.30			
				Bus Occupant	0.50	0.30			
				Truck Occupant	0.50	0.30			
				Motorcyclist	0.50	0.30			
				Pedal Cyclist	0.50	0.30			
				Pedestrian	1.00	1.00			
					P3b Values				
				Road user	lower risk	higher risk			
P_{3b}	Probability of driving into rockfall debris on road	Determined as 1 - P_{3a} - P_{3c}	Dimensionless - P(drive into|event & in BD)	Car Occupant	0.30	0.20			
				Bus Occupant	0.30	0.20			
				Truck Occupant	0.30	0.20			
				Motorcyclist	0.30	0.20			
				Pedal Cyclist	0.30	0.20			
				Pedestrian	0.00	0.00			
					V3b Values				
				Road user	lower risk	higher risk			
V_{3b}	Probability of death if drive into rockfall debris on road	Estimate based on NZ road crash data for relevant collisions	Dimensionless - P(death|collision)	Car Occupant	0.030	0.050	Based on NZ motor vehicle crash analysis		
				Bus Occupant	0.010	0.017	Assumed	3	x better than cars
				Truck Occupant	0.010	0.017	Assumed	3	x better than cars
				Motorcyclist	0.060	0.100	Assumed	2	x worse than cars
				Pedal Cyclist	0.015	0.025	Assumed	2	x better than cars
				Pedestrian	0.000	0.000			
					P3c Values				
				Road user	lower risk	higher risk			
P_{3c}	Probability of swerving to avoid rockfall debris on road	Simple estimation of reasonable range	Dimensionless - P(swerve|event & in BD)	Car Occupant	0.2	0.5	Assumed same for all road users		
				Bus Occupant	0.2	0.5	(except zero for pedestrians)		
				Truck Occupant	0.2	0.5			
				Motorcyclist	0.2	0.5			
				Pedal Cyclist	0.2	0.5			
				Pedestrian	0	0			

Table A1.9 Probabilities of various outcomes of swerving to avoid debris on Main Road.

					Values - NEAR side		Values - FAR side		
				Road user	lower risk	higher risk	lower risk	higher risk	**Notes**
P_{4a}	Probability of coming to a safe stop if swerve to avoid debris on road	Simple estimation/assumption; higher for FAR side of road from sea (swerving assumed to be to seaward to avoid debris)	Dimensionless - P(safe|swerve, event & in BD)	Car Occupant	0.7	0.5	0.6	0.2	Assumed same for all road users. Higher for NEAR side of road from sea as swerving assumed to be to seaward to avoid debris, and more room available from NEAR side
				Bus Occupant	0.7	0.5	0.6	0.2	
				Truck Occupant	0.7	0.5	0.6	0.2	
				Motorcyclist	0.7	0.5	0.6	0.2	
				Pedal Cyclist	0.7	0.5	0.6	0.2	
				Pedestrian	0.7	0.5	0.6	0.2	
					Values - NEAR side		**Values - FAR side**		
				Road user	lower risk	higher risk	lower risk	higher risk	**Notes**
P_{4b}	Probability of colliding with roadside/other object if swerve to avoid debris on road	Simple estimation/assumption; higher for FAR side of road from sea (swerving assumed to be to seaward to avoid debris)	Dimensionless - P(coll|swerve, event & in BD)	Car Occupant	0.2	0.3	0	0	Simple assumption made for all road users on NEAR side of road; on far side there is little to hit; the outcome of swerving significantly off the road would be to end up in the sea or on the beach
				Bus Occupant	0.2	0.3	0	0	
				Truck Occupant	0.2	0.3	0	0	
				Motorcyclist	0.2	0.3	0	0	
				Pedal Cyclist	0.2	0.3	0	0	
				Pedestrian	0.2	0.3	0	0	
					V4b values				
				Road user	lower risk	higher risk	**Notes**		
V_{4b}	Probability of death if collide with roadside/other object when swerve to avoid	Estimate based on NZ road crash data for collisions with most relevant types of object - worst case would be another car	Dimensionless - P(death|coll after swerve)	Car Occupant	0.03	0.05	Based on NZ motor vehicle crash analysis		
				Bus Occupant	0.01	0.02	Assumed	3	x better than cars
				Truck Occupant	0.01	0.02	Assumed	3	x better than cars
				Motorcyclist	0.06	0.10	Assumed	2	x worse than cars
				Pedal Cyclist	0.02	0.03	Assumed	2	x better than cars
				Pedestrian	0.00	0.00			
					P4c values - near side (to cliff)		**P4c values - far side (from cliff)**		
				Road user	lower risk	higher risk	lower risk	higher risk	**Notes**
P_{4c}	Probability of driving into sea if swerve to avoid debris on road	Simple estimation/assumption; higher for NEAR side of road to sea	Dimensionless - P(sea|swerve, event & in BD)	Car Occupant	0.1	0.2	0.4	0.8	Assumed same for all road users; cyclists' generally better road awareness balanced by worse stability if swerving rapidly
				Bus Occupant	0.1	0.2	0.4	0.8	
				Truck Occupant	0.1	0.2	0.4	0.8	
				Motorcyclist	0.1	0.2	0.4	0.8	
				Pedal Cyclist	0.1	0.2	0.4	0.8	
				Pedestrian	0.1	0.2	0.4	0.8	
					V4c values - west of Dean's Head		**V4c values - East of Dean's Head**		
				Road user	lower risk	higher risk	lower risk	higher risk	**Notes**
V_{4c}	Probability of death if drive into sea when swerve to avoid debris on road	High for cells adjacent to deep water; low for cells from Shag Rock to the east where shallow/sandy	Dimensionless - P(death|sea after swerve)	Car Occupant	0.2	0.5	0.05	0.1	Assumed same for all road users. Chance of significant head injury for cyclists considered to balance extra difficulty of escape from motor vehicles
				Bus Occupant	0.2	0.5	0.05	0.1	
				Truck Occupant	0.2	0.5	0.05	0.1	
				Motorcyclist	0.2	0.5	0.05	0.1	
				Pedal Cyclist	0.2	0.5	0.05	0.1	
				Pedestrian	0.2	0.5	0.05	0.1	

If the road user does not brake safely before encountering the debris then they must either drive into/onto it or swerve around it. At Deans Head swerving is almost certainly the more dangerous option as it is assumed to involve a good chance of driving into the sea, particularly for road users on the FAR (seaward) side of the road. Simple estimates have been made of the probability of swerving as opposed to driving into the debris, with the probability of swerving estimated in the range 20-50% for all road users. This takes into account that although swerving is the more dangerous behaviour it is also an instinctive response to a hazard appearing immediately in front of a road user.

It should be clear from the above discussion and Tables A1.8 and A1.9 that there is no great science behind these probability estimates; they provide a crude attempt to provide cautious (erring if anything on the side of pessimism) estimates of the risk from this second hazard. No attempt is made to distinguish between road users; in particular the greater nimbleness of motor or pedal cycles is assumed to offset the greater controllability under sudden braking/swerving of four-wheeled vehicles. All of these probabilities are entered as user inputs into the companion spreadsheet used to calculate road user risk so that they can be varied and their effects explored.

Moving onto the fourth stage of the event tree in Figure A1.6, there are three possible outcomes if the road user swerves to try and avoid the debris: 1) stopping safely; 2) driving into an object on the road; or 3) driving off the road (which in this instance means into the sea or onto the beach). The values assumed for the probabilities of each outcome are shown in Table A1.9; the most important probability is that of driving into the sea which it is assumed could be high for the road section to the west of Deans Head, but which would be considerably lower to the east where most of the beach is uncovered except at very high tides.

Finally we now need to consider vulnerabilities in the event of any of the three steps labelled 3b, 4b and 4c (driving into debris, into another object on the road or into the sea respectively). Again we have made simple estimates, informed initially by analysis of New Zealand motor vehicle crash data based largely on cars. Table A1.10 shows a summary of the numbers of crashes and the number which were fatal, aggregated over the three years 2010–2012, classified by the type of object with which a collision took place based on the NZ Ministry of Transport classifications provided on the left of the table (NZ MoT 2010, 2011, 2012).

On the right of Table A1.10 we have indicated which classes of crash targets have been aggregated to provide a starting point for estimating probabilities of death involving crashes into debris, other objects on the road and the sea respectively. Table A1.11 then shows the overall average % of collisions that are lethal for each of those aggregated categories of crash target. The figures for rural roads and all NZ roads are shown for comparison, while the values assumed for Deans Head are shown on the right of the table. For debris and objects on the road we have assumed a range roughly spanning that encountered on NZ roads generally. For driving into the sea we consider the road section to the west of Deans Head to be a particularly dangerous one because of the conjunction of closeness to the road, absence of a strong barrier, and deep and fast flowing water. We have therefore adopted 20% (the probability of crashes into water proving fatal on urban roads) as a lower value for this road section, and have used 50% as an upper probability of death. It is assumed that cyclists' probability of injury while crashing/swerving balances out the difficulty of escaping from four-wheel drive vehicles that fall into water, so that once again we have used the same vulnerability values for all road users.

Table A1.10 Lethality of motor vehicle crashes on New Zealand urban roads.

Objects Struck	Urban Roads 2010-2012 combined							Used as analog for
	Total No. of Collisions		No. of Fatal Collisions		Lethality			
	Total	In Darkness	Total	In Darkness	Overall	In Darkness	Not in Darkness	
Driven or accompanied animals	1	0	1	0	1.000			
Bridge or approach rails	40	20	2	1	0.050	0.050	0.050	
Upright cliff or bank	287	149	5	4	0.017	0.027	0.007	
Debris on the road	10	3	0	0	0.000	0.000	0.000	Debris
Over bank or cliff	153	77	11	4	0.072	0.052	0.092	
Fence letterbox hoarding etc	943	482	27	15	0.029	0.031	0.026	Object
Guard rail	154	87	5	3	0.032	0.034	0.030	
House or building	243	118	10	6	0.041	0.051	0.032	
Traffic island or median	206	113	12	7	0.058	0.062	0.054	Debris
Phone boxes bus shelters etc	102	49	1	1	0.010	0.020	0.000	
Kerb	393	185	16	11	0.041	0.059	0.024	Debris
Slip washout or flood	5	2	0	0	0.000	0.000	0.000	Debris
Parked vehicle	1519	623	17	9	0.011	0.014	0.009	Object
Train	17	6	1	0	0.059	0.000	0.091	
Pole or post	983	563	36	24	0.037	0.043	0.029	Object
Broken down or accident vehicles	293	57	1	0	0.003	0.000	0.004	Object
Roadworks signs or drums	12	6	0	0	0.000	0.000	0.000	Object
Traffic sign or signals	281	144	12	8	0.043	0.056	0.029	Object
Tree	690	394	27	18	0.039	0.046	0.030	
Stray or wild animals	12	5	0	0	0.000	0.000	0.000	
Ditch	118	54	5	3	0.042	0.056	0.031	
Into water river or sea	40	24	8	6	0.200	0.250	0.125	Sea
Other	173	88	3	3	0.017	0.034	0.000	
TOTALS	**6675**	**3249**	**200**	**123**	**0.030**	**0.038**	**0.022**	

Table A1.11 Collision lethality and assumptions for Deans Head.

Target Types	% NZ Collisions Fatal			Dean's Head assumptions	
	Urban Roads	Rural Roads	All Roads	lower risk	higher risk
Object on road	2.3%	5.6%	3.7%	**0.030**	**0.050**
Debris	4.6%	3.8%	4.4%	**0.030**	**0.050**
Sea	20.0%	11.2%	13.0%	**0.200**	**0.500**

A1.2.4.4 Example calculation

We return to cell 79209 in the source area 1 (mid volume) scenario. The debris height is 0.82 metres, so the criterion is met (>0.2 m) for the “Drive into/swerve avoiding debris on road” hazard to apply.

The probabilities to evaluate the event tree Figure A1.6 and work out the probability of death for a motorcyclist if present and then the contribution to risk per journey are shown in Table A1.12.

Table A1.12 Example calculation for Hazard 2 – drive into/swerve avoiding debris.

Parameter	Definition: Probability of ...	Lower risk value	Higher risk value
P3b	... driving into debris	0.3	0.2
V3b	... death if drive into debris	0.06	0.01
P3c	... swerving around debris	0.2	0.5
P4b	... swerving into object on road	0	0
V4b	... death if swerve into object on road	0.06	0.1
P4c	... swerving into sea	0.4	0.8
V4c	... death if swerve into sea	0.2	0.5
RESULT	P(death if present) = P3b.V3b + P3c(P4b.V4b + P4c.V4c)	= 0.0018 + 0.2(0+0.4x0.2) = 0.0178	= 0.002 + 0.5(0+0.8x0.5) = 0.202
x T2	Time at risk	2.4×10^{-9}	3.0×10^{-9}
x f	Frequency of source failure	0.01	0.01
FINAL RESULT	Contribution to P(death per journey) = P(death if present) x T2 x f	$\mathbf{4.3 \times 10^{-13}}$	$\mathbf{6.1 \times 10^{-12}}$

Table A1.12 illustrates what prove to be some general points for this assessment:

a. The contribution from swerving into the sea dominates this hazard for road sections to the west of Deans Head; and

b. The contribution to risk per journey from this hazard is significantly lower than that from Hazard 1 (directly impacted/inundated by debris).

A1.2.5 Road user risk per journey and risk parameters derived from it

The parameters shown in the above tables are uncertain. As in our previous work on road user risk from rockfall, inputs and outputs are presented as ranges from “reasonable lower” to “reasonable upper” values. No statistical significance is attached to these ranges; the results are regarded as providing a sensible range, given the associated uncertainties, within which to assume the actual risk might lie. Perhaps the single largest uncertainty is in the volume of material which flows from the debris sources; as for the dwelling risk assessments this has been explicitly considered by carrying out all assessments three times, for upper, central and lower estimates of debris source volumes.

The risk equation is evaluated for each cell in the grid for each slope-collapse scenario considered, as described in sections A1.1.5 and A1.1.6. The grid used was simplified relative to that used in modelling dwelling risk by excluding cells that did not form part of the roadway in order to streamline the calculation process; in all other respects the rockfall modelling used to estimate individual road-user risk was identical to that used to estimate individual dwelling occupant risk.

As in the dwelling occupant assessment, the set of scenarios modelled covers:

1. Four seismic trigger scenarios ranging from 0.1–0.4 g up to 2–5 g peak ground acceleration, with an increasing probability as shaking increases that cliff collapse will be triggered;
2. Four non-seismically triggered cliff collapse scenarios (corresponding to different severities of weather-induced rockfall); and
3. Source areas 1 and 2, each with lower, mid and upper volume options and frequency options from 0.05 to 0.005 per year.

The risk per journey in a given cell is then calculated by summing over all sources and both hazards (the “impacted/inundated by” hazard 1 and the “drive into/swerve avoiding” hazard 2).

The overall risks per journey were calculated by summing over all cells making up the NEAR (landward) side of Main Road and the FAR (seaward) side of Main Road, allowing the change in risk across the width of the road readily to be compared with each other and with the existing motor vehicle crash risk (based on average statistics for New Zealand urban roads, from Ministry of Transport publications on road crashes and casualties and on number of journeys and distance travelled by different road user groups.

The individual risk per journey is then used to calculate individual risk per year for heavy users of the road, the average expected fatalities per year, and the average time expected between fatal accidents as shown in Table A1.13. Note that this estimate of time between fatal accidents is based on the assumption that multiple fatality accidents do not contribute significantly to risk which is not valid for Deans Head. The predicted frequencies of fatal accidents obtained from Table A1.13 thus overstate how often actual fatal accidents (which on average will kill more than one person) occur; a better estimate of fatal accident frequency is provided by the Societal Risk assessment described in Section A1.1.7.

Current New Zealand road traffic accident statistics were used to provide comparison information on the risk road users would face in their ordinary travel up and down this section of Main Road for a journey of the same length (660 m) as that covered in the risk assessment model.

Table A1.13 Calculation of risk parameters of interest from single cell risk per journey.

Aggregation of Risk Parameters for Cells		
(a) Risk per journey		
Risk R_{ij} for road user j within cell i =	$R1_{ij} + R2_{ij}$	
Risk R_j per journey to road user j =	sum of R_{ij} for all relevant i	
	(all cells on uphill side or downhill side of road, as appropriate)	
(b) Other key risk parameters		
Annual Individual Fatality Risk for user j	$= Rj \times M_{j,ind}$	$M_{j,ind}$ = Journeys/year by individual heavy road user of type j
Average expected fatalities per year, user j	$= Rj \times M_{j,tot}$	$M_{j,tot}$ = Journeys/year by ALL road users of type j
Probability of 1 or more fatal accidents/year (road user type j)	$= Pj = 1 - (1-R_j)^{Mj,tot}$	
Probability of 1 or more fatal accidents/year (among ALL road users)	$= 1 - (1-P_{car}) \times (1-P_{motorcycle}) \times (1-P_{cycle}) \times (1-P_{pedestrian})$	

A1.2.6 Multiple fatality ("societal") risk estimation

An important consideration at Deans Head is that there is the possibility of many road users being killed in a single slope collapse incident, if either: 1) traffic is dense at the time of the accident; or 2) traffic has been stopped before the main slope collapse occurs. A simple, crude calculation has been carried out of how often different numbers of fatalities might occur, using a highly simplified model of how many people die under each slope collapse scenario, in combination with estimates of how often different traffic densities will be encountered (taking into account the possibility of a prior earthquake having blocked the road when the main slope collapse occurs as discussed in A1.1.3 above), to construct a chart showing how often a number of deaths in excess of a specified number might be expected.

The simplified modelling approach is as follows:

1. For each scenario, a length of road is calculated from the available data on debris passing each cell for which a simple "devastation" criterion is met. This is defined as 10 or more boulders per m cell for rockfall, or 20 kW power of loess per m cell.
2. The proportion of time for which different densities of traffic apply (Table A1.4) are used with the calculated devastated length of road for each scenario to estimate the number of vehicles in the devastation zone for each scenario.
3. Simple assumptions are then made that there are 1.5 occupants per vehicle and that the proportion of people killed if the rockfall criterion is met will be 90% for rockfall if next to deep water (70% elsewhere) and 80% for loess if next to deep water (40% elsewhere), to translate numbers of vehicles into numbers of deaths
4. The frequency with which each number of deaths is expected is obtained by multiplying the frequency of the relevant source failure event by the proportion of the time for which the relevant traffic density is expected to apply.

The whole calculational path is shown in Table A1.14.

A table is then constructed from the 3rd and 4th elements of Table A1.14 showing the frequency with which events involving N or more fatalities are expected, and is used to construct a complementary cumulative distribution function chart or “f/N curve” to graph the data.

Major approximations and assumptions made here include:

a. Ignoring road users other than motor vehicles;

b. Ignoring any road sections not meeting the “devastated” criterion;

c. Simple estimates of vulnerabilities for all road sections; and

d. Assumed proportions of time for which different traffic densities apply.

The resulting f/N curve should be treated as indicative rather than definitive.

Table A1.14 Multiple fatalities and frequency calculations.

1. Length of Road meeting "Devastated" Criterion

Deep Water or Elsewhere	Side of road	S1_low	S1_mid	S1_upper	S2_low	S2_mid	S2_upper	EQ Band 1	EQ Band 2	EQ Band 3	EQ Band 4	Non EQ Band 1	Non EQ Band 2	Non EQ Band 3	Non EQ Band 4
Deep water	NEAR	5	14	112	0	32	55	0	0	0	37	0	0	0	0
	FAR	0	12	120	0	23	45	0	0	0	0	0	0	0	0
Elsewhere	NEAR	12	55	128	0	0	0	0	0	85	304	0	0	0	0
	FAR	0	54	134	0	0	0	0	0	0	159	0	0	0	0
Road length assumed (deep water)		**2.5**	**13**	**120**	**0**	**27.5**	**50**	**0**	**0**	**0**	**18.5**	**0**	**0**	**0**	**0**
Road length assumed (elsewhere)		**6**	**54.5**	**134**	**0**	**0**	**0**	**0**	**0**	**42.5**	**231.5**	**0**	**0**	**0**	**0**

2(a) N vehicles affected in given scenario, DEEP WATER

Vehicle Separation (m)	% time	S1_low	S1_mid	S1_upper	S2_low	S2_mid	S2_upper	EQ Band 1	EQ Band 2	EQ Band 3	EQ Band 4	Non EQ Band 1	Non EQ Band 2	Non EQ Band 3	Non EQ Band 4
6	1.0%	0.4	2.2	20	0	4.6	8.3	0	0	0	3.1	0	0	0	0
20	2.0%	0.1	0.7	6	0	1.4	2.5	0	0	0	0.9	0	0	0	0
42	1.4%	0.1	0.3	2.8	0	0.6	1.2	0	0	0	0.4	0	0	0	0
58	8.3%	0	0.2	2.1	0	0.5	0.9	0	0	0	0.3	0	0	0	0
86	15.3%	0	0.2	1.4	0	0.3	0.6	0	0	0	0.2	0	0	0	0
225	72.0%	0	0.1	0.5	0	0.1	0.2	0	0	0	0.1	0	0	0	0

2(b) N vehicles affected in given scenario, ELSEWHERE

Vehicle Separation (m)	% time	S1_low	S1_mid	S1_upper	S2_low	S2_mid	S2_upper	EQ Band 1	EQ Band 2	EQ Band 3	EQ Band 4	Non EQ Band 1	Non EQ Band 2	Non EQ Band 3	Non EQ Band 4
6	1.0%	1	9.1	22.3	0	0	0	0	0	7.1	38.6	0	0	0	0
20	2.0%	0.3	2.7	6.7	0	0	0	0	0	2.1	11.6	0	0	0	0
42	1.4%	0.1	1.3	3.2	0	0	0	0	0	1	5.5	0	0	0	0
58	8.3%	0.1	0.9	2.3	0	0	0	0	0	0.7	4	0	0	0	0
86	15.3%	0.1	0.6	1.6	0	0	0	0	0	0.5	2.7	0	0	0	0
225	72.0%	0	0.2	0.6	0	0	0	0	0	0.2	1	0	0	0	0

3 Fatalities per event

Vehicle Separation (m)	% time (non-seismic)	S1_low	S1_mid	S1_upper	S2_low	S2_mid	S2_upper	EQ Band 1	EQ Band 2	EQ Band 3	EQ Band 4	Non EQ Band 1	Non EQ Band 2	Non EQ Band 3	Non EQ Band 4
6	1.0%	1	8	37	0	6	10	0	0	7	45	0	0	0	0
20	2.0%	0	2	11	0	2	3	0	0	2	13	0	0	0	0
42	1.4%	0	1	5	0	1	1	0	0	1	6	0	0	0	0
58	8.3%	0	1	4	0	1	1	0	0	1	5	0	0	0	0
86	15.3%	0	1	3	0	0	1	0	0	1	3	0	0	0	0
225	72.0%	0	0	1	0	0	0	0	0	0	1	0	0	0	0

4 Event Frequencies	*source failure=>*	*0*	*0*	*0.05*	*0*	*0*	*0.05*	*0.159094*	*0.025138*	*0.001594*	*4.899E-05*	*0.12*	*0.0044*	*0.0002*	*0.00001*
% time	% time (seismic triggers)	S1_low	S1_mid	S1_upper	S2_low	S2_mid	S2_upper	EQ Band 1	EQ Band 2	EQ Band 3	EQ Band 4	Non EQ Band 1	Non EQ Band 2	Non EQ Band 3	Non EQ Band 4
1.0%	10.0%	0.0E+00	0.0E+00	5.0E-04	0.0E+00	0.0E+00	5.0E-04	1.6E-02	2.5E-03	1.6E-04	4.9E-06	1.2E-03	4.4E-05	2.0E-06	1.0E-07
2.0%	1.8%	0.0E+00	0.0E+00	1.0E-03	0.0E+00	0.0E+00	1.0E-03	2.9E-03	4.6E-04	2.9E-05	8.9E-07	2.4E-03	8.8E-05	4.0E-06	2.0E-07
1.4%	1.3%	0.0E+00	0.0E+00	7.2E-04	0.0E+00	0.0E+00	7.2E-04	2.1E-03	3.3E-04	2.1E-05	6.4E-07	1.7E-03	6.3E-05	2.9E-06	1.4E-07
8.3%	7.5%	0.0E+00	0.0E+00	4.2E-03	0.0E+00	0.0E+00	4.2E-03	1.2E-02	1.9E-03	1.2E-04	3.7E-06	1.0E-02	3.7E-04	1.7E-05	8.3E-07
15.3%	13.9%	0.0E+00	0.0E+00	7.6E-03	0.0E+00	0.0E+00	7.6E-03	2.2E-02	3.5E-03	2.2E-04	6.8E-06	1.8E-02	6.7E-04	3.1E-05	1.5E-06
72.0%	65.4%	0.0E+00	0.0E+00	3.6E-02	0.0E+00	0.0E+00	3.6E-02	1.0E-01	1.6E-02	1.0E-03	3.2E-05	8.6E-02	3.2E-03	1.4E-04	7.2E-06

A2 APPENDIX 2: RESULTS FROM SURVEYS OF CADASTRAL AND MONITORING SURVEY MARKS

PlotID	Mark name	Source	Method
1	SR11	Aurecon	TS
2	SR14	Aurecon	TS
3	SR23	Aurecon	TS
4	SR200	Aurecon	TS
5	SR201	Aurecon	TS
6	SR300	Aurecon	TS
7	SR301	Aurecon	TS
8	SR302	Aurecon	TS
9	SR303	Aurecon	TS
10	MN V DP 35669	LINZ	RTK GPS
11	PEG	LINZ	RTK GPS
12	PEG	LINZ	RTK GPS
13	PEG	LINZ	RTK GPS
14	IS XVI DP 41827	LINZ	RTK GPS
15	PEG XXIV DP 41827	LINZ	RTK GPS
16	MN XIV DP 41827	LINZ	RTK GPS
17	PEG DP 7436	LINZ	RTK GPS
18	IT XXI SO 14067	LINZ	RTK GPS
19	IT VI DP 49638	LINZ	RTK GPS
20	PEG DP 49638	LINZ	RTK GPS
21	PEG DP 49638	LINZ	RTK GPS
22	PEG DP 2012	LINZ	RTK GPS

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

SURVEY MARKS
Index Map

Deans Head
Christchurch

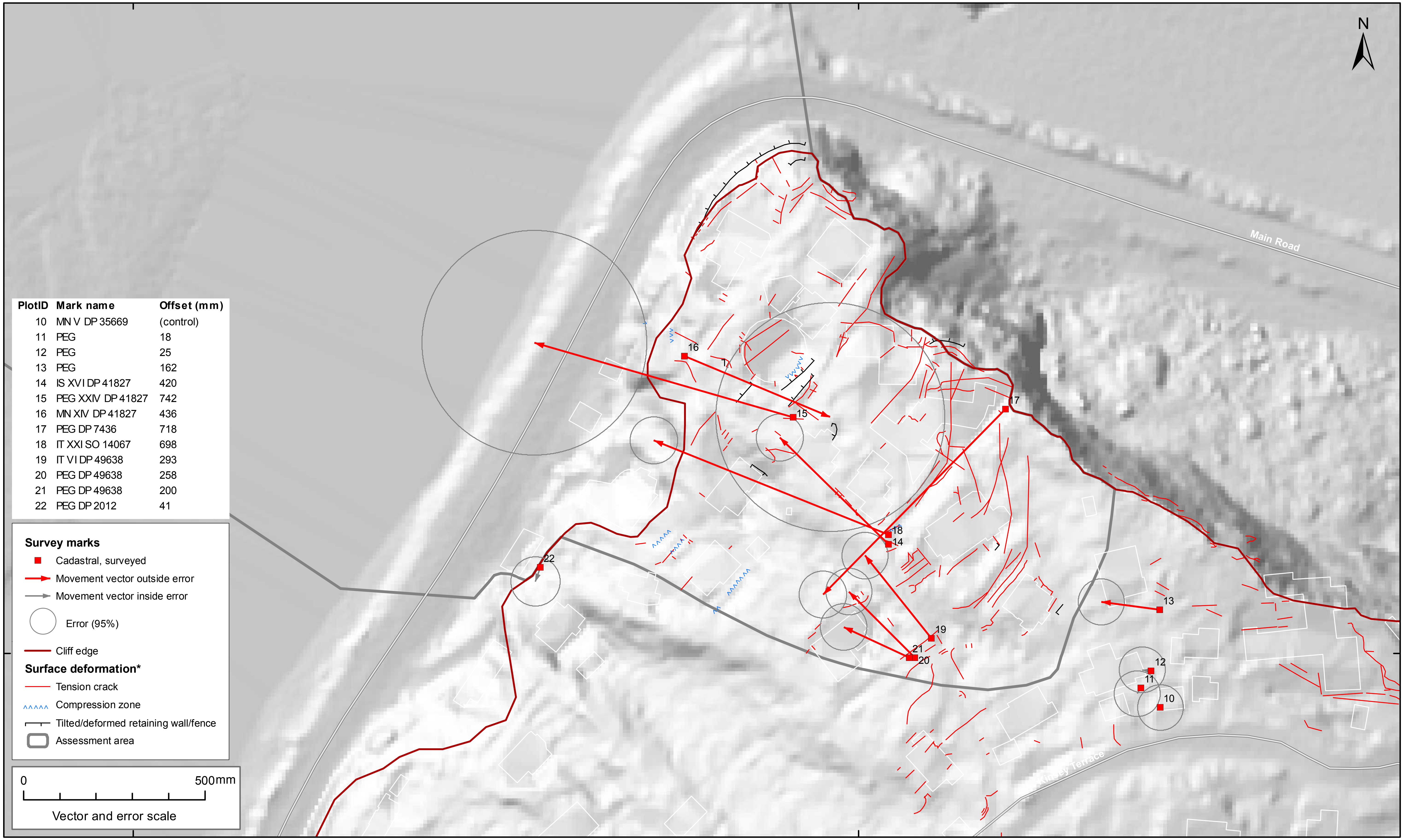
APPENDIX 2

Map 1

FINAL

REPORT: CR2014/77

DATE: June 2014



PlotID	Mark name	Offset (mm)
10	MN V DP 35669	(control)
11	PEG	18
12	PEG	25
13	PEG	162
14	IS XVI DP 41827	420
15	PEG XXIV DP 41827	742
16	MN XIV DP 41827	436
17	PEG DP 7436	718
18	IT XXI SO 14067	698
19	IT VI DP 49638	293
20	PEG DP 49638	258
21	PEG DP 49638	200
22	PEG DP 2012	41

Survey marks

Cadastral, surveyed

Movement vector outside error

Movement vector inside error

Error (95%)

Cliff edge

Surface deformation*

Tension crack

Compression zone

Tilted/deformed retaining wall/fence

Assessment area

0 500mm

Vector and error scale

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Cadastral Marks (Source: LINZ)
Total Movement - Pre 22-02-2011 to 30-10-2012

Deans Head
Christchurch

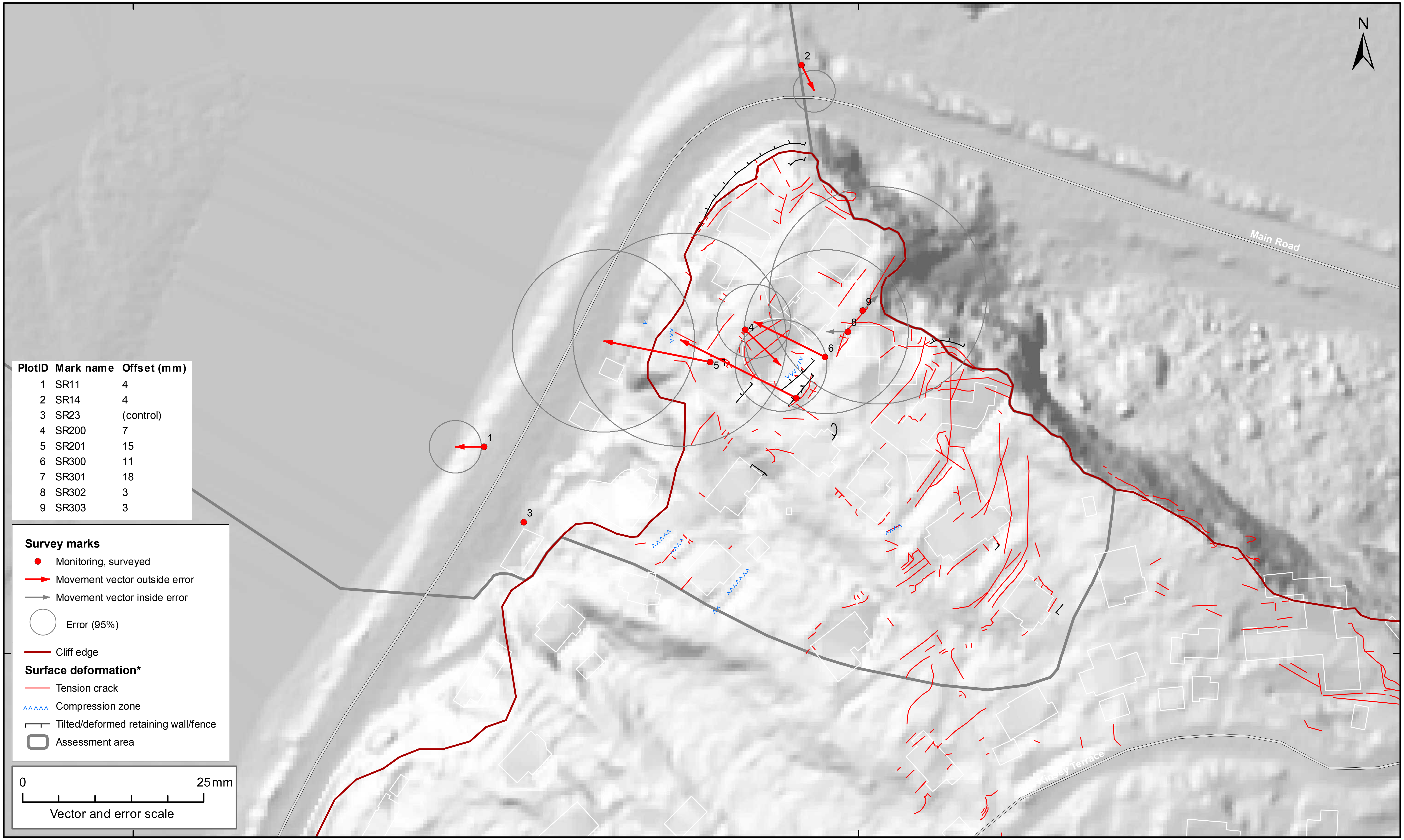
APPENDIX 2

Map 2

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Monitoring Marks (Source: Aurecon NZ Ltd)
Earthquake Related Movement - 16 Apr 2011 (M 5.3)

Deans Head
Christchurch

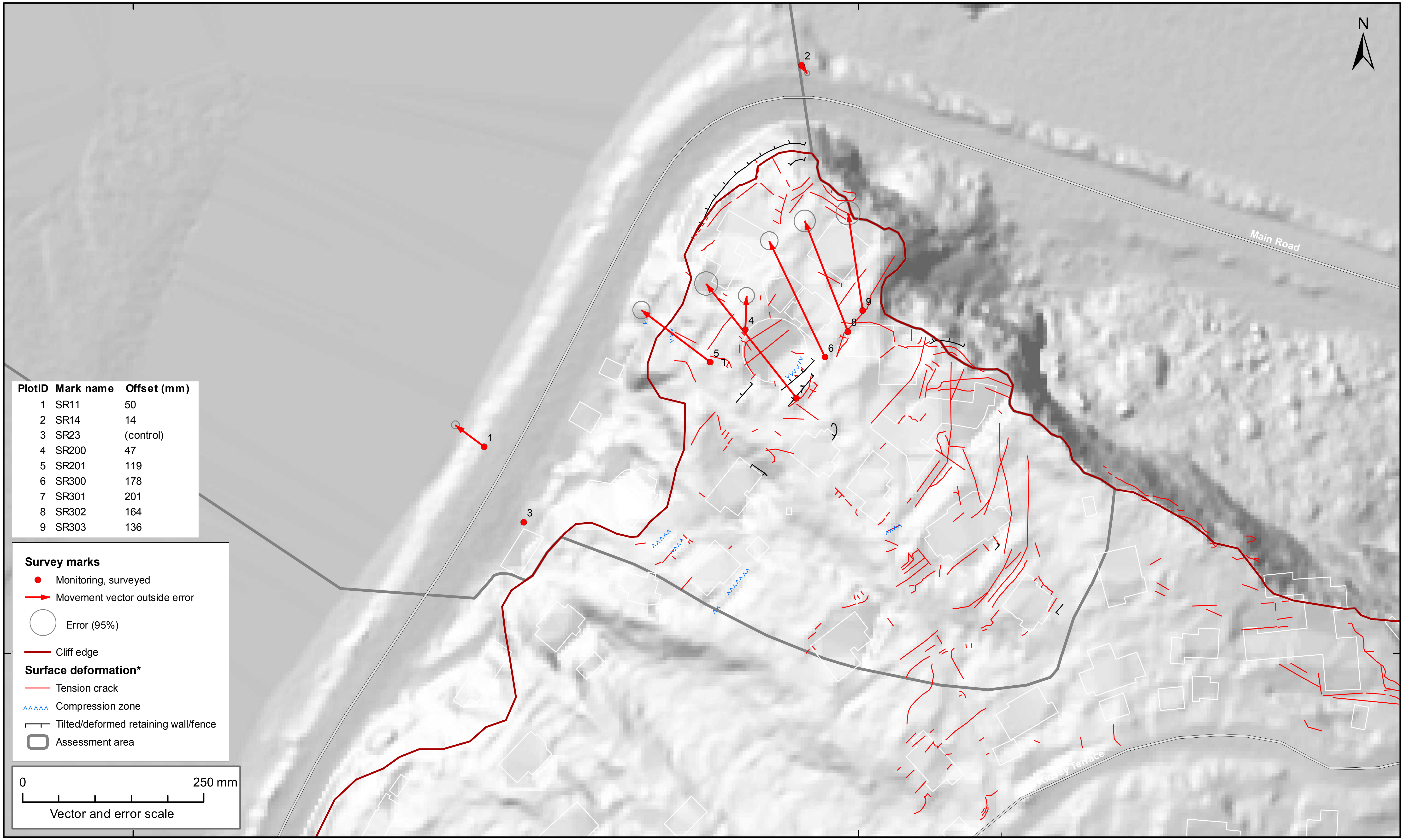
APPENDIX 2

Map 3

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Monitoring Marks (Source: Aurecon NZ Ltd)
Earthquake Related Movement - 13 Jun 2011 (M 6.0)

Deans Head
Christchurch

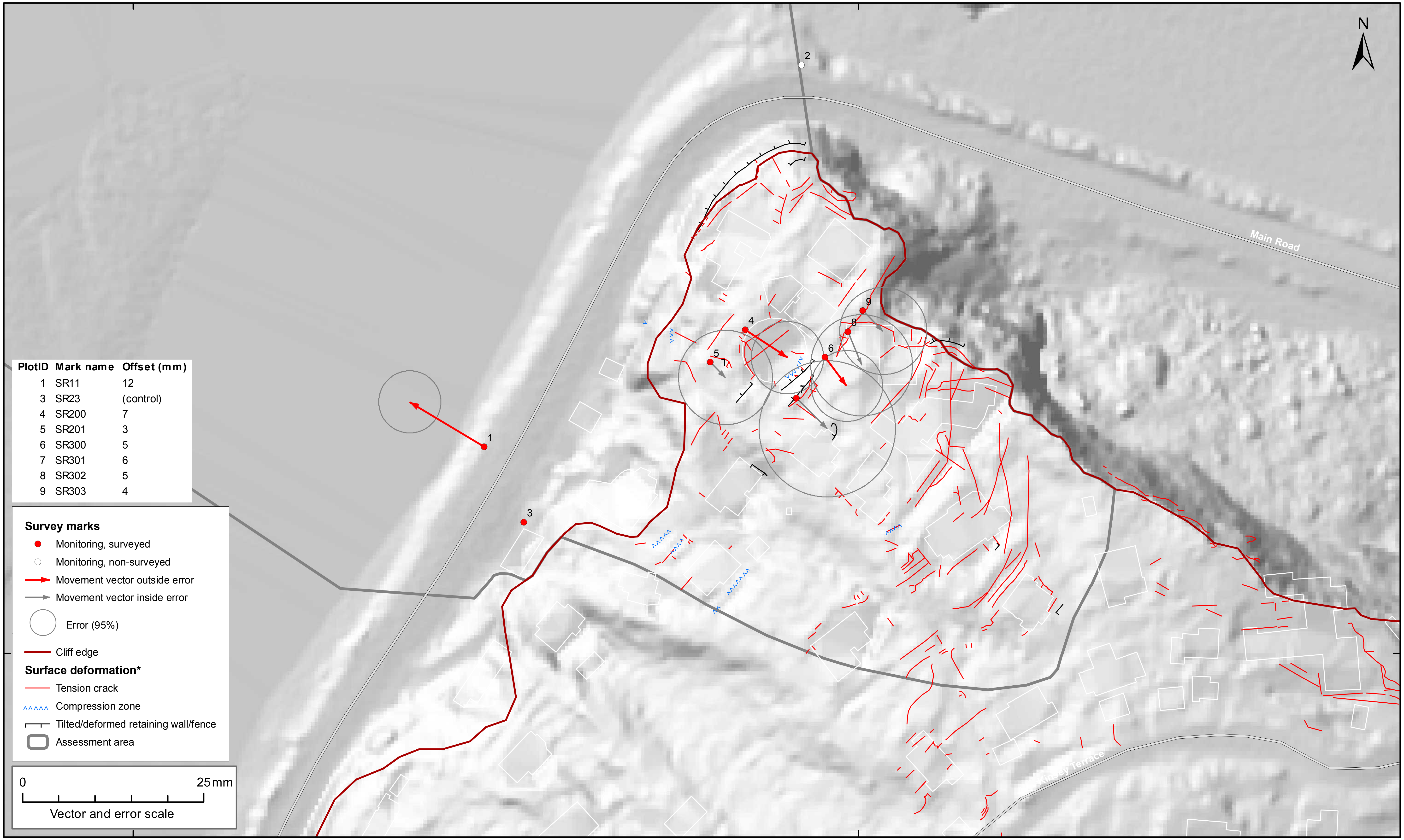
APPENDIX 2

Map 4

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Monitoring Marks (Source: Aurecon NZ Ltd)
Earthquake Related Movement - 23 Dec 2011 (M 5.9)

Deans Head
Christchurch

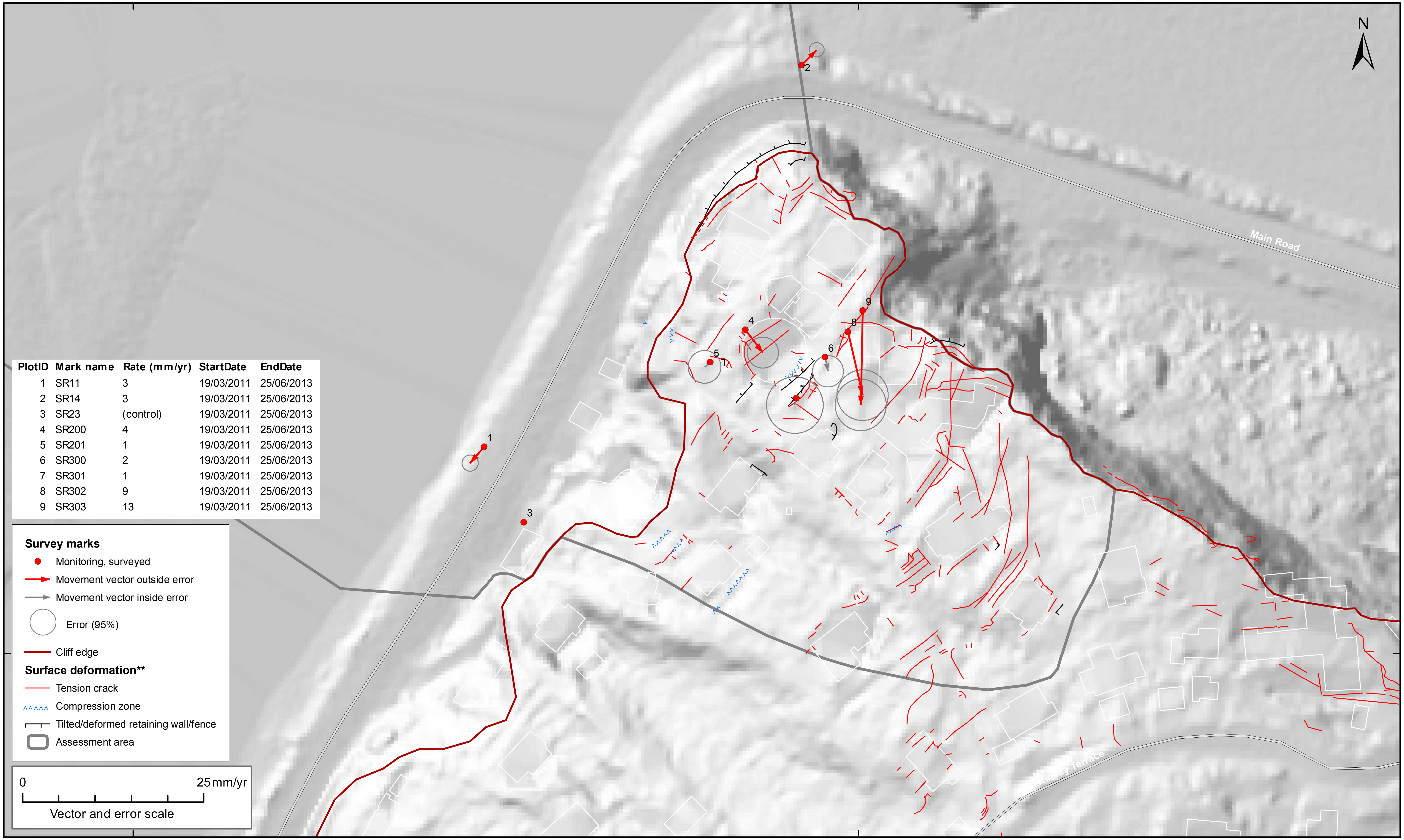
APPENDIX 2

Map 5

FINAL

REPORT: CR2014/77

DATE: June 2014



PlotID	Mark name	Rate (mm/yr)	StartDate	EndDate
1	SR11	3	19/03/2011	25/06/2013
2	SR14	3	19/03/2011	25/06/2013
3	SR23	(control)	19/03/2011	25/06/2013
4	SR200	4	19/03/2011	25/06/2013
5	SR201	1	19/03/2011	25/06/2013
6	SR300	2	19/03/2011	25/06/2013
7	SR301	1	19/03/2011	25/06/2013
8	SR302	9	19/03/2011	25/06/2013
9	SR303	13	19/03/2011	25/06/2013

SCALE BAR: 0 50 100 m

EXPLANATION:
* Movement with assumed earthquake induced landslide movement and tectonic (earthquake) movement removed. Movement estimated from least squares adjustment (assuming a linear trend).
** Taken from the report CR2012/317
Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints provided by Christchurch City Council (20/02/2012).
PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL

CHK: CM, GA, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Monitoring Marks (Source: Aurecon NZ Ltd)
Filtered Linear Movement*

Deans Head
Christchurch

APPENDIX 2

Map 6

FINAL

REPORT: CR2014/77

DATE: June 2014

A3 APPENDIX 3: PAST LANDLSIDES IN THE PORT HILLS AND BANKS PENINSULA

Past Landslides in the Port Hills and Banks Peninsula

Introduction

Not many landslides in loess occurred during the 2010/11 Canterbury earthquakes, and where they did occur they generally comprised small (<100 m^3) disrupted falls and avalanches of loess from steep slopes (adopting the terminology of Keefer 1984).

Several of the mass movements being investigated by GNS Science for Christchurch City Council are areas where the 2010/11 earthquakes caused significant cracking in loess, where the cracks are thought to relate to landslide processes, mainly coherent soil slides/slumps (Keefer, 1984) rather than shallow inelastic behaviour of the ground during shaking.

It is not well understood how these mass movements in loess will perform in the future, especially in the Class I areas (where the landslide, if it were to occur, could cause loss of life). The findings of work presented in this report suggest there is potential for earth/debris flows (a very mobile type of landslide where the debris resembles wet concrete) to occur from the loess slopes in these Class I areas.

Recent (past few decades experience) suggests such landslides are relatively small (< 100 m^3), but there is good geomorphological and historical evidence of much larger landslides, including some that have killed people in Banks Peninsular. This appendix presents a summary of the historical and pre-historic evidence of landslides in the Port Hills and Banks Peninsula.

Landslide types

Historical landslides in the Port Hills and Banks Peninsula have mainly been due to rainfall (Harvey, 1976; Bell and Trangmar, 1987; Goldwater, 1990; Elder et al., 1991; Udell, 2013; and McSaveney et al. 2014). There have been five deaths from landslides, (mainly earth/debris flows in loess or loess derivative materials) in Banks Peninsula reported in newspaper articles 1870-1938 (compiled by E. McSaveney 2012). Two people were killed while walking or camping; and the other three people were killed in their homes.

One well documented landslide event that affected the larger area of the Port Hills was reported by Harvey (1976). A total of 519 landslides, mainly earth/debris flows in loess in the Port Hills were mapped after a rainstorm. The rain occurred over 5 days between 19-23 August 1975. A total rainfall of 126 mm was recorded at the Christchurch Gardens Gauge, with a maximum daily rainfall of 69 mm on 21 August 1975. A daily rainfall of 69 mm has an annual frequency of once every 5 years and the 5-day rainfall occurs about once every 2 years (based on McSaveney et al, 2014), indicating the rain was unexceptional.

A study of landslides in the Akaroa area by Tonkin and Taylor (2008) identified three main types of landslide affecting the area: 1) bedrock landslides; 2) Active gullies encompassing tunnel erosion, surface erosion and small- to medium-scale landslides (about 1 to 5 m deep and 3-10 m wide); and 3) large loess/bedrock landslides (5 to 15 m in depth and 100-300 m wide/long). Tonkin and Taylor (2008) suggest that the generally accepted ideas on slope instability on the Port Hills include: 1) soil creep/shallow landslides triggered by rainfall; 2) tunnel gully erosion; 3) large-scale landslides are absent and 4) bedrock landslides are absent. Large-scale landslides and bedrock landslides were thought to be absent from the Port Hills, but present in the Akaroa area, because the climate in Akaroa is slightly wetter, and the materials more weathered than the Port Hills.

Landslide volumes

Harvey (1976) noted that most of the 519 landslides from August 1975 occurred in loess and mixed colluvium. Landslide volumes estimated using the mean data reported by Harvey (1976), range from a few tens to many hundreds of cubic metres. Estimated volumes of individual relict landslides (pre 1940) in loess and loess-derivative materials, such as colluvium in the Port Hills, were mapped by Townsend and Rosser (2012) from aerial photographs and field assessments. The distribution of 124 relict landslides, adopting the area depth relationships of Larsen et al. (2010) range from a few tens to tens of thousands of cubic metres. No landslides in the tens of thousands of cubic metres range have been documented in the Port Hills since European settlement

More recently, claims made to the earthquake Commission for landslip damage, over the period 1997 to 2012, were mainly triggered by rainfall (Udell, 2013; McSaveney et al., 2014). These claims generally relate to landslides with volumes of less than 100 m^3.

A large earth/debris flow, predominantly in loess, occurred in Lyttelton during the 5 March 2014 rainstorm. The volume of this landslide is estimated to be 1,000-2,000 m^3.

Factors contributing to past landslides

Bell and Trangmar (1987), based in part on the work by Harvey (1976), state that: i) most of the rainfall-induced landslides in the Port Hills area occurred on slopes inclined between 25° and 31°; ii) the angle of the back scarp varied between 31° and 45°; iii) most failures had rupture surfaces that corresponded to hydraulic boundaries e.g. loess/colluvial loess boundary; iv) the depth of failure was typically between 0.6 and 2.5 m deep (Bell and Trangmar, 1987) with a mean depth of about 1.0 m and length of 15-20 m (Harvey, 1976); and v) the landslides were generally translational in shape and their basal slide surfaces were sub parallel to the ground surface (Harvey, 1976).

Harvey (1976) found that slopes with relatively sunny (inferred to be drier) aspects had the lowest average failure slope angles, and shady (inferred to be wetter) aspects had steeper failure slope angles. However, most of the displaced debris (about 67%) came from landslides on the shady slope aspects. Results from slope stability back-analysis carried out by Elder et al. (1991) of several of the landslides mapped by Harvey (1976) suggest that the difference in slope angle between the sunny versus shady aspects was not particularly significant. The higher total volume of debris from landslides occurring on shady slope aspects would suggest that these landslides were larger in volume than those occurring on sunny slope aspects.

Elder et al., (1991) note that loess failures tend to trigger in the upper “S” (lower surface layer including topsoil, 0.2-0.7 m below ground surface) and “C” (compact layer 0.4 m to 1.3 m below ground level) layers. This is because the upper horizons are relatively weaker (in shear strength) than the underlying parent material, but principally this reflects a loss of capillary tension “suction” and the build-up of pore water-pressures above the relatively impermeable lower layers (Elder et al., 1991).

Potential earthquake effects contributing to future landslides

An initial assessment of the effects of seismically induced ground deformation and cracking caused by the 2010/11 earthquakes on the occurrence of localised landslides following rainfall, in the Port Hills was carried out by Udell (2013). Udell (2013) reports that there has been little difference in the numbers of claims made to the EQC for rainfall-induced landslide damage to dwellings following the 2010/11 earthquakes compared to those made before the earthquakes. This assessment is based on the number of claims made to the EQC for landslides triggered during the August 2012 rainstorm being comparable to those numbers made in response to pre-earthquake rainstorms in October 2000 and August 2006. These three rain events had 96-hour rainfalls with annual frequencies of about once every 5 years. The results reported by Udell (2013) are somewhat limited as:

- The August 2012 rainfall was unexceptional.
- There is no information relating to the volumes of the landslides that initiated the claims, and therefore the severity of the landslide hazards cannot be assessed, i.e. pre-2010/11 earthquake claims could have been made for relatively minor damage from relatively small landslides.
- Many areas of the Port Hills were not cracked during the 2010/11 earthquakes, and many areas only suffered superficial cracking and deformation unrelated to mass movement processes. Therefore, it would be unlikely that rainfall following the 2010/11 earthquakes would trigger more landslides and therefore claims in these areas. It is likely that the loess slopes in these areas were already cracked and fissured before the earthquakes, as such features, in loess, are quite common in loess.
- People were evacuated from the main areas where cracking caused by the 2010/11 earthquakes was thought to relate to mass movement processes (Massey et al., 2013). In the most affected areas (the Class I areas, Massey et al., 2013) many dwellings were purchased by the Canterbury Earthquake Recovery Authority and so it would be unlikely that claims would be made to the EQC in respect of land movement occurring after the 2010/11 earthquakes.
- It is too early after the 2010/11 earthquake to assess the long-term performance of the Class I mass movements with regards to rainfall. Initial inspections following the March 2014 rainstorm identified many small (less than 50 m^3) landslides, of predominantly loess, had occurred in these areas, even though the rainfall in these areas was unexceptional.

Summary of landslides in the Port Hills

Most recorded historical landslides in the Port Hills have comprised relatively shallow (less than 5 m deep) and small (less than 100 m^3 in volume) earth/debris flows, which have occurred in loess and loess-derived materials. Such landslides have occurred frequently and have resulted in many landslide claims to the EQC.

Results from geomorphological mapping suggest that large volume (>1,000 m^3) relict landslides have occurred in the Port Hills, but that these have been relatively infrequent – one recorded since European settlement in c. 1840.

Such large landslides have occurred historically in the wider Banks Peninsula area, and have killed five people (in four landslides).

It is too early to assess how the slopes that were significantly cracked, as a result of earthquake-induced mass movement (particularly the Class I areas) during the 2010/11 earthquakes, will perform in the future.

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Tonkin and Taylor Ltd. 2008. Slope hazard susceptibility assessment. Akaroa Harbour Settlements. A report prepared for Christchurch City Council. March 2008.

Udell, H. L. 2013. An initital assessment of the effects of seismically induced ground deformation on the occurrence of localised instability following rainfall in the Port Hills, Christchurch. Proceedings of 19th NZGS Geotechincal Symposium. Ed. CY Chin, Queenstown.

Locations of early landslides on Banks Peninsula reported in newspapers (1870-1923)

Eileen McSaveney

Landslides with fatalities

August 1870 –Little River road, somewhere near Akaroa (1 death)

July 1879 – bush at Pigeon Bay (1 death) (rain)

September 1904 - French Farm, Akaroa (1 death) (rain)

January 1923 - at Puaha, four miles from Little River (2 deaths) (flood/debris flow from breached landslide dam) (rain)

Other landslides

September 1870 – rockfall from cliff, Lyttelton Harbour, bay opposite the Pilot Station (Earthquake)

June 1881 – Tikau Bay, Akaroa (failure of landslide dam) (rain)

January 1884 – upper road to Lowry Bay and in gully three-quarters of a mile from Lowry Bay (rain)

May 1886 – small slip closed Sumner road traffic for a time

August 1886 – 1,000 ft high slip on headland between Port Levy and Pigeon Bay

August 1886 – wrecked Annandale Station at Pigeon Bay (eastern side of bay had many smaller slips) (rain)

July 1895 – Pigeon Bay (Holme’s Bay side) (caused tsunami) (rain)

August 1895 – Pigeon Bay (wrecked house of Knudson) (landslide near wharf?) (rain)

July 1896 – house wrecked at Lyttelton (rain)

May 1899 – between Lyttelton and Governor’s Bay? (boy trapped during attempted crossing of track of recent landslide)

July 1906 – house damaged at Little Akaroa Bay [NB There is no “Little Akaroa Bay”, did they mean Little Akaloa Bay?]

March 1907 – rockfall - Sumner Road cliffs between Shag Rock and Middle Rock

July 1923 – slips at Lyttelton and at Salt’s Gully (Lyttelton township) (rain) (eight years earlier at same location a landslide covered a cowshed, smothering eight cows)

Newspaper articles from 1870 to c. 1938

Banks Peninsula landslides

Papers Past online archive – compiled by Eileen McSaveney

LANDSLIDES WITH FATALITIES

Grey River Argus, Volume IX, Issue 717, 23 August 1870, Page 2

A man named Duerden has been killed by a landslip on the Little River road, near Akaroa. When found, his body was fearfully mutilated, both legs being broken in several places, his ribs smashed, and numerous other injuries, which must have caused instantaneous death. A man named Walker, living at Little River, had a narrow escape. He was conversing with Duerden, and saw the slip coming, but was overtaken by it, and buried up to the hips, fortunately receiving no injuries.

****

Timaru Herald, Volume XXXI, Issue 1491, 2 July 1879, Page 2

Christchurch, June 29. A man named William Bamford, while working in the bush at Pigeon Bay, was killed last night by a landslip. He was asleep in his tent at the time and was completely buried. A terrific easterly gale was experienced here last night, but fortunately no particular damage was done.

****

Wanganui Herald, Volume XXXVIII, Issue 11366, 23 September 1904, Page 7

THE WEATHER.

Gales in the South.

Landslip Fatality.

(Per United Press Association.)

CHRISTCHURCH, September 22.

A very severe south-west gale, with heavy showers of rain, raged last night and this morning, doing minor damage to trees and fences. The low-lying parts of the city and surrounding country were temporarily flooded. A landslip at French farm, Akaroa, killed a resident, Mr William Giddens, 70 years of age.

****

Auckland Star, Volume LIV, Issue 23, 27 January 1923, Page 7

BURIED UNDER LANDSLIP.

ONE KILLED TWO INJURED.

AN EXTENSIVE SLIDE.

(By Telegraph—Press Association.)

CHRISTCHURCH, this day. A big landslip occurred at Puaha, four miles from Little River, shortly after midnight, owing to heavy rain. A party of grass seeders was caught in the slips, and Griffiths Pidgeon, a married man, 30 years of age, was killed, and his brother, Frederick Pidgeon, a single man, and James Howard were injured. Howard had to be dug out, and was seriously injured.

The constable at Little River, in telephoning for assistance to dig the men out, stated that the debris extended for two miles. A party of constables has gone out.

Auckland Star, Volume LIV, Issue 24, 29 January 1923, Page 4

CANTERBURY LANDSLIDE.

HOWARDS BODY FOUND. MAN WASHED INTO LAKE.

(By Telegraph - Own Correspondent)

CHRISTCHURCH, Saturday.

The landslide at Puaha near Little River, dammed the waters of the creek, which follows the course of the Puaha Valley. This torrent broke through and swept everything before it. A whare containing a camping party which had been engaged in grass-seeding, was swept down the valley for a mile. One man was killed outright and his brother was seriously injured and had a very narrow escape from death. The third man is still missing, and is believed to have been carried into the flood waters of Lake Forsyth.

The names of the campers are as follows: Griffiths Pidgeon. aged 30. married, killed; Fred Pidgeon, brother—seriously injured; James Howard—missing. Howard's wife is living at Westport, from which place Howard arrived only yesterday.

The slide took place from the top the hill, and blocked the valley below, damming up the creek, which by that time was swollen into a roaring river. The force of the pent waters gradually broke down the resistance of the fallen earth, and with a tremendous rush and roar, the angry torrent swept down the valley.

The force of the current lifted the whare in which the camping party was sleeping and rushed it down the valley for a mile. The body of Griffiths Pidgeon was recovered this morning, and his brother was found to be very seriously injured. He managed to struggle to a whare situated further down the valley, the light from which had attracted his notice. The body of Howard has not yet been recovered. Possibly it is buried or the raging stream may have carried it into Lake Forsyth.

HEAVY FLOODS REPORTED

BRIDGES WASHED TO SEA. (By Telegraph.—Press Association CHRISTCHURCH, this day

The body of James Howard, the second man lost in the Little River landslide, was found on Sunday evening, covered with debris, in the centre of Puaha Creek, two miles from the camp and eight chains from the spot where Pidgeon's body was found. Howards was badly mutilated and almost un recognisable. Howard's wife resides at Westport.

Rain was very heavy throughout Bank's Peninsula and floods are reported at various places, washing bridges out to sea. Over five inches in 24 hours were recorded at Akaroa.

OTHER LANDSLIDES

Cave near Sumner? – July 1875

Timaru Herald, Volume XXIII, Issue 1232, 21 July 1875, Page 3

The Lyttelton Times says:—The excavations that have lately been made have brought to light many curiosities, such as greenstone tomahawks, skeletons of Maoris, and different kinds of bones. The other day, on Dr Turnbull's section, was found amongst the soil, a bone of the Moa, which was pronounced by Dr Von Haast to be the right metatarsal (or lower leg bone) of a very small species of Dinornis. During the process of removing the soil from the base of the hills, skeletons of Maoris were found in different positions, one with his head on his knees, another with his arms stretched out; remains of what apparently were cooking utensils and places where fires, had been made. The general opinion of those who examined it was, that the locality had been originally a Maori camp, and that the people had been buried alive, probably through a landslip. The bones of young children were also found. There were four of five tomahawks, one a beautiful specimen of greenstone, which is now in the possession of the finder, Mr Murphy.

****

The Christchurch Star, Sunday Sept 3 1870

In a letter published in a morning paper, Dr Haast requests that all who have any information regarding the recent earthquake will communicate with him. We hear that the chimneys in Mr Rhodes' house on the Papanui Road will have to be rebuilt. Mr Rhodes' house at Purau has also been considerably damaged. Colonel de Renzie Brett writes as follows from Kirwee, Courtenay, on Sept. 1: "About a quarter-past six o'clock yesterday evening we experienced a severe shock of earthquake. It produced a rocking motion, which caused the dwelling house built of wood and roofed with galvanised iron to make a noise as if a heavy piece of ordnance were passing by over a pavement. I feel confident that had the house been built of stone or brick it would have been seriously damaged. The motion lasted about three seconds, and appeared to be from east west."

A Leeston correspondent gives the direction of the wave there as about south or south-easterly. He also notes that previous to the shock there was a low rumbling sound, which was followed by a vibratory motion. The time is given as about 25 minutes past six o'clock. No damage is recorded beyond a few breakages at the Irwell and Leeston hotels, and a few shaken chimneys.

The recent earthquake was very severely felt in the neighbourhood of the Pilot Station, Lyttelton Harbour**. It appears that several tons of loose overhanging rock were seen to fall into the sea on the side of the bay opposite the Pilot Station.**

A South Rakaia correspondent writes: On Wednesday evening at 19 minutes past six(by our time) we were visited by a very severe shock of earthquake, which seemed to pass from N.W. by W. to S.E. by E., and lasted nearly one minute, and could distinctly he heard for a considerable time afterwards. It was preceded by a rumbling or roaring, which became almost deafening, and then died away slowly. It shook the store belonging to Mr Middleton so severely as to stop the clock and displace a quantity of goods, pitching jars,pots, and parcels from the shelves, and shifting bags of grain from the stacks. The horses which were feeding outside started away affrighted, and the whole neighbourhood was thrown into a state of confusion for some time. The evening was fine and moonlit, but a heavy gale rose about 9.30, which lasted till morning.

An Ashburton correspondent writes; "A severe shock of earthquake was felt here on Wednesday evening last at 25 minutes past 6. It was preceded by a loud rumbling noise, and resembled the earthquake of Saturday, June 5, 1869. It appeared to pass in an E.or S.E. direction. It caused much fear among the inhabitants here, for hitherto they had not felt any of the shocks that have been experienced farther north. I have not heard of any damage being done. Some two or three clocks were stopped at the time mentioned. A smart shock was felt at Waimate, about 6.15 p.m. It appeared to take a southwesterly direction.

The following items are from the Timaru and Gladstone Gazette of Friday last: A severe shock of earthquake was felt in Timaru on Wednesday evening last at about twenty minutes past six. The direction appeared to be from north to south. Several buildings appeared to be shaken, but no material damage has fortunately been done. At the Brown street brickyard several men were employed at the time in stacking bricks Preparatory to their being burnt; they were, however, disturbed in their work by some of the bricks falling down, and hearing the bricks knocking together, and afraid that there might be danger in their remaining in the kiln, speedily left it. A shed about fifty feet long, belonging to Mr Barnfrede, was also much shaken. The vessels in the roadstead also felt the shock. On board the Ottawa the vessel was thought to be dragging, but on observations being taken, it was found not to be so. As soon as the shock was over, groups were observed collected in various parts of the town, evidently expecting a repetition of the shock, and as might be supposed, rumours were rife as to several buildings being injured, but as is generally the case, turned out to be mere idle reports. We have heard of several extraordinary freaks having taken place, but which are hardly worth enumerating.

Our Temuka correspondent reports as follows: "This morning the inquiry was, Did you feel the earthquake?" and there was no mistake hut it was felt, and that pretty severely last night. About half-past six p.m. a tremendous rumbling noise was heard, and in a very few seconds the houses and buildings began to shake about in a manner that was certainly anything but pleasant. The motion lasted some seconds, giving unmistakeable evidence as to what it was, and causing the occupiers of houses to vacate the same with all possible speed. The first observation I heard on reaching the road was evidently from a son of the Sister Isle who observed "Faith, this is the first earthquake I ever saw, and I never saw such a big one in my life." But joking apart, the shock was pretty severe, and caused considerable alarm. Most of the brick buildings have sustained damage, and the new store erected by Mr Mendleson has been cracked in many places, rendering it necessary to secure the same by bracing it with iron; and Mr Collins shop felt the effects of the shock. A picture in Dr. Rayner's house was shaken from the wall and the glass broken to pieces, but I do not hear of any real serious injury being the result. A variety of Opinions are expressed as to the direction from which the earthquake proceeded, but I should imagine it was from the north-west and proceeded south east.

Our Waihi Crossing correspondent says: At about a quarter to seven p.m. a severe shock was felt in the neighbourhood of the Waihi Crossing, causing great alarm to the inhabitants, and a sickening sensation was felt by them after the shock, as was plainly visible on their countenances as they flocked together to relate the circumstances. At the Clarendon Hotel the bottles and glasses rattled together on the shelves. It was preceded with a loud rumbling noise, and appeared to move from north-west to southeast. From Oamaru we learn that two very perceptible shocks were felt at about half-past six. Several substantial buildings the Bank of New Zealand among others were visibly shaken, but we have not heard of any actual damage. From Dunedin we learn that there was a smart shock at twenty minutes past six. It lasted for several seconds. The direction was from north to south. No damage done only rang bells and jingled glasses.

Landslide dam failure at Tikau Bay, Akaroa – June 1881

Otago Witness, Issue 1546, 25 June 1881, Page 9

A rather distressing occurrence in connection with the late storm (says the Christchurch Press) took place on the property of Mr A. C. Knight, Tikau Bay, Akaroa. An employe of Mr Knight was living with his wife in a small house near the creek, which it seems had been blocked up with a landslip, thereby causing a stoppage and allowing a large pool of water to get together. The heavy rain of Friday night swelled the creek into a raging torrent, and, the dam giving way, carried the house down the gully, breaking it to pieces with all its contents, the occupants barely escaping with their lives. The poor man not only lost all his clothes and furniture, but £18 in money, which was in his purse. While searching amongst the debris for his money, he discovered his watch, which he had left on a nail in the house, hanging on the branch of a tree, And, strange to say, the watch was going.

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Landslides – Lowry Bay – January 1884

Evening Post, Volume XXVII, Issue 19, 23 January 1884, Page 2

[Wellington]
A very heavy landslip is reported on the upper road to Lowry Bay, entirely blocking it up, and compelling all traffic to deviate to the lower or tidal road. Our informant estimates that the work of clearing a passage must occupy several days even if a strong staff of men should be employed.

Two Italian fishermen had a very narrow escape from sudden death yesterday. They live in a small hut erected at the mouth of a deep gully about three quarters of a mile from Lowry Bay. Owing to the excessive rain of Monday, a heavy landslip occurred during the night in the gully just above this hut. The men were awakened by the rush of the earthy and rocky avalanche that was descending and absolutely brushing past their hut, but, strange to say, without injuring it, although had it been struck fair by any one of the massive boulders, several feet in diameter, which came down in regular volleys, it is morally certain that the building and its inmates would have been crushed to jelly. The two men listened in the utmost terror to the appalling sounds, which they supposed to indicate a tremendous earthquake, and momentarily expected to be dashed into atoms, but the landslip left them unhurt. In the morning they found the face of the immediately adjacent country extraordinarily changed, and were devoutly thankful for their hairbreadth escape.

Star, Issue 5619, 15 May 1886, Page 3

Sumner.

TRAFFIC STOPPED BY A SLIP. [Special to the Star."]
SUMNER, May 15.
A slight slip has taken place on the Sumner road, which has stopped traffic for a time. It is still raining here (12.30 p.m.) Some parts of the township are completely flooded.

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Pigeon Bay landslide – August 1886
(NB Produced large wave)

North Otago Times, Volume XXXI, Issue 6132, 19 August 1886, Page 2

CHRISTCHURCH.
August 19.

A serious landslip has occurred at Pigeon Bay, completely wracking Mr Hay's house, which afterwards caught fire. No lives were lost, all the family managing to make their escape. Every assistance was rendered by the settlers. The roads on the Peninsula are impassable, and to-night great damage was feared unless the rain abated.

Timaru Herald, Volume XLIII, Issue 3708, 20 August 1886, Page 2

THE PIGEON BAY LANDSLIP.

(By Telegraph.) Christchurch, Aug. 19.

Further details to hand with reference to the landslip at Pigeon Bay show that the whole of Messrs Hay Bros., Annandale Station, has been swept away. Mr Thomas Hay heard the slip coming about 9.15 a.m. on Wednesday. He called his men to take out the four children. Mrs Hay also had to be carried. They ran as fast as they could for the road. Thomas Hay stayed to see all out of the house, and then ran himself, the slip nearly overtaking him. Another slip followed, shifting the chimneys and setting fire to the house, and some time afterwards a third slip carried away the whole of the buildings into the sea and creek. The slips came from the top of the range about 1 1/2 miles from the house. The beach and the bed of the creek are strewn with debris, and about twenty men were working today picking up what they could out of the silt. Mr Hay estimates his loss at £8000. The weather is again thick and reigning. [sic]

Messrs Hay Bros. house, woolshed, and outbuildings, which were destroyed by landslip and fire at Pigeon Bay, were insured in the South British Office for £2600.

Hawera & Normanby Star, Volume VIII, Issue 1418, 6 September 1886, Page 2

THE LAND-SLIP IN CANTERBURY.
(Christchurch Press)

The late continuous rain has been the cause of a disaster at Pigeon Bay, the result of which in a small way reminds one forcibly of the late eruptions in the North Island. Fortunately, however, no loss of life occurred, though had the accident happened at night or earlier in the morning, it is probable we should have bad to chronicle a sad disaster. As it was the escape of Mr. Thomas Hay and his family from death may be regarded as almost miraculous. There are few of the older settlers who do not know the homestead of Annandale well. Here it was that some forty-three years ago Mr. Ebenezer Hay settled down, and it has since become one of the most noted of the estates of Canterbury. The house itself, which has been added to and modernised, as it were, since its first building, stood back from the road a little, the mountain spur rising at the back. It was not far from the shores of the bay, and when seen, as it was, by the writer not many months ago, was the beau ideal of a peaceful and happy rural retreat. Now all is desolation, not a vestige either of the house itself or the outhouses surrounding it being left. The destruction is complete, and so sudden was the calamity which overtook the family that it was with the utmost difficulty that they made their escape, merely with the clothes they were wearing at the time.

The letter sent by the messenger from Mrs. Hay to her relatives here contained a most graphic account of the disaster. Between eight and nine on Wednesday morning the men who were working on the farm heard a roar, and looking towards the hills which rise up at the back of Annandale, saw the mountain, as it were, rending in two over their heads, and a gigantic landslip coming down. The alarm was at once given, with praiseworthy promptitude and coolness, each one seized a child and rushed down the path from the house to the road. As they fled along in terror a second slip came down, crushing the house to atoms, and the debris fell all round the flying fugitives, so close to them that the fall of earth was, as it were, upon them. Fortunately, they were enabled to gain the road in safety, and ultimately took refuge in the store. In the meanwhile, the house, which had been flattened to the earth by the fall of the slip, took fire. This was caused by fires in different parts of the house, which were log fires, the one in the kitchen being raised up above a large colonial oven. So soon as the debris crushed on to the house the fire was thrown out in contact with the boards, and the remains of Annandale were destroyed altogether in this way. The family passing, scantily clad, through torrents of rain, ultimately managed to reach the hotel, wet through and almost exhausted from the terrible scene through which they had passed.

We were working in the creek," said Mr. James Hay, whom I met up to the knees in soft mud superintending the work of picking out the relics from the soil, "when I heard a most tremendous roar. We had been on the look-out for slips, and therefore were to some extent prepared. Those in the house ran for their lives, and as I went at top speed towards the house to aid I looked up. There above me, coming down the mountain side at railroad speed was a wall of earth some forty or fifty feet high throwing up as it came high in the air a kind of spray. I thought at first it was an eruption. We all got out of the house and down to the bottom by the fence. As the mass of earth came on it struck a very strong fence which we had put up above the house, breaking the 6 x 4 posts abort off like matches. This I think prevented it carrying "away the house. I then rushed up to the house to see if all were out, and supposing they were so turned to leave, when just then I saw the little head of one of the children. This was a little boy about two years old who had been into the store room taking the sugar. I grabbed him and turned to ran. As I did so I heard a second slip coming, and had hardly got away when it came with a rush and a roar, right on to the house crushing it as one would an eggshell. So close was it behind me that I felt the spray of the earth "striking me in the back as I ran. The house then took fire, and burned for quite two hours. The two eldest of the youngsters ran themselves, and we managed to get the rest out and away on to the bridge over the creek only just in time to see our home disappear as if it had never existed. The gardener had a narrow escape. He was in a small shanty in the garden and heard the roar. He started out and had hardly gone a chain before the shanty was buried under ten feet of earth. We lost nine dogs and about fifty sheep. Some of the carcases of the latter we have found in the soil. By the bye a most singular occurrence took place with regard to one of the dogs. The first slip buried him completely, but after the second one I was surprised to see him join us on the bridge. To give you an idea of the way in which the various things in the house were scattered, continues Mr. Hay, "We found my brother's purse containing £18 down by low water mark. This had been placed in a drawer in one of the rooms. The heavy safe was also carried down, to low water mark, and stranger than all we found the kitchen store and the kettle on it near the safe."

The insurances amount in the whole to £2620, distributed as follows :— £l500 on the dwellinghouse, £400 on the woolsbed, £65 on the dairy and cheese house, £135 on the slaughter-houses, £20 on the men’s house, and £500 on the furniture. All these insurances are in the South British Company.

Te Aroha News, Volume IV, Issue 169, 11 September 1886, Page 5

A TERRIBLE LANDSLIP

DESTRUCTION OF A CANTERBURY HOMESTEAD.

Narrow Escape of Sixteen Persons**.**

The late continuous rain has been the cause of a disaster at Pigeon Bay, which has swept away completely one of the oldest residences in Canterbury, and converted what was a charming spot into perfect desolation. Fortunately, however, no loss of life occurred, though, had the accident happened at night or earlier in the morning, it is probable we should have had to chronicle a sad disaster. As it was the escape of Mr Thomas Hay and his family from death may be regarded as almost miraculous. There are few of the older settlers who do not know the homestead of Annandale well. Here it was that some forty-three years ago Mr Ebenezer Hay settled down, and it has since become one of the most noted of the estates of Canterbury. The house itself which has been added to and modernised, as it were, since its first building, stood back from the, road a little, the mountain spur rising at the back. A letter sent by Mrs Hay to her relatives in Christchurch contained a most graphic account of the disaster. Between eight and nine on Wednesday morning, 18th August the men who were working on the farm heard a roar, and looking toward the hills which rise up at the back of

Annandale, saw the mountain, as it were, rending over their heads, and a gigantic land slip coming down. The alarm was at once given, and with praiseworthy promptitude and coolness, eahc [each] one seized a child and rushed down the path from the house to the road. As they fled along in terror a second slip came down crushing the house to atoms, and the debris fell all around the flying fugitives, so close to them that the fall of earth was as it were upon them. Fortunately they were enabled to gain the road in safety, and ultimately took refuge in the store. In the meanwhile the house, which had been flattened to the earth by the fall of the slip, took fire. This was caused by the fires in different parts of the house which were log fires, the one in the kitchen being raised up above a large colonial oven. So soon as the debris crushed on to the house, the fire was thrown out in contact with the boards and the remains Aunanaale [Annandale] were destroyed altogether in this way. The force of the slip may, be imagined when it is stated that the remains of the furniture, &c, were swept right out into the bay.

The family than [sic] made an attempt to get round to the hotel, but owing to the large land slips which had fallen on the road between the hotel and the store, they were unable to do so. The only method by which they could reach the shelter of the hotel was by boats. This, owing to the sea running in the bay, was a work of some danger. Added to this the rain was descending in torrents, and they possessed little or nothing in the shape of covering to keep out the wet. Ultimately they managed to reach the hotel, wet through and almost exhausted from the terrible scene through which they had passed. Once at the hotel Mr and Mrs Bridges did all in their power to make them comfortable. It may be noted that there were at the time of the accident some sixteen persons at Annandale including Mr and Mrs Hay and family and those employed on the farm. The other settlers in the Bay were so much alarmed after the calamity that they too left their houses and sought refuge in the hotel.

Otago Witness, Issue 1814, 27 August 1886, Page 15

THE PIGEON BAY LANDSLIP.

EXTRAORDINARY EXPERIENCES.
An interesting account of the landslip in the Pigeon Bay district is given by the special reporter of the Christchurch Press, who says : —

The scene along the coast was exceedingly fine, the waves beating against the rockbound shore with great force, and sending up clouds of spray. An excellent view of what is known as " The Blow Hole," close to Port Levy rocks, was obtained. This is a cavity in the rocks open to the sea, with an orifice on the landward side, through which the spray is sent high in air with great violence. Yesterday it was in full operation, and resembled one of the geysers in the North Island, the column of spray being some 30ft or 40ft high.

As we steamed slowly down Pigeon Bay the effects of the late rains were noticeable on either side. The face of the mountains sloping down to the sea were scarred deeply in numerous places with heavy slips, many tons of earth, in parts taking with them trees, having fallen on the beach. Of course the scene of the late disaster was the one to which the eyes of all on board turned at once, and as we drew near the full extent of what had occurred was enabled to be realised. Where once was a beautiful garden, with well-appointed house, stables, dairy, wool shed, and the usual outbuildings of a large farm, was now a blank. The steamer having moored to the wharf, I set off on an

INSPECTION OF THE SCENE.

To reach this by way of the road was, as I subsequently found out, a work not only of difficulty but also in parts of danger. Once on terra firma, my troubles were by no means over, as the rain had almost entirely demolished the road, and what was left was simply quagmire. However, after a little trouble, I reached the bridge over the creek, the creek opposite where Annandale once was, and I will now endeavour to describe

WHAT THE SLIP LOOKED LIKE.

From where I stood looking up the mountain, some 1300 ft or 1400 ft high, the whole of the centre of the face, from top to bottom, was scarred with a great wide rent. At the top was a cup-like crater, as if the top of the mountain had fallen in and pushed out the soil underneath. With the cloud of mist hovering about the top of the hill, and the wide rent made more conspicuous by the chocolate colour of the soil, there seemed to me to be a singular resemblance to the rent in Tarawera — a resemblance which the steam-like appearance of the mist made more complete. This rent, down which the hundreds of tons of soil which overwhelmed Annandale travelled on that eventful morning with lightning speed, is about 100 or 150 ft wide. The hill rises behind the spot where the house is, but is not particularly steep until near the top. A clump of bluegums slightly to the right of the track of the slip, and therefore not exposed to the full force of it, one solitary walnut tree, and another bluegum near the bottom of the garden facing the road, are all that remains of a highly cultivated fruit and flower garden and 10 buildings, including a thirteen - roomed house and large wool shed. The site occupied by these now resembles nothing o much as a newly ploughed field with fragments of debris of all kinds mixed in the soil. At the spot where the house stood there is now from 12ft to 15ft of earth piled up, and at the bottom by the road it is some 3ft or 4ft above the 6ft fence. Beyond this latter, and covering the 8ft stone wall which divided the garden, the debris of the slip has gone right out into the bay, reclaiming the land from the sea for some yards below low water mark. Some idea of the force with which the mass of earth came down the hill may be gathered from the fact that the large wool shed referred to was taken bodily some chains and hurled into the creek, the massive timbers being splintered up, and the whole fabric dispersed like a house of cards. The creek is now filled with remnants of timber, iron, &c, whilst the shores of the bay from opposite Annandale to Holmes' Bay is also strewn with the wreckage of the house, furniture, &c. The scene is one of the utmost desolation. At one part was to be seen a quantity of household goods, books, and clothing, heaped together amidst the soil; in another, scattered along the beach was a mass of every conceivable article, strewn far and wide, as though some demon in a fit of destructive rage had hurled them right and left. When it is remembered that the house stood some 40ft above low water mark, and some four or five chains distant therefrom, some idea may be formed of the enormous amount of earth which fell in so short a time. Having endeavoured to convey an idea of the scene as it presented itself to me, let me note some of the

INCIDENTS OF THE EVENT.

"We were working in the creek," said Mr James Hay, whom I met up to the knees in soft mud superintending the work of picking out the relics from the soil, " when I heard a most tremendous roar. We had been on the look-out for slips, and therefore were to some extent prepared. Those in the house ran for their lives, and as I went at top speed towards the house to aid I looked up. There, above me coming down the mountain side at railroad speed, was a wall of earth some 40 or 50 feet high, throwing up, as it came, high in the air, a kind of spray. I thought at first it was an eruption. We all got out of the house and down to the bottom by the fence. As the mass of earth came on it struck a very strong fence which we had put up above the house, breaking the 6by 4 posts short off like matches. This, I think, prevented it carrying away the house. I then rushed to the house to see if all were out, and supposing they were so, turned to leave, when just then I saw the head of one of the children. This was a little boy about two years old, who had been into the store-room taking the sugar. I grabbed him and turned to run. As I did so I heard a second slip coming, and had hardly got away when it came with a rush and a roar, right on to the house, crushing it as one would an egg shell. So close was it behind me that I felt the spray of the earth striking me in the back as I ran. The house then took fire and burned for quite two hours. The two eldest of the youngsters ran themselves, and we managed to get the rest out and away on to the bridge over the creek only just in time to see our home disappear as if it had never existed. The gardener had a narrow escape. He was in a small shanty in the garden and heard the roar. He started out, and had hardly gone a chain before the shanty was buried under ten feet of earth. We lost nine dogs and about fifty sheep. Some of the carcasses of the latter we have found in the soil. By-the-bye, a most singular occurrence took place with regard to one of the dogs. The first slip buried him completely, but after the second one I was surprised to see him join us on the bridge. He was so coated with the soil that until we washed him we had no idea which of the dogs it was. What was the roar like? " says Mr Hay in answer to a question. "Well, I can hardly say. It was a most unearthly noise, and so loud that all the people in the bay heard it and ran out of their houses, thinking there was an eruption on the mountain and that an earthquake was about to take place. To give you an idea of the way in which the various things in the house were scattered," continued Mr Hay, " we found my brother's purse, containing £18 down by low water mark. This had been placed in a drawer in one of the rooms. The heavy safe was also carried down to low water mark, and stranger than all, we found the kitchen stove and the kettle on it ! near the safe."

Later.

The following additional particulars of the extraordinary landslip at Pigeon Bay were supplied by a resident to the Lyttelton Times :—

The women and children hurried down the lane, and over the bridge, to the store, and all were safe there before the fourth and dreadful avalanche. Mr Scott now rode down the main road. He saw the wreck. He heard the roar. He spurred his horse, and just cleared the bridge as the fourth avalanche came down with deafening sound, carrying the large wool shed, borne on cubic yards of liquid mud, right across the main road, into the creek above the big bridge, and hurling the burning house over the sea wall on to the sea beach below, obliterating every trace of the once extensive Annandale. The main road was now impassable. A pedestrian climbed up the hillside, just above the dreadful gully, and describes the scene as being awful. He climbed to the hilltop, above the slip, and I came down on the Holmes' Bay side, only to find himself hemmed in there. He describes the starting place as being like what he pictures the crater of a volcano to be. A huge precipice, about 80ft long and 30ft deep, opens down to a small table land, about the eighth of an acre in extent. The hillsides are all worn bare by the water. There are several smaller slips into the large one. A roaring sound like Niagara preluded a stream of liquid mud. The force of the fourth avalanche may be imagined, when it shook the store, 400 yards away, like an earthquake. At that moment several people were being conveyed from Feirrie Glen to the hotel in a boat, and the amount of mud forced into the sea on this occasion caused quite a tidal wave to sweep over the bay, and if the boat had not just reached the island it would probably have been swamped. The beach presented a most lamentable appearance. Timber, trees, grass seed, &c, were piled up and floating about as if two vessels had been wrecked in the bay.

Taranaki Herald, Volume XXXV, Issue 7152, 24 August 1886, Page 2

HEAVY FLOODS DOWN SOUTH.
THE LANDSLIP AT PIGEON BAY. Tue [sic] floods in Canterbury have done enormous damage, and the roads will not be passable for the coach for some time. There are tremendous slips everywhere, and though fifty or sixty men are at work they can make little head way. Tho disaster at Pigeon Bay is the most serious one. The whole top of the hill above Messrs Hay Bros. homestead slipped on to the house, woolshed, and offices, carrying them out to sea.

Sergeant Brooks, who had visited the scene of the landslip at Pigeon Bay, supplied the following :—About 915 on Wednesday morning Mr. Thomas O. Hay, Mr. Robert Hay, Mr. Husband, and three station hands were cleaning away the mud that had washed into the house on the previous night, when they heard a noise, and looking up the hill at a distance of about a mile they say a landslip coming straight towards the house, and Thomas Hay sang out, "All hands clear and run." Some ran into the house, where were Mr. James Hay and Mrs. Robert Hay with four children. The station hands carried a child each. Mr Robt. Hay and Mr Husband carried Mrs. Hay out of the house, making all haste to get clear of the slip. Mr. T. Hay was the last to leave, staying to see that there were no people left in the house. The slip was close on to his heels when he got to the road. For a short time the slip stopped, a portion of it resting against the house, but only for a minute, when it started a second time, twisting the chimneys of the house, which then took fire. A short time after, and while the house was still burning, a third slip came, carrying the large woolshed, stables, outbuildings and dwelling-houses of the station hands with the burning residence of the Messrs Hay Bros, a distance of 200 yds from where they originally stood, across a road and a creek on to the sea beach, leaving the whole corner section quite bare, the only thing left to mark the spot being part of the fowl house. The sea beach is all strewn with wreckage from the buildings, from amongst which was found the iron safe containing the papers of the Messrs Hay Bros. Some sacks of cocksfoot which were stored in the shed where found on another section 400 yards away. The Messrs Hay Bros. reckon their loss at fully £8000. The house and furniture were insured in the South British for £2000.

Mr. Ebenezer Hay, who was the first settler in Pigeon Bay, came to Wellington from Scotland in 1840, and after living for three years in Wellington, went to Pigeon Bay, where he built his first hut near the creek. He afterwards built a house on the site of that which has just been destroyed, which was erected about 14 years ago. The latter was a large two-storeyed building, and was surrounded by all the buildings required for carrying on the work of the station — a wool-shed, stable, slaughter-house, dairy, wash-house, and other structures, forming almost a small township. These stood on a slope about 120 yards from the Bay Creek, which ran past the front of the house, and about 180 yards from the sea, above which they stood 50ft. At the back of the House the ground ascended with a gradual slope to a precipitous knob, about a mile distant from which a small creek found its way to the sea. The slip was evidently caused by the breaking off of a portion of this knob, which rolled down the water-course, destroying everything in its path.

From the situation of the house, it might have been supposed to be entirely safe from all danger of landslips, while Mr. James Hay's residence, the Glen, which has escaped, appeared to be in a far more dangerous location. It is fortunate that the catastrophe did not occur at night, when tho occupants of Annandale were sleeping. Had it done so, not a single person would have escaped with life.

The startling event seems to have caused quite a panic in Pigeon Bay, as none of the residents could be sure that their houses were safe from a similar fate. No particulars are to hand as to any loss of live-stock that may have been occasioned, but it is supposed that this was not very great.

The rains have caused an immense amount of damage to the public roads and to private properties there. In some places the main road has been carried away bodily, pedestrians having to cross private properties to continue their journey. Many of the settlers were on watch all night dreading landslips. Many chains of fencing and acres of good land have been destroyed. Several narrow escapes of loss of life have occured.

Wellington, August 24. — It is still raining here more heavily than usual. There has only been one day without rain this month, and not three that could be called fine. No damage has been reported as yet, but the streets of the town are in fearful condition, and great complaints are being made against the city authorities.

Taranaki Herald, Volume XXXV, Issue 7153, 25 August 1886, Page 2

The stormy weather which has prevailed during the past month has been very severely felt in the South Island, and the accounts in our exchanges of the destruction there is to property are very sad to read. There have been several land slips, but the one in Canterbury has been the worst. Ordinarily, when a landslip is referred to, is [sic] is supposed to represent a fall of so many tons of earth, but the Pigeon Bay landslip, which, last week destroyed Annandale, the homestead of Mr. Thomas Orr Hay, cannot be estimated by the number of tons— it can only be adequately measured by-its number of acres. To give some idea of the power of the landslips, Mr. Hay states that he picked up his safe on the beach, half-way high and low -water marks, and about a couple of chains from the creek. It weighs half a ton, would take four or five men to roll it over. The big posts of the stock-yard, which were as thick as a man's body, were cut off at the ground as pieces are cleared off a chess-board. Mr. Hay in describing the landslip says. "My brother timed the fall of the third slip. I reckon that the hill is 1300 or 1400 ft high and a mile away, and my brother found the slip was just a minute and a-half from the time it started till it reached the sea. The biggest fall came even quicker than that. I don't know how many acres of the sea must have been filled up, but it must be three or four acres, and besides there is all the stuff that is left round the house."

Hawera & Normanby Star, Volume VIII, Issue 1412, 30 August 1886, Page 2

TELEGRAMS.
(PER UNITED PRESS ASSOCIATION.)
CHRISTCHURCH, August 30.
On Friday the captain of the steamer Akaroa, when passing the headland between Port Levy and Pigeon Bay, discovered a big slip on the northern side of the mountain, extending from the summit to the base, a height of 1000 feet. A strange rumbling heard at Lyttelton on Friday morning is supposed to have been caused by the slip.

Hawera & Normanby Star, Volume VIII, Issue 1414, 1 September 1886, Page 2

"Puff," in the Wellington Press : — "
Great landslip between Port Levy and Pigeon Bay ! The face of the mountain 1000 feet high tumbled into the sea ! Why skip ye so, ye little hills ! Banks Peninsula on the rampage ! Flopping about anyhow ! What does it mean ? There's been nothing like it since the first settlers arrived ! No ; the fact is there have been the heaviest spring rains for 25 years, and the Peninsula being stripped of the bush, the steep places have given way ! That's what will happen periodically in all the mountainous parts of the colony ! Only another of the evil results of wanton destruction of natural forests ! Oh yes, the colonists will have to pay pretty dear for their folly before they have done with it !"

****

Many years later

Evening Post, Volume CXXXV, Issue 59, 11 March 1943, Page 5

MR. EBENEZER HAY

(P.A.) CHRISTCHURCH, This Day.

The death has occurred of Mr. Ebenezer Hay, of Annandale, Pigeon Bay, at the age of 67. A well-known runholder and sportsman, he was a son of Mr. and Mrs. T. O. Hay, and was named after his grandfather, who sailed from Glasgow in the ship Bengal Merchant in 1839. Arriving in Wellington in January, 1842, his grandfather, with Captain Sinclair, built a small vessel on the Petone beach, and in it they set out to explore the South, Island, finally deciding to settle at Pigeon Bay. The old Annandale homestead, including the woolshed and outbuildings, was carried away by a huge landslide and the present homestead was erected in 1884. Originally the estate comprised some 7500 acres, carrying upwards of 10,000 sheep and 1500 head of cattle.

Caves at Moncks Bay - Report: April 1890

New Zealand Tablet, Volume XVII, Issue 51, 11 April 1890, Page 19

WATSONVILLE, SUMNER.

(From an occasional Correspondent.)

About two miles from Sumner proper, and opposite the rough-level tract of land, about forty-five acres in extent and known as Monck's Flat, there is a bay or broad flat valley that contains close upon fifty acres. The estuary formed by the union of the river Heathcote and the river Avon fronts this valley, and the hills on each side shade it completely from the east and south-west winds. The valley formerly formed one property and then belonged to the late Mr. Watson. ….The next valley towards Sumner belongs to Mr. Monck. Several months ago, when some men were getting stones for the roads from the face of a steep rock that is on Mr. Monck's property, and at the end of the spur that divides the two valleys, a cave consisting of two dome-shaped compartments, was suddenly and unexpectedly discovered. The apex of the outer cave, which now consists of but half a dome, is about eighteen feet high, and the apex of the inner cave is from eight to nine feet. The outer cave is also about twenty feet long, and fifteen broad, while the inner cave is nigh forty-two feet long and twenty-four wide. To advance into the inner cave—inside of which it is so intensely dark that to see anything a person must be provided with one or more candles — it was necessary to crawl on the knees, as the entrance is not more than two feet high. But Mr. Monck has cut a deep central trench, and there is now a walk from one end of the cave to the other. On the floor there was an accumulation of ashes and shells several yards in depth. This accumulation proves that the cave must have been a famous camping place for a very long time before the entrance to the outer cave was centuries ago accidentally covered and concealed by an earthquake or a landslip. The cave, like the larger one known as the Maori Point Cave, was originally simply an air bubble in a stream of lava, and it is very probable that there are several undiscovered caves at Sumner. Many articles of interest, such as a canoe paddle, and a bailer fashioned from a solid block of wood were found in the caves. Sinkers, fishing-hooks, and spears, parts of wooden combs, knots of skinned native flax, greenstone chisels and axes and a variety of bones were also discovered. In one place a large quantity of beautiful black curled glossy, human hair was found. This hair seemed as perfect as hair recently cut from the head of some Maori. Mr. Monck was anxious to preserve the caves as when first found, but when their discovery became known a whole army of persons rushed from the city of the plains, and these Cockney geologists soon destroyed what centuries had spared.

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Another landslip at Pigeon Bay (large wave) July 1895

Grey River Argus, Volume XXXVII, Issue 9182, 11 July 1895, Page 3

NEW ZEALAND TELEGRAMS

(PER PRESS ASSOCIATION)

Dunedin, July ,9.

Reports from Banks Peninsula state that most of the roads are blocked with landslips, caused by recent heavy rains.

A landslip of extraordinary dimensions at Pigeon Bay started at six o'clock, and rushed into the sea with such force as to raise a tremendous wave, which swept across Pigeon Bay (from Holme's Bay side), and swamped the road to a distance of nearly a mile. A number of families living in Pigeon Bay locality have left their homes; fearing further slips, the hills being dangerously fissured.

Timaru Herald, Volume LVIII, Issue 1803, 11 July 1895, Page 3

Reports from Bank's Peninsula state that most of the roads are blocked by a landslip caused by the recent heavy rains. Last night there was a slip of extraordinary dimensions at Pigeon Bay. The slip started at 6 o'clock and rushed into the sea with such force as to raise a tremendous wave which swept across Pigeon Bay (from the Holmes Bay side) and swamped the road for a distance of half a mile. A number of families in the Pigeon Bay locality are leaving their homes, fearing further slips, the hills being dangerously fissured.

Yet another landslip at Pigeon Bay August 1895

Star , Issue 5326, 2 August 1895, Page 3

LANDSLIP.

A HOUSE CARRIED AWAY.

[from our own correspondent]

AKAROA, August 2,

This morning another large landslip occurred at Pigeon Bay, which carried away Mr Knudsen's house and completely blocked the road to the wharf, to which communication can only be made at present by boat at high tide.

Wanganui Herald, Volume XXIX, Issue 8615, 3 August 1895, Page 2

Christchurch. 2nd August.

By a landslip at Pigeon Bay this morning the house of Knudson was swept away, and the road to the wharf completely blocked. No lives were lost, Knudson having removed his furniture and family about three weeks ago, when fissures appeared in the hillside above his place.

Star, Issue 5328, 5 August 1895, Page 4

The Landslip.

FURTHER PARTICULARS.

The steamer Jane Douglas ran an excursion trip to Pigeon Bay yesterday for the purpose of affording anyone sufficiently interested a view of the huge landslip which took place in the bay on Friday morning last. Between 80 and 100 persons, including a representative of this journal, availed themselves of the opportunity. The slip was not altogether a surprise to the residents, for during the heavy and continued rains of last month deep fissures had been noticed on the hill, and the settlers whose houses were below these had removed their belongings and left their homes. When the weather broke it was considered that all was then safe, and that the ground would settle down, as it has done in many other places, but last week's heavy rain and snow caused the worst fears of the residents to be realised. Steaming up Pigeon Bay harbour, numerous small slips, chiefly on the eastern side of the bay, were observable, and on nearing the wharf the heavy slips of three weeks ago came within sight. That which occurred almost abreast of the wharf, when the debris was hurled into the sea with sufficient force to create the huge wave which swept across the harbour (a distance of fully half a mile), was viewed with considerable interest. The site of the disastrous slip of nine years ago, when Messrs Hay's fine homestead was completely wrecked, also attracted attention, for on the same spot another slip had recently occurred. Here a portion of a plantation of gums had been uprooted and swept with the debris into the sea. All these huge slips, large as they undoubtedly were, pale with utter insignificance when compared with

LAST FRIDAY'S DISASTER.

Reaching the wharf, the majority of the party at once commenced the work of inspecting the ruins. The writer was fortunate in early obtaining the assistance of Mr Frank Dunkley, the young man who narrowly escaped losing his life by the slip. With the idea of obtaining a better view, the high hill from where the slip started was scaled, and on the climb up it was observed that for several chains on the northern side of the slip the earth showed deep fissures, which might at any time come away, and probably would do so in the event of heavy rain or frost. Arriving at the uppermost end of the slip, the sight well nigh baffles description. From here right into the sea, a distance of probably 850 or 900 yards, is one mass of ruin, fences being swept away, great slumps of trees lying strewn about, growing trees being uprooted and hurled in every direction amongst the clay. It is only in looking down into the great gulf which has been formed that any idea can be got of the magnitude of the disaster. Fully 900 feet wide; with an average depth of 50 feet, and for a length of about 2000 feet is the extent of the country that has suffered. In some places the depth extends to 70 feet, and in many places marks resembling huge plough furrows are visible where the volume of earth has forced its way down the hill. Little hillocks with their accompanying valleys have been formed here and there, while in many places the surface soil and even the snow are still visible, having simply slid perhaps a hundred yards from their previous position.

MR KNUDSON'S HOUSE,

which was a substantially built dwelling of five rooms, was situate on a spur dividing two gullies. The slip started on Mr Hay's land, and coming on into Mr Knudson's section, divided at the top of the spur behind the house. The volume was of such extent, however, and moved with such rapidity that a portion of it swept over the spur, and in its course demolished the house and garden. A portion only of the matchwood left was to be seen, for some of the timbers and sheets of galvanised iron were swept into the sea below. Just below where the house stood the debris again left the spur and joined the main volume in the gullies, and crossing the road swept into the sea close to what residents of the Bay call The Island. At its entrance into the water the face of clay, &c, was estimated to be fully seventy feet high, and fences, trees, &c, have been forced over the mud flats of the bay for hundreds of yards, so that at low tide it is almost possible to reach the other shore on dry land.

AN EYE WITNESS.

The Rev A. Blakiston, who was an eye witness to this awe-inspiring scene, has kindly supplied a few particulars. He states that at about 9.15 a.m. his attention was directed to sheep, horses and cattle running out of the gullies. He then saw that a slip was taking place. The surface about half-way up what subsequently turned out to be the slip appeared to be sliding down the hill, taking with it trees, just as they stood. Mr Blakiston called to one or two neighbours, and as they stood watching the scene, the whole hill appeared to tremble and shake, and then immediately, with a loud rumbling noise, the millions of tons of earth commenced to move. With one terrific rush the whole mass of earth, taking before it anything which came within its course, was hurled into the sea. The young man Dunkley was standing close to the water's edge, watching the small slip, when Mr Blakiston and others called to him. He had "a distance of fifty or sixty yards to run, and only just managed to get away from the line of the avalanche when it swept; at a great rate over the ground where a second or two before he had stood. The debris appeared comparatively dry, and residents of the Bay, who can now claim a good deal of experience of these matters, state that all previous slips have been much more sloppy.

Great sympathy is felt for Mr Knudson and his family. Mr Knudson has resided at the Bay for thirty-one years. He has a family of nine—five daughters and four sons—and the homestead which was so quickly demolished on Friday has been his home for over a quarter of a century.

Messrs A. Cuff and Co. very generously devoted the net proceeds of yesterday's trip of the Jane Douglas to the fund which is being organised for the assistance of the sufferers by the slip.

Star , Issue 5329, 6 August 1895, Page 3

The Pigeon Bay Landslip — The special trip run by the Jane Douglas for the benefit of the sufferers by the landslip at Pigeon Bay resulted in the sum of 8£ 2s 6d being taken. The whole of this will be handed over to the relief fund by the Lyttelton and Peninsular Steamship m Company.

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Landslide at Lyttelton – July 1896

Poverty Bay Herald, Volume XXIII, Issue 7689, 29 July 1896, Page 2

Friday's Christchurch Press gives an account of the landslip at Lyttelton, briefly mentioned in our telegrams last week. A two-storied semi-detached house, containing about six rooms in each division, the property of Mr John Mclntosh, of the Peninsula, had the back wall smashed in by a heavy slip. One division of the house was occupied by Mrs Adams and her family, and the other division by Mrs Fenton and a large family, including several grown-up daughters. The hill behind the house is very steep, and, as it faces the south-west, small slips have been frequent, but hitherto they have not done much damage beyond piling up against the back wall of the house and smothering whatever happened to be in the back yard. On Thursday morning, however, a considerable area of the surface, which had become sodden with water, slipped off, and coming down with great force smashed in the back of the house and carrying all before it broke through into the front room. As may be imagined the inmates received a great fright. Every article of furniture in the back rooms was smashed and many of those in the front part of the house. The back rooms of the houses are frequently occupied as bedrooms, but on this occasion they were fortunately unoccupied. Had anyone been sleeping there they must have been killed as the back wall was driven in and the rooms filled to the ceiling with heavy wet clay. All exit from the house by the back way was cut off, and, as the stairs were smashed and filled up with, earth, the inmates had considerable difficulty in making their escape. Eventually a rope was obtained, and the occupants were lowered out of the top windows. The morning was pitch dark and the rain coming down in torrents, and, as may be imagined, the experience was a most unpleasant one. Added to the wretchedness was the doubt that at any moment another and larger slip might come down and hurl the building out on to the street or possibly over the cliff on the other side. At the first appearance of daylight carts were obtained, and the remains of the wrecked furniture were removed elsewhere, that from upstairs having to be lowered through the windows by ropes.

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Star, Issue 6493, 23 May 1899, Page 2

AN ALARMING EXPERIENCE.

A young man, one of a party that walked from Christchurch to Governor's Bay on Sunday, had an alarming experience. When nearing the main road leading from Lyttelton to the bay the party left the Pass Road, and intended taking a short cut on to the road below. They ran down the hill near the spot where the recent landslip occurred, and one of the party attempted to cross the clay surface over which the slip had passed. He had not gone far when he began to sink, till nothing, but his head remained in view. His mates went in search of assistance, and found a resident, who accompanied them to the spot. By the aid of clods placed as stepping-stones the rescuers were able to reach the entombed youth. Their efforts to pull him out of the semiliquid clay were unsuccessful, and it was only by the aid of a large fork that the unfortunate man was dug out of the trap into which he had fallen. But for the loss of one of his boots he was none the worse for his adventure.

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Northern Advocate - 7 July 1906, Page 2

A Landslip Ruins a Home.

TONS OF EARTH AND ROCK.

Christchurch, July 7.

A rather serious landslip occurred at Little Akaroa Bay, Banks Peninsula, on Tuesday night, about seven o'clock. Some tons of earth slid down the mountain side and came in contact with a dwelling-house and some refreshment rooms kept by an elderly couple named Bennett, their home being completely ruined. One part of the house was turned round, and the other was driven partly over some rocks. Tons of mud, stone, and other matter were accumulated round the house and garden.

Auckland Star, Volume LIV, Issue 179, 28 July 1923, Page 7

LANDSLIDES IN LYTTELTON.

SLIPS IN THE HILLS.

(By Telegraph.—Special to "Star.")

CHRISTCHURCH, this day.

Continuous rains during the month have caused a number of land slips of varying sizes in Lyttelton. On Thursday a portion of a clay bank over Captain S. S. Horn's house gave way, and about four tons of earth fell perilously close to the back door just after a previous fall of two tons had been cleaned away.

Water surging from the hills disappeared under the foundations of the house and found an outlet at the garden gate several feet below. At the same level the undercurrent made a cave about twelve feet deep, something like a shell hole in the lawn adjoining the house.

Yesterday a land slip of several tons occurred in Salts Gully. Starting on the hill side it swept all before it for about eighty yards, carrying away two fowl houses, overturning a substantially built shed, and uprooting a number of fruit trees. Later a further slide of soft mud covered the side entrance to the house. It is recalled that about 3 four years ago in the same locality a large landslide occurred in the early hours of the morning completely covering a cowshed and smothering eight cows.

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Sumner Road rockfalls - 1907

Star, Issue 8891, 30 March 1907, Page 7

GREAT LANDSLIP.

SUMNER CLIFFS "TAILING."

TRAFFIC COMPLETELY BLOCKED.

POSSIBLE DANGER TO CLIFTON RESIDENCES.

The cliffs on the Sumner Road have been a source of anxiety to the authorities and the public ever since the road was first opened by the Provincial engineer, and periodically there have been falls of rock, more or less serious. The cliff, of course, is constantly "tailing." That is to say, the steep face tends to wear down with the weather, and if the falling debris were left undisturbed it would, in course of time, form a moderately easy slope. The process is for the most part a very slow one, but the heavy rains of the past few days, with rather severe changes of temperature, apparently hastened the breaking-away, and last evening an enormous mass of rock and earth came down without warning.

The locality is familiar to everyone who has journeyed to Sumner, and the overhanging rocks always look threatening. The slip occurred just beyond the Shag Rock corner, between the Shag Rock and what is called the Middle Rock, and according to the estimate of the Sumner engineer, Mr W. J. O'Donnell, between 3000 and 5000 tons of stuff came down.

The fall occurred just before the seven o'clock tram from Christchurch reached the Shag Rock. Indeed, the tram is said to have been within a chain or two of the Rock when the enormous mass came thundering down on to the road. Fortunately there was no one very near, but Mr O'Donnell's son and daughter, who were on the road, saw the fall in the moonlight. The debris buried the roadway for perhaps a couple of chains, in places to a depth of fifteen or twenty feet. It smashed the water mains which supply the borough of Sumner, carried away the overhead gear of the electric tramway and played havoc with the permanent way. One mighty rock lies on the outer side of the road, and in its fall it has torn up rails and sleepers. The lines are bent and broken, and the permanent, way will have to be reconstructed.

So many false alarms have been, raised in connection with the cliffs that the report of a great slip did not at once receive credence. But the non-arrival of the seven o'clock tram made it clear to Sumner folk that the line was blocked, and news was sent through promptly to Christchurch. Vigorous measures were demanded, and emergency gangs were hastily organised at both ends. The Sumner Borough Council, concerned for the road, but more immediately still for its water supply, engaged five and twenty men forthwith, to connect the upper reservoir with the lower main, so that a supply might be available at the earliest moment. At the Christchurch end, the tramway authorities at once sent down a gang of men to clear the line. It was hopeless to think of getting trams through, however, and arrangements were hastily made to carry passengers between Sumner and Monck's by motor launch. This service worked very well, the last batch of passengers getting through to Sumner by midnight.

In the meantime Mr F. H. Chamberlain, the Tramway Board's engineer, went down to investigate. He returned late last night, and it was understood that a gang of thirty men would be put on at once to clear the line and carry out repairs. The Sumner Borough Council expected last night that a dray might be able to get through by midday to-day, but there seems to be no prospect of tramway communication being restored before to-night at the earliest. A fervent hope was expressed, however, that daylight would prove the obstacles to be less formidable than they appeared by moonlight. Still, there are some enormous pieces of rock in the debris, and these will not easily be shifted, even with the appliances available to the Tramway Board's staff.

It was rumoured last night that one of the houses on the hill-top was unsafe, but inquiry showed that the fall had occurred from the face of the cliff, and there was no reason to suppose that the ground at the back was affected.

Star , Issue 8892, 2 April 1907, Page 2

[Editorial]

THE SUMNER ROAD.

The recent landslip on the Christchurch-Sumner road has naturally directed public attention to the need for protection against similar accidents. It is felt that Friday's slip might, under different circumstances, have been attended by loss of life, and that unless a repetition of it is prevented the next fall may be much more serious in its consequences. The public confidence is indeed gravely disturbed, and it rests with the authorities to take immediate steps to restore it. The precise nature of the action to be taken is not, of course, for a layman to decide. It should be left for the decision of expert engineers, and the engineers should be the cleverest procurable. And when the experts have given their opinion as to the nature of the measures to be taken to render the road absolutely secure against further falls, it will be the duty of the authorities to carry them into effect without loss of time. If there is any difference of opinion as to the local body or which the responsibility of doing the work rests, it should be settled at once. There may not be another fall for years but on the other hand the cliff may give way again at any moment, and it is the duty of the authorities to make provision for the possibilities of the immediate, not the distant future. Considerations of expense should not be allowed to stand in the way. The safety of the public is of more important than saving the rates, and no expense in reason should be spared to ensure the public safety. The mere removal of the debris that fell last week, and the widening of the road under the cliffs though necessary for the convenience of traffic, would be of little avail as permanent solution of the problem. It is possible that the top of the cliffs will have to be removed or the estuary bridged and the road diverted from under the cliffs. It is possible, even, that still more drastic measures will be necessary to ensure the safety of traffic. But whatever steps are shown to be expedient must be taken no matter what the cost may be. Sumner is the principle watering-place of North and Mid-Canterbury; it has a large resident population, and it is patronised by hundreds of visitors daily. To leave the road in its present position would be to set up a perpetual menace to life and limb, and to endanger the popularity of the borough both as a place of residence and as a holiday resort. We have no desire to be alarmist, but we certainly think that the various authorities interested ought to co-operate in providing a safe access to the borough with as little loss of time as possible.

Wanganui Herald, Volume XXXXI, Issue 12130, 2 April 1907, Page 5

THE SUMNER LANDSLIP.

A Dangerous Cliff.

(Per United Press Association.)

CHRISTCHURCH, April 2.

The work of clearing the Sumner landslip was suspended yesterday, there being ample room for vehicles to pass. Large rocks have to be blasted, and it will five or six days to clear the road altogether. The general opinion is that the upper overhanging cliff will have to be brought down and the face sloped back, but even though the road and the tramline be moved further out into the estuary there is still the danger of a fall from the cliff, which at present seems as if hanging just over the road. If something be not done a terrible accident may happen.

Evening Post, Volume LXXIII, Issue 79, 4 April 1907, Page 6

There is a difference of opinion whether the Sumner Borough Council or the Christchurch Tramway Board is responsible for the roadway running under the Cliffs, the scene of the recent landslip. The board maintains that its duty is to make tramlines and not to form roadways. Meanwhile no steps have been taken to remove the source of a very great danger from the overhanging rocks.

A4 APPENDIX 4: RESULTS FROM THE TWO-DIMENSIONAL SITE RESPONSE ASSESSMENT FOR CROSS-SECTION 1

The results from the two-dimensional site response modelling are shown for cross-section 1. The maximum acceleration (A_{MAX}) at the slope crest derived from the modelling of each synthetic earthquake time history has been plotted in Figure A4.1. The slope crest is defined as the convex break in slope between the lower steeper slope and the upper less steep slope. Each point on the graph represents the response of this location to a given synthetic free field rock outcrop earthquake input motion (Table A4.1).

The extent of the coseismic landslide, inferred from survey data, crack mapping and slope morphology, is about 100 m in length and about 50 m wide. The modelled peak ground accelerations at the surface across the landslide are, therefore variable given the extent of the landslide. The highest modelled peak ground accelerations during the 22 February 2011 earthquake coincide with the convex break in slope (A_{MAX}).

The fundamental frequency of the slope varies from 2.8 to 4.2 Hz based on the equation in Bray and Travasarou (2007), where frequency = $1/(4 \times H/V_S)$, and H = slope height of 36 m, and V_S = average shear wave velocity for the slope of 400–600 m/s. The dominant frequency of the input motions is between 3.6 Hz and 5.7 Hz. The "tuning ratio" defined as the ratio between the dominant frequency of the input motion and the fundamental frequency of the slope (Wartman et al., 2013), is about 1.3–2.1 for a shear wave velocity of 400 m/s, and 0.9–1.4 for a shear wave velocity of 600 m/s.

Results from the seismic response assessment suggest that the peak ground acceleration amplification factors (S_T) for cross-section 1 is about 2.6 (±0.3) for horizontal motions, and 2.4 (±0.1) for vertical motions – errors at one standard deviation (Figure A4.1).

Table A4.1 Results from the two-dimensional site response assessment for cross-section 1, using the out-of-phase synthetic free-field rock outcrop motions for the Maffeys Road site by Holden et al. (2014) as inputs to the assessment. PGA is peak ground acceleration.

Earthquake (2011)	Free-field input PGA (horizontal) – A_{FF} (g)	Free-field input PGA (vertical) – A_{FF} (g)	Maximum PGA (horizontal) at convex break in slope – A_{MAX} (g)	Maximum PGA (vertical) at convex break in slope – A_{MAX} (g)
22 February	0.87	0.77	2.48	1.89
16 April	0.05	0.03	0.09	0.10
13 June	0.44	0.30	0.71	0.71
23 December	0.19	0.14	0.43	0.31

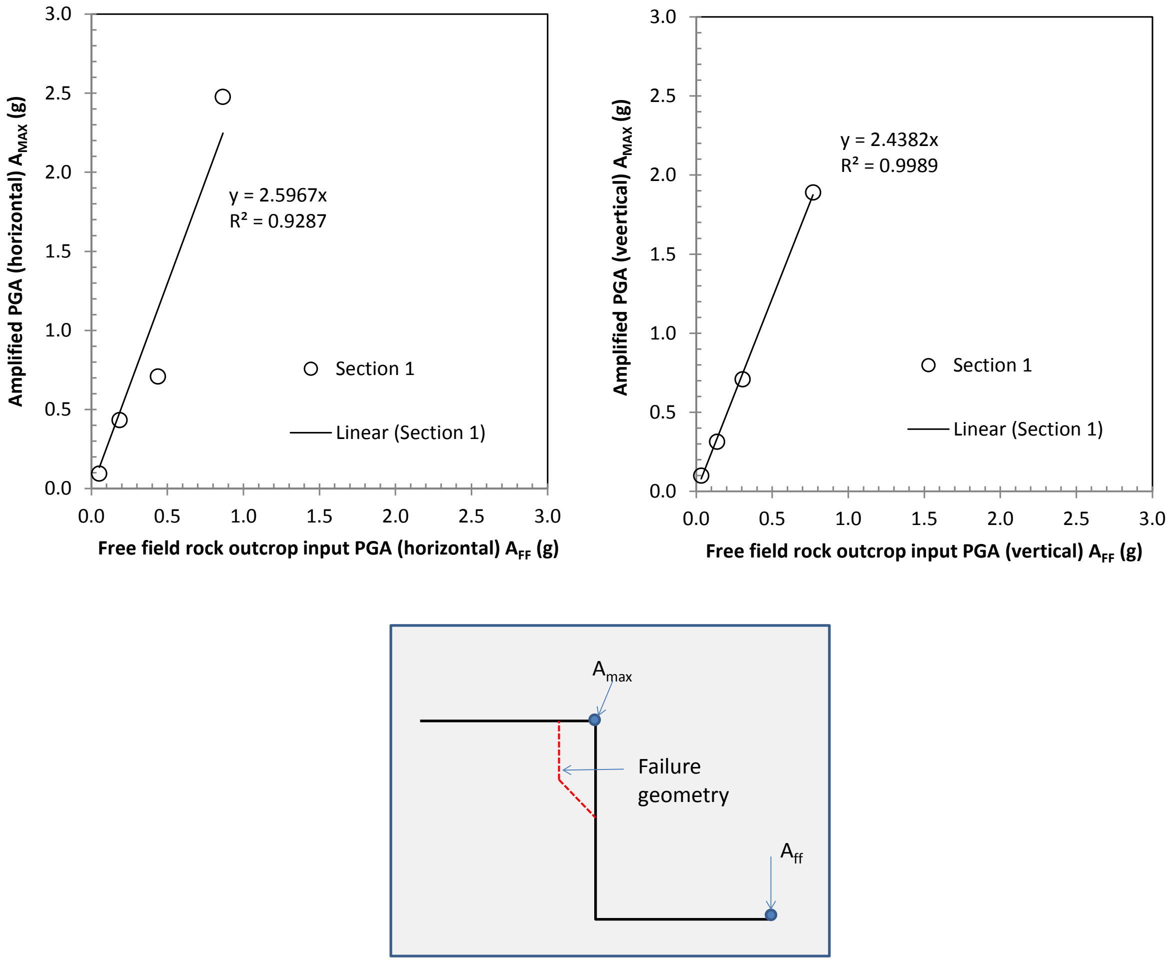


Figure A4.1 Amplification relationship between the synthetic free-field rock outcrop input motions (A_{FF}) and the modelled cliff crest maximum accelerations (A_{MAX}) for cross-section 1. A schematic diagram showing the locations of the various recorded accelerations is shown.

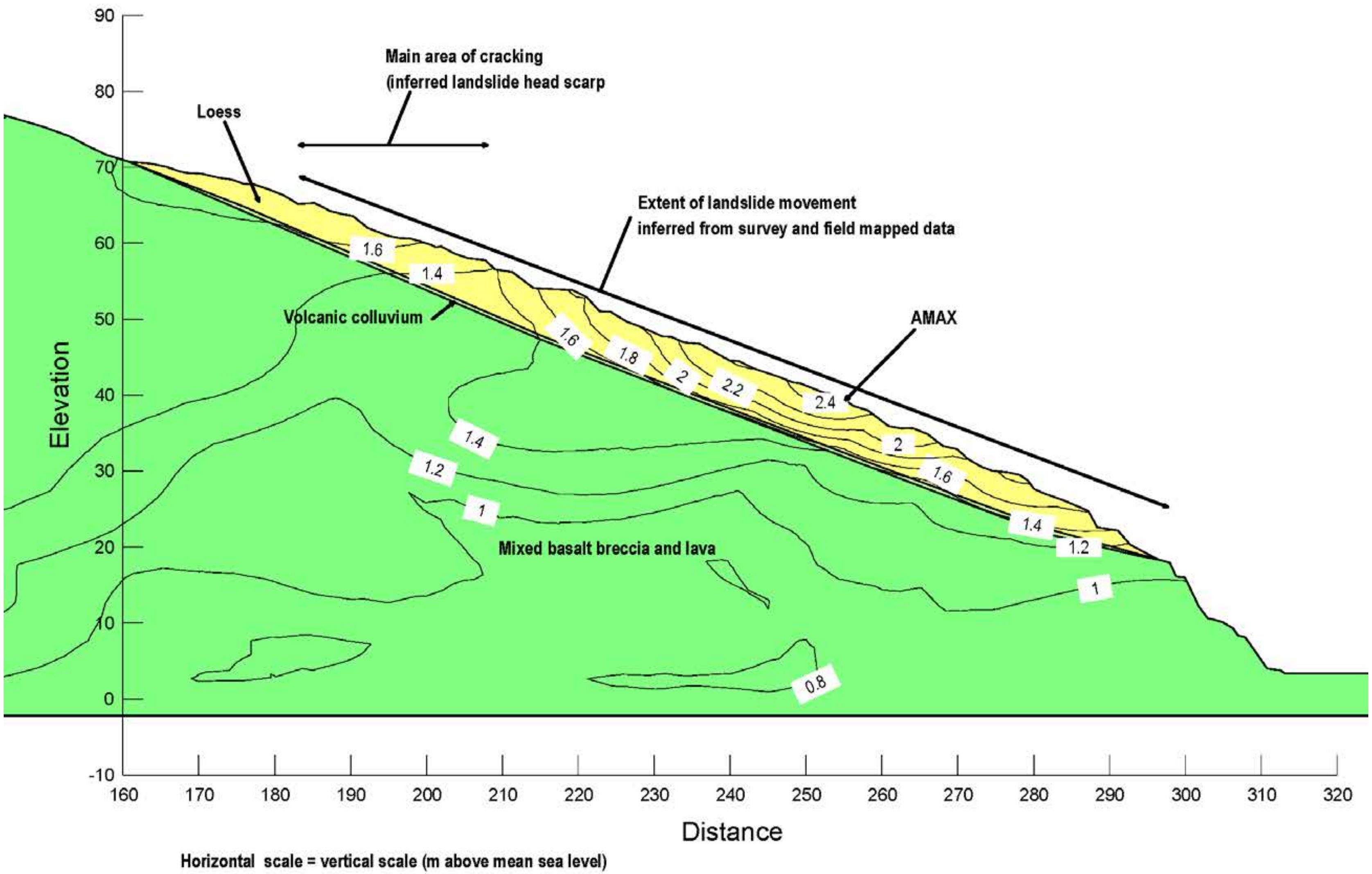


Figure A4.2 Modelled peak horizontal ground acceleration contours for the 22 February 2011 earthquake at Maffeys Road, cross-section 4.

The relationship between the modelled vertical and horizontal peak ground accelerations recorded at the slope crest (A_{MAX}) is shown in Figure A3.3. The gradient of the linear fit is 0.78 (±0.04) – errors at one standard deviation. The relationship between horizontal and vertical peak ground accelerations appears linear.

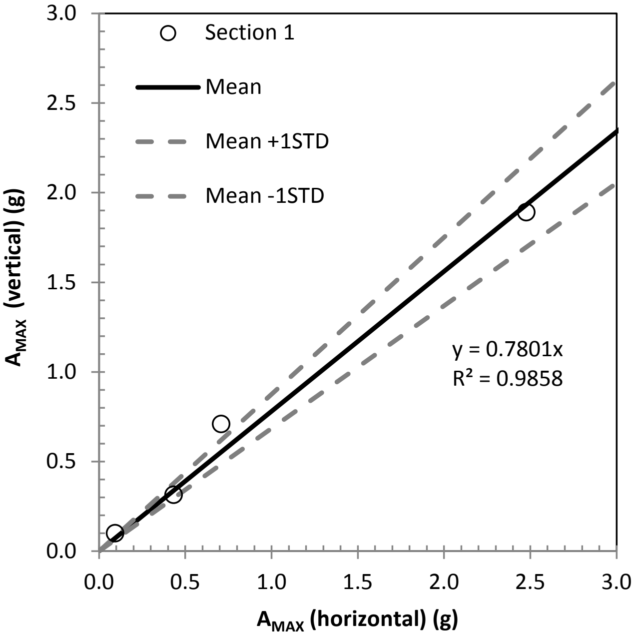


Figure A4.3 Relationship between the modelled horizontal and vertical maximum accelerations modelled at the convex break in slope (A_{MAX}) for cross-section 1, using the synthetic free-field rock outcrop motions for the Deans Head site by Holden et al. (2014) as inputs to the assessment. The mean and standard deviation trend lines are fitted for A_{MAX} all data. Errors are shown as the mean ± one standard deviation (1 STD).

Results from this assessment have shown that the relationship between the peak ground acceleration of the free-field input motion and the corresponding modelled peak acceleration at the slope crest (A_{MAX}) is approximately linear. In the range of modelled peak horizontal accelerations, the horizontal amplification factor (S_T) is typically in the order of about 2.6 times the input free-field peak horizontal acceleration.

Cross-section 1 comprises about 6 m of loess and colluvium overlying mixed basalt lava and breccia, where the mean shear wave velocities of the materials change from 600–1,200 m/s in the basalt breccia and lava, to 200–400 m/s in the loess and colluvium (Figure A3.2). The results suggest that the impedance contrasts between the materials contribute most to the amplification of shaking, but that the peak horizontal accelerations (for all modelled earthquakes) concentrate around the convex break in slope, defined as A_{MAX}.

These results are similar to those reported by others (e.g., Del Gaudio and Wasowski, 2010), where material impedance contrasts have been shown to have a significant effect on the amplification of shaking. Given the increased amplification of shaking within the loess and colluvium, coupled with the coseismic landslide displacement inferred from surveying, it likely that the basal slide surface is coincident with the boundary between the colluvium and the underlying rock.

In experimental data, as the slope displaces during an earthquake, the slide surface can "base isolate" the mass above, resulting in lower levels of shaking and displacement. Therefore, the reported amplification factors are near the upper bound of published topographic amplification factors. However, given the impedance contrasts between the loess/fill and rock are so high, this contrast could lead to the trapping of seismic waves within the loess and colluvium. Assessment of this is outside the scope of this report.

Eurocode 8, Part 5, Annex A, gives some simplified amplification factors for the seismic action used in the verification of the stability of slopes. Such factors, denoted S_T, are to a first approximation considered independent of the fundamental period of vibration and, hence, multiply as a constant scaling factor.

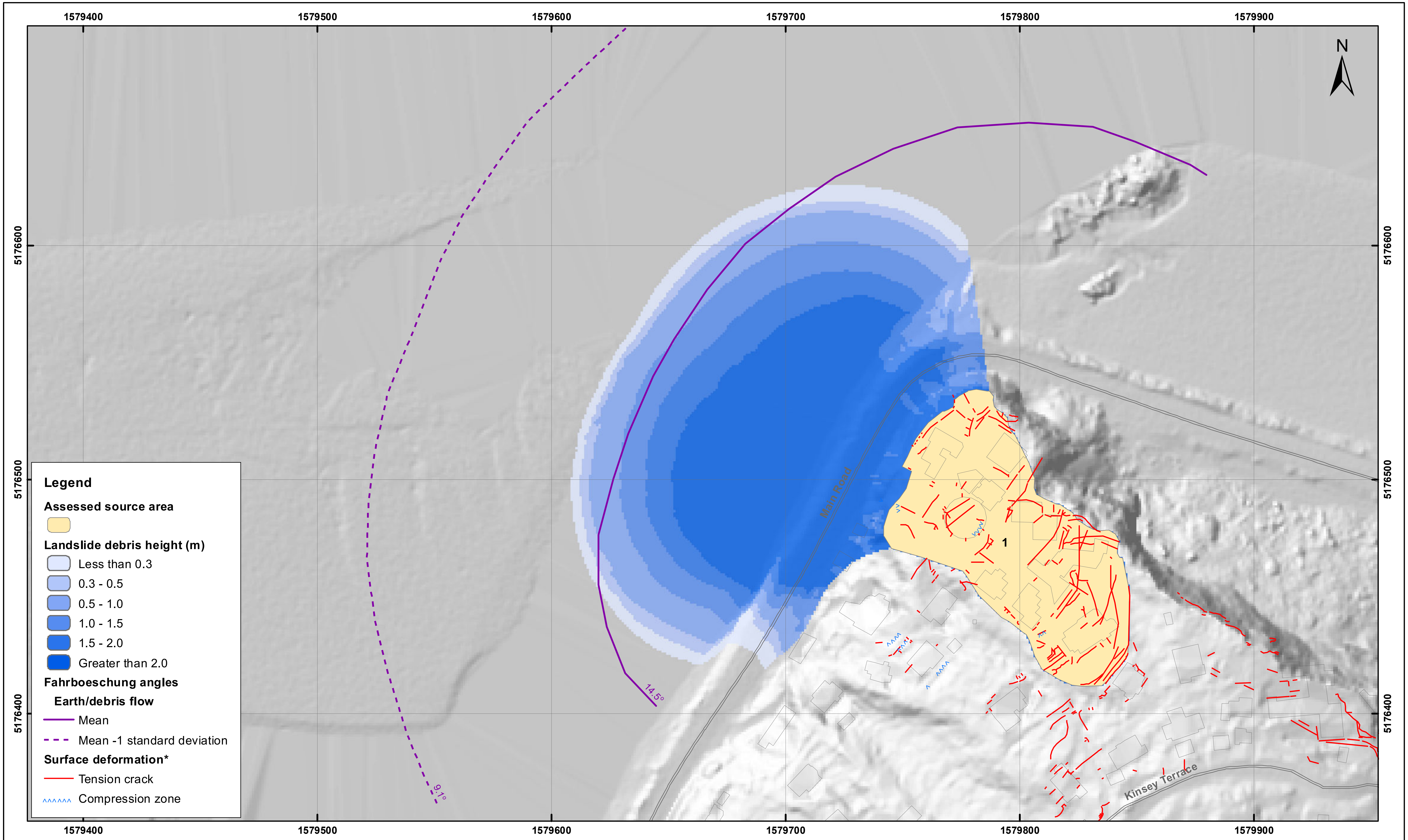
Eurocode 8, Part 5, Annex A recommends:

1. Isolated cliffs and slopes. A value $S_T \geq 1.2$ should be used for sites near the top edge;
2. Ridges with crest width significantly less than the base width. A value $S_T \geq 1.4$ should be used near the top of the slopes for average slope angles greater than 30° and a value S_T >1.2 should be used for smaller slope angles;
3. Presence of a loose surface layer. In the presence of a loose surface layer, the smallest S_T value given in a) and b) should be increased by at least 20%;
4. Spatial variation of amplification factor. The value of S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base; and
5. These amplification factors should in preference be applied when the slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m.

Ashford and Sitar (2002) recommend an S_T of 1.5 be applied to the maximum free-field acceleration behind the crest based on their assessment of slopes in homogenous materials, typically >60° to near vertical and of heights (toe to crest) of typically >30 m. This factor is based on the assessment of slopes that failed during the 1989 Loma Prieta M_W 6.9 earthquake.

Results from the seismic response assessment suggest that the horizontal peak ground acceleration amplification factors (S_T) for Deans Head are about 2–3 (cross-section 1) times greater than the free field input motions. These are larger than those values reported by Ashford and Sitar (2002), and are in part a function of the impedance contrasts within the slope, which are not reported to occur in the slopes assessed by Ashford and Sitar (2002). These higher factors may also be a function of the site to earthquake source distances. In the case of Deans Head, the site is within 5 km of the epicentres of the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, making them all "near-field" earthquakes.

A5 APPENDIX 5: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1 AND 2; ESTIMATED LANDSLIDE RUNOUT HEIGHT



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake
2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 1 - Upper Volume (40,000 m^3)

Deans Head - Port Hills
Christchurch

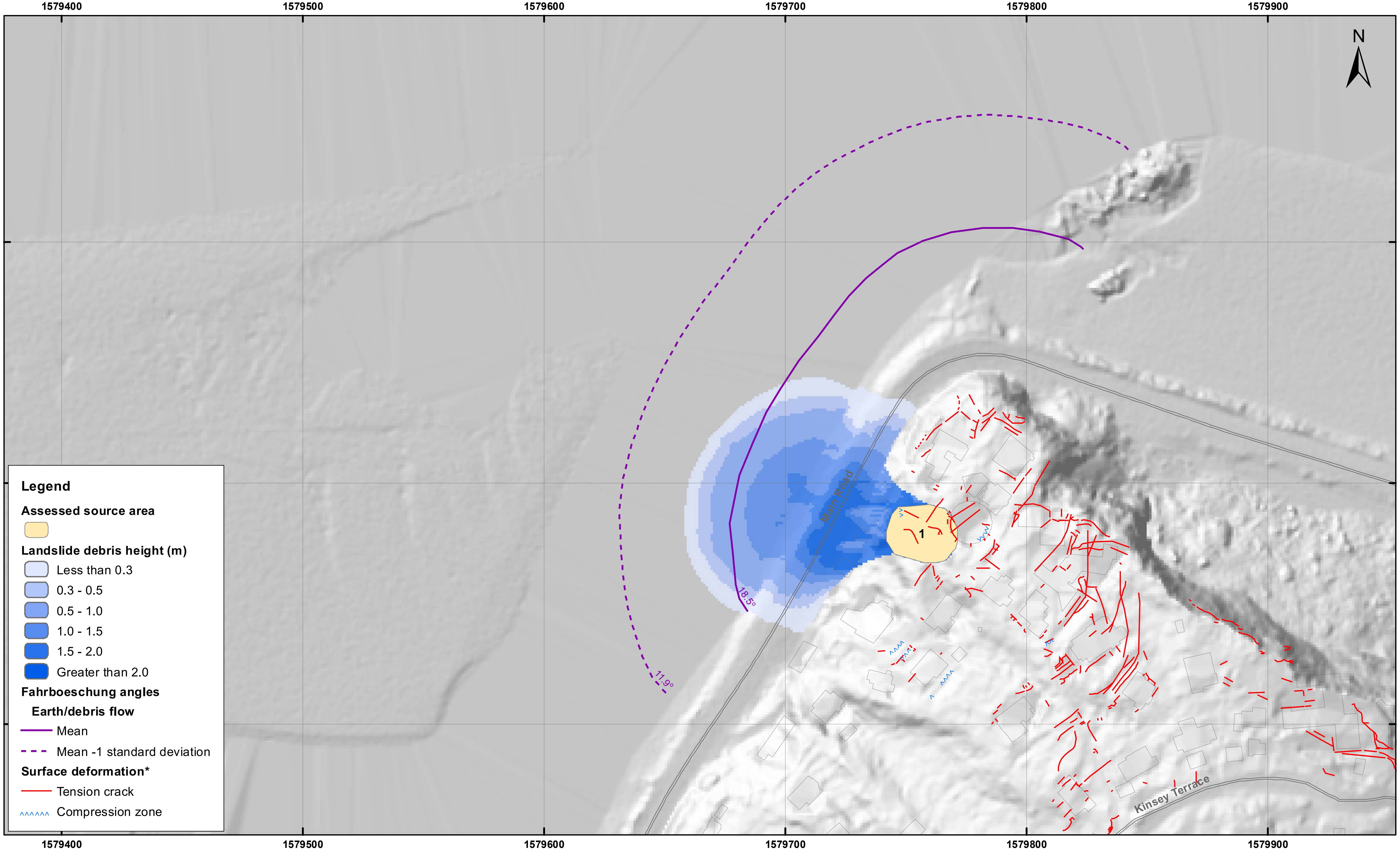
APPENDIX 5

Map 1

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 1 - Middle Volume (3,500 m^3)

Deans Head - Port Hills
Christchurch

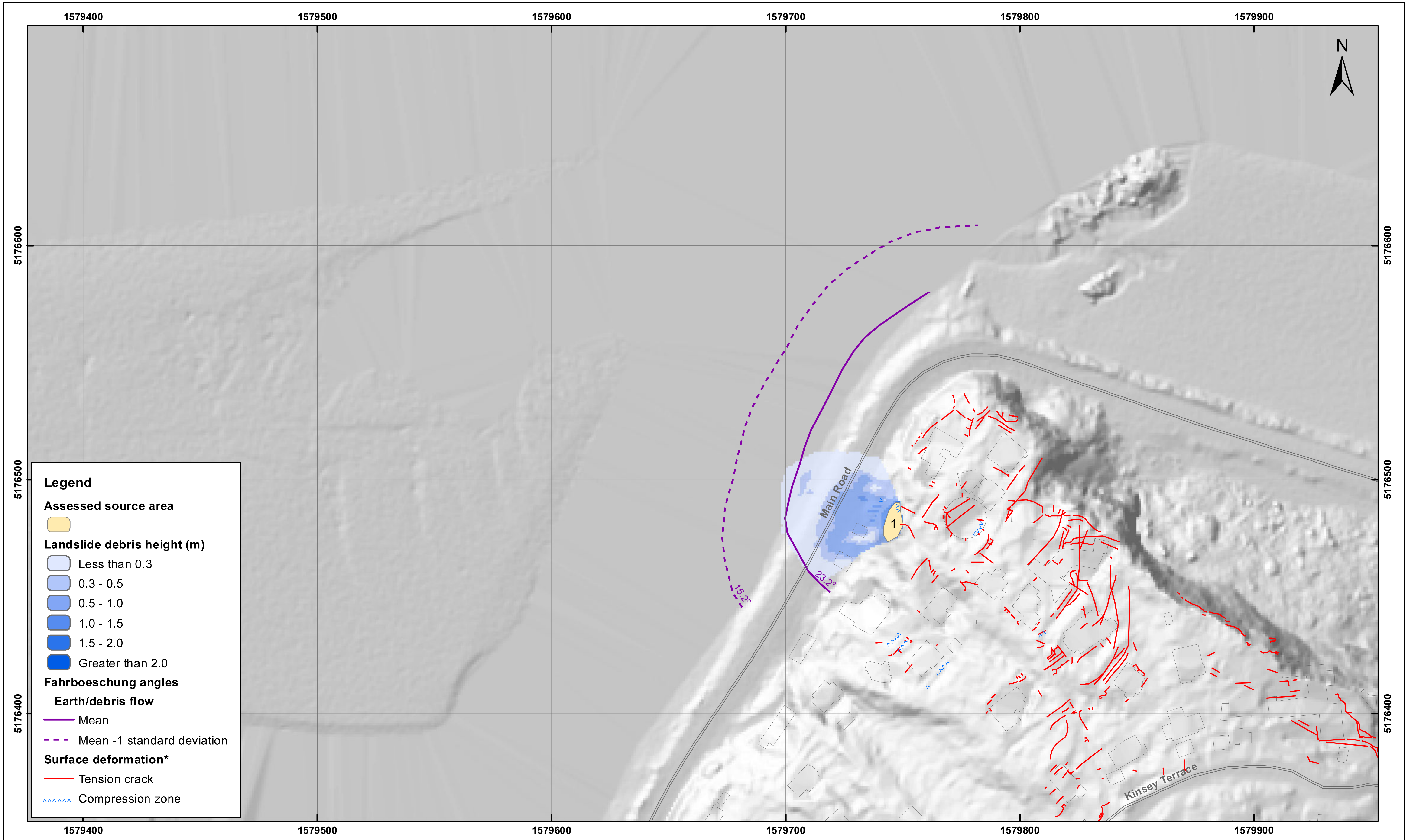
APPENDIX 5

Map 2

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 1 - Lower Volume (330 m^3)

Deans Head - Port Hills
Christchurch

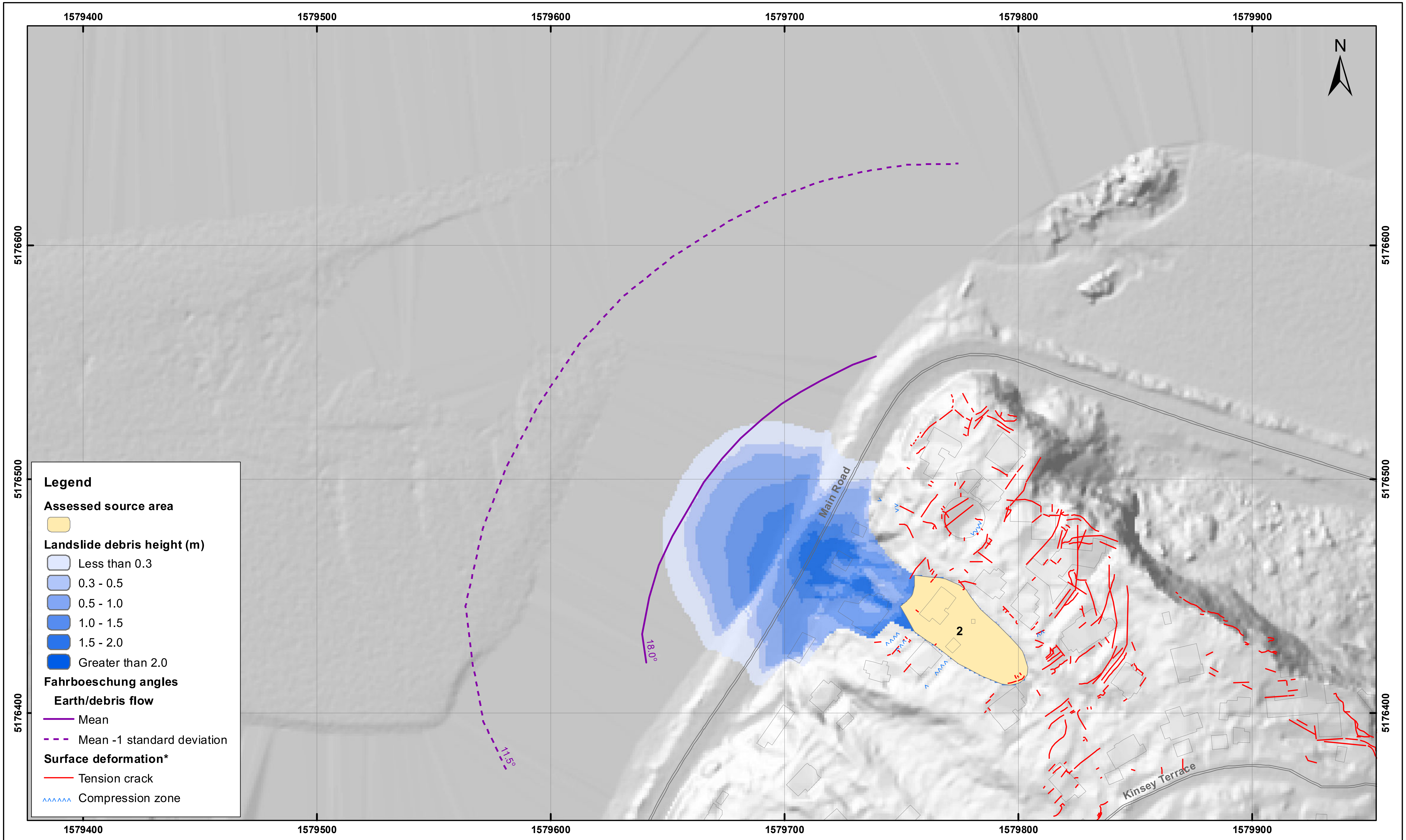
APPENDIX 5

Map 3

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 2 - Upper Volume (4,500 m^3)

Deans Head - Port Hills
Christchurch

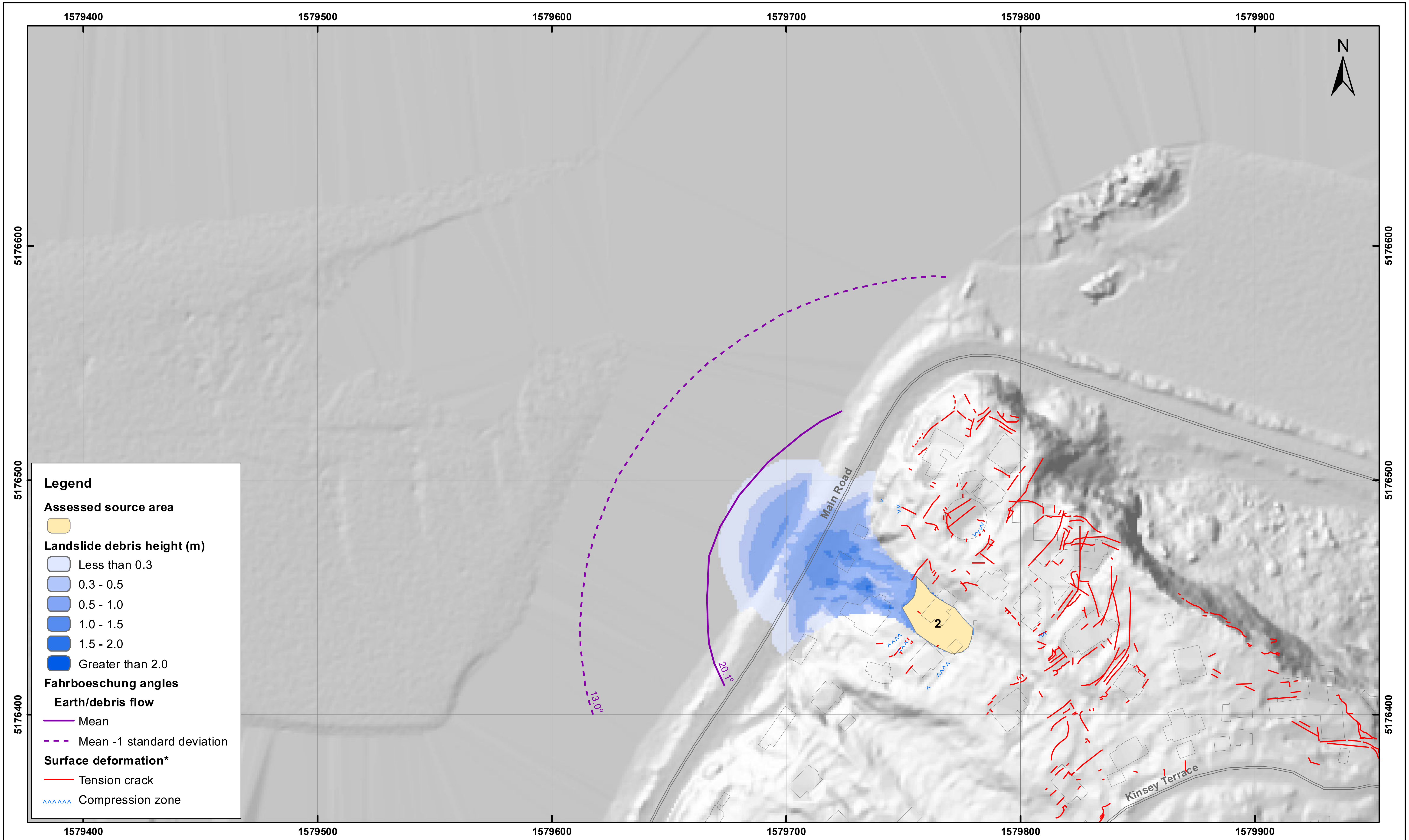
APPENDIX 5

Map 4

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
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CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 2 - Middle Volume (1,500 m^3)

Deans Head - Port Hills
Christchurch

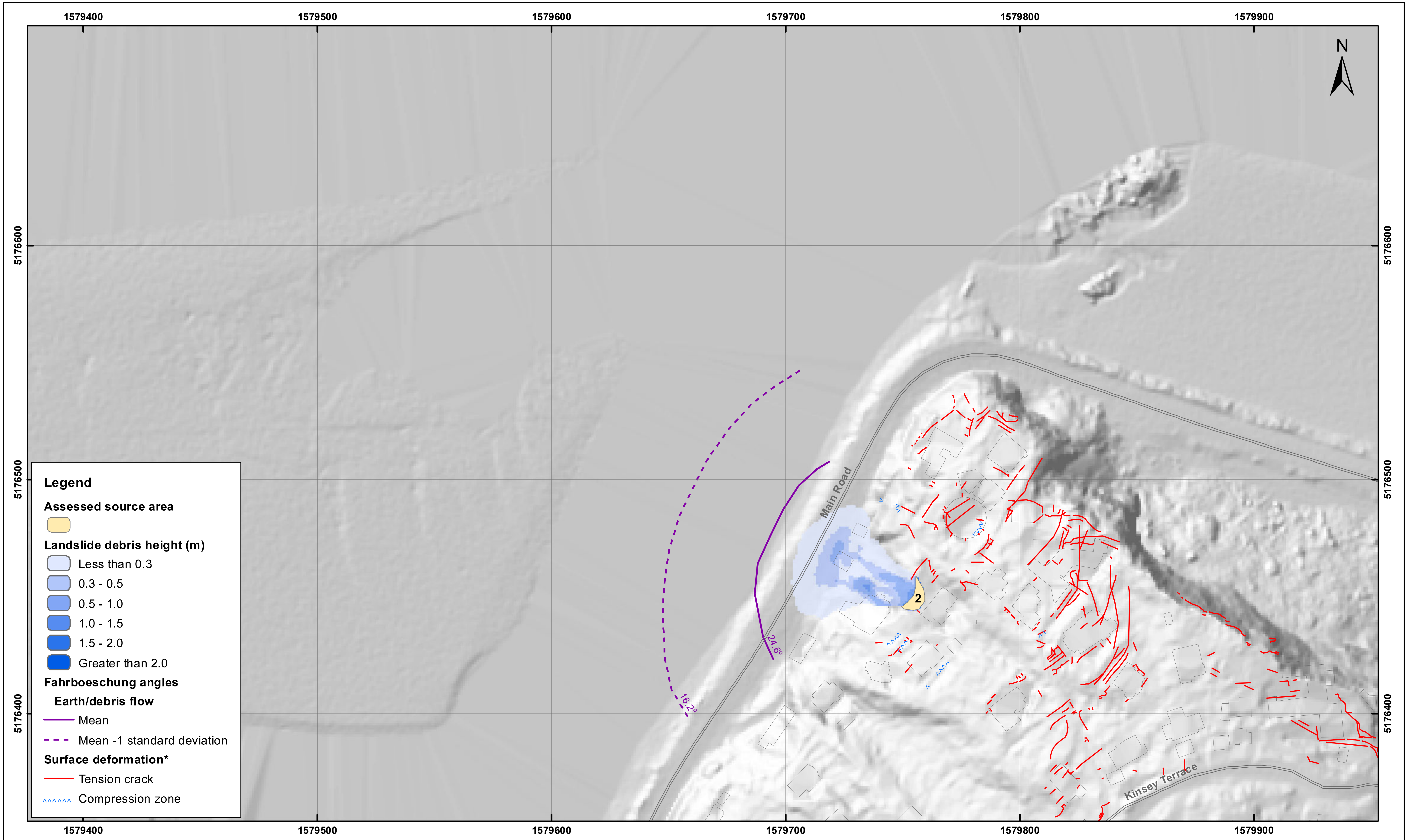
APPENDIX 5

Map 5

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
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PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 2 - Lower Volume (180 m^3)

Deans Head - Port Hills
Christchurch

APPENDIX 5

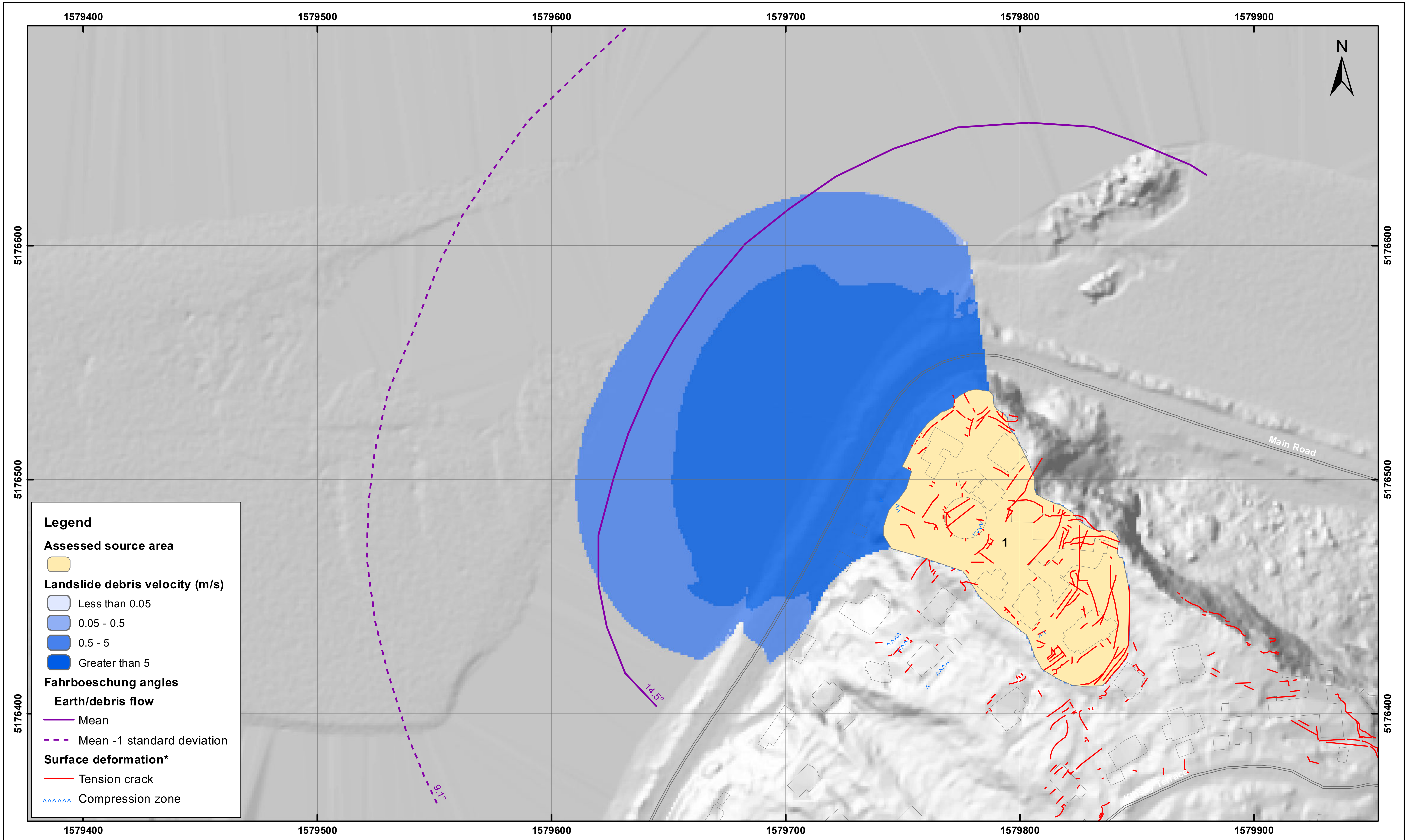
Map 6

FINAL

REPORT: CR2014/77

DATE: June 2014

A6 APPENDIX 6: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1 AND 2; ESTIMATED LANDSLIDE RUNOUT VELOCITY



SCALE BAR: 0 50 100 m

EXPLANATION:

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CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 1 - Upper Volume (40,000 m^3)

Deans Lane - Port Hills
Christchurch

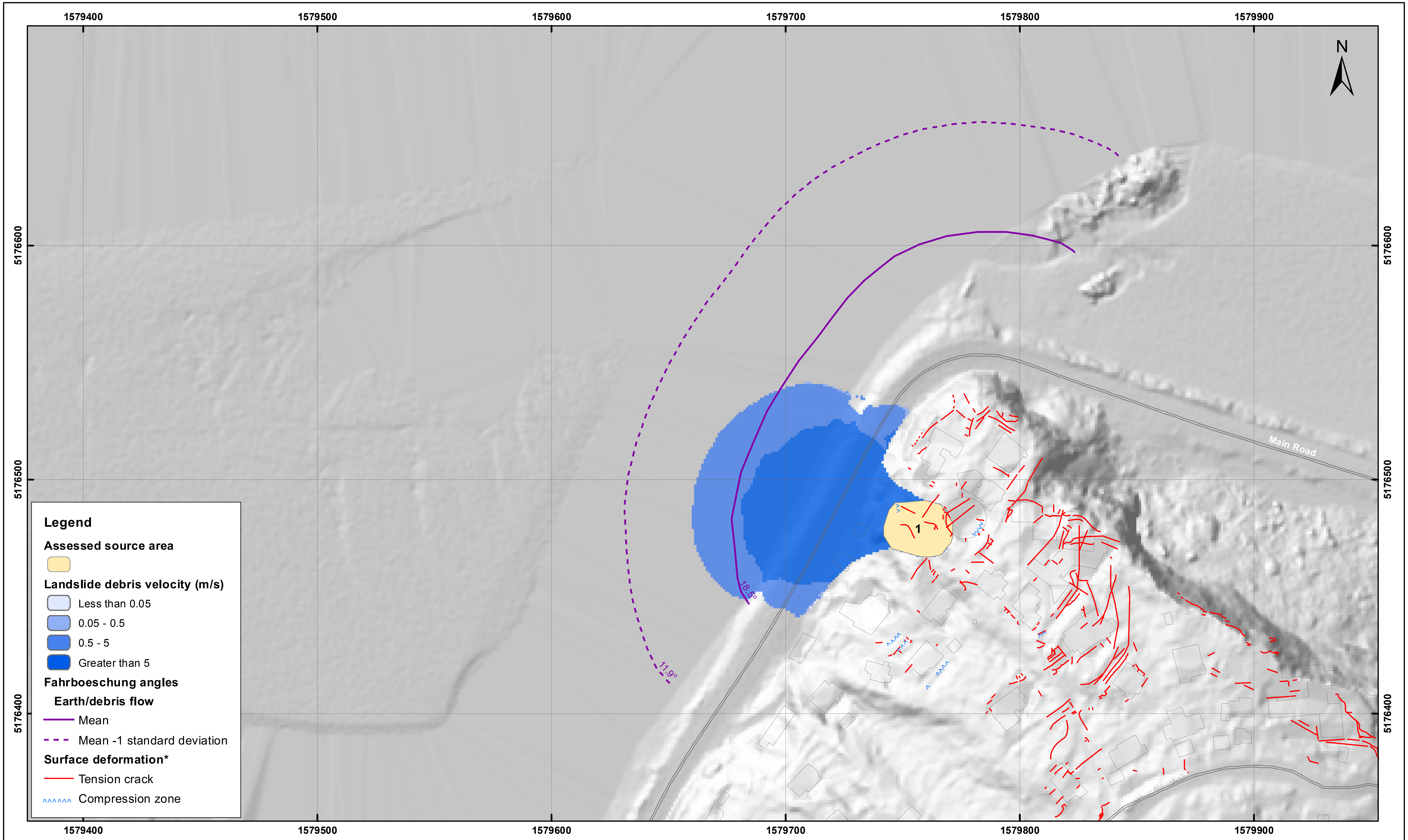
APPENDIX 6

Map 1

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 1 - Middle Volume (3,500 m^3)

Deans Lane - Port Hills
Christchurch

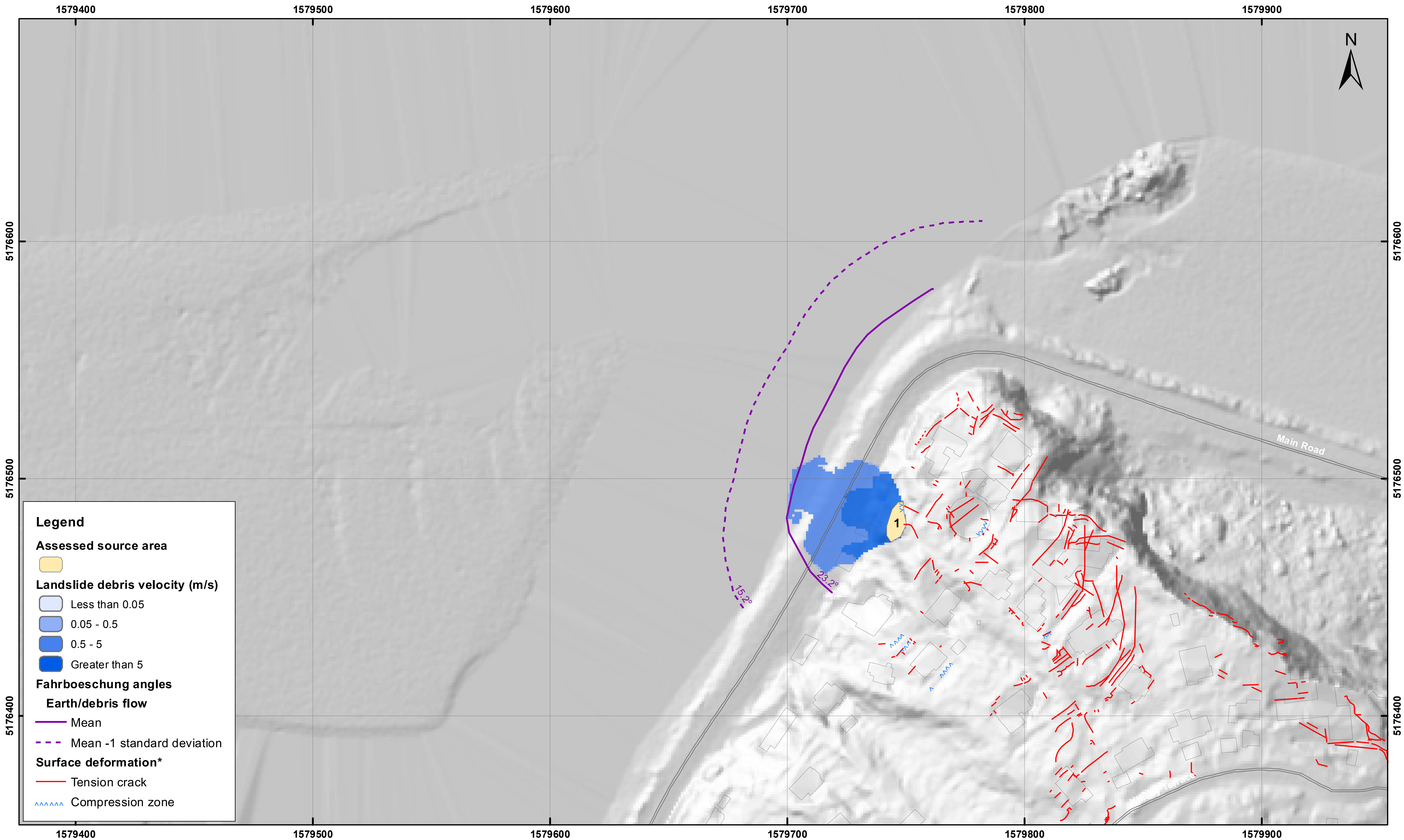
APPENDIX 6

Map 2

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake
2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
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PROJECTION: New Zealand Transverse Mercator 2000

DRW:
BL, WR

CHK:
CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 1 - Lower Volume (330 m^3)

Deans Lane - Port Hills
Christchurch

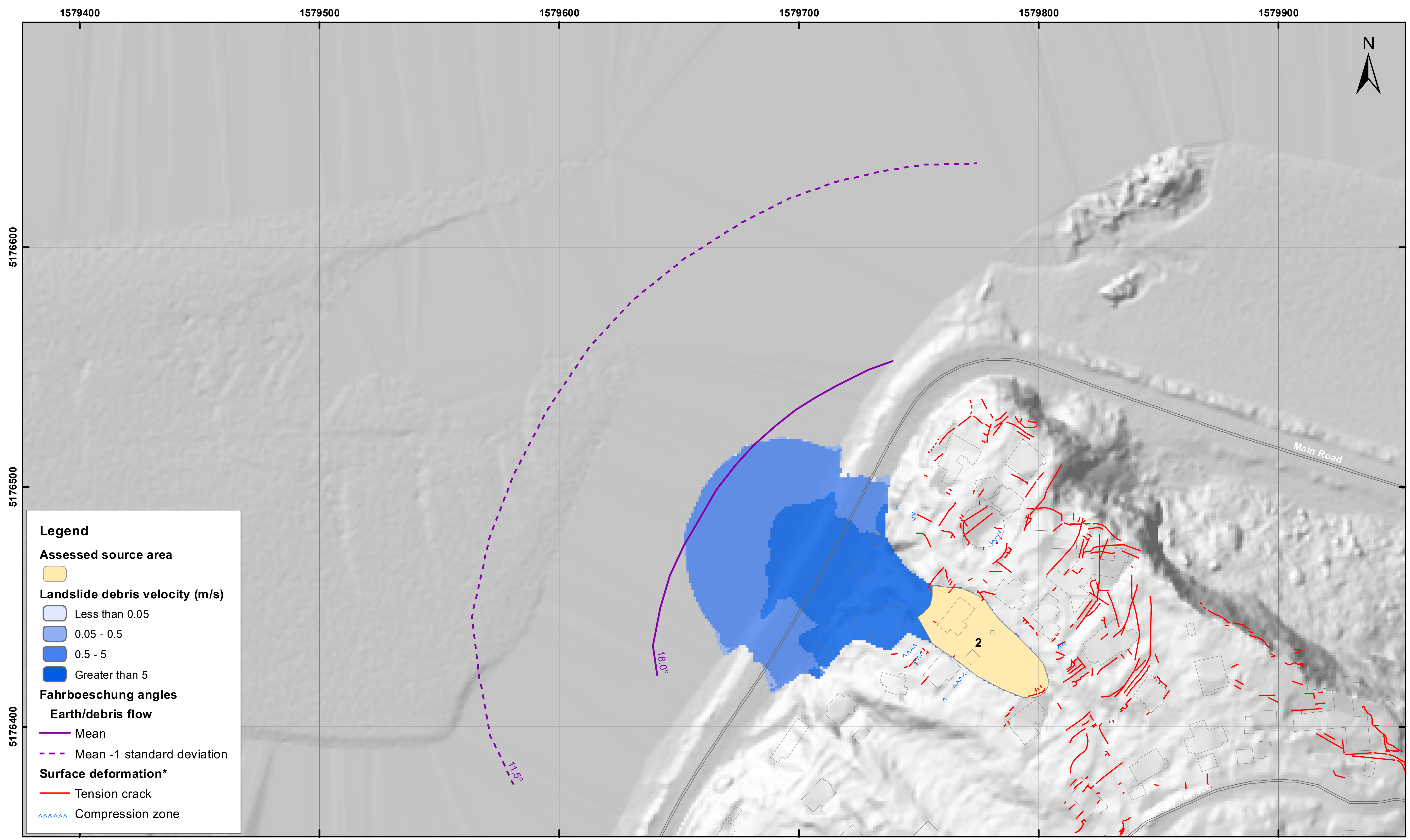
APPENDIX 6

Map 3

FINAL

REPORT:
CR2014/77

DATE:
June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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PROJECTION: New Zealand Transverse Mercator 2000

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CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 2 - Upper Volume (4,500 m^3)

Deans Lane - Port Hills
Christchurch

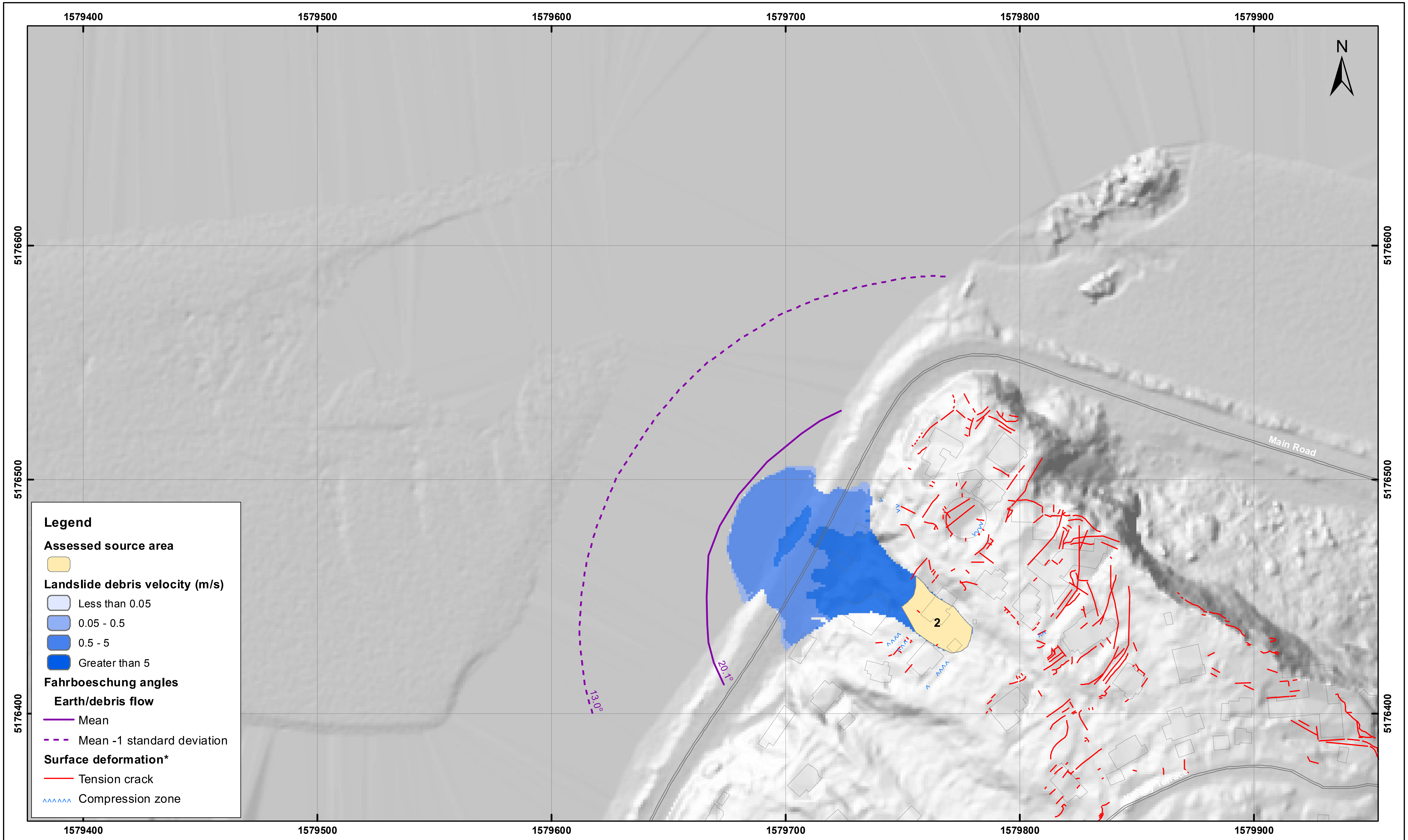
APPENDIX 6

Map 4

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 2 - Middle Volume (1,500 m^3)

Deans Lane - Port Hills
Christchurch

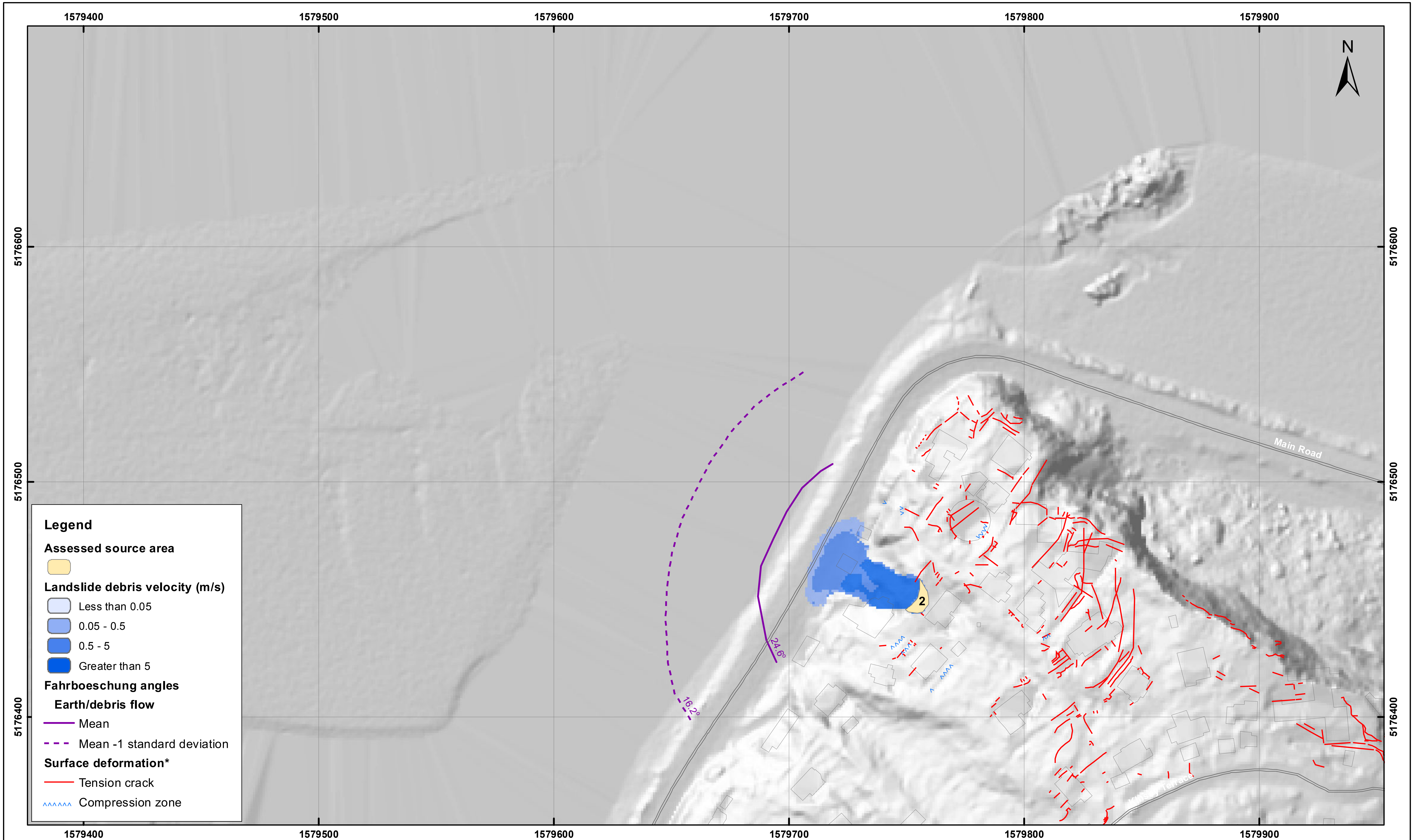
APPENDIX 6

Map 5

FINAL

REPORT: CR2014/77

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 2 - Lower Volume (180 m^3)

Deans Lane - Port Hills
Christchurch

APPENDIX 6

Map 6

FINAL

REPORT: CR2014/77

DATE: June 2014

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