Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Risk assessment for Cliff Street

C. I. Massey F. Della Pasqua B. Lukovic
W. Ries D. Heron

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Risk calculations were independently checked by T. Taig (TTAC Limited) and R. Buxton (GNS Science).

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EXECUTIVE SUMMARY

ES 1 INTRODUCTION

This report combines recent field information collected from the Cliff Street 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. The results in this report supersede those in an earlier cliff-collapse study (Massey et al., 2012a).

Following the 22 February 2011 earthquakes, extensive cracking of the ground had 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 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.

The Cliff Street mass movement was assessed in the Stage 1 report as being in the highest relative hazard exposure category (Class I). The risk to life of people in dwellings at the slope crest and toe from debris avalanche and cliff top recession hazards associated with the steep rock slopes (collectively termed cliff collapse) has already been estimated for the Cliff Street mass movement and is contained in Massey et al. (2012a).

Following the 22 February 2011 earthquakes significant localised cracking was also noted in the loess (soil) above the steep rock slope at the Cliff Street mass movement and a persistent low-angle discontinuity – along which movement of the mass occurred – was observed daylighting (exposed) in the slope face. The significance of the cracking and discontinuity, with respect to slope instability hazards, was not assessed by Massey et al. (2012a).

The loess and rock slopes in the Port Hills have always been susceptible to landslides (e.g., Bell and Trangmar, 1987; McSaveney et al., 2014). Debris from loess landslides, in particular earth/debris flows (Cruden and Varnes, 1996) has the potential to travel further than debris from cliff collapse. Debris from large cliff collapses – larger than those assessed by Massey et al. (2012a) – has the potential to fall from the cliff, with the resultant debris travelling further than occurred in the 2010/11 Canterbury earthquakes.

These are the reasons for the Cliff Street mass movement being included in the Class I (high priority for further investigation) mass movements, and for the commissioning of this report.

This report, as part of the Stage 3 investigations, presents the risk assessment results for the Cliff Street Class I mass movement.

ES 2 INVESTIGATION PROCESS AND FINDINGS

Detailed investigations of the site and its history show no evidence for large slope failures from the time of European settlement (about 1840) to the present. There is possible evidence of a substantial relict landslide scar dating from earlier times, and of relatively regular small earth/debris flows occurring in the loess.

The slopes at Cliff Street were significantly cracked in the 22 February 2011 earthquake, though relatively little movement was observed in the subsequent earthquakes. Overall ground displacement through the 2010/11 Canterbury earthquakes is not known as there were no markers in place to enable before and after measurements to be made.

The present condition of the slopes is that they are significantly cracked, with numerous open cracks in the loess covering and in the rock-slope face, allowing rapid water ingress into the ground. By mapping and linking cracks and other features of the site it has been possible to identify a further two potentially significant landslide hazards, which were not addressed in the earlier cliff collapse study. These landslide hazards are: 1) earth/debris flows occurring in the loess and fill (soil) materials; and 2) larger cliff collapses (comprising debris avalanches and cliff-top recession volumes that are larger than those previously assessed) occurring in the volcanic rock.

Numerical models have been used to assess the stability of the Cliff Street slopes, in particular the two potential landslide hazard types. Analyses have considered both:

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

Earth/debris flows

The main triggering mechanism for the assessed earth/debris flows 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, especially in the eastern loess dominated source area (source area 1).

The water contents of the loess 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, and therefore more susceptible to failure.

For the assessed earth/debris flow 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 earthquake aftershocks since the 22 February 2011 earthquake. Although small (less than 50 m^3 in volume) earth/debris flows were triggered at the site by rain following the 2010/11 Canterbury earthquakes, no larger landslides have occurred.

Cliff collapse (debris avalanches and cliff-top recession)

The main triggering mechanism for the assessed cliff collapses is considered to be earthquakes, although rainfall could trigger smaller volumes of rock to fall from the slopes.

The dynamic analysis used the proprietary Slope/W model to calibrate the slope parameters, by simulating the 22 February and 13 June 2011 earthquakes, then adjusting the parameters to give the best fit with the observed slope movement and cracking. These best-fit parameters were then used in the model to explore the susceptibility of each of the newly identified potential source areas to different levels of ground shaking.

The findings of the dynamic analyses suggest that large cliff collapses are only likely to be triggered under significant earthquake shaking.

Failure volumes and triggering frequencies

For the two assessed landslide hazards, the likely volume of material mobilised during a slope failure event and the frequency of the slope failure triggering event are both uncertain.

The volumes of material involved in, and the frequency of, slope failure events from the newly identified sources have been assessed. Both are highly uncertain; the frequency particularly so. On the one hand the slopes have survived some substantial aftershocks and two substantial rainfall episodes since the 22 February 2011 earthquake without major failure. On the other hand:

a. the strength of the slopes has been weakened by cracking; and in particular

b. the cracking has made the slopes more vulnerable to water ingress, which would be expected to weaken them further (possibly critically so in a severe weather episode).

The risk assessment carried out in the earlier cliff collapse study was modified to include an assessment of the risk from: 1) earth/debris flows; and 2) large cliff collapses (larger than those assessed previously by Massey et al., 2012a), using a range of frequency and landslide volume parameters to reflect the associated uncertainties. The overall annual individual fatality risk for dwelling occupants from the combined landslide hazards has been assessed.

The key findings of the risk assessment are that the newly identified hazards increase the risk level at dwellings already identified (Massey et al., 2012a) as being at high levels of risk.

ES 3 REPORT CONCLUSIONS

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

ES3.1 Hazard

1. There is the potential for large cliff collapses, larger than those assessed by Massey et al., 2012a) and earth/debris flows to occur at the site:
    a. at the west end of the site, there is potential for cliff collapses of around 1,000–2,500 m^3, largely of rock (corresponding to assessed source area 1);
    b. on the eastern slope, there is the potential for volumes ranging up to several hundreds of cubic metres of earth/debris flows of mixed soil and rock (corresponding to assessed source area 2); and
    c. much of the rock-slope face appears unstable and rocks fall from the slope with no apparent trigger, indicating that parts of the slope face are only marginally stable to unstable, with factors of safety much less than those assessed for the deep-seated failures a) and b) above.
2. The most likely triggers for these newly identified landslide sources are earthquake ground shaking for the cliff collapses and prolonged heavy rainfall for the earth/debris flows.
3. The frequency of landslide events from these sources is difficult to estimate and could be anything from once in a few tens to once in many hundreds of years.

ES3.2 Risk

The results from the risk assessment, taking into account the debris avalanche hazard (at the west of the site, source area 1) and the earth/debris flow hazard (at the east of the site, source area 2), have only increased the level of risk at those dwellings that were already within the original cliff-collapse risk areas, presented in Massey et al. (2012a). No additional dwellings are within the revised risk zones.

ES3.3 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.
    c. There is currently no precedent data for rates of change of groundwater or water content of loess to provide reliable alert criteria.
2. It should be noted that slope material strengths, and thus factors of safety, may be expected to deteriorate with time, weathering and any further earthquakes.

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 will be regarded as tolerable.
2. Decide how Council will manage risk on land 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);
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 not recommended as it 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:

a. 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, the damaged storm water systems at the crest of the assessed site, and should if possible be relocated away from this area; and

b. Control surface water seepage by filling the accessible cracks on the slope and providing an impermeable surface cover to minimise water ingress. However, it is not thought that such works alone are sufficient to reduce the risk.

ES4.3 Long-term actions

ES4.3.1 Engineering measures

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;

d. Withdrawal and rezoning of the land for non-residential use; or

e. Any proposed engineering works would require a detailed assessment and 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. It should be noted that the uncertainty relating to the failure mechanisms at source area 1 requires further investigation before engineering solutions can be designed.

ES4.3.2 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 changes in ground conditions; or

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

1.0 INTRODUCTION

This report combines recent field information collected from the Cliff Street 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. The results in this report supersede those in an earlier cliff collapse study (Massey et al., 2012a).

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.

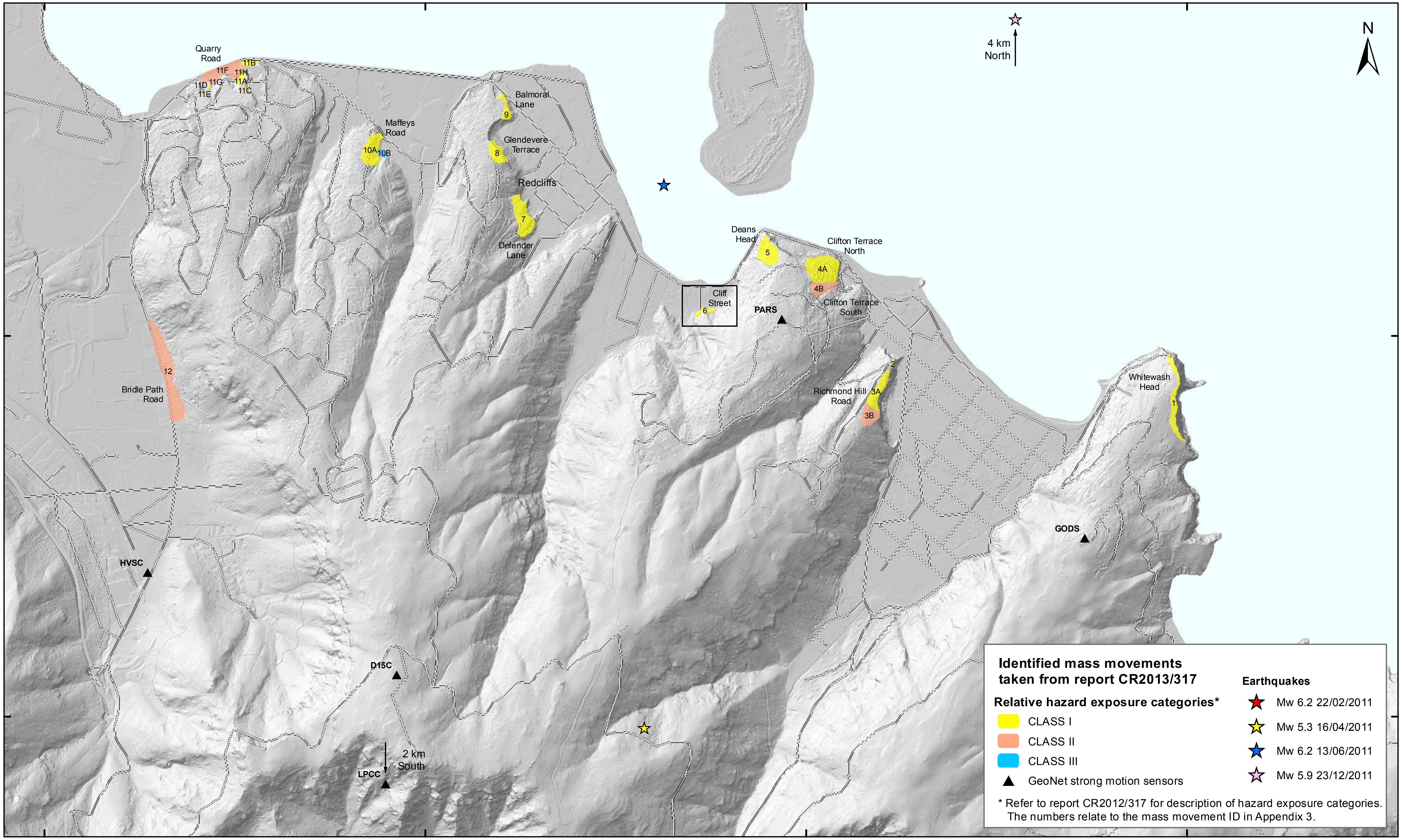
Christchurch City Council contracted GNS Science to carry out detailed investigations of the identified areas of mass movement, in order to assess the nature of the hazard, the frequency of the hazard occurring, and whether the hazard could pose a risk to life, a risk to existing dwellings and/or a risk 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, that were initiated by the 2010/11 Canterbury earthquakes. This is to assist Christchurch City Council land-use and infrastructure planning and management in the areas, as well as to establish procedures to manage on-going monitoring and investigation of the hazards and for civil defence emergency management procedures.

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 interpreted boundaries of these areas (as understood at the time of reporting); and 3) an initial "hazard-exposure" assessment (Table 1) to prioritise the areas with regards to future investigations and what type of investigations could be appropriate. Stages 2 and 3 comprise detailed assessments of individual mass movements in order of decreasing 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 (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 Stage 2, which includes this Stage 3 report on the Cliff Street Class I mass movement. Mass movements assessed as being 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.

The Stage 1 report recommended that mass movements in the Class I relative hazard-exposure category should be given a 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

GNS SCIENCE TE PŪ AO

LOCATION MAP

Port Hills Christchurch

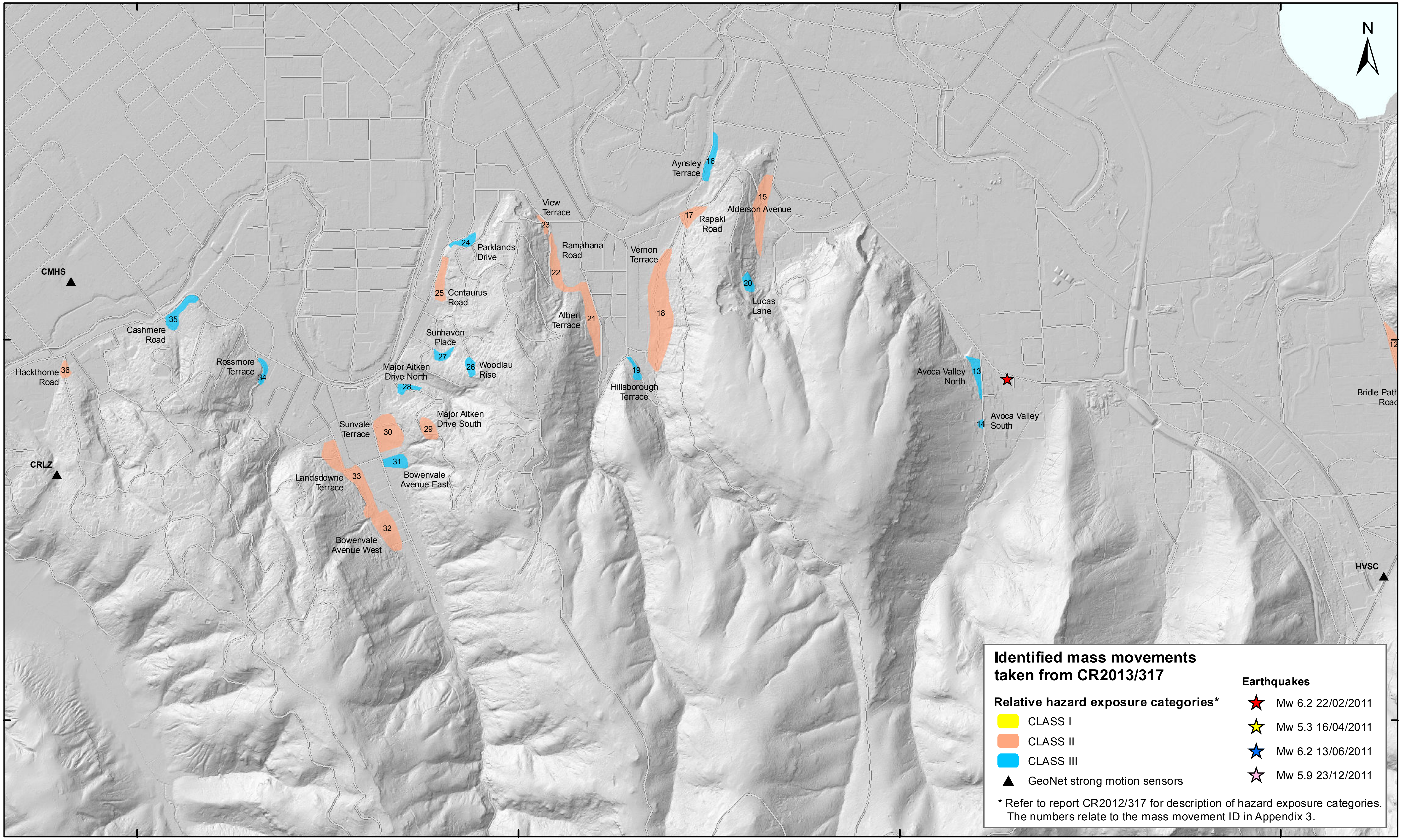
FIGURE 1

Map 1

FINAL

REPORT: CR2014/73

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

GNS SCIENCE TE PŪ AO

LOCATION MAP

Port Hills Christchurch

FIGURE 1

Map 2

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Table 1 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 mass movement are inferred by adding together the mapped crack apertures (openings) along cross-sections through the mass movement. 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 mass movements has been appropriately assessed as "Class II". It is likely that many of the 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 CLIFF STREET MASS MOVEMENT

The Cliff Street mass movement area is shown in Figures 1 and 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 Cliff Street Class I mass movement. The map in Figure 2 outlines the assessed potential landslide source areas within the assessment area, while Figure 3 shows the approximate boundary of the assessment area.

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 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

MASS MOVEMENT LOCATION MAP

Cliff Street
Christchurch

FIGURE 2

FINAL

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1.3 PREVIOUS WORK AT THE CLIFF STREET SITE

Following the 22 February 2011 earthquakes, significant localised cracking was noted on the slope surface within the Cliff Street assessment area (Figure 3). Previous investigations of the site comprised:

1. The risk to life of people in dwellings at the cliff crest and bottom 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. (2012a).
2. The Christchurch City Council engaged URS New Zealand Ltd. (hereafter referred to as URS Ltd.), to provide further geotechnical data to allow GNS Science to fully assess any potential life risks and lifeline impacts from mass movement at Cliff Street. The URS Ltd. work commenced in mid-August 2013 and results were delivered to GNS Science in October–November 2013.
3. The purpose of the URS Ltd. investigation works comprised: 1) engineering geological and geomorphological mapping of the site at a scale of 1:1,000; 2) construction of two geological cross-sections through the site area at a scale of 1:500; 3) interpretation of aerial photographs from 1946–2011; and 4) assessment of available LiDAR data for the site and the construction of a digital terrain model. Other investigations included review and assessment of movement monitored since 6 June 2013 (by M. Yetton at the request of Christchurch City Council).

At the time of this report, no sub-surface investigations have been carried out at the site.

Figure 3 Aerial view of the Cliff Street mass movement study area number 6 (within the yellow dashed line), looking south.

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 assessed landslide hazards, within the shown assessment area in Figure 2.
2. 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 results contained in this report supersede the preliminary results contained in the Working Note 2013/08 (Massey and Della Pasqua, 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. This was followed by analysis of the results.

1.6.1 Engineering geology assessment methodology

The findings presented in this report are based on engineering geological models of the site developed by URS New Zealand Ltd. (Yetton, 2014), in consultation with GNS Science.

In addition to the work carried out by URS Ltd., GNS Science has carried out:

- Field mapping of the site;
- Laboratory testing (ring-shear) of clay material found in discontinuities present on the site; and
- Field-based strength testing (point load testing) of the main rock type (tuff) found at the site.

1.6.2 Hazard assessment methodology

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:

1. Earth/debris flow source area at the east of the site; and
2. Large deep-seated cliff collapse source area at the west of the site.

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) represent the potential locations of the source areas for the assessed hazards that could occur in the assessment area. These are additional to the smaller volume cliff collapses previously assessed by Massey et al. (2012a). For these two additional landslide hazards:

- 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 from the earth/debris flows and cliff collapses assessed in this report. Cliff-collapse hazards (comprising debris avalanches and cliff-top recession) within the assessment area were previously assessed by Massey et al. (2012a), and these results are combined with the results in this report, to present combined risk estimates for all of the assessed landslide hazards.

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:

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

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.

1.6.4.1 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.

1.6.4.2 Scenarios assessed

Three scenarios for each source area were assessed in the risk assessment. This was done to take into account the main uncertainty relating to estimated source volumes used in the risk assessment. The assessed volumes are listed in Table 3. Uncertainties relating to the other input parameters used in the risk assessment and their impacts on the estimated risk are discussed in Section 6.

Table 2 Scenarios modelled in the risk assessment.

Scenario	Cliff collapse (source area 1)	Earth/debris flow (source area 2)
A	Upper estimate of source volume	Upper estimate of source volume
B	Middle estimate of source volume	Middle estimate of source volume
C	Lower estimate of source volume	Lower estimate of source volume

2.0 DATA USED

The data and the sources of the data used in this report are listed in Table 3.

Table 3 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 (10 cm 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, 1975, 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 (2003)	Digital Elevation Model derived from LiDAR survey carried out in 2003; resampled to a 1 m ground resolution.	AAM Hatch	2003	Used as the pre-22 February 2011 ground model.
LiDAR digital elevation model (2011a)	Digital Elevation Model derived from post-22 February 2011 earthquake LiDAR survey; re-sampled to 1 m ground resolution.	NZ Aerial Mapping	8–10 March 2011	To generate change models (between the 2003 and 2011a surveys) to determine the locations, extents and volumes of material.
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.

Data	Description	Data source	Date	Use in this report
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.
Synthetic earthquake time/ accelerations	Earthquake time acceleration history's 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.
Downhole shear wave surveys carried out at different sites in the Port Hills	Downhole shear wave velocity surveys carried out in the Aurecon 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	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.
Port Hills Land Damage Studies Cliff Street Field Investigations	Field mapping, aerial photograph interpretation and assessment of engineering geological slope hazards present at the site	URS Ltd.	Final report April 2014	Used as the basis for the hazard and risk assessments.

3.0 SITE ASSESSMENT RESULTS

The site assessment and engineering geological conceptual models for the site were developed by URS Ltd. in consultation with GNS Science. The URS Ltd. engineering geological map, cross-sections and conceptual model for the site are presented in Figure 4–Figure 7. These figures are reproduced from Yetton (2014), and the main results are summarised below.

3.1 SITE HISTORY

3.1.1 Before the 2010/11 Canterbury earthquakes

- The Cliff Street mass movement is part of an abandoned pre-historic (pre-1840 AD) coastal cliff. Erosion by the sea at the base of the cliff probably ceased about 3,500–3,700 years ago (based on data in McFadgen and Goff, 2005). Parts of the slope have been modified by excavations for construction of access to residential properties (Nos. 11A and 12 Cliff Street) and a new subdivision (Emily Heights) (Figure 4).
- There has been no evidence of large slope deformation recorded at the site since European settlement began (c. 1840 AD).
- There is geomorphic expression of a possible large relict landslide scar apparent in the earliest available aerial photographs (1946), shown in Figure 4.
- Several small earth/debris (loess) flow failures are apparent in the 1946 aerial photographs.
- There is some evidence of local slope cracking and compression in the middle slope that occurred sometime in the period after landscaping work; this is apparent in the 1973 aerial photographs.
- In January 2003 (Google Earth Imagery) Emily Heights road is only half its current width. The western flank of the slope below 12 Cliff Street (and above 24 Cliff Street) appears natural and not yet modified for road construction.
- In December 2004 (Google Earth Imagery) construction of Emily Heights road appears to be underway, where the natural slope appears to have been modified (cut) for road construction. Construction of retaining walls beneath 12 Cliff Street are underway.

3.1.2 During the 2010/11 Canterbury earthquakes

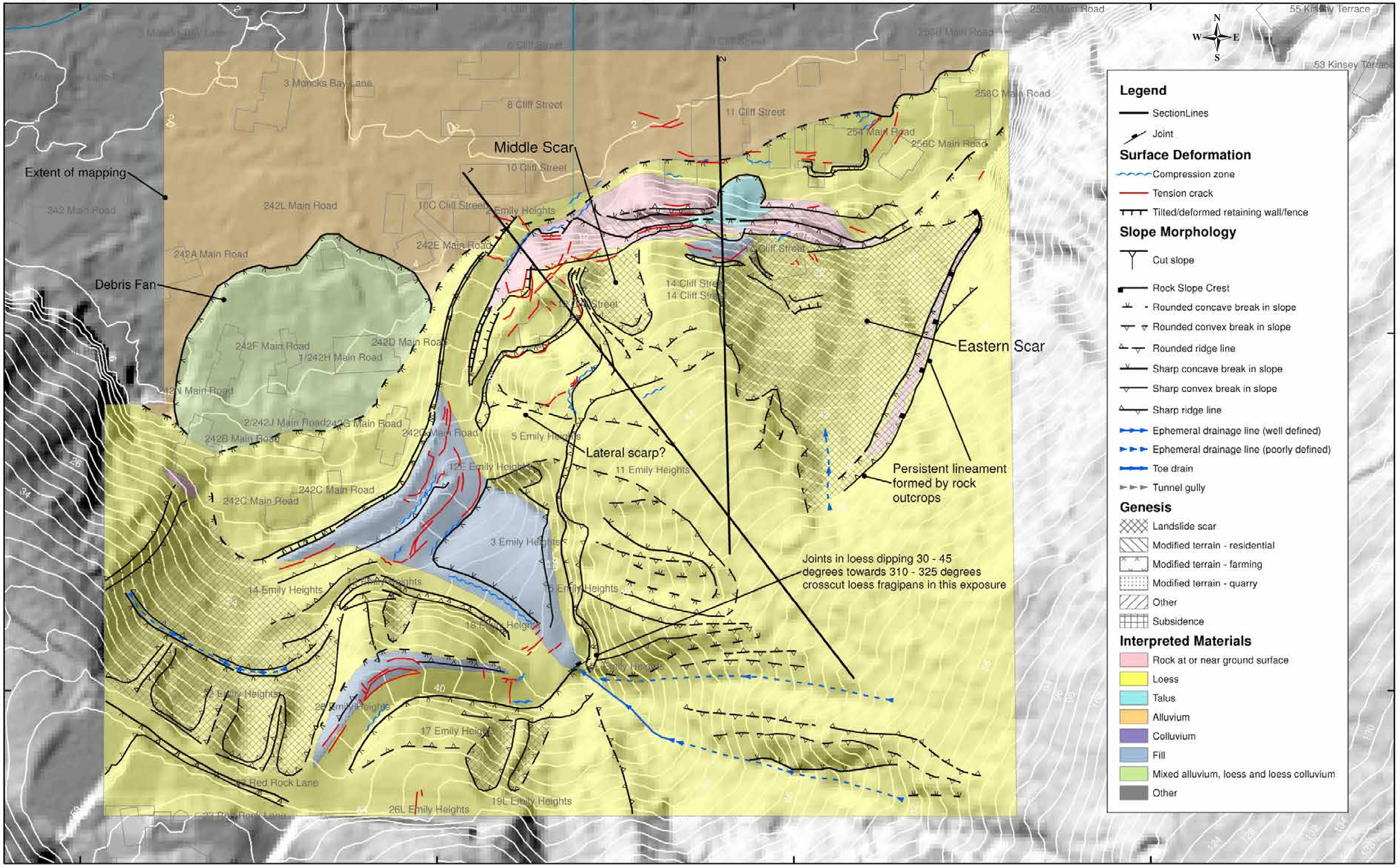
- 4 September 2010 (Darfield) earthquake – No cracks reported or measured.
- 22 February 2011 earthquakes – Cracks in loess and rock at the cliff crest were mainly generated during the 22 February 2011 earthquakes. The total displacement across measured cracks (including horizontal and vertical components) behind the cliff top is estimated to be 60–100 mm (horizontal) and 50–260 mm (vertical) along cross-sections 1 and 2 (Yetton, 2014) (Table 5). However, the total displacements in cliff crest areas in response to the 22 February 2011 earthquakes are unknown due to a lack of cadastral survey stations in the area. The cut slope in the parking area for 12 Cliff Street has a prominent overhang along a discontinuity suggesting partial block sliding over a weak zone. Total displacement of parts of this cut slope were about

70 mm (measured by GNS Science post-23 December 2011). However, it is not known whether these displacements were only in response to the 22 February 2011 earthquakes, or include displacements in subsequent earthquakes.

- 16 April 2011 earthquake – No cracking or slope displacement was reported to GNS Science.
- 13 June 2011 earthquakes – Minor movement is likely to have occurred due to the 13 June 2011 earthquakes (and possibly the 23 December 2011 earthquakes) but displacements were not measured.

3.1.3 After the 2010/11 Canterbury earthquakes

- The loess and rock slopes within the Cliff Street mass assessment area are currently cracked with many cracks being open to inflow of surface water. Seepage outflow points have been identified by URS Ltd. and GNS Science on the cliff face in the car port for 12 Cliff Street (arrow in Figure 8).
- Falls of talus and loess in the road cut drive to 11A Cliff Street occurred during the winter of 2012, approximate accumulated volume is about 50–100 m^3.
- Slow outward movement of the cliff face (in the cut slope parking area for 12 Cliff Street only) appears to have occurred in wet weather between 5 June and 13 September 2013, as indicated by recently installed improvised crack displacement indicators (metal straps and pins referred to as "tell-tales") (Figure 8). Movement was measured along the laterally persistent, partially clay filled discontinuity, but no further opening of the cracks at the cliff crest was identified. Movement measured by the "tell-tales" were 0.01–0.02 m.
- Further falls of talus and loess from areas along the road cut drive to 11A Cliff Street occurred between 5 June and 7 August 2013. Some of this debris travelled downslope into the garden of 11 Cliff Street.
- A small earth/debris flow (less than 50 m^3) was recorded in loess from the slope adjacent to 12 Cliff Street. This was triggered by the March 2014 rainstorm.



0 5 10 20 30 40
Meters
NZGD 2000 New Zealand Transverse Mercator

The drawing is reproduced from URS (2014), Port Hills land damage studies Cliff Street field investigations. Report prepared for Christchurch City Council. (Yetton, M. 2014).

DRW:
PC

CHK:
CM

GNS SCIENCE
TE PŪ AO

URS LTD. ENGINEERING GEOLOGY MAP
Reproduced from Yetton (2014)

FIGURE 4

Cliff Street
Christchurch

FINAL

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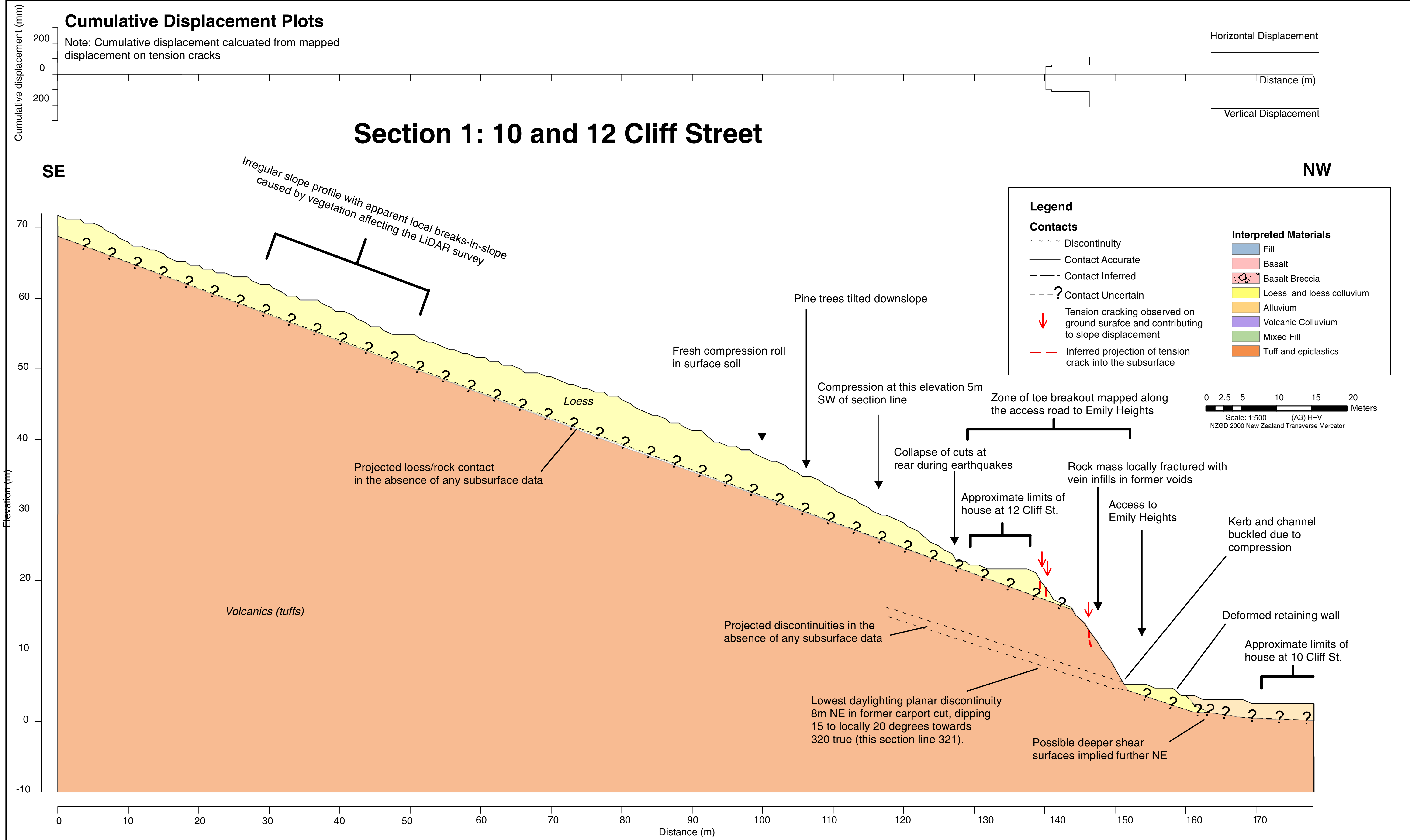
CR2014/73

DATE:
June 2014



The drawing is reproduced from URS (2014), Port Hills land damage studies Cliff Street field investigations. Report prepared for Christchurch City Council. (Yetton, M. 2014).

DRW: PC

CHK: CM

URS LTD. ENGINEERING GEOLOGY CROSS SECTION 1
Reproduced from Yetton (2014)

Cliff Street
Christchurch

FIGURE 5

FINAL

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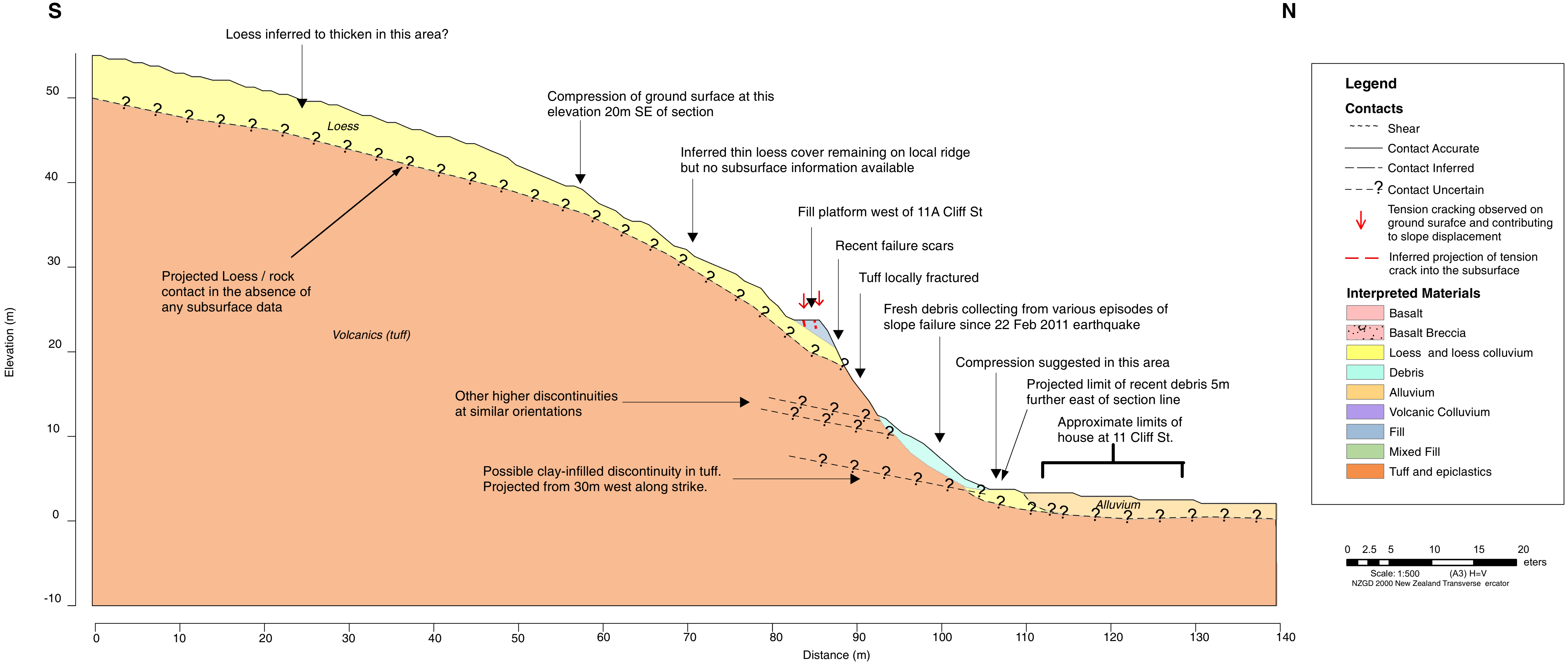
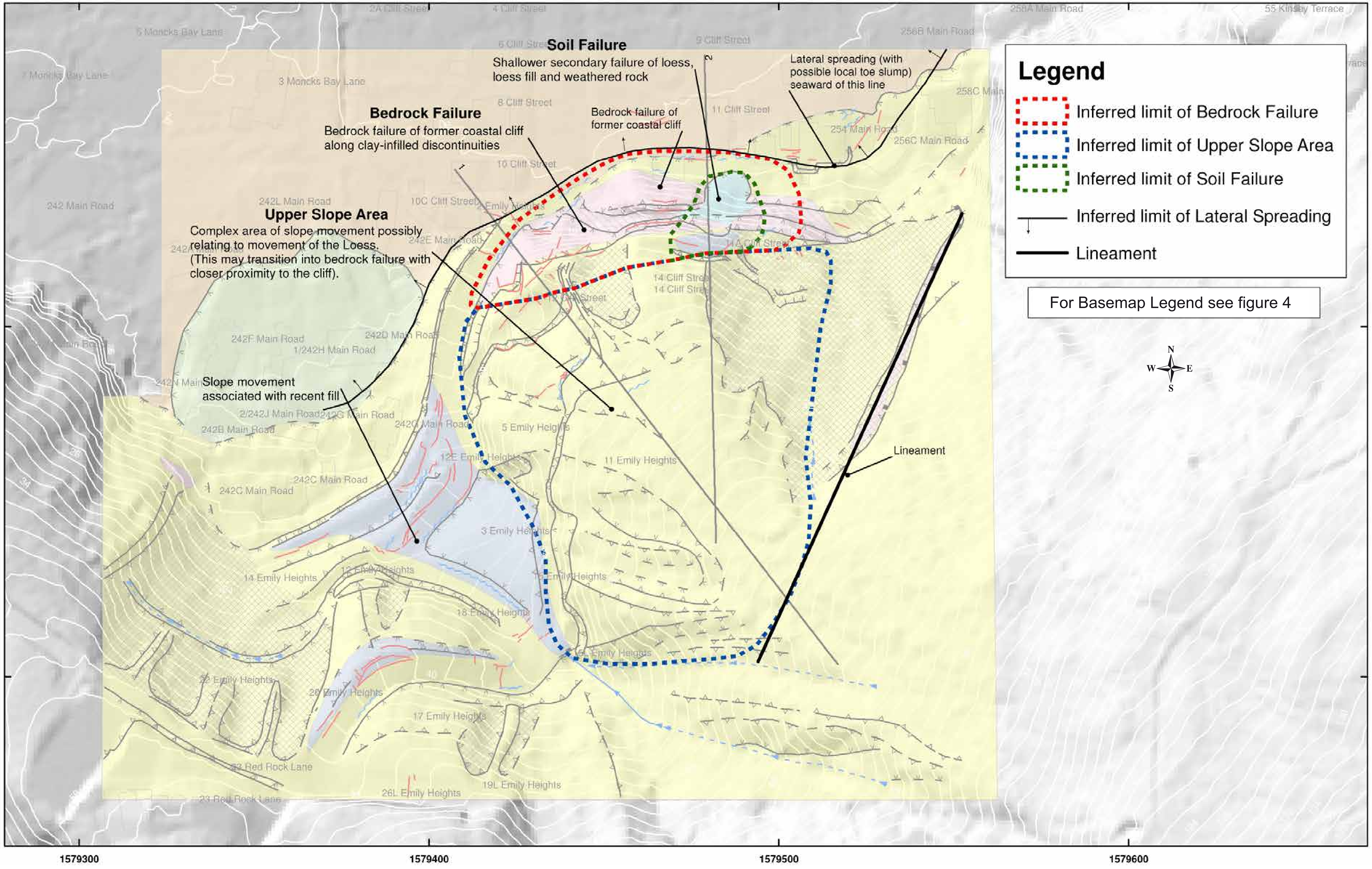
Cumulative Displacement Plots

Note: Cumulative displacement calculated from mapped displacement on tension cracks.

Section 2: 11 Cliff Street



The drawing is reproduced from URS (2014), Port Hills land damage studies Cliff Street field investigations. Report prepared for Christchurch City Council. (Yetton, M. 2014).	DRW: PC CHK: CM	GNS SCIENCE TE PŪ AO	**URS LTD. ENGINEERING CROSS SECTION 2 Reproduced from Yetton (2014)**	**FIGURE 6**
			Cliff Street Christchurch	**FINAL** REPORT: CR2014/73 DATE: June 2014



The drawing is reproduced from URS (2014), Port Hills land damage studies Cliff Street field investigations. Report prepared for Christchurch City Council. (Yetton, M. 2014).

DRW: PC

CHK: CM

URS LTD. ENGINEERING GEOLOGY MODEL
Reproduced from Yetton (2014)

Cliff Street
Christchurch

FIGURE 7

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Figure 8 Earthquake-induced cracking in volcanic tuff with post-earthquake movement shown along a zone of discontinuous soft clay within a laterally persistent sub-horizontal discontinuity (yellow arrow). However, it is not known whether these displacements were only in response to the 22 February 2011 earthquakes, or include displacements in subsequent earthquakes.

3.2 SITE INVESTIGATIONS

3.2.1 Geomorphological mapping

The results from field mapping of slope morphology, interpreted surface materials and their genesis and surface deformation mapping carried out by URS Ltd. (Yetton, 2014) are shown in Figures 4–7.

3.2.2 Subsurface trenching and drilling

No subsurface investigations have been carried out at the site.

3.2.3 Surface movement

3.2.3.1 Inferred slope displacement from crack apertures

Total cumulative displacement of the slope inferred from crack apertures along cross-sections 1 and 2, in response to the 2010/11 Canterbury earthquakes, is in the order of about 0.1–0.3 m (Table 4).

Table 4 Total cumulative displacements across crack apertures (in mm) after the 22 February and 13 June 2011 earthquakes, measured by the Port Hills Geotechnical Group (Yetton, 2014). 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 (°)	
Section 1					
1	260	140	295	62	20
Section 2					
2	50	60	78	40	30–35

The vector of displacement (direction and angle of movement from the horizontal, inferred from crack apertures) for cross-section 1 is generally steeper than the dip of the loess/colluvium and rock interface (Figure 4), suggesting displacement occurred though the underlying rock mass.

The vector of displacement for cross-section 2 is only slightly steeper than the dip of the loess/colluvium and rock interface (Figure 5), suggesting displacement of the mass occurred along this interface.

3.2.3.2 Surface movement monitoring

No surface movement monitoring is currently being carried out. “Tell-tales” have been installed across the main discontinuity exposed in the car port of 12 Cliff Street.

3.2.4 Subsurface movement

No subsurface monitoring is currently being carried out.

3.2.5 Groundwater

There is no groundwater monitoring or soil water content data available for the Cliff Street mass movement. During site visits in the winter of 2013 made by GNS Science and URS Ltd., evidence of water seepage was noted from the main defect exposed in the carport, 12 Cliff Street. This is not unexpected as there is a considerable catchment area above the site.

3.3 ENGINEERING GEOLOGICAL MODEL

3.3.1 Slope materials

Slope materials were interpreted from field mapping by Yetton (2014) (Figures 4–7). A summary of the main materials present at the site is given below.

3.3.1.1 Fill

In some areas, particularly around 11A Cliff Street the slope crest has been backfilled for construction of residential homes. The nature and extent of the fill is not known, although its approximate location is shown in Figure 4.

3.3.1.2 Loess

The loess mantling the cliff at the Cliff Street mass movement is similar to that in 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 slope creep processes and by construction activities for the residential dwellings (Figure 9).

The loess in the main zone of cracks at the cliff top (12 Cliff Street) appears to be unsaturated and relatively firm where exposed. It forms recessive slopes above the underlying volcanic breccia of approximately 30°, locally 60°. The loess is highly hygroscopic and when exposed to water it quickly disintegrates into muddy silt.

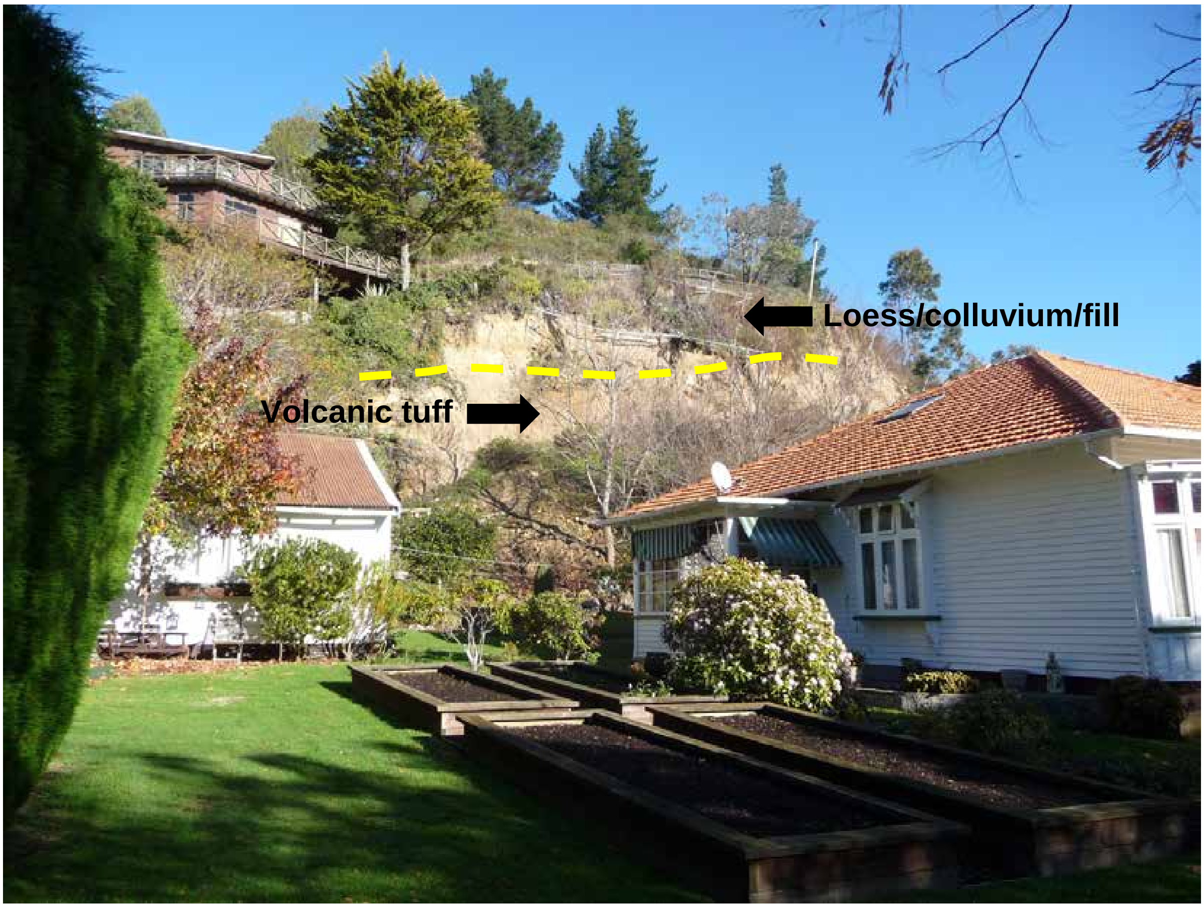


Figure 9 View to the south of the eastern end of the site. 11A Cliff Street is visible in top left of the photograph and 11 Cliff Street is in the foreground. Loess forms the upper part of the exposed (bare) slope and overlies volcanic tuff.

3.3.2 Loess colluvium

A layer, of sandy silt containing occasional small boulders and gravel with minor clay was logged in the field. Where exposed, this material is highly variable and dominated by silt with minor gravel and cobbles. The thickness of the colluvium varies but typically appears to be less 0.3 m. It is thought to represent the deposits of debris from past landslides and other erosion processes. The material derives mainly from reworked loess.

3.3.2.1 Bedrock (volcanic tuff)

The Cliff Street slope is underlain by a thick and relatively uniform massive volcanic tuff. The material is weak to moderately weak in hand specimen, and has a primary layering that makes it weaker in shear parallel to the layering. Where exposed, the layering dips at approximately 15°, locally 20°, towards the northwest (320°); at a small-scale (0.1 m) the layering undulates locally. Movement of the slope in the west of the mass movement has occurred along a number of these layers, particularly along a lower more laterally persistent defect containing discontinuous clay filling. The genesis of the defect in the rock is not known. It does not appear to be a tectonic joint, shear or fault, but rather a primary fabric relating to the emplacement of the tuff. There is no subsurface information available to describe the shape of this defect within the slope, so its shape and persistence have been inferred from rock outcrops.

As well as those discontinuities that are exposed near the base of the former coastal cliff, there is a buried zone of compression close to Cliff Street road level shown by buckled seal for the first 30–40 m northeast from Emily Heights (Figure 5). These features may relate to movement along a deeper (buried) discontinuity (Yetton, 2014).

Figure 10 View southeast of the western end of the site. Emily Heights Road is in the foreground. Note the toe break out of the rock slope failure evident along the edge of Emily Heights Road (yellow arrow). This is an extension of the main discontinuities exposed in the car port for 12 Cliff Street (Figure 3).

3.3.3 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.

3.3.3.1 Loess

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 (Carey et al., 2014).

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 11). 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, or that the outer face of the slope is naturally drier (Table 5).

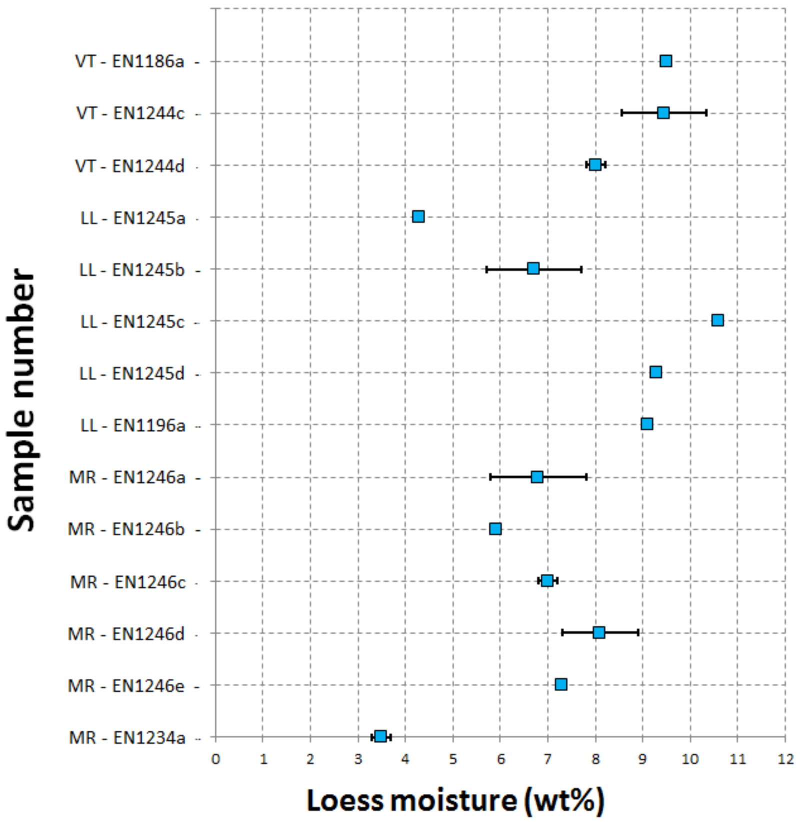


Figure 11 In-ground moisture (water, wt%) content of collected loess samples.

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 on type of direct shear equipment (Carey et al., 2014). The results are summarised in Table 5 and Table 6 and plotted in Figure 12. 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 which yield residual shear strengths of c = 0–6 kPa and ϕ = 27–37°.

The shear strength results in Table 5 are considered to be more representative of the bulk residual strength parameters for the loess slope rather than peak strength parameters.

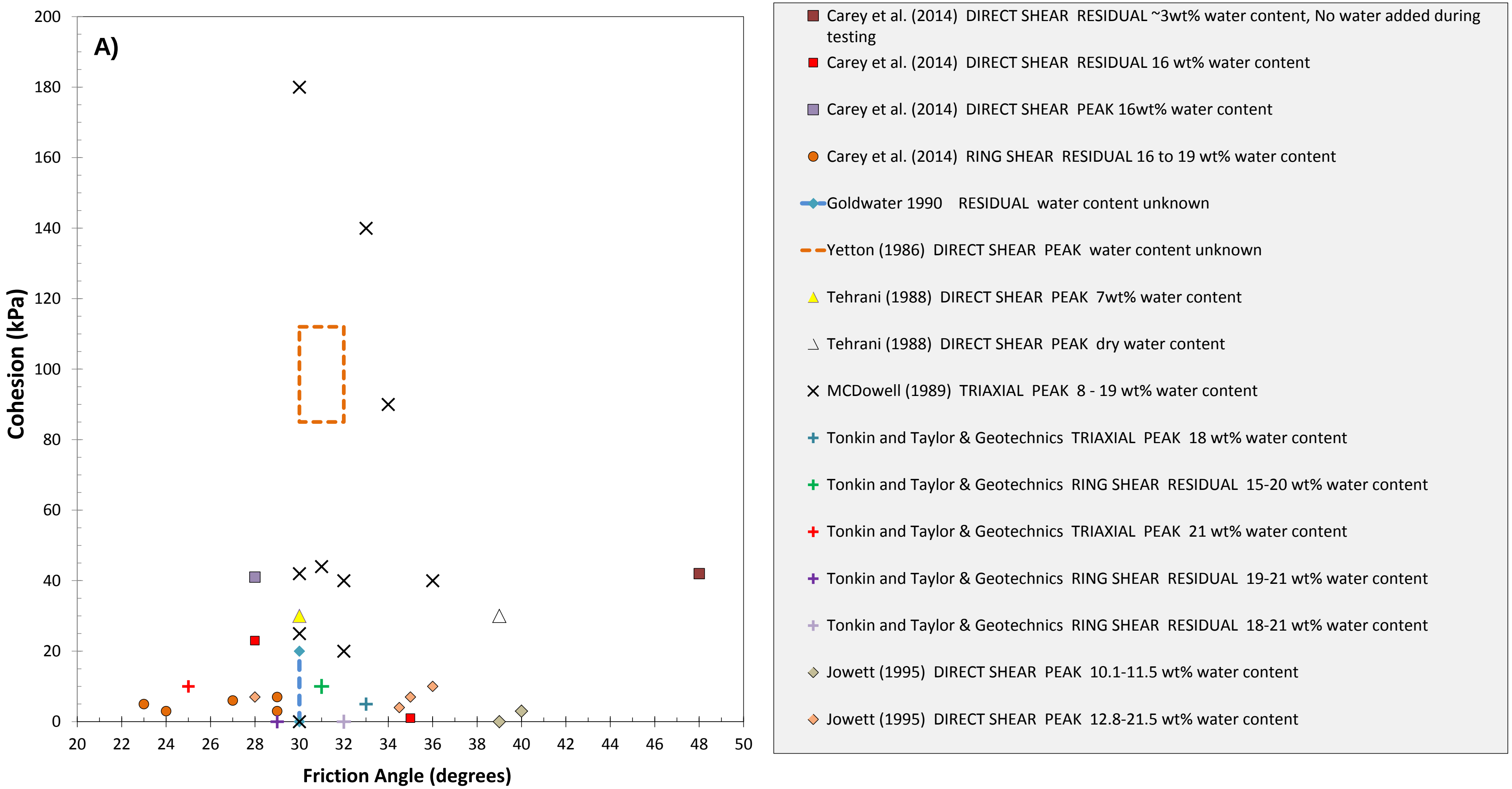
Effect of moisture content on loess shear strength

Comparison can be made with shear strength results from other published Port Hills investigations (Table 7) by plotting them alongside the results of the GNS Science testing (Figure 12).

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 12 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)
200
180
160
140
120
100
80
60
40
20
0
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

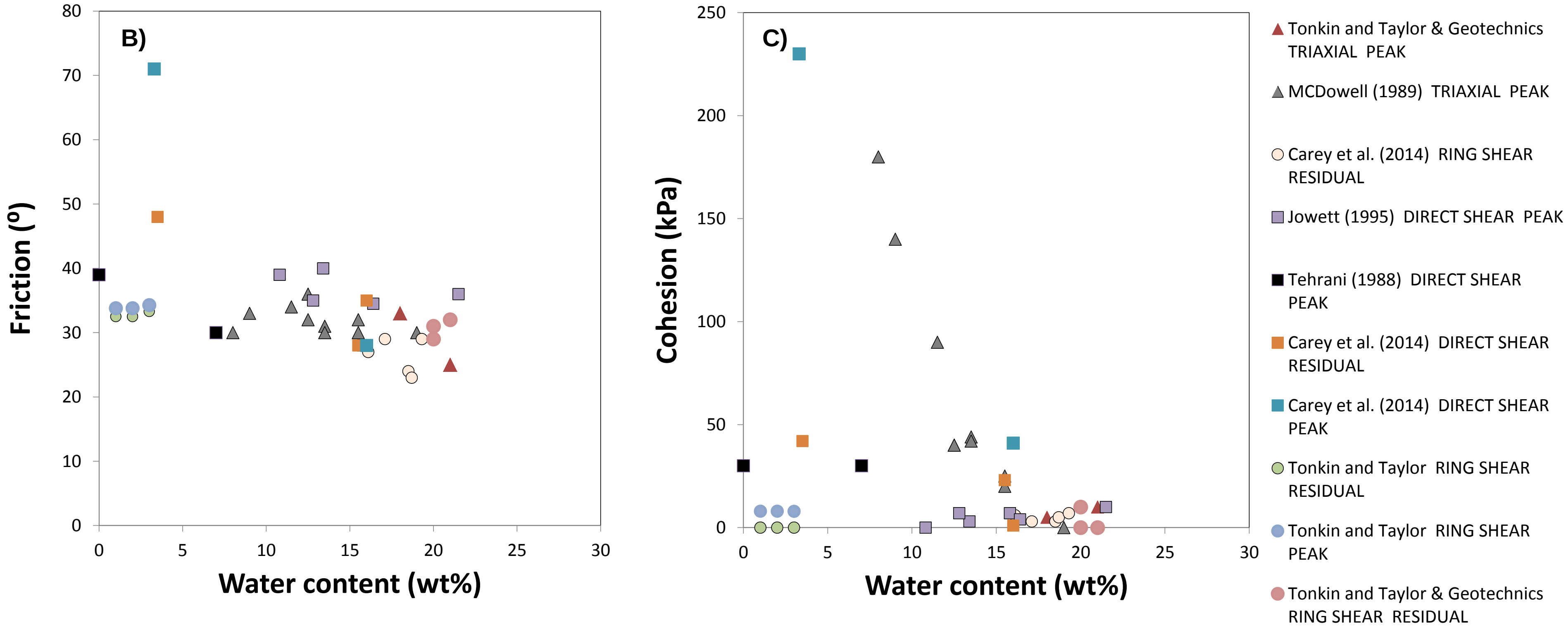


Figure 12 Loess residual shear strength results (from Table 8 and Table 9). A) Cohesion and friction laboratory results plotted for loess. B) Loess residual cohesion plotted against water content. C) Loess residual friction plotted against water content.

Table 5 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 6 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)

3.3.3.2 Loess compressive strength

A summary of unconfined compressive strength tests carried out by GNS Science on samples of loess from the Port Hills are shown in Table 7. Unconfined compressive strength tests were carried out to constrain the range in loess Young's modulus value for finite element numerical models.

Figure 13 shows the range in Young's modulus and compressive strength with sample water content. There is a trend where Young's modulus and the unconfined compressive strength decrease with increasing water content. This relationship is also shown in Figure 14, for the different sites tested.

These values of Young's modulus are comparable to tests results reported for Port Hills, Ahuriri loess by Jowett (1995) (Table 8), where test results show Young's modulus values of 44.4 MPa at 2.4 % water content.

Table 7 Unconfined compressive strength test results carried out by GNS Science (Carey et al., 2014).

Location	Lab number	Water content (%)	Dry Density (t/m^3)	Saturation ratio (%)	Compressive Strength (MPa)	Axial tangent modulus (Young's modulus) (MPa)	Remark
Vernon Terrace	EN1244a	9.5	2.03	77	0.35	26	
Vernon Terrace	EN1244c	9.0–9.9	1.74	49	0.19	7	
Vernon Terrace	EN1244d	7.9–8.1	1.55	29	0.37	30	
Lucas Lane	EN1245a	4.3			0.17	11	High clay content
Lucas Lane	EN1245b	6.2–7.2					
Lucas Lane	EN1245c	10.6	1.91	69	0.2	9	
Lucas Lane	EN1245d	9.3					
Maffeys Road	EN1246a	6.3–7.3	1.62	25	0.5	32	
Maffeys Road	EN1246b	5.9					
Maffeys Road	EN1246c	6.9–7.1	1.8	40	0.87	60	
Maffeys Road	EN1246d	7.7–8.5	1.78	40	0.71	56	
Maffeys Road	EN1246e	7.3					

Table 8 Loess Young's modulus tests results of Jowett (1995).

Location	Water content (%)	Dry density (t/m^3)	Compressive strength (MPa)	Axial tangent modulus (Young's modulus) (MPa)	Data source
Ahuriri	2.4	1.8	1.73	44.4	Jowett (1995)
Timaru	?	?	1.71	46.3	Jowett (1995)

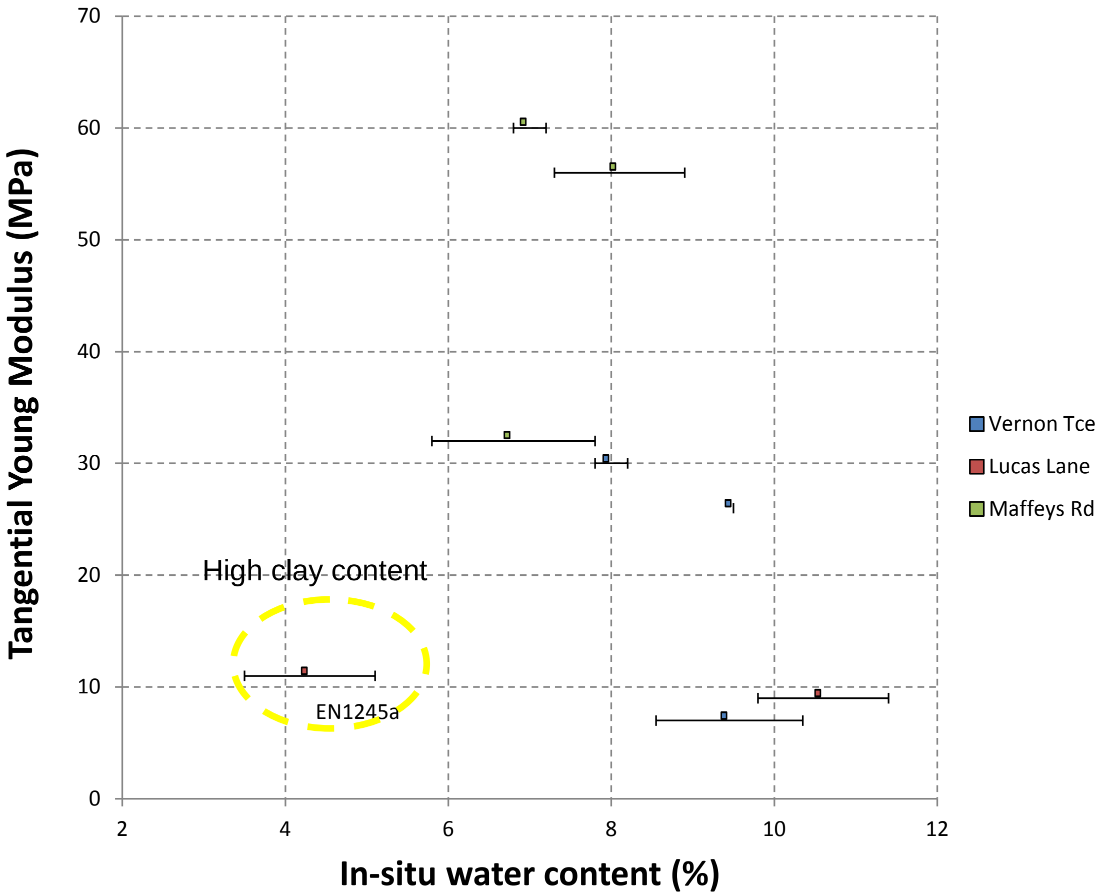


Figure 13 Loess Young's modulus versus water content (moisture content).

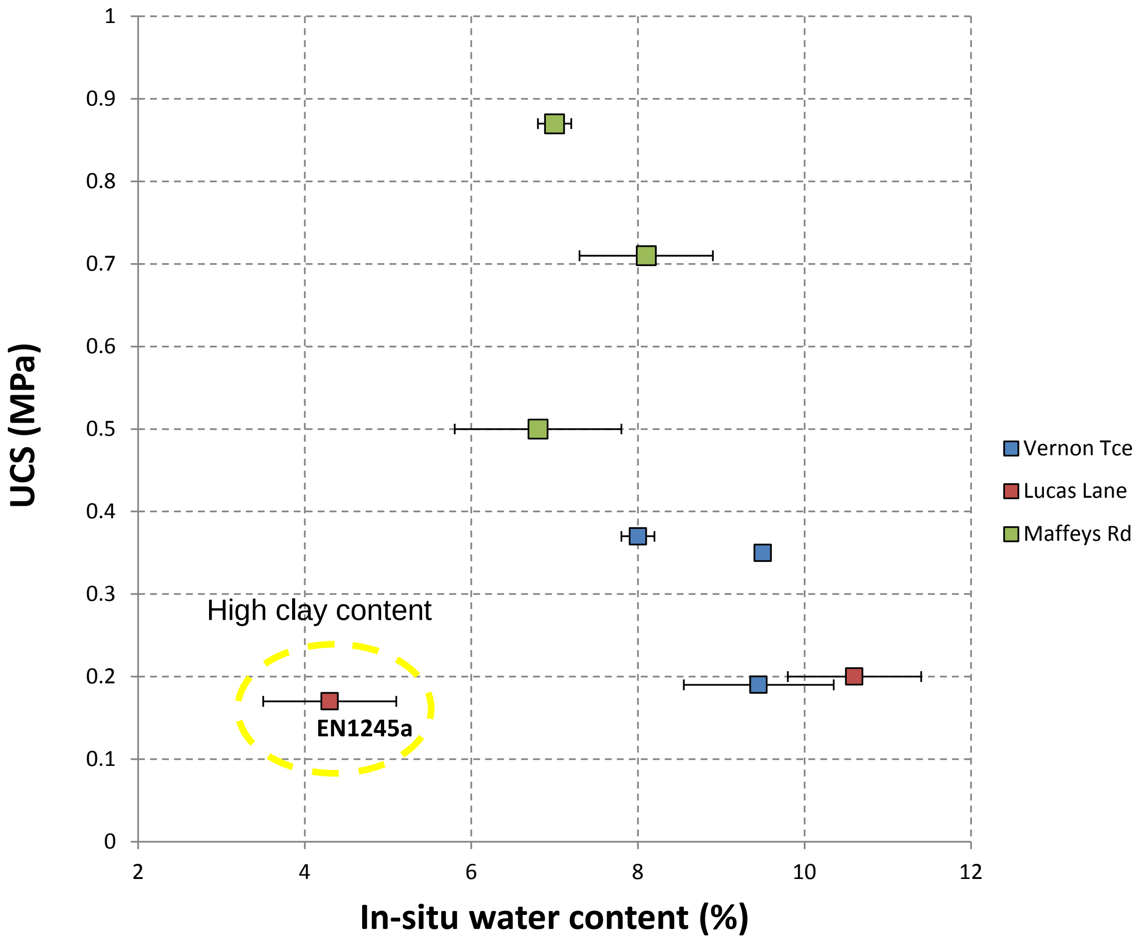


Figure 14 Loess unconfined compressive strength (UCS) versus water content.

3.3.3.3 Volcanic tuff point load strength

No core or block samples of *in situ* tuff were taken or tested in the laboratory. Point load strength testing of samples of the tuff was carried out on site by GNS Science. Twenty seven samples of the tuff were tested, adopting the procedure of Ulusay and Hudson (2007). The samples were typically "irregular lump" shaped, and tests were carried out perpendicular to any observed layering, where observed in the sample. Samples ranged in diameter from 28 to 64 mm, with a mean of 44 mm (±9 mm, at 1 standard deviation), and a mode of 42 mm. For all samples the depth (*D*) to width (*W*) ratios were all between 0.7 and 1.2 (mean of 0.9 ±0.1 at one standard deviation), and the length (L) was at least 0.5*W*.

The size corrected point load strength index $I_{S(50)}$ was obtained using the size correction factor given in Ulusay and Hudson (2007), for tests near the standard 50 mm size. The mean $I_{S(50)}$ for the tuff was 0.4 (±0.1 at one standard deviation). This is based on 23 test results and excludes the two highest and lowest values from the 27 valid tests (Figure 15).

The point load strength test results have been used to predict the likely range of the uniaxial compressive strength of the tuff. On average, uniaxial compressive strength is 20–25 times the point load strength (Ulusay and Hudson, 2007). This would give the mean uniaxial compressive strength of the tuff of between 7.7 (±2.2) and 9.6 (±2.7) MPa, errors at one standard deviation. However, this factor could be as low as 13 times the uniaxial compressive strength (Kohno and Maeda, 2011), giving a uniaxial compressive strength of 5 (±1.4) MPa.

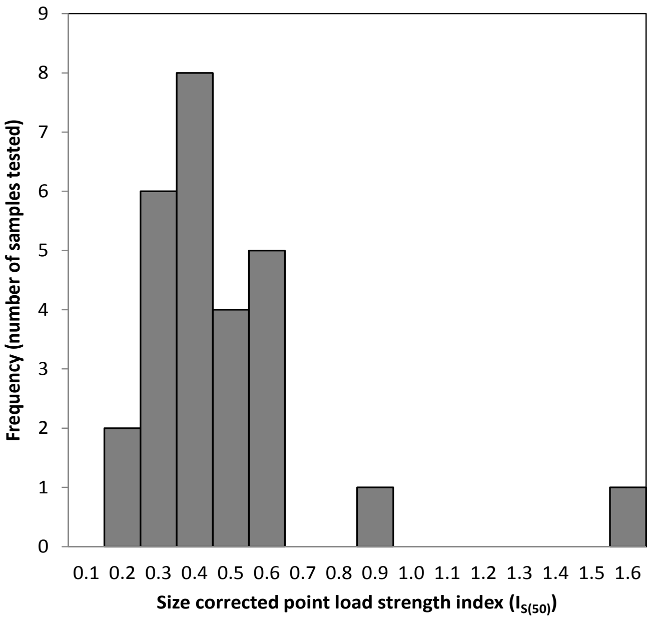


Figure 15 Size corrected point load strength testing results on samples of the Cliff Street tuff.

3.3.4 Shear strength of the clay in-filled discontinuity

Movement of the rock mass at the western end of the site during the 2010/11 Canterbury earthquakes (mainly the 22 February 2011 earthquakes) involved sliding along several pre-existing low angle (~15° dipping out of the slope) discontinuities that were partially filled with a low strength clay (up to several cm thick) (Figure 4).

The shear strength of the clay exposed in the cut-slope parking area for 12 Cliff Street was tested in-house using ring shear equipment. Test results gave residual shear strength values of friction (ϕ) of 12° (±2°) and cohesion (c) of 3 kPa (±3), errors represent one standard deviation. Based on field observations and optical microscope assessment of the clay, the clay appears to be a secondary infill. No comminuted (pulverised) grains were apparent under the microscope, which suggests that the material is not a gouge.

It should be noted that the clay material is discontinuous. The discontinuities in-filled by the clay are not smooth and where the clay is not present, there are rock-on-rock contacts in parts (the proportion of which is unknown), which would contribute to overall higher shear strength of the discontinuity.

To estimate the shear strength of the main discontinuity the criterion suggested by Barton (2008) was adopted. Where the shear strength (τ) of the discontinuity is given by:

$$\tau = \sigma_n \tan\left[JRC\, log_{10}\left(\frac{JCS}{\sigma_n}\right) + \emptyset_r\right]$$

Equation 3

Where: σ_n is the normal stress range of the discontinuity estimated as the overburden stress (50 – 400 kPa); *JRC* is the joint roughness coefficient and was estimated in the field as being between 10 and 14 (using the typical joint profiles shown in Figure 8 of Barton, 2008); and *JCS* is the joint wall compressive strength. The latter was taken from the point load test results adopting a uniaxial compressive strength of 5.5 MPa (equivalent to the mean minus one standard deviation, adopting a ratio between the point load strength and a uniaxial compressive strength of 20). Variable residual shear strengths (friction ϕ_r) of the infill material (clay) were adopted between 12° and 30°, where 12° was the residual strength derived from ring-shear testing. This was thought to be at the lower end of the range and did not take into account the observed variations of the infill.

Using Equation 3, the shear strength (cohesion and friction) of the discontinuity was estimated for the range of normal stress (50–400 kPa) at different values of joint wall unconfined compressive strength and joint roughness coefficients (Table 9).

Table 9 Range of parameters derived for the main discontinuity. JRC is Joint roughness coefficient. UCS is uniaxial compressive strength.

Infill residual shear strength ϕ_r (°)	JRC	Joint wall UCS (kPa)	Normal stress range σ_n (kPa)	Cohesion c (kPa)	Friction ϕ (°)
12	-	-		0	12
12	12–14	4,000	50–400	20–25	22–24
12	12–14	5,500	50–400	21–26	24–26

The values shown in Table 9 are uncertain, as the joint compressive strength could be lower because of surface weathering and the point load test to unconfined compressive strength ratio may be lower. Conversely, based on outcrop observations, the proportion of the defect that is filled with the clay appears to be lower in comparison to the proportion that is rock-on-rock. However, the relative proportions in the slope are unknown and cannot be readily determined by further investigation.

3.3.5 Adopted parameters for numerical models

For the purpose of stability assessment, material strength parameters were selected as follows:

3.3.5.1 Loess shear strength

The bulk value of cohesion and friction for the loess slope was assessed by numerical back-analysis of slope stability for cross-section 2 (Figure 16). The stability of the slope under static conditions is assessed by assuming the slope is drained with no permanent water table. However, the friction and cohesion parameters were varied to take into account the changing shear strength parameters with different *in situ* water contents.

Figure 16 shows an example of the back-analysed friction and cohesion values that yield different factors of safety. Factors of safety greater than 1 are shown as circles and factors of safety less than 1 are shown as crosses. Each circle and cross represents a modelled slide surface. If a slope has a static factor of safety of 1, then the slope is assessed as being unstable.

Figure 16 illustrates that the factor of safety of the slope can be highly sensitive to variations in cohesion, supporting the results from the laboratory testing. For example at a friction angle of 30°, a drop in cohesion from 10 to 5 kPa represents a decrease in factor of safety from greater than 1.3 to about 1.1.

The range of the laboratory test results (Table 5) when used in the stability assessment yield factors of safety of less than 1. These values are considered to be too low, considering that the laboratory testing was carried out under added water conditions, where the water contents of the tested samples were higher than the measured *in situ* water contents.

Typical lower estimates of the bulk shear strength parameters of loess in the Port Hills adopted by local Geotechnical consultants are friction (ϕ) = 30° and cohesion (c) = 10 kPa (Port Hills Geotechnical Group, personal communication 2012). These data are shown as a red hollow circle on Figure 16.

Although there are no direct measurements of *in situ* water contents of the loess at the Cliff Street site, the sensitivity of the loess slope to changing water contents was addressed in this study by adopting a range of strength parameters for the stability assessments (friction angle (ϕ): 25–35° and cohesion (c): 5–30 kPa).

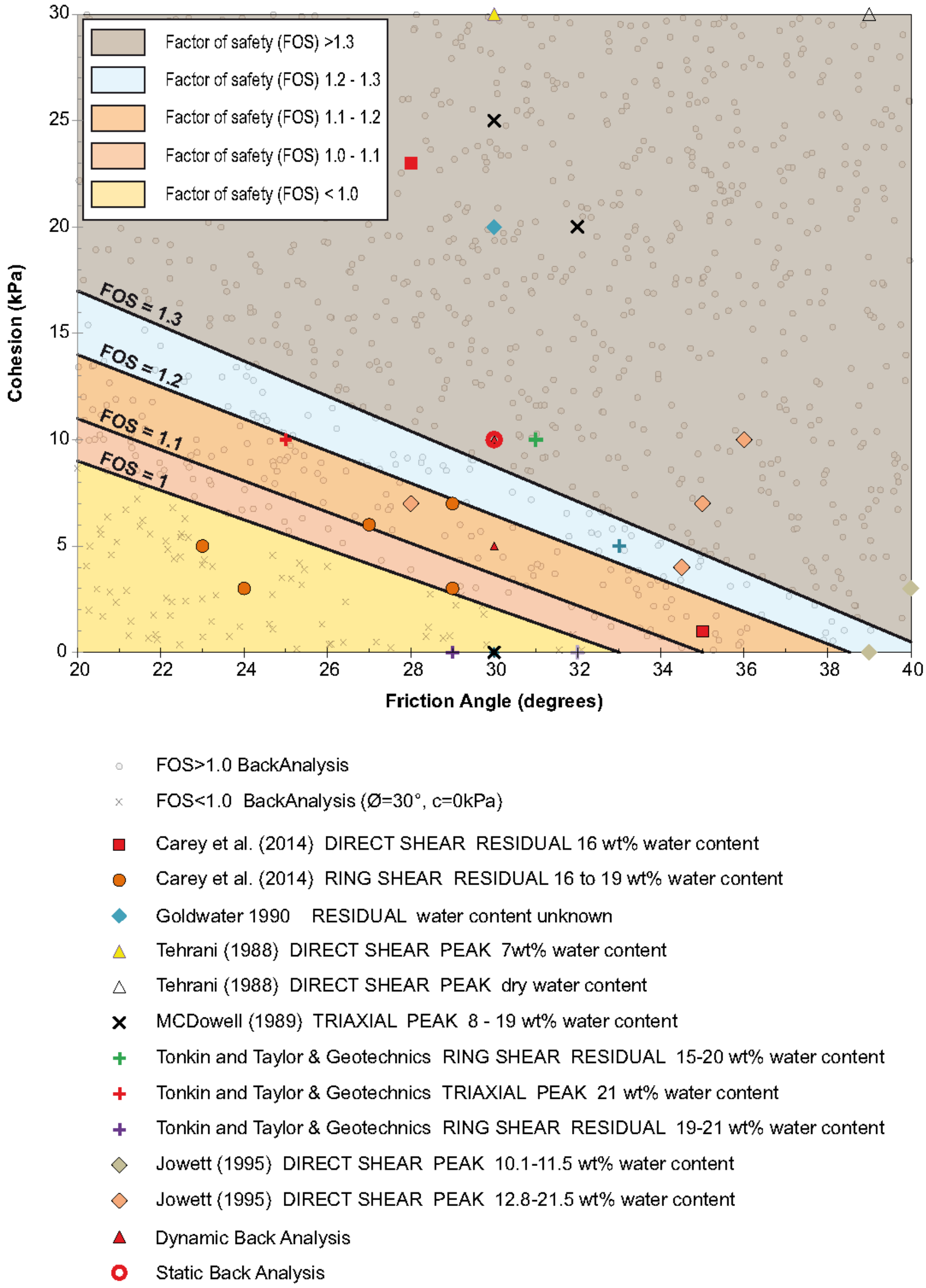


Figure 16 Sensitivity assessment of the loess slope (cross-section 1) bulk shear strength parameters of friction and cohesion.

3.3.5.2 Other loess parameters

A Young's modulus of 30 MPa was adopted based on laboratory testing results from the loess. This represents the midpoint of the range of test results (10–60 MPa) and *in situ* loess water contents of 6–10%.

A Poisson's ratio (the negative ratio of transverse to axial strain measured during compression testing) of 0.3 was adopted for numerical assessments. This was based on published values as shown in Table 10.

Table 10 Published Poisson's ratio values.

Material	Poisson's ratio	Data source	Remarks
Ma Lam Loess	0.3	Liu et al. (2013)	
Argentinean Loess	0.2	Rinaldi and Santamaria (2008)	28% water content
Argentinean Loess	0.31	Rocca et al. (2006)	
Nebraska Loess	0.35	Sharma (2011)	

3.3.5.3 Loess Bulk Shear Modulus

The *in situ* shear modulus of the materials was derived from the downhole shear-wave velocity surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) based on nearby survey available from Redcliff drillholes BH-MB-02, and results from the dynamic probing carried out by Tonkin and Taylor for the Earthquake Commission at Clifton Terrace (Tonkin and Taylor, 2012a). The results from the dynamic probing are summarised in Figure 17. The mean shear wave velocity is 306 m/s (±93 m/s at one standard deviation) and the mode is 222 m/s.

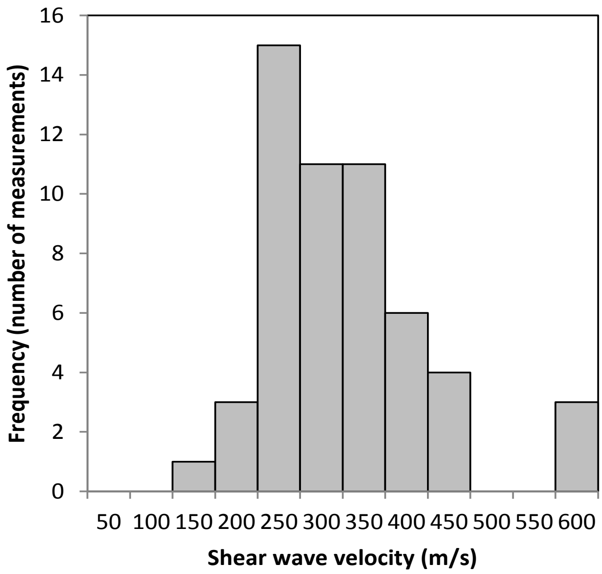


Figure 17 Loess shear wave velocity results from dynamic probing reported by Tonkin and Taylor (2012a) for the loess at Clifton Terrace.

The corresponding shear wave velocity for the loess intersection (0.6–2.7 m) measured in the drillhole BH-MB-02 was 288 m/s, which is consistent with the results from the dynamic probing.

These values are also consistent with shear wave velocity trends defined by Rinaldi et al. (2001) for other loess as a function of normal stress and moisture content (Table 12) 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 water contents of 6.4 %.

Applying the relationship for shear wave velocity:

$$G_S = \rho \; . \; V_s^2 \qquad \textbf{Equation 4}$$

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 12 Shear wave velocity profiles from Port Hills and other loess.

Material	Shear wave velocity V_S (m/s)	Data source
Port Hill loess from Redcliff borehole MB02 Inferred moisture content 6–10 wt%	288	Southern Geophysical Ltd. (2013)
Port Hills loess from Clifton Terrace dynamic probing	126–582	Tonkin and Taylor (2012a)
Argentinian Loess moisture content ~16 wt%	280–300	Rinaldi et al. (2001)
Argentinian Loess moisture content 6.4 wt%	300–320	Rinaldi et al. (2001)

3.3.5.4 Volcanic bedrock

In order to derive rock mass strength parameters for the tuff that take into account the nature of the discontinuities, the geological strength index (Hoek, 1999) was adopted to reduce the strengths derived from the point load testing of intact samples by using the Rocscience RocLab software. The geological strength index values adopted for the tuff are shown in Figure 18.

The *in situ* shear modulus of the materials was derived from the downhole (drillhole) shear-wave surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) in drillholes across the Port Hills. No shear-wave surveys were carried out at Cliff Street, and so the adopted values were taken from a compilation of shear wave records from similar materials elsewhere in the Port Hills. The adopted shear-wave velocities were measured for the epiclastic rocks in the Port Hills. The epiclastic rocks tend to be massive in outcrop, and have similar uniaxial compressive strengths (typically 4–10 MPa) to the tuff. The range of shear-wave velocities recorded in the drillholes for the epiclastic materials varies between 680 and 2,040 m/s. For the assessment, a mean shear-wave velocity of 1,000 m/s (± 300 m/s at one standard deviation) was adopted.

A summary of strength parameters adopted for the Cliff Street assessment are shown in Table 13.

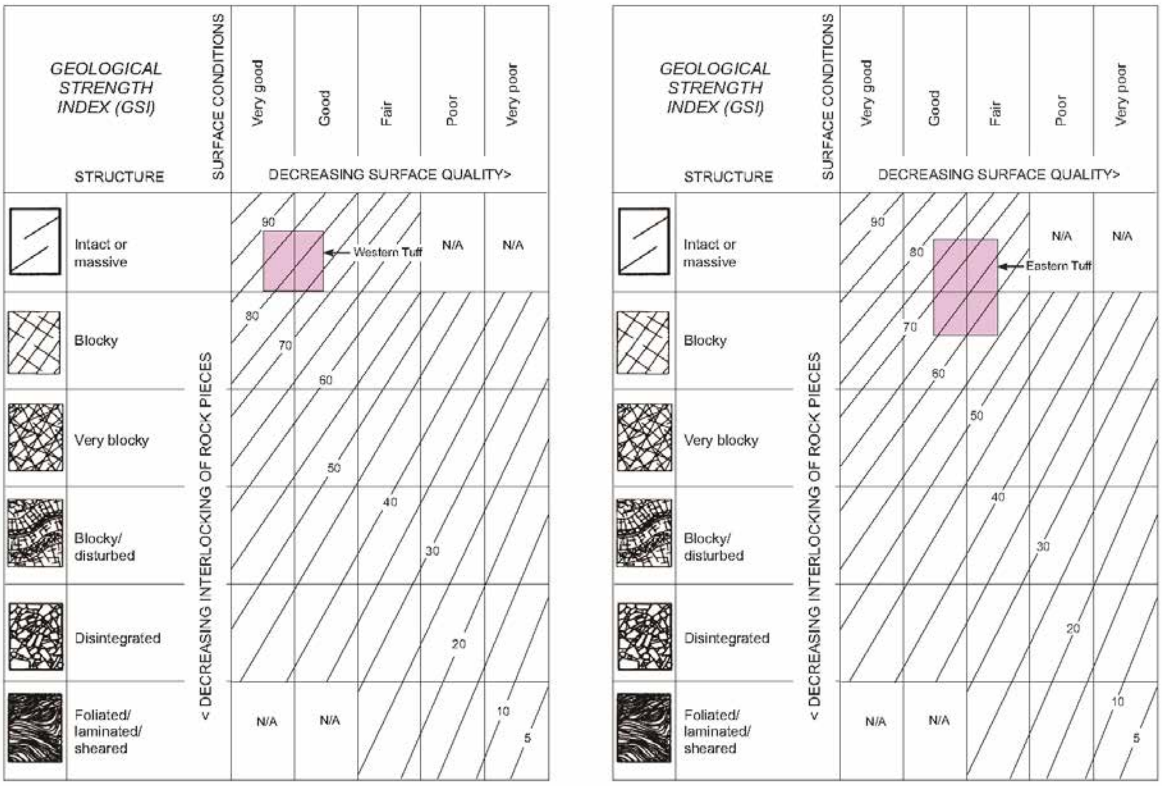


Figure 18 Geological strength index plot for tuff exposed at Cliff Street.

Table 11 Bulk (mass) geotechnical material parameters derived from testing and field surveys and used for the modelling.

Unit	Cohesion c (kPa)	Friction ϕ (°)	Tensile strength (MPa)	Unit weight (KN/m^3)	Young's modulus (MPa)	Poisson's ratio	Shear modulus G_S (MPa)
Loess	5–35	25–35	0–0.01	17	30	0.3	160–270
Tuff (rock mass) strength)	100–270	44–46	0.6	19	1,100	0.1	850–1,700
Clay in-filled discontinuity (Table 10)	20–45	24–43	-	-	-	-	-
Clay in filled discontinuity	0	12	-	-	-	-	-

3.3.6 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 corresponding 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 (mechanism 2) 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. Mechanism 1 is thought to be the most important for rock slope stability, as the open tension cracks in the overlying loess would allow water to readily infiltrate any open cracks in the underlying rock mass. It should be noted that there is currently no monitoring of groundwater levels at the site.

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 2.

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 much of 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 19).

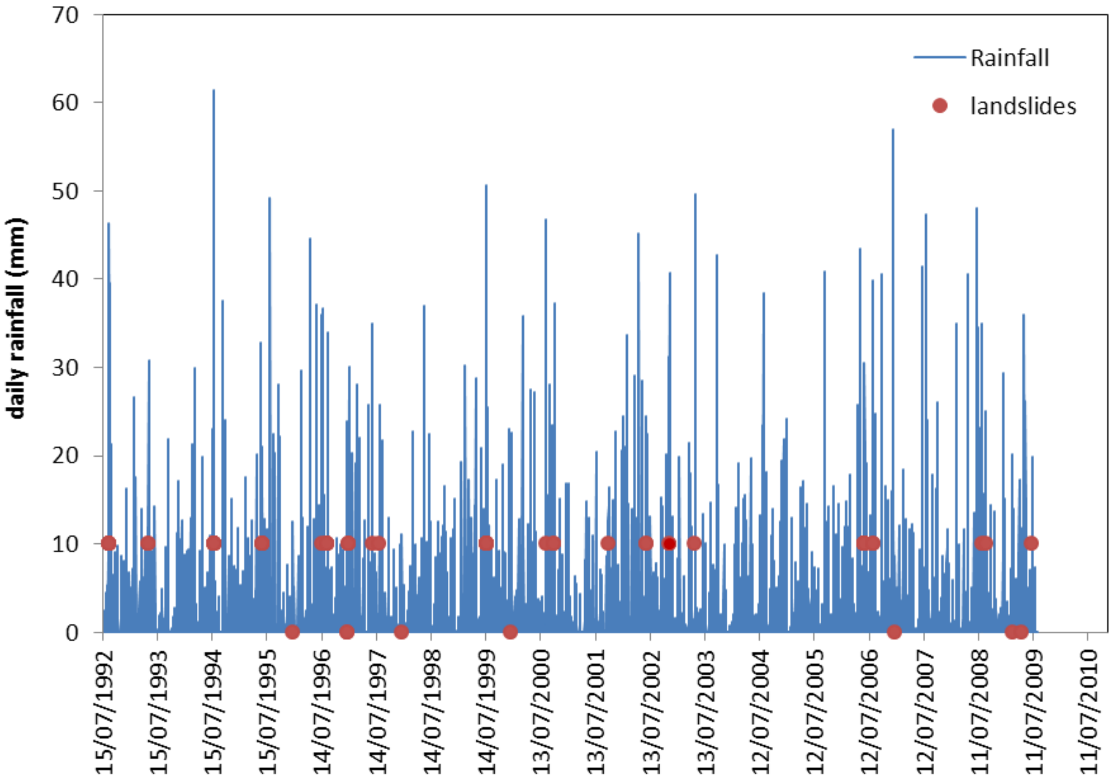


Figure 19 Daily rainfalls at Christchurch Gardens and landslides in the Port Hills. Daily rainfalls at Christchurch Gardens and landslides in the Port Hills investigated by Geotechnical Consulting Ltd, or listed by the Earthquake Commission as causing damage to homes. Landslides without rain are plotted 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 12):

- 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 4 km west of Cliff Street). 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 20, 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 12. 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 12 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)

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 0.05–0.1 (once every 10 years), it occurred at the end of summer when the ground would have had a seasonally low water content.

The maximum daily rainfall that occurred during the period of recorded movement of the western slope (5 June–13 September 2013), and when the earth/debris flow was thought to have occurred from the eastern slope (5 June and 7 August 2013), was unexceptional. However, the cumulative rainfall for these periods was 142 mm and 108 mm respectively. 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.

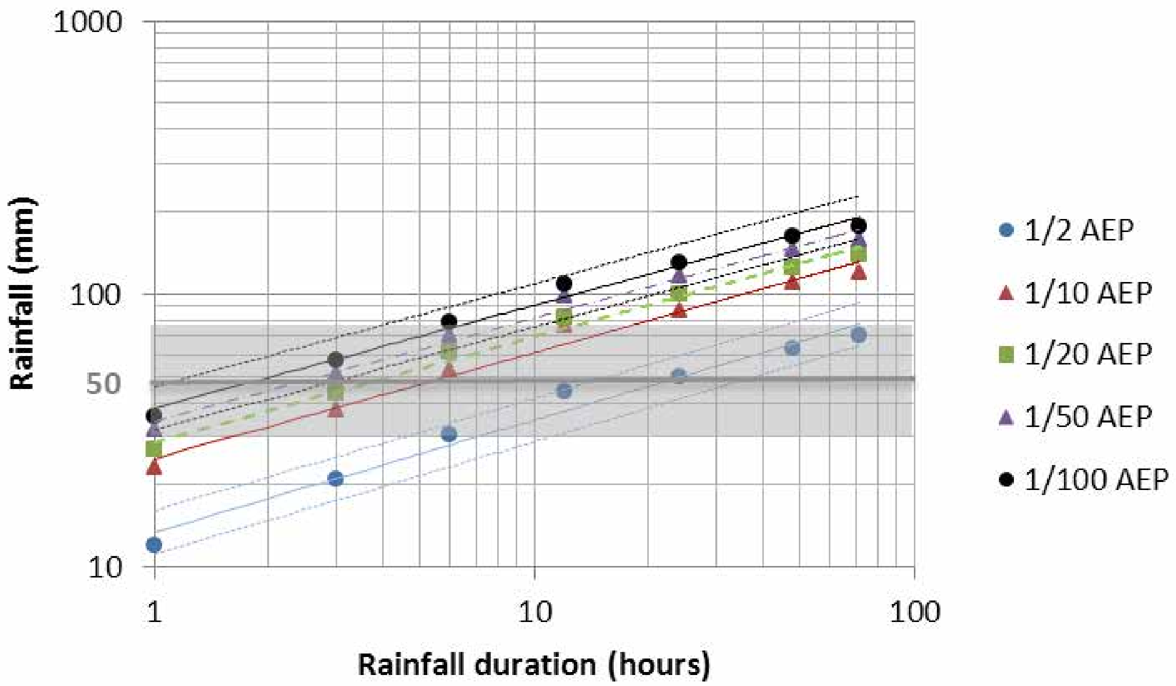


Figure 20 Rainfall depth-duration-return period relations estimated by Griffiths et al. (2009) using recorded rainfall data. Error limits of 20% are shown by dotted lines for the 1/2 and 1/100 AEP curves. Shaded area covers the range of 30–75 mm of rainfall over which the expected number of soil landslides 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

The engineering geological models of the site presented by Yetton (2014) have formed the basis for numerical modelling of the stability of the slopes within the assessment area. Two main identified slope failure mechanisms have been assessed based on the field observations in Yetton (2014) and are shown on Figure 7:

- Cliff collapse (comprising many boulders) and rockfalls (comprising individual boulders) falling from the steep rock cliff and associated cliff-top recession;
- Earth/debris flows originating in loess above the cliff crest;

Relatively shallow displacement of the loess in the upper; and slumping of loess (and fill) at the slope toe have not been assessed in this report.

3.4.1.1 Static conditions

Potential failure mechanisms occurring at the Cliff Street mass movement under static conditions comprise:

- *Loess earth/debris flows:* there have been several documented loess earth/debris flows from these slopes before the 2010/11 Canterbury earthquakes (Yetton, 2014). Geomorphological evidence also suggests that earth/debris flows in loess have been frequently occurring at this site. The strength of the loess has been shown to be sensitive to changes in moisture content, and the earthquake-induced cracks in the slope caused by the 22 February and 13 June 2011 earthquakes, now allow surface water to more easily enter the soil mass, changing the water content of the loess over time. It is therefore reasonable to assume that the slopes at Cliff Street are now more susceptible (vulnerable) to future triggering events such as rain, potentially resulting in larger earth/debris flows (volumes >100 m^3) moving down the slope with longer runout of debris.
- *Rock slope failure:* movement of the cracked and dilated rock slope along the persistent clay in-filled discontinuity at the western end of the site was recorded by Yetton (2014) during the winter of 2013. However, it was noted that the cracks at the cliff crest (above the area of movement) showed no further dilation. These measurements and observations suggest that parts of the rock slope could displace during periods of rain. Two failure mechanisms have been assessed; 1) failure through the rock mass; and 2) failure along the persistent defect. It should be noted, however, that given the lack of ground investigation data (drillholes) the failure mechanism at this site is uncertain.
- *Ravelling (relatively shallow failures of rock and soil):* from the steep rock and loess slope face and crest areas involving relatively small volumes (<100 m^3) that would form rockfalls and earth/debris flows with limited runout of debris. Ravelling of rock from the cliff is an on-going process and can occur during periods of rain and at other times without an obvious trigger.

3.4.1.2 Dynamic conditions

The magnitude of any permanent earthquake-induced displacement in the slope will be dependent upon its stability as well as the magnitude and duration of the earthquake-induced peak ground accelerations. If the Cliff Street mass movement slope is subjected to peak ground accelerations similar to those recorded during the 2010/11 Canterbury earthquakes, it is likely that the magnitude of permanent slope displacement could be comparable or greater than recorded during the 2010/11 Canterbury earthquakes as the slope is now cracked and degraded.

In addition to the failures under static conditions, it is likely that any future earthquakes would trigger:

- *Small rockfalls and cliff collapses*: from the steep rock (and soil) slopes, similar to those triggered during the 22 February and 13 June 2011 earthquakes, and that these could occur randomly from anywhere along the cliff face.
- *Larger, deep-seated cliff collapses*: now that the rock slope is cracked and dilated it is possible that future earthquakes could cause a localised large failure of the rock slope at the western end of the site (source area 1). The cumulative displacement of the rock mass in this area (inferred from crack apertures) in response to the 22 February 2011 earthquakes is about 300 mm. At present, displacement is confined to several discrete defects with no indication of the disintegration of the overall rock mass. It is not known how much cumulative displacement the rock mass could undergo before failing catastrophically to form a debris avalanche.

3.4.2 Failures mechanisms adopted for modelling

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

- *Mechanism 1:* Local deep-seated failure through the volcanic tuff, of the outside edge of the rock slope, resulting in cliff collapse (debris avalanches and cliff-top recession), cross-section 1.
- *Mechanism 2:* Local deep-seated failure of the volcanic tuff along the persistent discontinuity dipping out of the slope.
- *Mechanism 3*: Local shallow failure of loess over rockhead, at the eastern part of the slope, forming a discrete landslide source area, cross-section 2.

4.0 HAZARD ASSESSMENT RESULTS

4.1 SLOPE STABILITY – STATIC CONDITIONS

Two cross-sections were assessed across the Cliff Street mass movement (cross-sections 1 and 2 in Yetton, 2014) (Figures 5 and 6). Geotechnical material strength parameters used in the modelling are listed in Table 15. A range of shear strength parameters were used for the key materials to assess the sensitivity of the slope – along a given cross-section – to failure.

4.1.1 Source area 1 (cross-section 1)

The results from the static limit equilibrium modelling and finite element modelling for cross-section 1 are shown in Table 13.

Table 13 Example results from the static limit equilibrium model (LEM) and finite element modelling (FEM) slope stability assessment of cross-section 1 (western slope).

Mechanism	Failure geometry	Minimum LEM FoS[1]	FEM SRF[2]	Main material		Slide mass depth[2] (m)	Slide mass length[2] (m)
				Friction (°)	Cohesion (kPa)		
1	Through the rock mass	3.4	3.7	44	100	11–15	16–26
2	Basal[3] discontinuity	2.4	2.1	24	20	11–15	16–26
2	Basal[3] discontinuity	1.5	1.3	12	0	11–15	16–26
3	Loess only	1.9	1.9	30	10	1–1.5	5–7

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

2 Estimated depth (perpendicular to slide surface) and length (crown to toe of failure) of failure based on the slide surface geometry.

3 Modelled as a joint in Phase2, adopting the given shear strength parameters

Limit equilibrium and finite element models were run to assess three failure mechanisms: 1) failure though the rock mass; 2) deep-seated failure of the rock slope along the basal discontinuity; and 3) failure through the overlying loess.

4.1.1.1 Rock slope failure: Mechanism 1

Models were run adopting the lower and upper strength estimates adopting the parameters in Table 13. Results show that the minimum static factor of safety based on the adopted parameters, is about 3.4 from the limit equilibrium modelling and 3.7 from the finite element modelling.

4.1.1.2 Rock slope failure: Mechanism 2

Models were run adopting the lower and upper strength estimates of the basal discontinuity adopting the parameters in Table 13. Results show that the minimum factor of safety based on the adopted parameters, is about 2.4 from the limit equilibrium modelling and 2.1 from the finite element modelling. Models were also run assuming the defect is entirely in filled with clay. These gave factors of safety of 1.5 (limit equilibrium modelling) and 0.9 (finite element modelling), where the finite element modelling results would suggest the slope should fail under static conditions, however, these results are not thought to be representative of the current slope conditions.

Sensitivity of the rock slope to changes in pore pressure acting both in tension cracks and within the basal discontinuity has been assessed, as movement of at least part of the slope was identified after 2010/11 Canterbury earthquakes during a long period of wet weather. The results from the sensitivity assessment are shown in Figure 21, for different shear strengths of the basal discontinuity (from Table 9). The modelled slide surfaces are shown on Figure 22 and were calculated when pore pressure head levels were assumed to be 0 m (i.e., the defect was assumed to be dry).

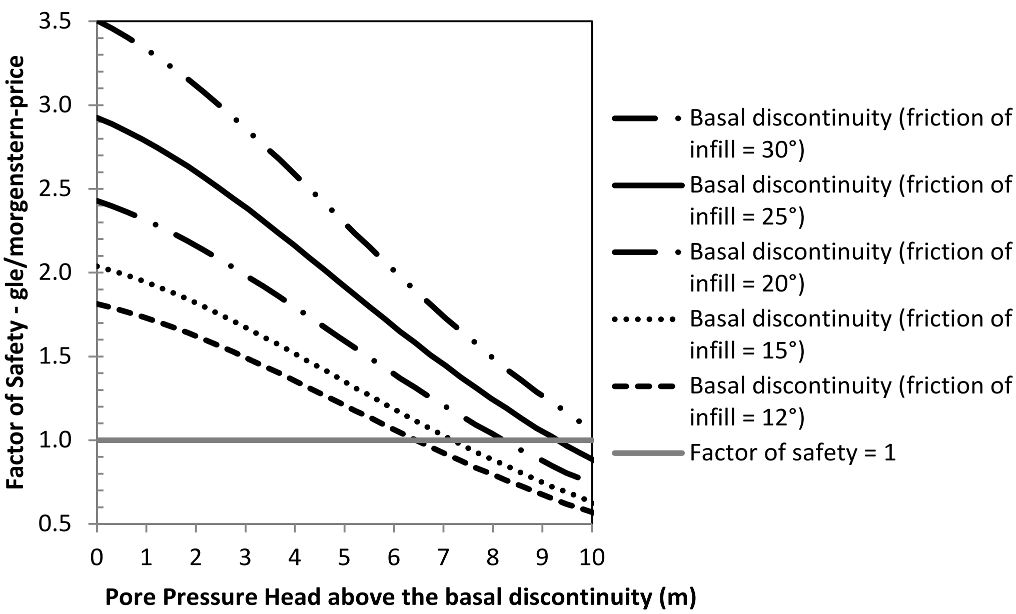


Figure 21 Sensitivity of the slope along cross-section 1 to changes in pore pressure. A pore pressure head of 0 m corresponds to the lower basal discontinuity. The pore pressure head was varied at different levels above the basal discontinuity, with the corresponding pore pressure head level being used to establish the amount of water acting in the tension crack. The shear strength of the basal discontinuity was derived using the relationship of Barton (2008) for various friction angles of the clay material infilling the discontinuity.

Results from the limit equilibrium and finite element modelling (Table 13 and Figure 22 and Figure 23) show that there is a good correlation between the shape and location of critical slide surfaces derived from the limit equilibrium model and the zones of increased shear strain (displacement) from the finite element model assessment. The static factors of safety and the shear strength reduction factors are also comparable.

The results show that, as expected, the sensitivity of the slope to changes in pore pressure is governed by the shear strength of the basal discontinuity. At the lowest modelled values of shear strength, about 6 m of pore pressure head acting along the basal discontinuity would be needed to initiate failure. At the highest values of shear strength, about 10 m of pore pressure head would be needed to initiate failure. Failure is defined as when the slope factor of safety is ≤1.0. These results are for the assumed failure geometry shown in Figure 22 and Figure 23.

Given the lack of subsurface information to constrain the presence, shape and persistence of the basal discontinuity within the slope (back from the slope face), and the pore pressure levels, it is not known whether such a failure mechanism is credible. The geometry and persistence of the discontinuity within the slope is critical to its stability.

4.1.1.3 Loess slope failure: mechanism 3

The results from the assessment, adopting loess shear strength parameters of cohesion (c) 10 kPa, and friction angle (ϕ) 30° suggest that the slope factor of safety for cross-section 1 is about 1.9. The dimensions of the simulated failure surfaces with factors of safety of less than two, are relatively small (Table 13) and confined to the outside face of the slope.

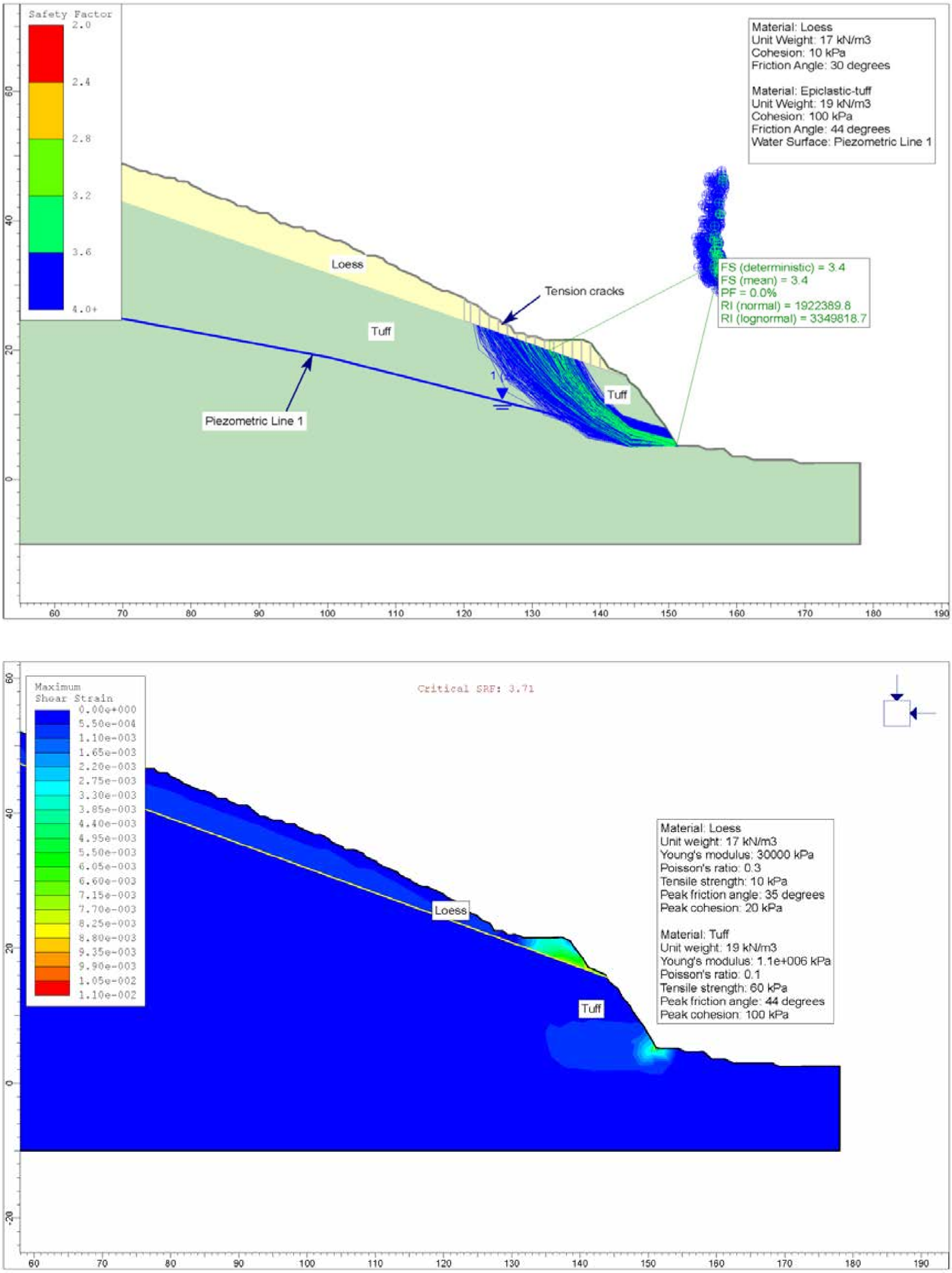


Figure 22 Example limit equilibrium and finite element modelling results for cross-section 1, mechanism 1.

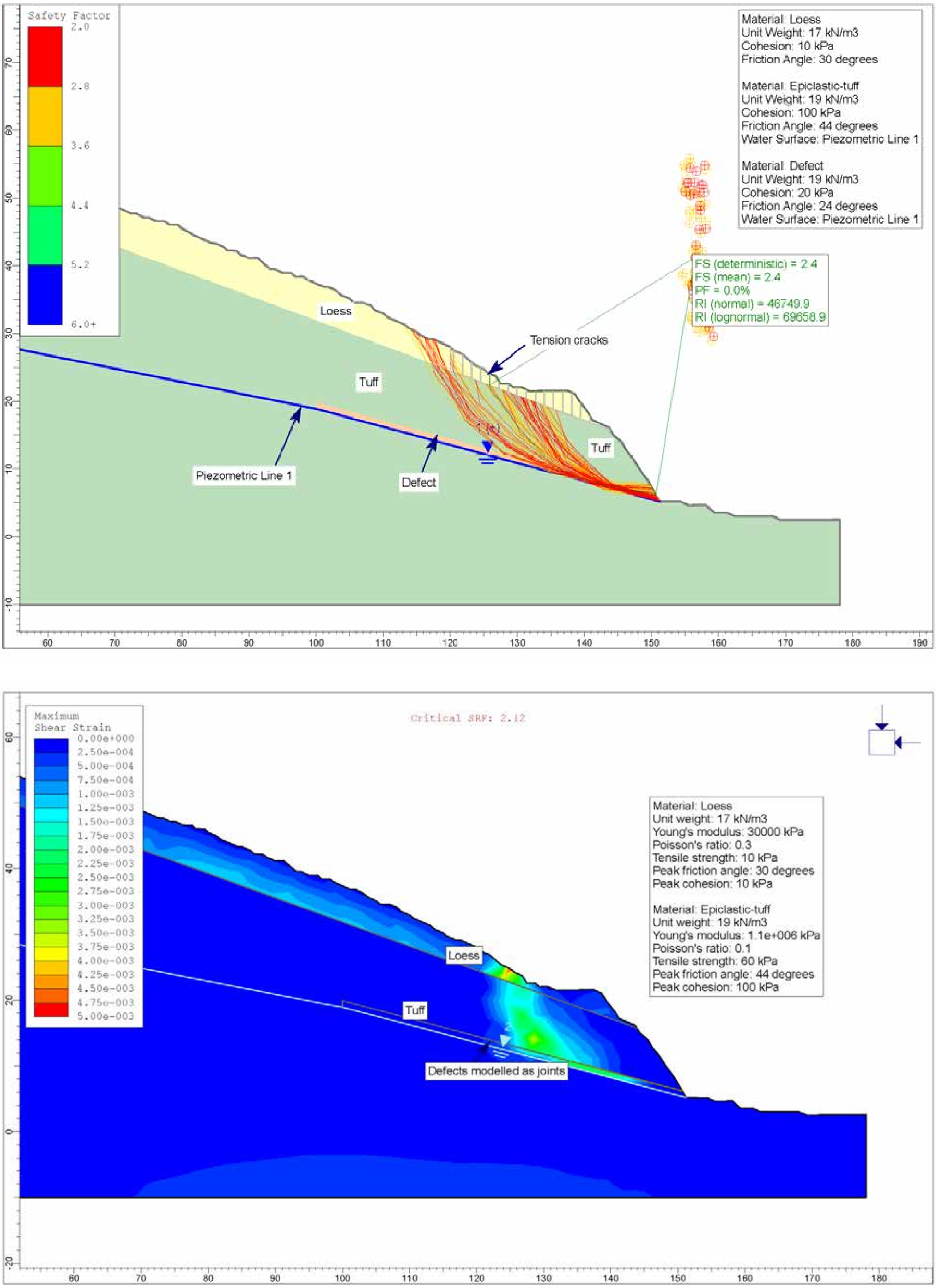


Figure 23 Example limit equilibrium and finite element modelling results for cross-section 1, mechanism 2.

4.1.2 Source area 2 (cross-section 2)

The results from the static limit equilibrium and finite element modelling for cross-section 2 are shown in Table 14, adopting failure mechanism 3. These results are shown for loess shear strength parameters of friction angle (ϕ) = 30° and cohesion (c) = 10 kPa.

Table 14 Example results from the static limit equilibrium (LEM) and finite element modelling (FEM) slope stability assessment of cross-section 2 (eastern slope).

Mechanism	Failure geometry	Minimum LEM FoS[1]	FEM SRF[2]	Main material		Slide mass depth[2] (m)	Slide mass length[2] (m)
				Friction (°)	Cohesion (kPa)		
1	Through the rock mass	3.4	3.5	44	100	11–15	16–26
2	Loess only	1.4	1.4	10	30	1–3	12–23

1 FoS is the factor of safety derived using the general limit equilibrium method of Morgenstern and Price (1965). 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 or factor of safety value that brings a slope to the verge of failure (Dawson, 1999).

2 Estimated depth (perpendicular to slide surface) and length (crown to toe of failure) of failure based on the slide surface geometry.

Results from the limit equilibrium and finite element modelling (Table 14) show that there is a good correlation between the static factors of safety and the shear strength reduction factors are also comparable. The sensitivity of the results to variation in cohesion is illustrated in Figure 24, where a relatively small reduction in cohesion from 10 to 2 kPa results in a drop in the factor of safety to below 1.

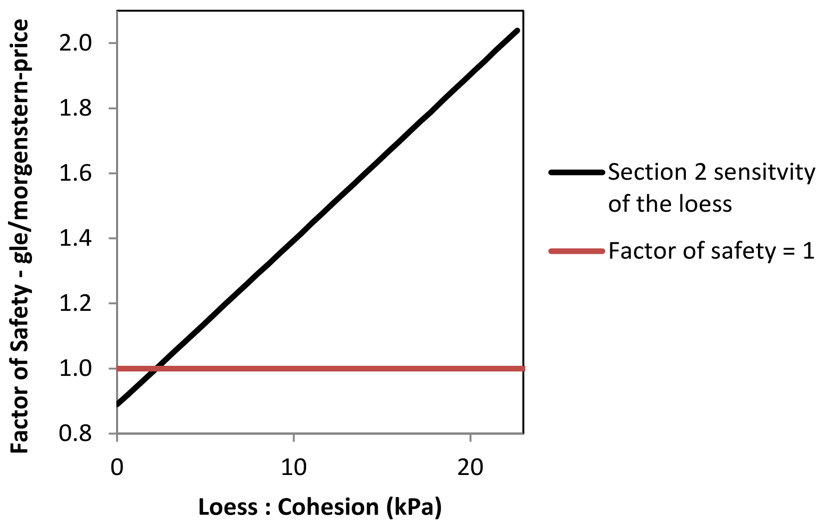


Figure 24 Sensitivity of the loess to changes in cohesion, for cross-section 2. A constant friction (ϕ) for the loess, of 30° was used in the assessment.

The results show that, as expected, the sensitivity of the slope model to changes in pore pressure is governed by the shear strength of the loess. The example shows that with a shear strength of cohesion = 10 kPa and friction = 30° a 2 m rise in pore pressure head above the base of the loess (above rockhead) would initiate failure. At higher values of shear strength, about 4 m of pore pressure head would be needed to initiate failure (Figure 25).

Additionally, infiltration from prolonged rainfall, leading to the deepening of the wetting band accompanied by a rise in water content of the loess and a decrease in matric suction could cause a loss of cohesion (Figure 25). These results are for the assumed failure geometry shown in Figure 26.

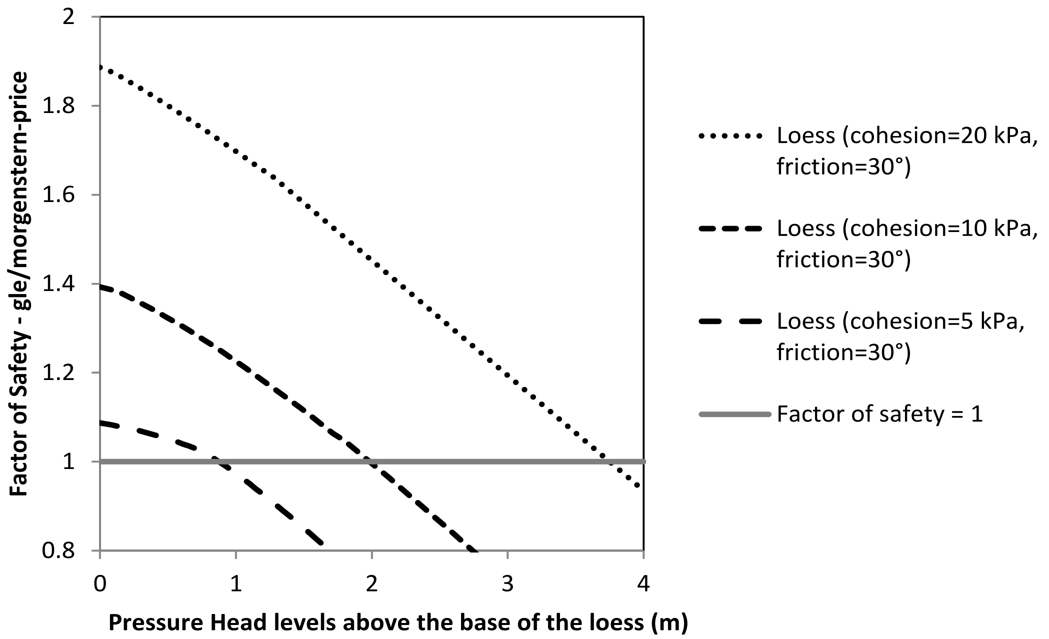
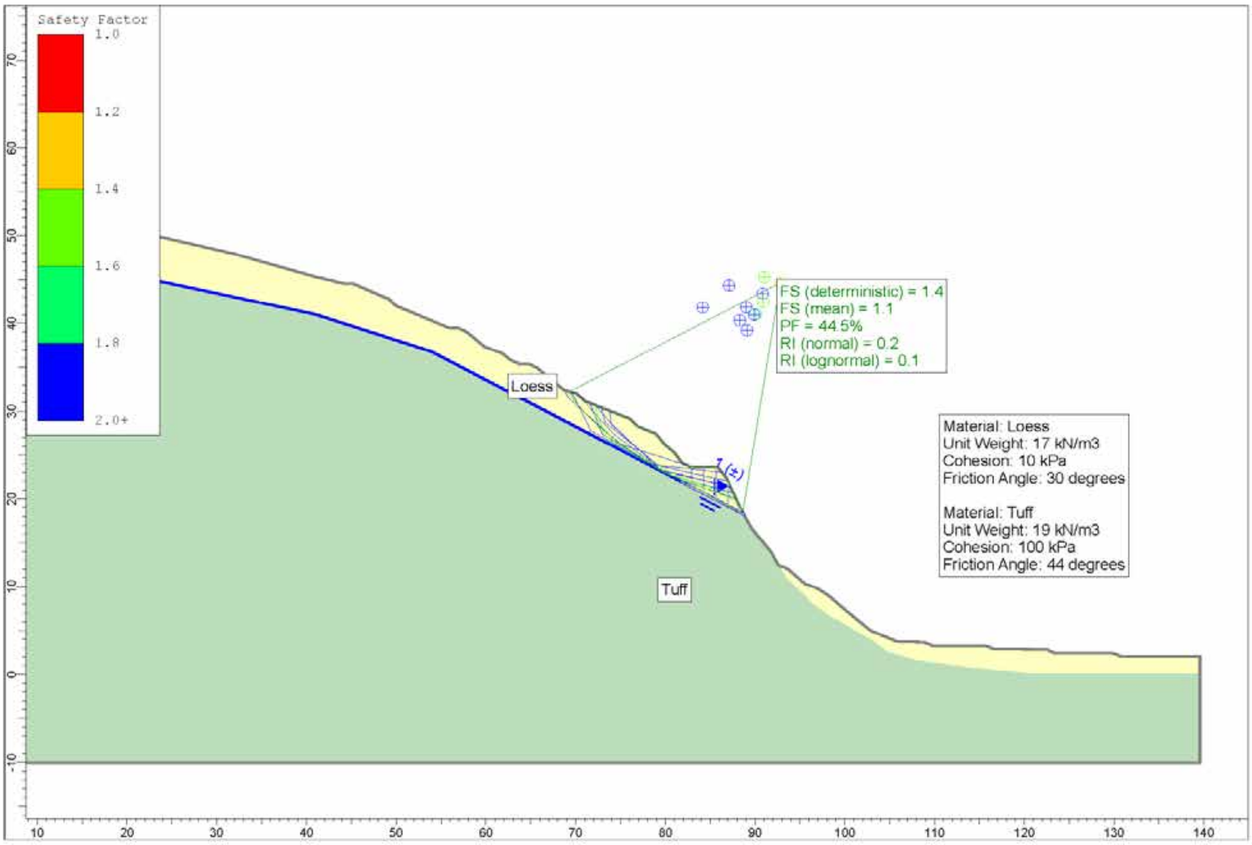


Figure 25 Sensitivity of the slope to changes in pore pressure, for cross-section 2. A pore pressure head of 0 m corresponds to the base of the loess. The water level acting in the modelled tension cracks is taken from the pore pressure head levels.

Results from the limit equilibrium and finite element modelling (Figure 26) show that there is a good correlation between the shape and location of critical slide surfaces derived from the limit equilibrium model and the zones of increased shear strain (displacement) from the finite element model assessment.



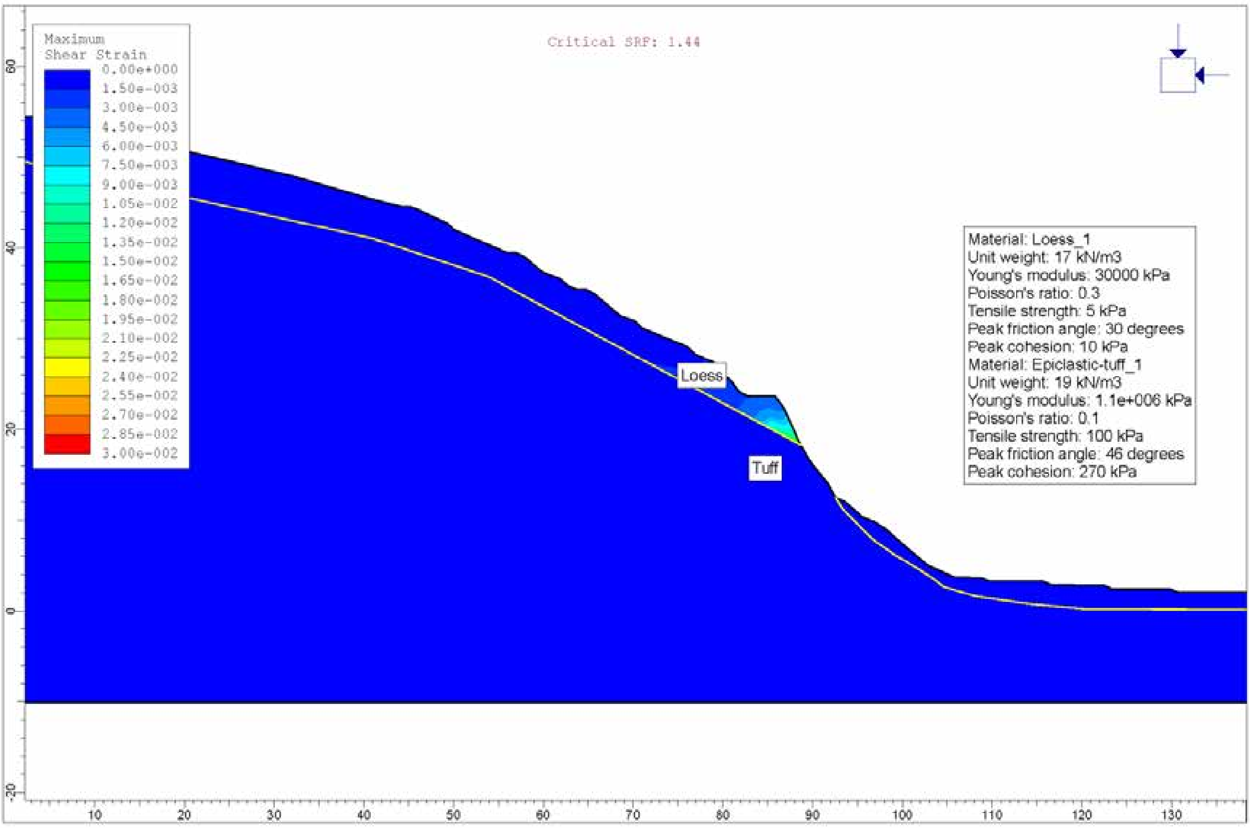


Figure 26 Example limit equilibrium and finite element modelling results for cross-section 2.

4.1.2.1 Summary of static slope stability results

For source area 1 (cross-section 1), the results from the stability assessment indicate that:

- Mechanism 1, rock slope: It is unlikely that the modelled deep-seated failure geometry would fail as a single failure under static conditions given the factor of safety for the assessed mechanisms are greater than 2. Results show that these assessed failure mechanisms are not particularly sensitive to groundwater increases. However, it is possible that local small failures of the slope could occur at lower pore pressure levels, especially in those areas where the basal discontinuity contains predominantly clay, and where it is exposed in the slope face. Such failures are likely to be small in volume.
- Mechanism 2, rock slope: Given the lack of subsurface information to constrain the presence, shape and persistence of the basal discontinuity, it is not known whether such failure mechanisms are credible. It is likely that failure of the rock slope is occurring both through the rock mass and along the basal discontinuity where observed in outcrop.
- Mechanism 3, loess slope: the loess slope overlying the rock has the lowest factor of safety of about 1.9 when drained, if the loess shear strength parameters that are typically used in the Port Hills are adopted (friction (ϕ) = 30° and cohesion (c) = 10 kPa). These results suggest that the loess slope is relatively stable under static conditions. Water ingress into the slope via the open tension cracks could cause flow-failures to develop, but given the slope geometry these are unlikely to be large in volume.

For source area 2 (cross-section 2), mechanism 3: the results from the stability assessment indicate that:

- If the loess shear strength parameters that are typically used in the Port Hills are adopted (friction angle (ϕ) = 30° and cohesion (c) = 10 kPa), then the factors of safety of the assessed slope would be about 1.4 when drained, but could reduce to less than 1 with a 2 m rise in pore pressure levels.
- A relatively small reduction in the cohesion of the loess from 10 to 2 kPa results in a drop in the factor of safety to less than 1. It is probably more reasonable to assume that the cohesion of the loess would reduce in response to increasing water content linked to rainfall infiltration, rather than the development of a continuous pore pressure surface within the slope leading to a reduction in the effective stress within the saturated loess.
- If water contents increase, then the range of shear strengths, derived from testing, could feasibly represent the strength of the loess in the slope. Under such conditions, results from the numerical slope stability back-analysis suggest that failure of the loess is likely.
- Earth/debris flows have occurred at the site recently, historically, and pre-historically, however, these have tended to be relatively small in volume (<100 m^3).
- Given the now cracked nature of the slopes at Cliff Street, it is feasible that loess water contents could increase in response to rainfall, as water can more readily enter the slope via the cracks.
- It is therefore possible future earth/debris flows at the site could occur more frequently and be larger in volume, than those have occurred in the past.

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.

The stability of cross-section 1 (source area 1) and cross-section 2 (source area 2) under dynamic conditions is assessed by assuming a drained slope with no permanent water table.

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 3.

Results from this assessment show that the relationship between the peak ground acceleration of the free field input motion and the corresponding modelled peak acceleration at the cliff crest (A_{MAX}) is approximately linear. Over the range of modelled peak horizontal accelerations the amplification factor is about 1.7 times the input free field peak acceleration (Appendix 3). The input peak accelerations are those derived from the synthetic free field rock outcrop earthquake time acceleration histories (Holden et al., 2014).

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. This was done for a range of slide surfaces, with: 1) different yield accelerations (Ky); and 2) different ratios of yield acceleration (Ky) to the maximum average acceleration of the slide mass (K_{MAX}). Permanent displacements were estimated along the slide surface, where the displacing mass was treated as rigid-plastic body and no internal plastic deformation of the mass was accounted for. Also, 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 Cliff Street 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.

The results from each modelled scenario were then compared to the recorded coseismic permanent slope displacements for each earthquake, for each cross-section (Figure 27–Figure 29). The modelled slide-surface geometries are presented in Figure 30 and Figure 31.

Table 15 Material strength parameters used for modelling permanent coseismic displacements for cross-sections 1 and 2. Coseismic displacements are inferred from measurements of the rock mass made at the slope toe (cross-section 1) and inferred from mapped crack apertures. Yield accelerations 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.

Cross-section	Earthquake	Failure mechanism and critical material	Friction (ϕ) (º)	Cohesion (c) (kPa)	Total inferred coseismic displacement (m)	Modelled coseismic displacement (m)	Yield acceleration (g)
1	22 February 2011	Intact tuff	44	100	0.3	0	0.9
1	22 February 2011	Basal discontinuity	24	20	0.3	0.03	0.4
		Basal discontinuity	12	0	0.3	0.2	0.1
1	22 February 2011	Loess	30	5	0.3	0.2	0.4
		Loess	30	10	0.3	0.03	0.6
1	13 June 2011	Basal discontinuity	24	20	-	0	0.5
		Basal discontinuity	12	0	-	0.1	0.1
2	22 February 2011	Loess	30	5	0.1	0.9	0.1
		Loess	30	10	0.1	0.1	0.3
		Intact tuff	44	100	0	0	0.9

4.2.2.1 Source area 1 (cross-section 1)

Results show that the minimum yield acceleration for the slope adopting a failure mechanism through the discontinuity at the base of the displacing masse ranges from 0.1 to 0.4 for the range of parameters modelled (Figure 22) for both the 22 February and 13 June 2011 earthquakes. The range of measured and inferred displacements are shown on Figure 27 for the 22 February 2011 earthquake and Figure 29 for the 13 June 2011 earthquake. The inferred displacements are from crack apertures at the slope crest and the lower displacements are measured acorss the discontinutiy at the toe of the slope, mainly fo the 22 February 2011 earthquake, as slope displacements in response to the 13 June 2011 earthquake are unknown and are inferred to be less than 0.05 m, on the assumption that any displacements greater than these would have been noticed, as the areas is populated and the defect in the slope toe is clearly visible from the road.

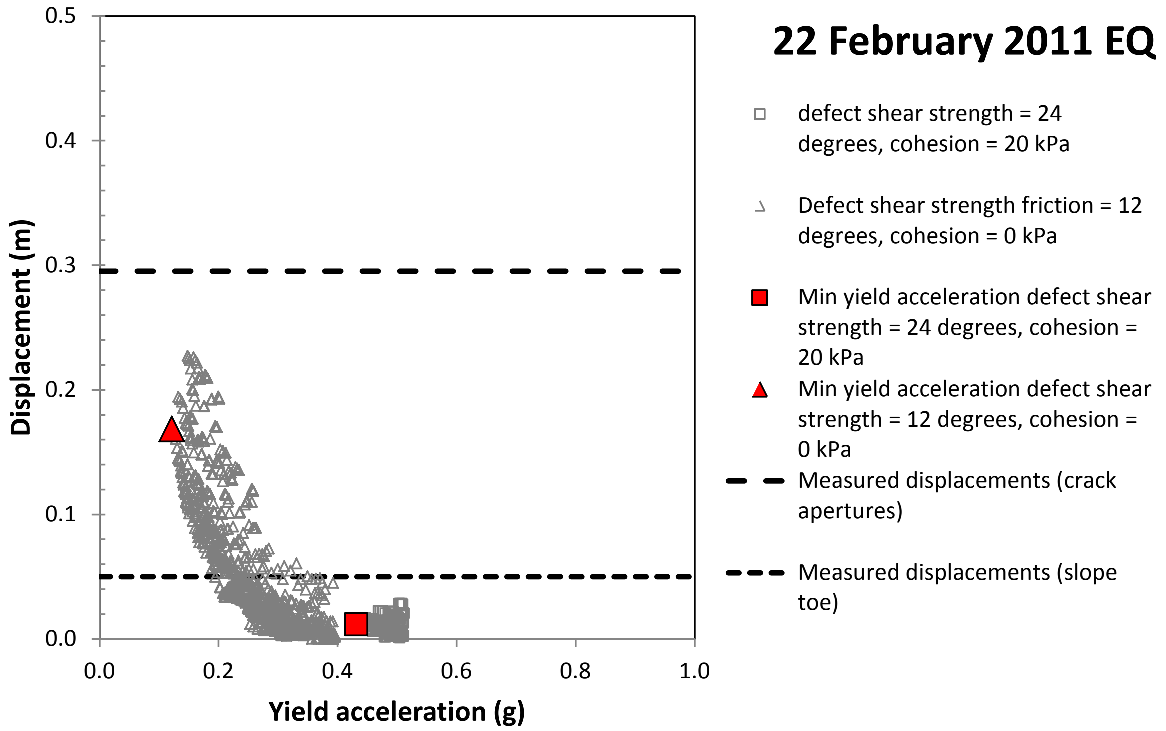


Figure 27 Modelled Slope/W decoupled displacements for cross-section 1, adopting variable estimates of the material strength of the basal discontinuity. Each datapoint represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the estimated coseismic permanent displacement for the cross-section.

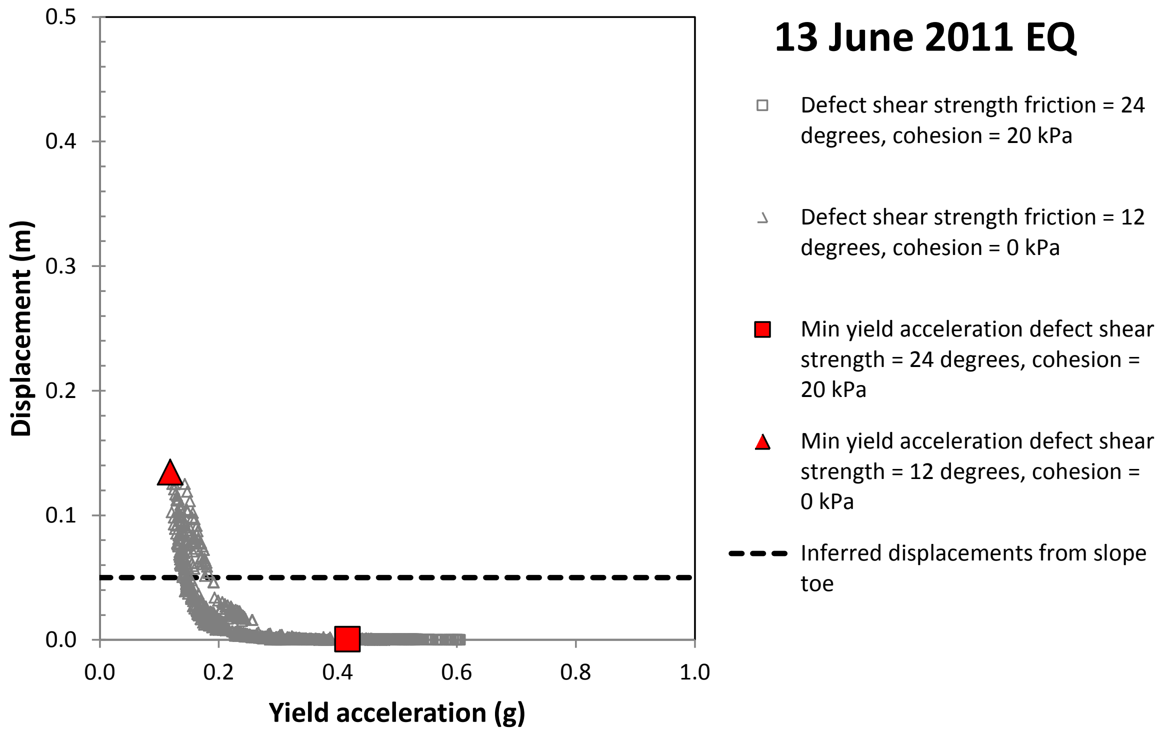


Figure 28 Modelled Slope/W decoupled displacements for cross-section 1, adopting variable estimates of the material strength of the basal discontinuity. Each datapoint represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 13 June 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic displacement for the cross-section.

From numerical back-analysis, the joint shear strength parameters in Figure 27 and Figure 28 give the best correlation between the modelled and inferred displacements for the 22 February and 13 June 2011 earthquakes, are those adopting discontinuity shear strengths between friction (ϕ) of 12–24° and cohesion (c) of 0–20 kPa .

4.2.2.2 Source area 2 (cross-section 2)

Results show that the minimum yield acceleration for the slope ranges from 0.1 to 0.4 for the range of parameters modelled (Figure 29) for the 22 February 2011 earthquake. The range of inferred displacements are also shown and are inferred from crack apertures at the slope crest.

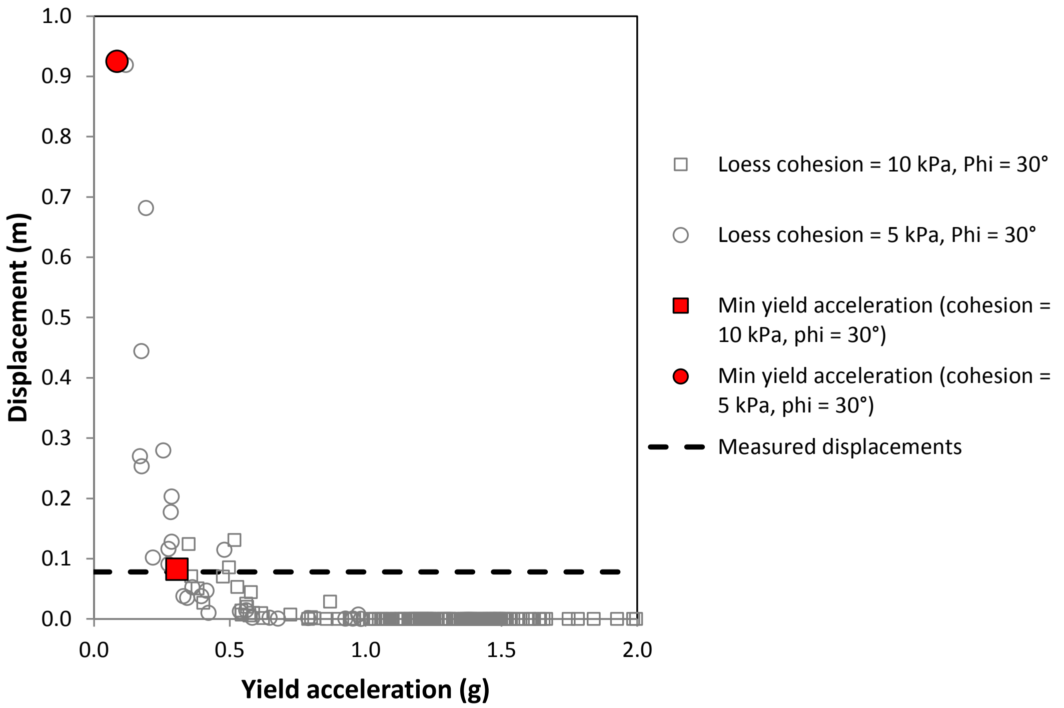


Figure 29 Modelled Slope/W decoupled displacements for cross-section 2 adopting variable estimates of the material strength of the loess. Each data point represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred displacement for the cross-section.

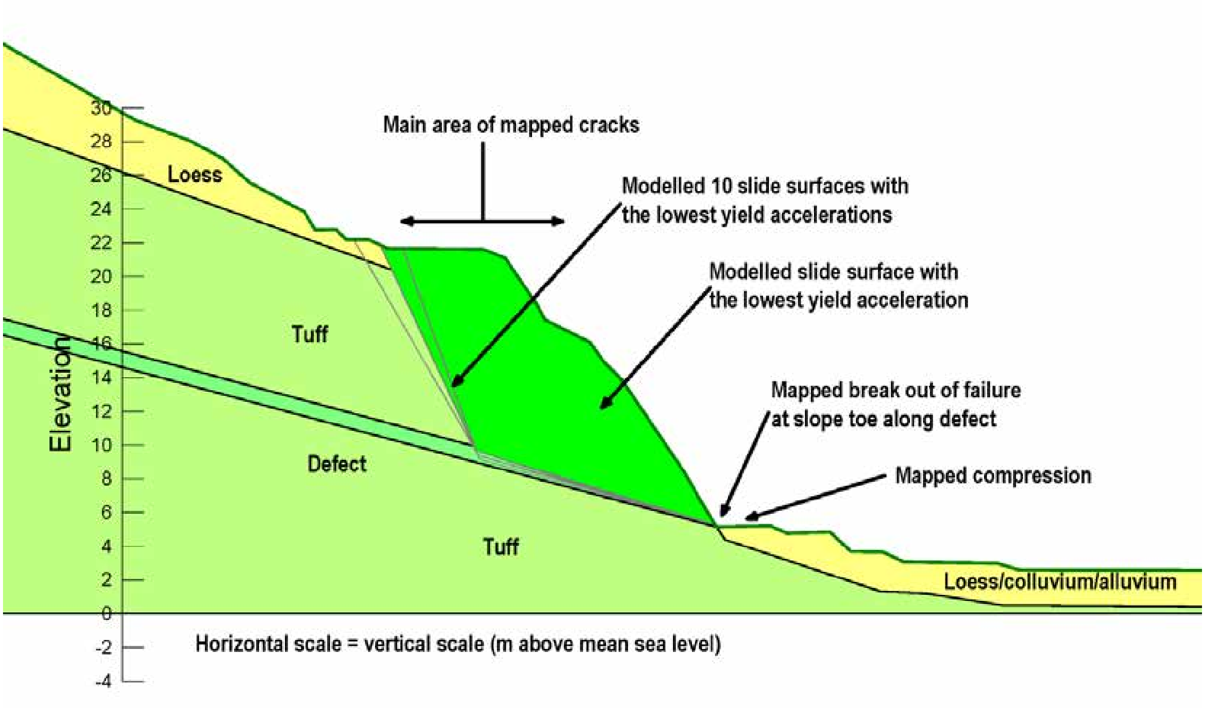


Figure 30 Cross-section 1 seismic slope stability assessment for the 22 February 2011 earthquake, for failure though the rock mass (mechanism 2).

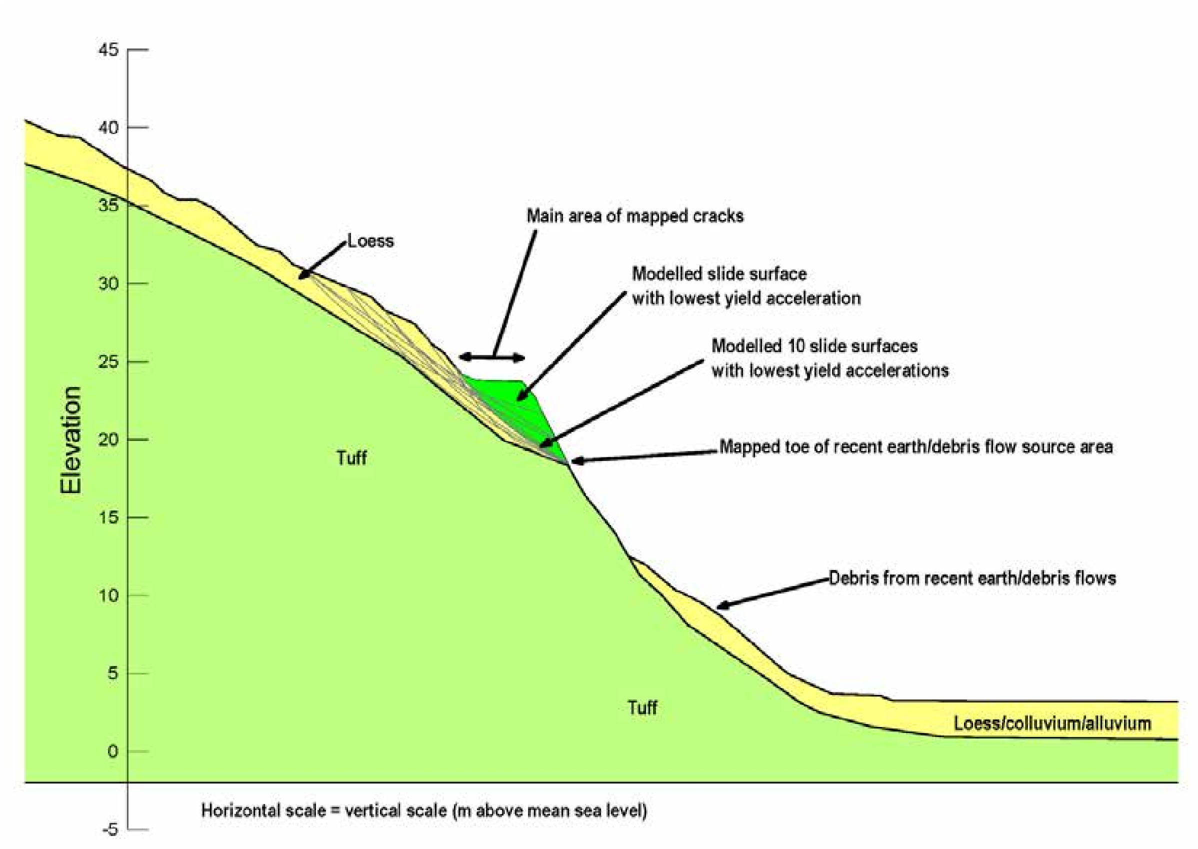


Figure 31 Cross-section 2 seismic slope stability assessment for the 22 February 2011 earthquake, for failure though the loess, above rockhead (mechanism 3).

4.2.2.3 Results from the assessment

The results from the assessment show that the magnitude of permanent slope displacement during an earthquake at the Cliff Street mass movement will vary in response to the shear strength of: 1) the basal discontinuity (source area 1, western slope), and 2) the loess (source area 2, eastern slope), at the time of the earthquake, as well as the duration and amplitude of the earthquake shaking.

Although displacement and failure of the loess along cross-section 2 (eastern slope) could occur under dynamic conditions, the volumes of any such failures are relatively small in comparison to failures that could be triggered by rain. For the western slope (source area 1), a large failure of the rock slope is more likely under dynamic loading during a strong earthquake.

The results show that:

1. For source area 1:
   - Failure though the rock mass (mechanism 1) is unlikely given the high yield acceleration of the slope 0.9 or above, unless there are a persistent discontinuities present in the slope:
   - For mechanism 2 the yield accelerations for the modelled slide-surface geometries, are about 0.1–0.4 g based on the modelled parameters. A good correlation between the recorded permanent coseismic displacements and modelled displacements for the 22 February 2011 earthquake was obtained for modelled slide surfaces adopting the range of shear strength parameters for the basal discontinuity of cohesion (c) 0–20 kPa and friction (ϕ) 12–24°.

- For mechanism 2, the magnitudes of permanent slope displacements from the decoupled results suggest that it is unlikely that the entire source area would fail as a result of strong earthquake shaking. Given the lack of subsurface information there is considerable uncertainty relating to the failure mechanism of the rock mass in this area.
- For mechanism 3, the yield accelerations for the modelled slide-surface geometries and material parameters are between 0.4 and 0.6. Permanent displacement of the loess is likely to have occurred during the 22 February 2011 earthquake, as well as displacement of the rock. However, it is unlikely that a large failure of the loess (overlying the rock) would occur under dynamic conditions, although small falls are possible.

2. For source area 2:
   - A good correlation between the recorded permanent coseismic displacements and modelled displacements for the 22 February 2011 earthquakes was obtained for modelled slide surfaces adopting loess shear strength parameters of cohesion (c) 10 kPa and friction (ϕ) of 30°. These are representative of the "typical" loess strength parameters used by engineering consultants working in the Port Hills (M. Yetton, personal communication 2013).
   - The lowest yield acceleration for the modelled slide-surface geometries is about 0.1–0.3 g and corresponding to slide surfaces within the loess, however displacements are likely to be relatively small and local.
   - Deep-seated failure through the rock mass is unlikely as the yield accelerations are typically greater than 0.9 g, and weak discontinuities have been identified in the slope from field mapping (Yetton, 2014).

For these assessments the displacements measured from field exposures and crack apertures are assumed to represent the coseismic permanent displacement of the slope during the 22 February 2011 earthquakes.

The selected slide-surface geometries shown in Figure 30 and Figure 31 are consistent with the results from the field mapping of crack apertures, where the locations of the modelled slide masses, with the lowest yield accelerations, correspond to the locations of the mapped cracks at the slope crest.

The estimated displacements of the slope for the 16 April and 23 December 2011 earthquakes, adopting the failure mechanisms and strength parameters in Table 17, were zero metres. No displacements of the slope were identified as a result of these earthquakes

4.2.2.4 Forecast modelling of slope deformation

For source area 1 (cross-section 1), Permanent displacements, from the decoupled assessment results from the 22 February 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 20 slide surfaces were chosen to represent the results from the earthquake input motion, adopting different estimates of the shear strength of the basal discontinuity (listed in Table 15).

The results show that between Ky/K_{MAX} values of 0.2–0.6, and Ky/A_{FF} values of 0.1–0.7, the data are well fitted to a straight line (exponential trend line) in semi-log space. The coefficient of determination (R^2) is 0.97 for Ky/K_{MAX} and 0.81 for Ky/A_{FF}, and includes all of the plotted data (N = 41). The gradients of the fitted lines are different, with the Ky/K_{MAX} line having a slightly steeper gradient, indicating, as expected, that for the same magnitude of displacement the ratio of Ky/K_{MAX} is lower than the corresponding Ky/A_{FF} ratio.

The peak ground acceleration of the input motion (A_{FF}) does not take into account amplification effects caused by the slope geometry and contrasting materials (Appendix 3). From the data in Figure 32, the mean ratio of K_{MAX} to A_{FF} for source area 1 is 1.5 (±0.3 at one standard deviation), meaning that K_{MAX} is on average 1.5 times greater than the peak horizontal ground acceleration of the input motion.

For ratios of Ky/K_{MAX} (Figure 32), the estimated magnitudes of displacement are consistent with those reported by Jibson (2007), where the data from Cliff Street plot between the ranges of data for earthquakes of M6.5–7.5 reported by Franklin and Chang (1977) and Ambraseys and Menu (1988) and plotted by Jibson (2007).

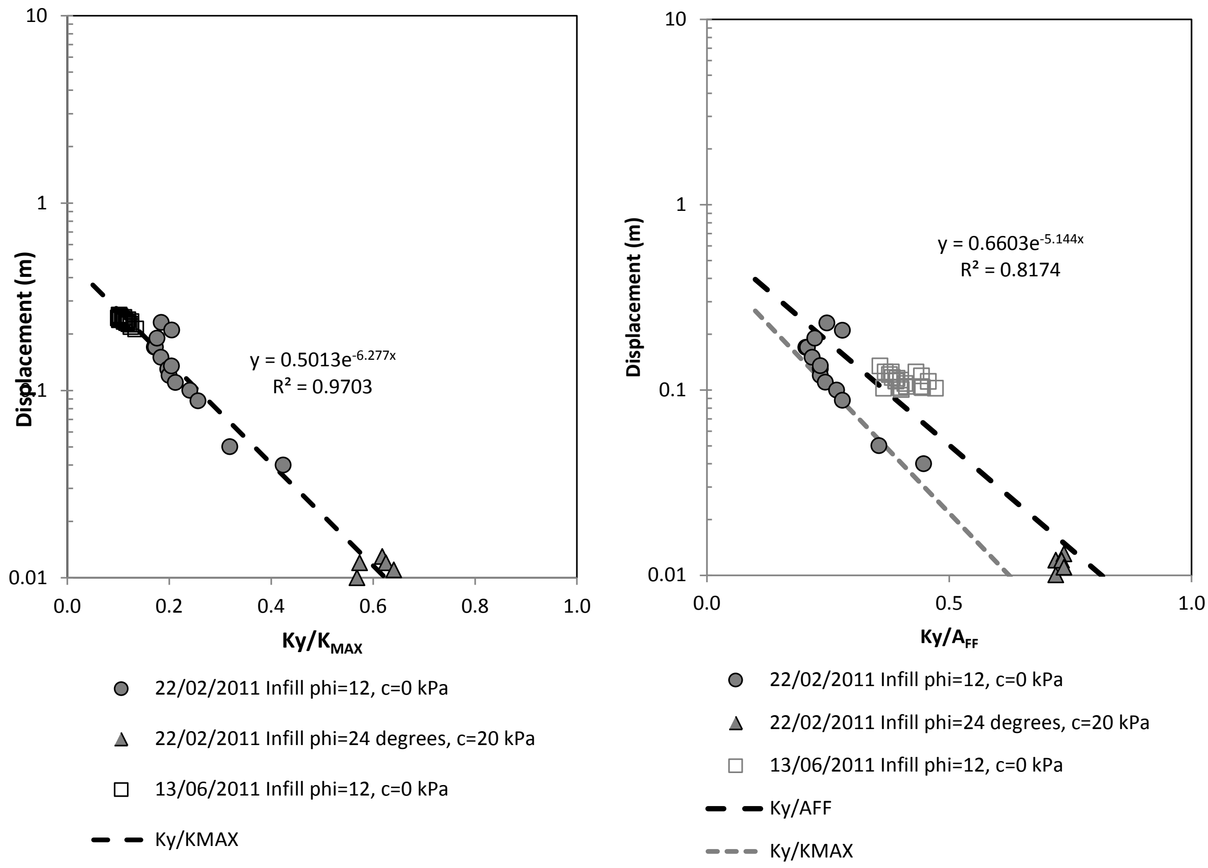


Figure 32 Decoupled Slope/W displacements calculated for cross-section 1, for different ratios of yield acceleration to maximum average acceleration of the mass (Ky/K_{MAX}), and maximum acceleration of the mass (Ky/A_{FF}), for selected slide-surface geometries, and given material shear strength parameter models 2 and 3. 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 = 41). 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:

1. the shear strength of the rock mass at the time of the earthquake;
2. pore pressures within tension cracks and the rock mass, at the time of the earthquake; and
3. duration and amplitude of the earthquake shaking.

For the western slope (source area 1, cross-section 1), the relationship between the yield acceleration and the maximum average acceleration (from Figure 32) has been used to determine the likely range of displacements of a given failure mass with a range of yield accelerations (*Ky*) at given levels of maximum average ground acceleration (K_{MAX}). This has been done using the four earthquake event bands, used to represent the range of earthquake events the slopes could be subjected to in the future. The four peak ground acceleration bands are the same as those originally adopted for the cliff collapse risk assessment by Massey et al. (2012a).

The results are shown in Table 16. Conservative yield accelerations have been adopted to take into account the possibility that the current shear strength of the materials is now degraded as a result of the past movement.

Table 16 Cross-section 1 (source area 1) forecast modelling results from the dynamic slope stability assessment. Estimated displacements are rounded to the nearest 0.1 m. PGA is peak ground acceleration.

PGA band (A_{FF})[1]	**1**	**2**	**3**	**4**
PGA (A_{FF}) band (g)	0.1–0.4	0.4–1.0	1.0–2.0	2.0–5.0
Midpoint of PGA (A_{FF}) band (g)	0.25	0.7	1.5	3.5
Adopted K_{MAX} to A_{FF} ratio	1.8 (mean + 1 standard deviation)			
Equivalent K_{MAX}	0.3	1.2	2.6	6.1
Yield acceleration (Ky) (g)	**Estimated displacements (m)**			
0.15	**0.1**	**0.2**	**0.3**	**0.4**
0.30	**0.0**	**0.1**	**0.2**	**0.4**
0.40	**0.0**	**0.1**	**0.2**	**0.3**

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

[2] The relationship between the yield acceleration and the average acceleration (from Figure 32) has been used to determine the likely magnitude of displacement for different yield accelerations (*Ky*) for a given slide surface at given levels of average ground acceleration (K_{MAX}). The values of K_{MAX} represent the midpoint of each peak ground acceleration band used in the risk assessment. For example, the midpoint peak acceleration of the 0.1–0.4 g band would be 0.25 g.

The geometries of the different failure masses adopted for the assessment are the same as those shown in Figure 30. 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 basal discontinuity and any pore pressures present at the time of the earthquake.

Based on the results it is unlikely that the entire mass of source area 1 would fail catastrophically at low ground accelerations. At higher ground accelerations, similar or higher than those recorded during the 22 February 2011 earthquake, it is possible that failure of at least part of source area 1 would occur. The likelihood of such a failure initiating is discussed in Section 5.

4.2.3 Slope stability – summary of results

The main results from the static and dynamic stability assessment are:

1. In their current state, it is possible for failure of the example 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 reduce with time, and the occurrence of further strong earthquakes.
2. Two main mechanisms of failure that could generate large volumes of debris have been identified: 1) failure of the rock slope in the west of the site (source area 1) mainly under dynamic loading conditions; and 2) failure of the loess slope in the east of the site (source area 2) mainly under static (rain) conditions due to a decrease in cohesion in the loess/fill/colluvium or increase in pore pressure.
3. It should also be noted that the stability assessment results presented are for deep-seated slide surfaces through the rock mass. However, much of the slope face appears unstable and rocks fall from the slope with no apparent trigger, indicating that parts of the slope face are only marginally stable to unstable, with factors of safety much less than those assessed for the deep-seated failures.
4. Other failures (in addition to the modelled failure mechanisms and geometries) of rock and soil could occur from elsewhere on the slope, however, these volumes are considered to be smaller than those that could be generated by assessed source areas 1 and 2, and to be adequately covered within the stated range of uncertainty associated with the risk assessment by Massey et al. (2012a).

4.3 RUNOUT DISTANCE

4.3.1 Potential future source volume estimation

The likely locations and volumes of potential debris avalanche (western slope, source area 1) and earth/debris flow (eastern slope, source area 2) sources have been estimated based on:

1. Numerical stability analyses;
2. Surveys;
3. Mapped crack distributions relating to the 2010/11 Canterbury earthquakes; and
4. 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 as quarter-ellipsoids (half-spoon shaped) (following the method of Cruden and Varnes, 1996) (Figure 33) and estimated volumes are shown in Table 17.

Figure 33 Estimation of landslide volume assuming a quarter-ellipsoid shape.

Table 17 Example earth/debris flow source volumes (the first digit in the number is significant).

Source area	Main landslide type	Volume scenario	Estimated volume (m^3)
1	Debris avalanche	Lower	1,100
		Middle	1,800
		Upper	2,300
2	Earth/debris flow	Lower	180
		Middle	460
		Upper	560

The credibility of the estimated debris avalanche failure volumes was evaluated by comparing them against volumes of individual debris avalanches that fell from the rock slopes at Richmond Hill Road, Shag Rock Reserve and Redcliffs (Massey et al., 2012b) during the 13 June 2011 earthquakes (Figure 34). These volumes were derived from terrestrial laser scan change models.

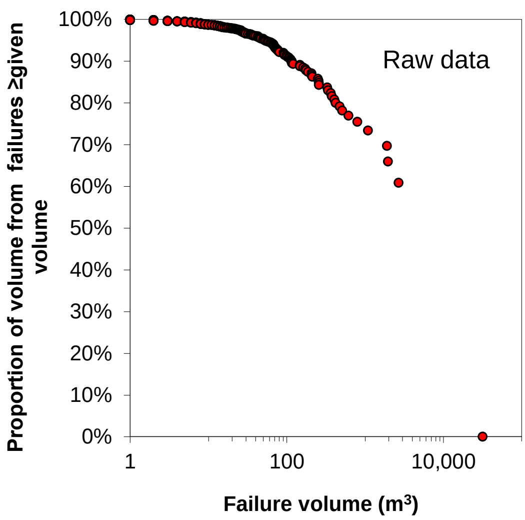


Figure 34 Proportion of volume from debris avalanches in the Port Hills greater than or equal to a given volume. Data from the 2011 landslide volumes triggered by the 13 June 2011 earthquakes, derived from terrestrial laser scan change models of Richmond Hill, Shag Rock Reserve and Redcliffs.

The credibility of the earth/debris flow potential failure volumes was 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 35, 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 17 is well within the range of these two datasets.

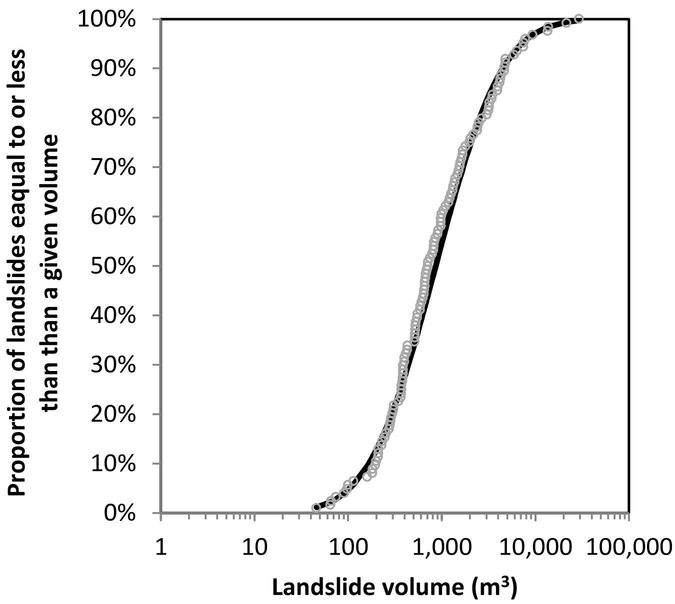


Figure 35 Estimation of landslide volumes in the Port Hills loess from Townsend and Rosser (2012) adopting the area depth relationships of Larsen et al. (2010).

4.3.2 Runout modelling

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

4.3.2.1 Empirical method

Debris avalanches (source area 1)

A total of 45 sections through Port Hills debris avalanches that were triggered by the 22 February and 13 June 2011 earthquakes have been assessed. For each section the fahrboeschung (angles) for debris avalanche: 1) "talus" (where the ground surface is obscured by many boulders); and 2) "boulder roll" (individual boulders) have been defined based on field mapping. The results for each are shown in Figure 36 as ratios of H/L where H is the height of fall and L is the length, or runout distance, of the mapped rockfalls and debris avalanche deposits (talus).

These two fahrboeschung relationships are based on debris avalanches that fell from many cliffs in the wider Port Hills area during the earthquakes. They therefore reflect all of the different types of slope shape that could affect the debris avalanche runout.

The mean and mean minus one standard deviation fahrboeschung angles for debris avalanches (talus and boulder roll) from each potential source volume (lower, middle and upper estimates), are shown in Table 18.

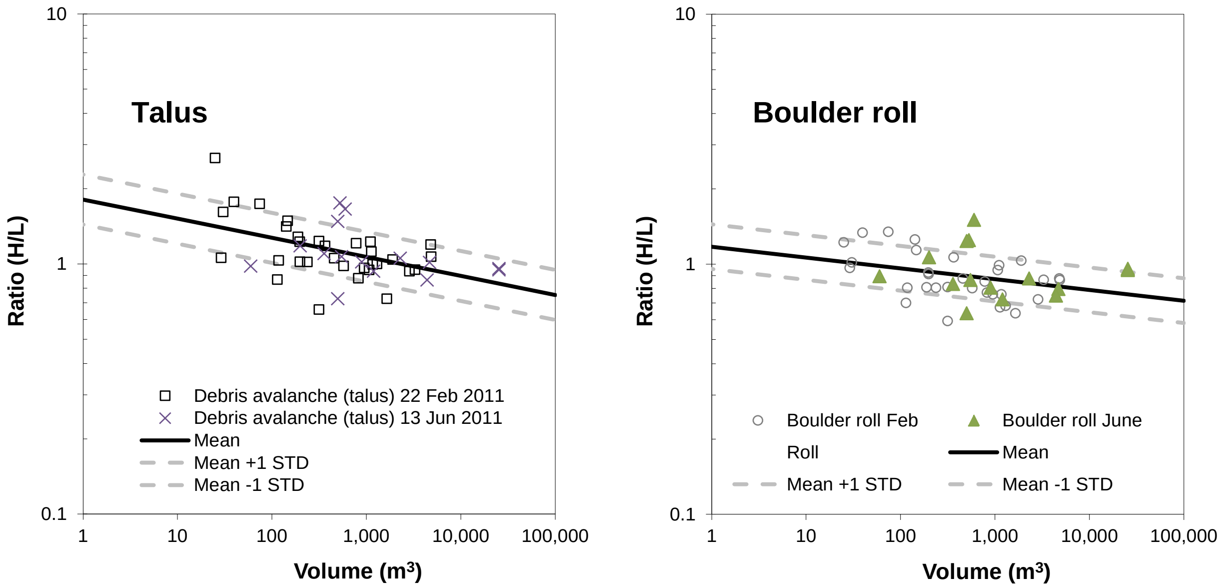


Figure 36 The empirical fahrboeschung relationships, expressed as the ratio of height (H) to length (L) for both debris avalanche talus and boulder roll (rockfalls), recorded in the Port Hills. N = 45 sections. Errors are expressed as the mean ± one standard deviation (STD).

Table 18 The mean and mean-minus-one-standard-deviation (-1 STD) fahrboeschung angles (F-angles) for source area 1 lower, middle and upper volume estimates, based on the compilation of debris avalanche and rockfall runout data from the Port Hills debris avalanches.

Source area	Volume (m^3)	Talus F-angle (°)		Boulder roll F-angle (°)	
		Mean	Mean – 1 STD	Mean	Mean – 1 STD
1 lower	1,100	46.7	40.1	41.0	35.3
1 middle	1,800	45.6	39.0	40.4	34.7
1 upper	2,300	45.1	38.5	40.1	34.4

Earth/debris flows (source area 2)

For source area 2, the fahrboeschung angles have been defined applying the method described in Massey and Carey (2012) as shown in Figure 37; using published debris flow records reflecting different types of slope shape that could affect the debris runout.

The mean and mean minus one standard deviation fahrboeschung angles for each potential source volume (lower, middle and upper estimates) are shown in Table 19.

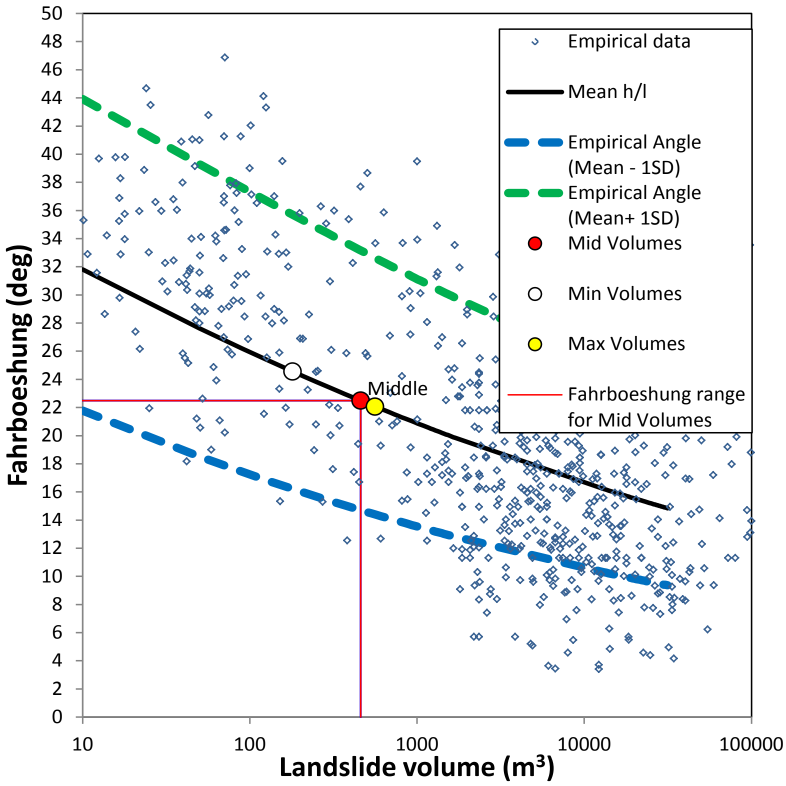


Figure 37 Estimation of fahrboeschung angles for volumes shown in Table 21, based on empirical runout data presented by Massey and Carey (2012).

Table 19 The mean and mean-minus-one-standard-deviation (-1 STD) fahrboeschung angles (F-angles) for source area 2 lower, middle and upper volume estimates, based on the compilation of earth/debris flows in Massey and Carey (2012).

Source area	Volume (m^3)	F-angle (°)	
		Mean	Mean – 1 STD
2 lower	180	25	16
2 middle	460	23	15
2 upper	560	22	14

4.3.2.2 Numerical method – RAMMS

Debris avalanche (source area 1)

It is noted, as detailed by Massey et al. (2014), that the fahrboeschung method does not take into account the shape of the slope below the source area, which can have a significant effect on the actual runout of the debris. The RAMMS software (RAMMS, 2011) takes into account the site slope geometry when modelling debris runout. 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). The RAMMS model parameters were calculated from the back-analysis of 23 debris avalanches (ranging in volume from 200 to 35,000 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. The modelled parameters mu (µ) and xi were optimised to obtain a good correlation between the modelled versus actual runout and deposited debris heights (Figure 38).

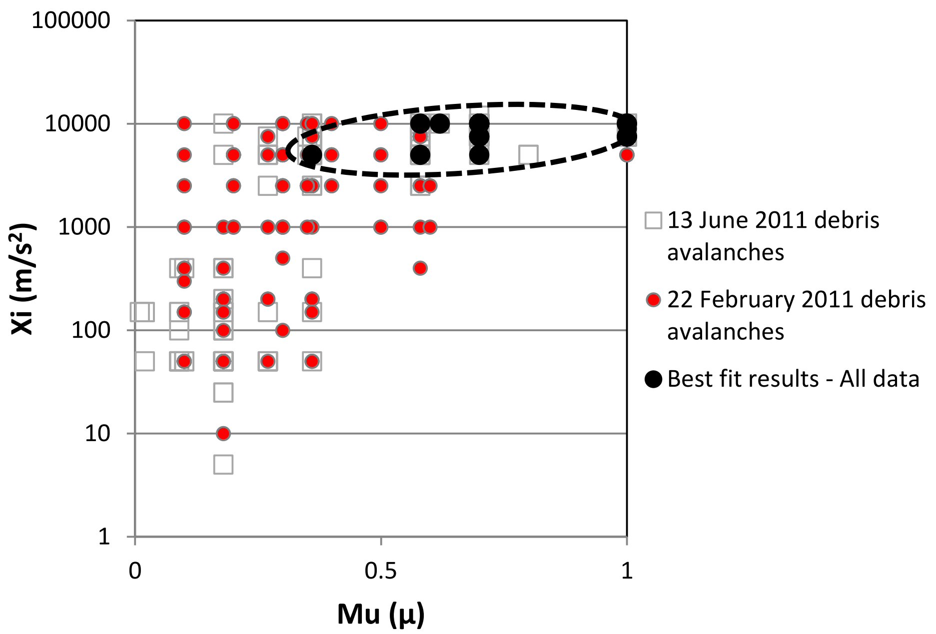


Figure 38 Range of parameters used to back-analyse the runout of debris avalanches in the Port Hills triggered by the recent earthquakes using the RAMMS software (RAMMS, 2011).

The model parameters that gave the "best fits" between modelled and actual runout distances and heights when: μ = 0.7 and xi = 7,500 m/s^2. The xi values are comparable to results from other assessments compiled by Andres (2010) for rockfalls (debris avalanches), but the μ values are larger than those shown by Andres (2010), possibly because the Port Hills debris avalanches are more clast-dominated (Figure 39).

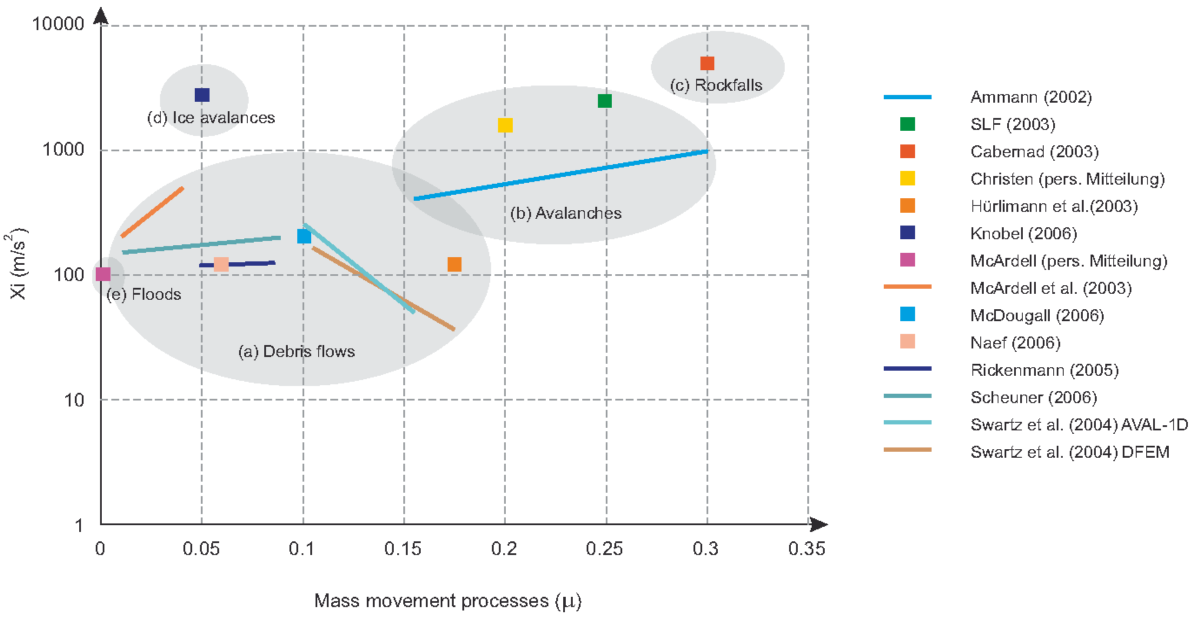


Figure 39 Range of parameters for different mass movement processes: a) debris flows, b) snow avalanches, c) snow avalanches, d) ice avalanches, e) debris floods. Modified from Andres (2010).

For each back-analysed debris avalanche, the modelled final debris thicknesses were compared to the actual deposit thicknesses interpolated from difference models derived from the airborne LiDAR surveys using a 1 m grid. For debris avalanches triggered by the 22 February 2011 earthquakes the deposit thicknesses were estimated from differences between the 2011a (March 2011) LiDAR survey and the 2003 LiDAR survey. For debris avalanches triggered by the 13 June 2011 earthquakes the 2011c (July 2011) and 2011a LiDAR surveys were used. Statistics from the comparison give a mean difference of 0.5 (±0.4) m^3, with a mode of 0.2 m^3 (Figure 40) for the 1 m^2 grid cells.

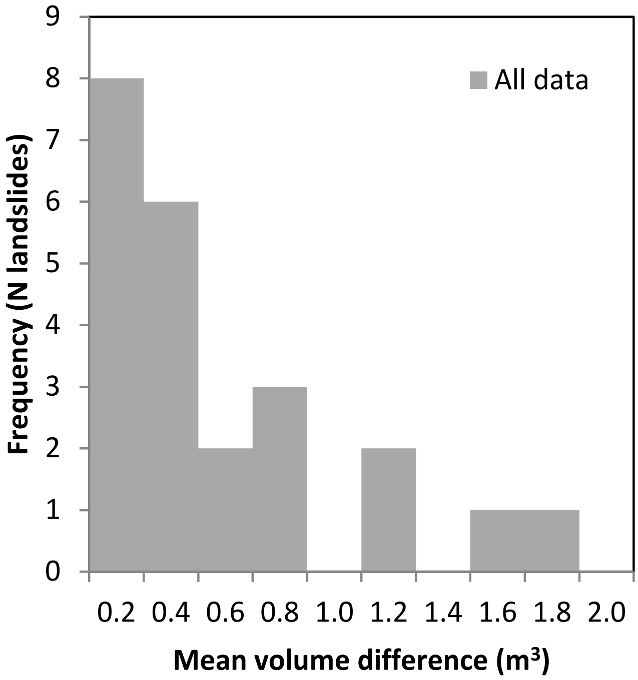


Figure 40 Mean volume difference between the RAMMS modelled volumes and the actual recorded volumes per 1 m^2 grid cell. N = 23 debris avalanches triggered by 22 February and 13 June 2011 earthquakes.

For the 23 debris avalanches, the performance of the RAMMS and fahrboeschung models (based on the compiled 45 sections shown in (Figure 36) were assessed against the actual field mapped runout distances. The RAMMS model performed well with a gradient of 1.01 (±0.04) at one standard deviation and coefficient of determination (R^2) of 0.3 indicating the data are scattered. The empirical fahrboeschung model performed about the same as the RAMMS model, where the gradient was 1.06 (±0.05) at one standard deviation but the coefficient of determination (R^2) of 0.5 indicates less scatter (Figure 41).

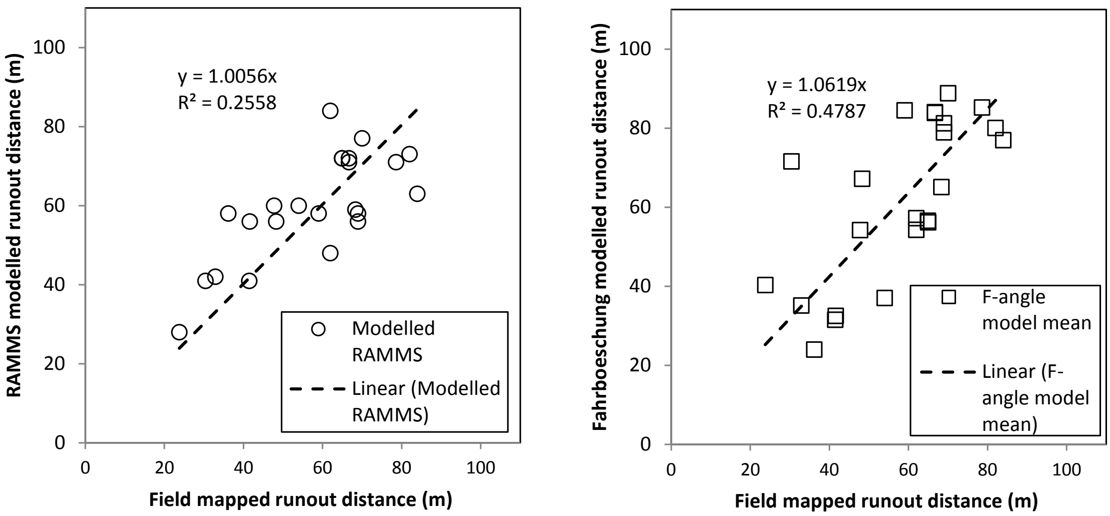


Figure 41 Comparison between the RAMMS modelled and the empirical-modelled debris runout (Figure 36) and the actual recorded runout for debris avalanches triggered by the 22 February and 13 June 2011 earthquakes. N = 23 debris avalanches.

Earth/debris flows (source area 2)

The RAMMS model parameters were calculated from the back-analysis of four Port Hills debris flows. The modelled parameters μ and *xi* 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 x_i = 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 x_i values are comparable to results from other assessments compiled by Andres (2010) for debris flows (Figure 42).

4.3.3 Forecast runout modelling

A hazard map (Figure 42) presents the empirical and numerical runout limits from the modelling. The mean and mean minus one standard deviation fahrboeschung angle for each source area assuming the upper volume estimates, are shown. The estimated runout distances from RAMMS are shown in Appendix 4 (debris height) and Appendix 5 (debris velocity), for sources 1 and 2 (upper, middle and lower source volume estimates), along with the corresponding mean and mean minus one standard deviation fahrboeschung angle

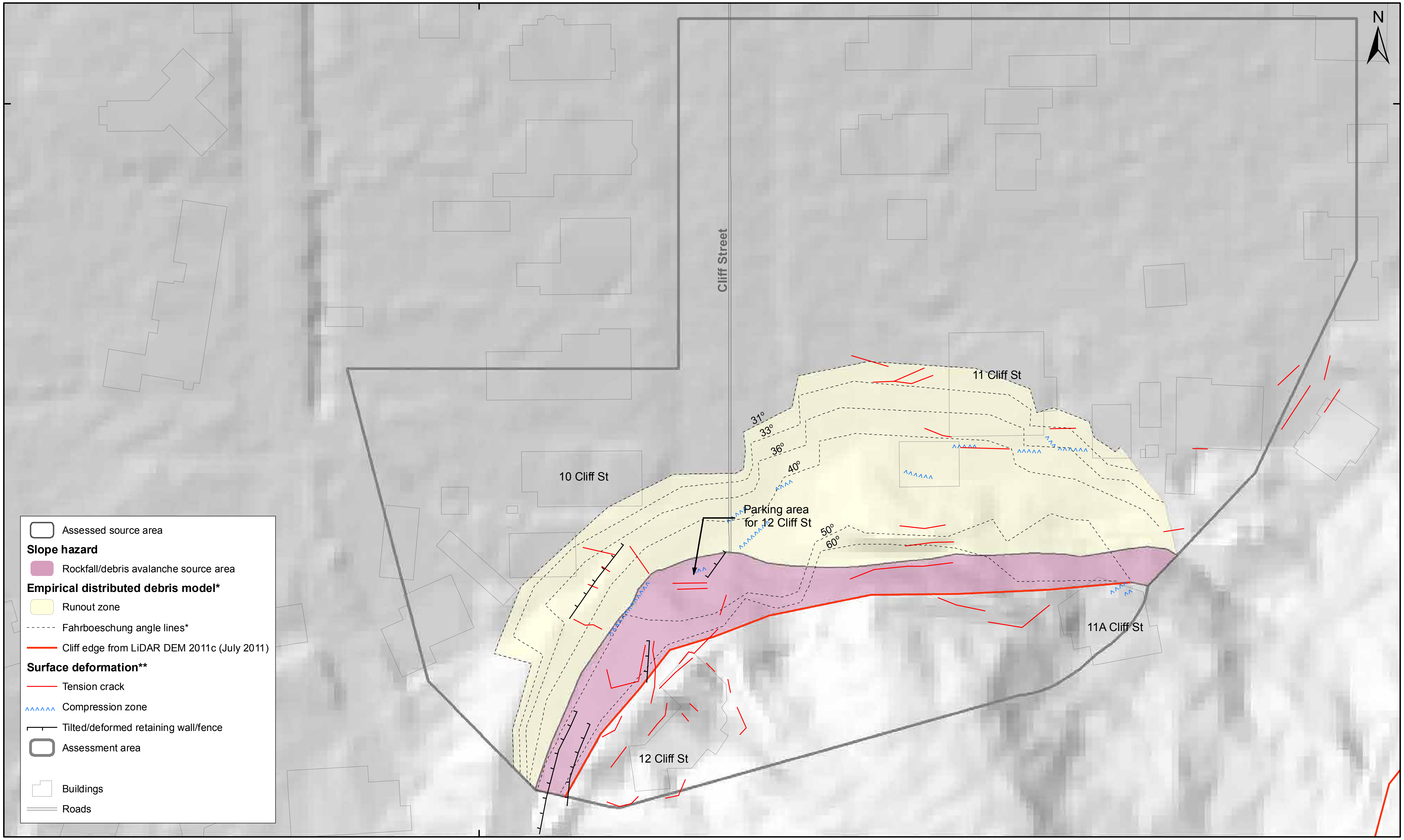
A hazard map (Figure 42) presents the empirical and numerical runout limits from the modelling. Figure 42, Map 1, shows the cliff collapse hazard map for the randomly distributed source areas and debris and the fahrboeschung angles from 60° to 31°, as described by Massey et al. (2012a). The 31° fahrboeschung angle is the runout limit of rocks from debris avalanches triggered by the 2010/11 earthquakes from the assessed cliffs (Redcliffs, Shag Rock Reserve, and Richmond Hill/Wakefield Avenue) in Massey et al. (2012a).

Figure 42, Map 2, shows the cliff collapse hazard map for source area 1 and the upper volume estimates. The mean and mean minus one standard deviation fahrboeschung angles for the upper debris volume estimates, are also shown. The estimated runout distances from the RAMMS modelling for the same source area is also shown for the upper debris volume estimates.

Figure 42, Map 3, shows the earth/debris flow hazard map for source area 2 and the upper volume estimates. The mean and mean minus one standard deviation fahrboeschung angles for the upper debris volume estimates, are also shown. The estimated runout distances from the RAMMS modelling for the same source area is also shown for the upper debris volume estimates

RAMMS runout models are contained in Appendix 4 (debris height) and Appendix 5 (debris velocity), for sources 1 and 2 (upper, middle and lower source volume estimates), along with the corresponding mean and mean minus one standard deviation fahrboeschung angles.

In general, there is a good correlation between the fahrboeschung angles and RAMMS runout limits for the assessed source areas.



SCALE BAR: 0 25 50 m

EXPLANATION:

* Modified from report 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

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CLIFF COLLAPSE HAZARD MAP
(Randomly distributed debris)

Cliff Street
Christchurch

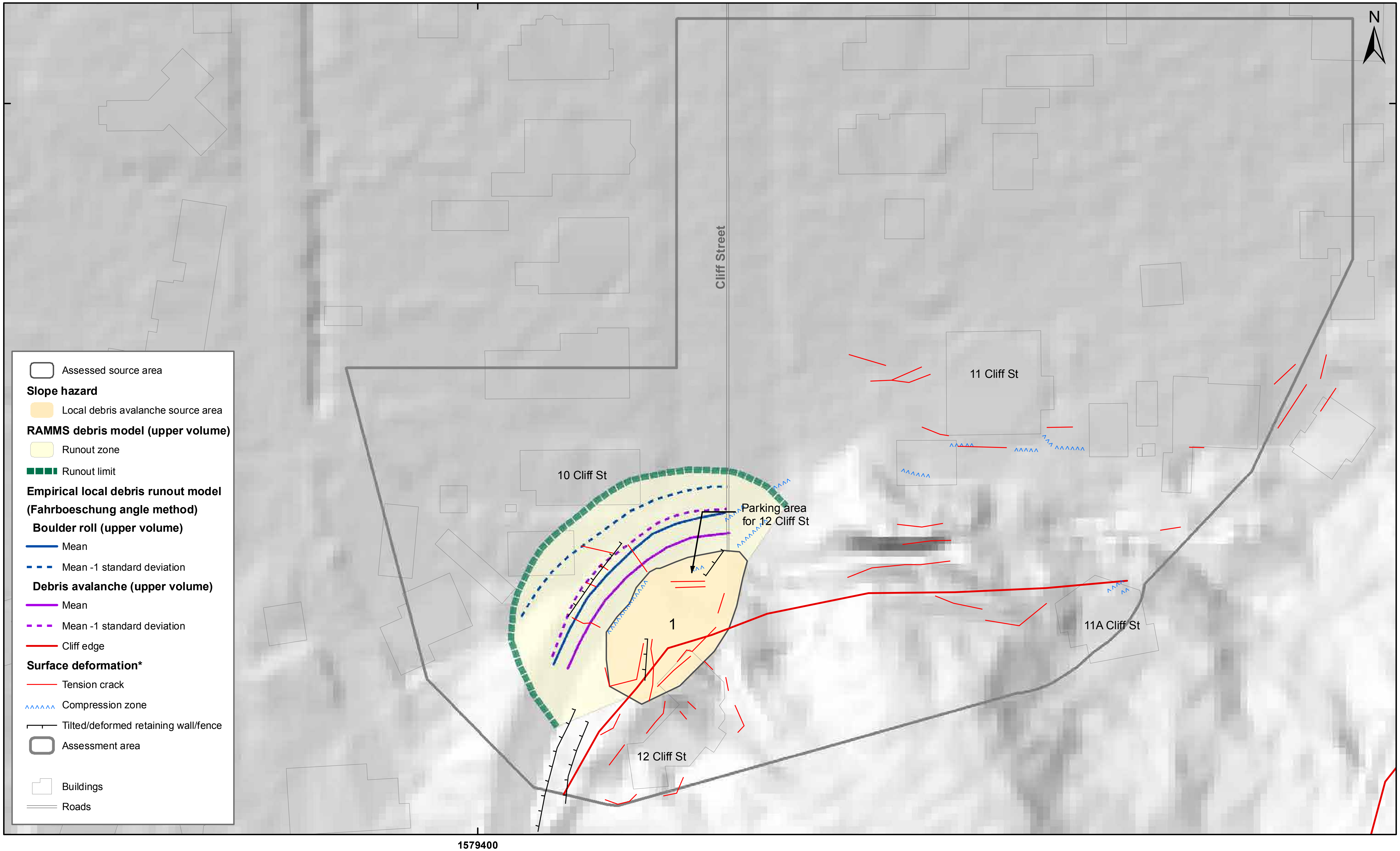
FIGURE 42

Map 1

FINAL

REPORT: CR2014/73

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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DEBRIS AVALANCHE HAZARD MAP
(Source area 1)

Cliff Street
Christchurch

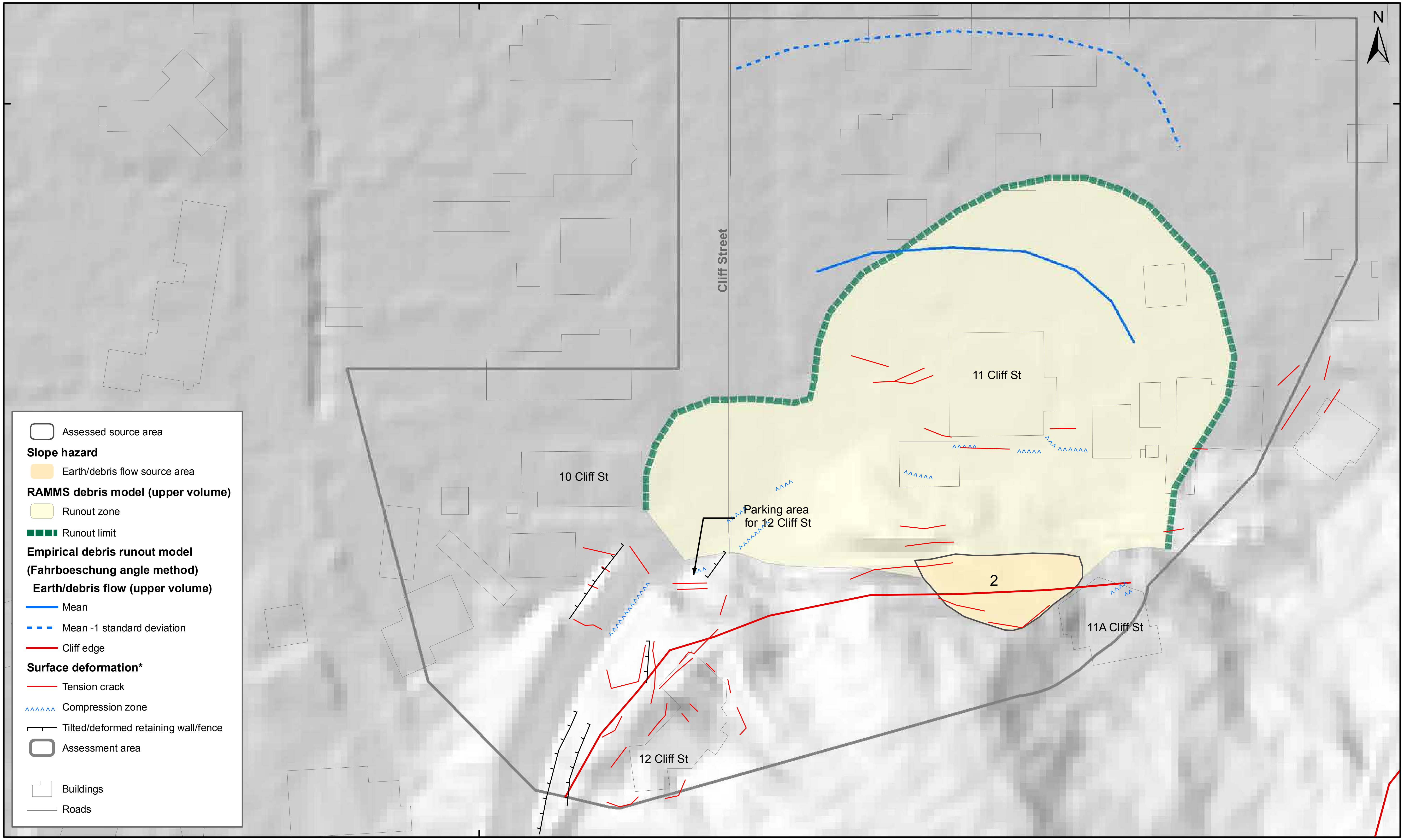
FIGURE 42

Map 2

FINAL

REPORT: CR2014/73

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 (Source area 2)

Cliff Street Christchurch

FIGURE 42

Map 3

FINAL

REPORT: CR2014/73

DATE: June 2014

5.0 RISK ASSESSMENT RESULTS

5.1 TRIGGERING EVENT FREQUENCIES

Failure of the assessed source areas could be triggered by earthquakes (dynamic conditions) or by water ingress (static conditions). Earthquakes are thought to be the main triggering event for source area 1, and rainfall for source area 2.

5.1.1 Source area 1 – frequency of earthquake triggers

For earthquake triggers, the frequency of a given free-field peak ground acceleration (A_{FF}) occurring is obtained from the New Zealand National Seismic Hazard Model (Table 20) (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).

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

Table 20 The annual frequency of a given peak ground acceleration (PGA) band occurring on rock (site class B) for different years from the 2012 seismic hazard model for Christchurch (G. McVerry, personal communication 2014). Note: these are free field rock outcrop peak ground accelerations (equivalent to A_{FF}).

Annual frequency of the representative earthquake event in a given PGA band				
PGA Band (g)	**0.1–0.4**	**0.4–1.0**	**1.0–2.0**	**>>2**
Year 2016 annual frequency of event	0.16	0.03	0.0016	0.00005
Next 50-year average annual frequency of event	0.08	0.01	0.0007	0.00002

5.1.1.1 Peak ground acceleration and permanent slope displacement

The probability of source area 1 being triggered in a given earthquake was based on the calculated permanent displacement, estimated from the decoupled results.

It is difficult to estimate the probability of triggering failure, leading to catastrophic slope collapse, where the debris runs out down slope forming a debris avalanche. It is also possible that permanent slope displacements could cause catastrophic damage to dwellings located at the cliff crest, even if the debris does not leave the source. The level of displacement chosen to differentiate between safe and unsafe behaviour (Abramson et al., 2002) differs between authors. 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.

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 32). Different levels of peak ground acceleration were adopted based on the four earthquake event bands, 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 0.7 g would have an equivalent K_{MAX} of 1.2 g, assuming a ratio of 1.8 (Table 21).

5.1.1.2 Permanent slope displacement and likelihood of catastrophic slope failure

The probability of occurrence of each local source area (1–3) was based on the estimated permanent displacement, estimated from the decoupled results (Figure 32), as follows:

- If the estimated displacement of the source area is ≤0.1 m then the probability of catastrophic failure = 0.
- If the estimated permanent displacement of the source area is ≥1.0 m then the probability of catastrophic failure = 1.
- If the estimated permanent displacements are between 0.1 m and 1 m then the probability of failure (P) is calculated based on a linear interpolation between P = 0 at displacements of 0.1 m, and P = 1, at displacements of 1 m.

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 near field earthquakes of short duration but high amplitude. The calculated displacements in Figure 32 represent displacements in response to these earthquakes. Earthquakes of longer duration will affect the site in different ways. For example, the response of the slope (at higher water contents representative of winter conditions) may be non-linear, and could lead to larger cumulative 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 rock slope.

Table 21 Probability of failure for source area 1. Estimated displacements are rounded to the nearest 0.1 m, and are based on the relationship between Ky/K_{MAX} shown in Figure 32.

Adopted yield acceleration (*Ky*) (g)	0.15			
PGA band (A_{FF})[1]	1	2	3	4
PGA (A_{FF}) range of band (g)	0.1–0.4	0.4–1.0	1.0–2.0	2.0–5.0
PGA of representative event in band (g) (A_{FF}):	0.25	0.7	1.5	3.5
Year 2016 annual frequency of representative event in band	0.16	0.03	0.0016	0.00005
Adopted K_{MAX} to A_{FF} ratio	1.8 (mean + 1 standard deviation)			
Representative equivalent maximum average acceleration (K_{MAX}) of each band (g)[2]	0.3	1.2	2.6	6.1
Estimated displacements (m)	0.0	0.2	0.3	0.4
Probability of failure	**0.0**	**0.1**	**0.2**	**0.4**

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

2 K_{MAX} represents the maximum average acceleration of the failure mass.

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 10 km, of the Cliff Street site.

The annual frequencies of a given level of peak ground acceleration occurring in the area are given by the National Seismic Hazard Model of New Zealand (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 zone; 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.

Buxton and 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 National Seismic Hazard Model. These earthquakes are similar to the 22 February, 16 April 13 June and 23 December 2011 earthquakes.

5.1.2 Source area 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 area 2. For rainfall (static) triggers:

- For the risk assessment, various return periods of 20, 50, 100 and 200 years for the rainfall 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 at the cliff top.

5.2 RISK ASSESSMENT RESULTS

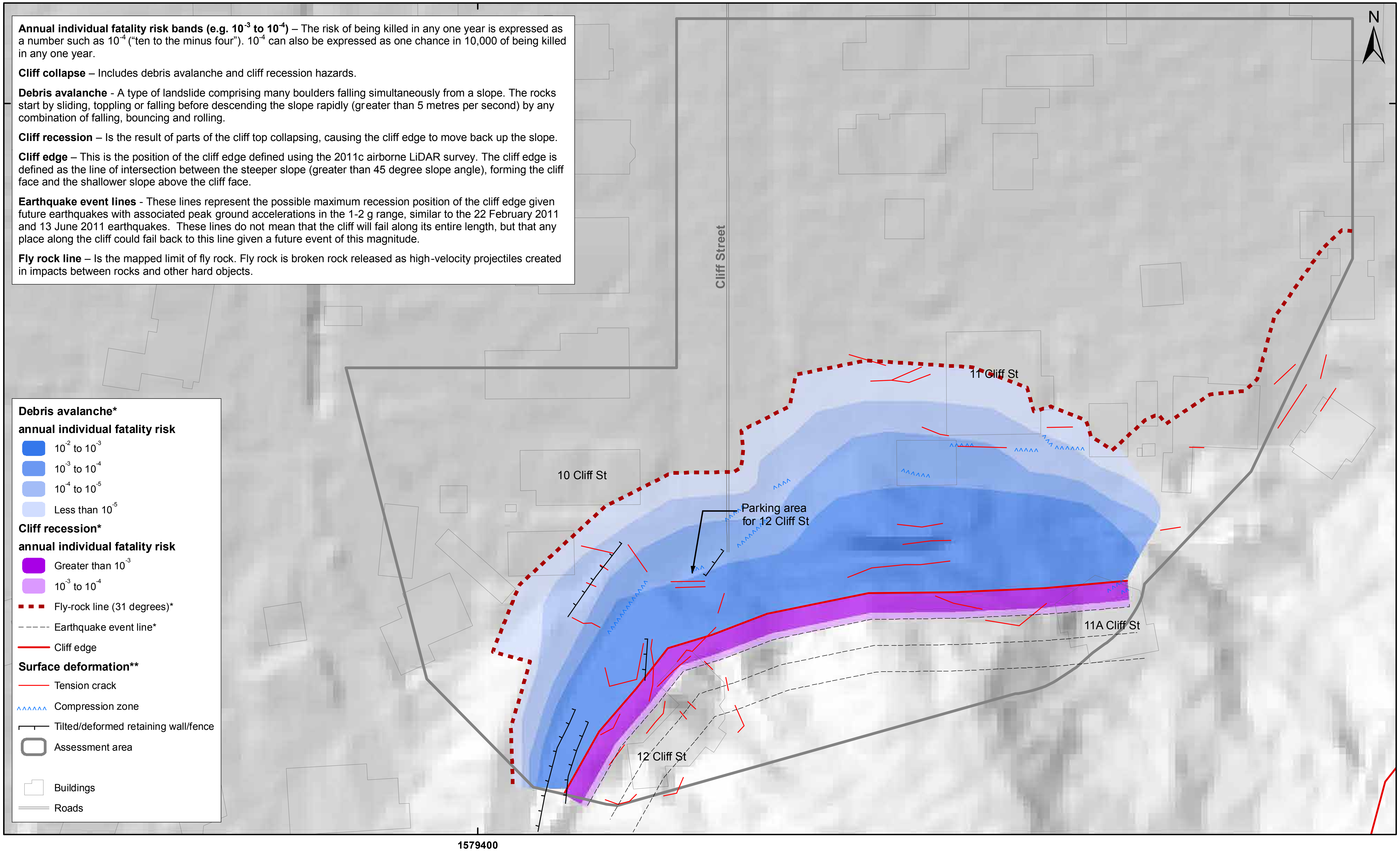
The results from the risk assessment are shown in Figure 43 (Maps 1–4) as the annual individual fatality risk. Map 1 shows the original annual individual fatality risk estimated for cliff collapses by Massey et al. (2012a). Map 2 shows the estimated annual individual fatality risk from debris avalanches associated with source area 1. Map 3 shows the estimated annual individual fatality risk from earth/debris flows associated with source area 2.

Map 4 in Figure 43 shows the annual individual fatality risk from combining the results shown in Maps 1–3, to produce a map showing the total risk from the combined different hazards present at the site.

5.2.1 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 43 adopt the following main variables:

- $P_{(H)}$ for earthquake triggers the annual frequency of the triggering event adopt the 2016 seismic hazard model results, which include aftershocks.
- $P_{(S:H)}$ 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 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.
- $V_{(D:T)}$ for debris avalanches hazards, the vulnerability of a person, if present and inundated by debris, is a constant vulnerability factor of 70% has been adopted for this risk assessment. A constant vulnerability value is thought reasonable as the velocity of the boulders, even in the distal runout zone are still relatively high with people unlikely to be able to get out of the way. For earth/debris flow hazards, a variable vulnerability has been used based on the velocity of the debris (Appendix 1).



SCALE BAR: 0 25 50 m

EXPLANATION:
* Modified from report 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

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CLIFF COLLAPSE
ANNUAL INDIVIDUAL FATALITY RISK
(From GNS Science Report CR2012/124)

Cliff Street
Christchurch

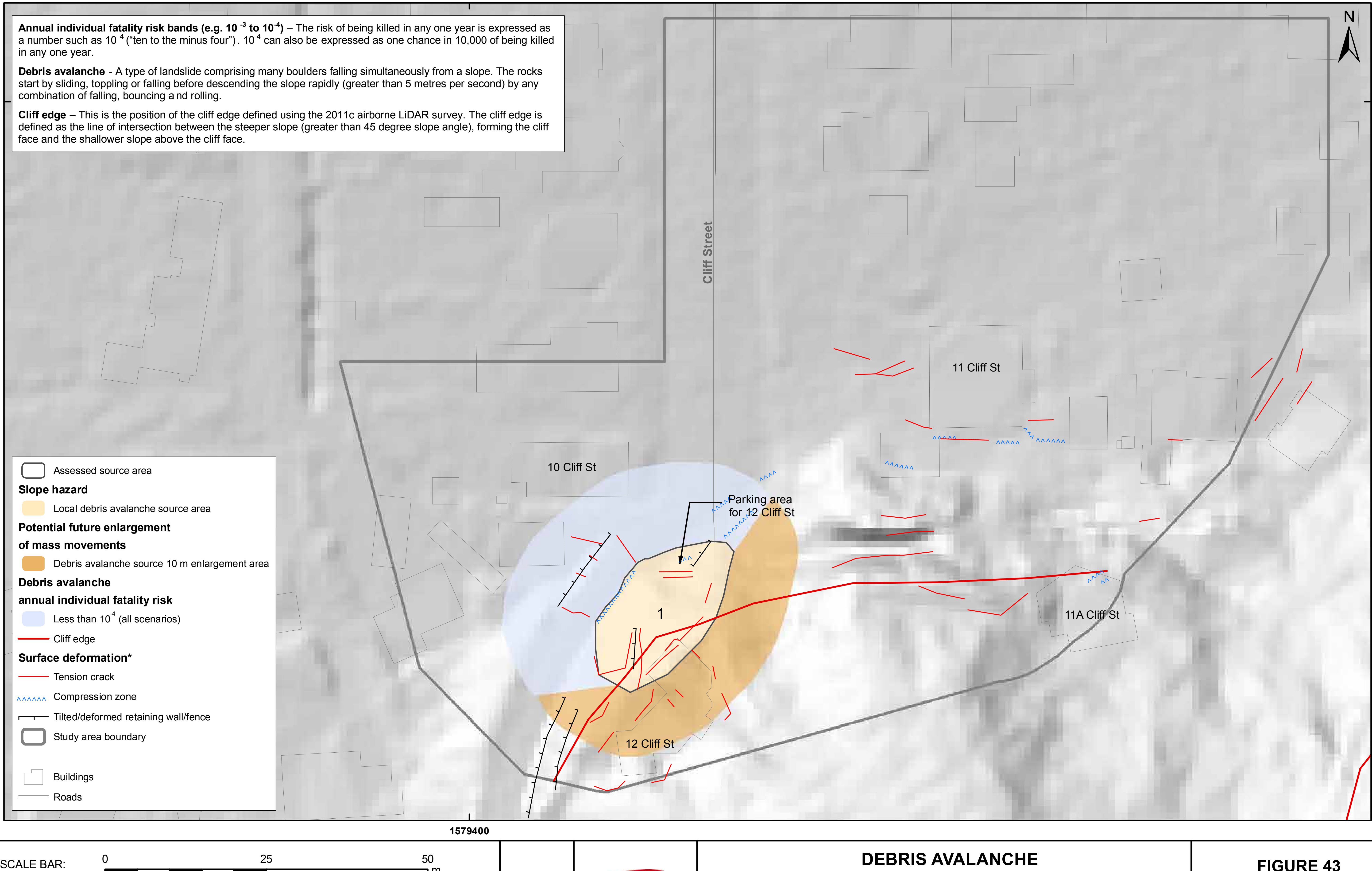
FIGURE 43

Map 1

FINAL

REPORT: CR2014/73

DATE: June 2014



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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DEBRIS AVALANCHE
ANNUAL INDIVIDUAL FATALITY RISK
(Source area 1)

Cliff Street
Christchurch

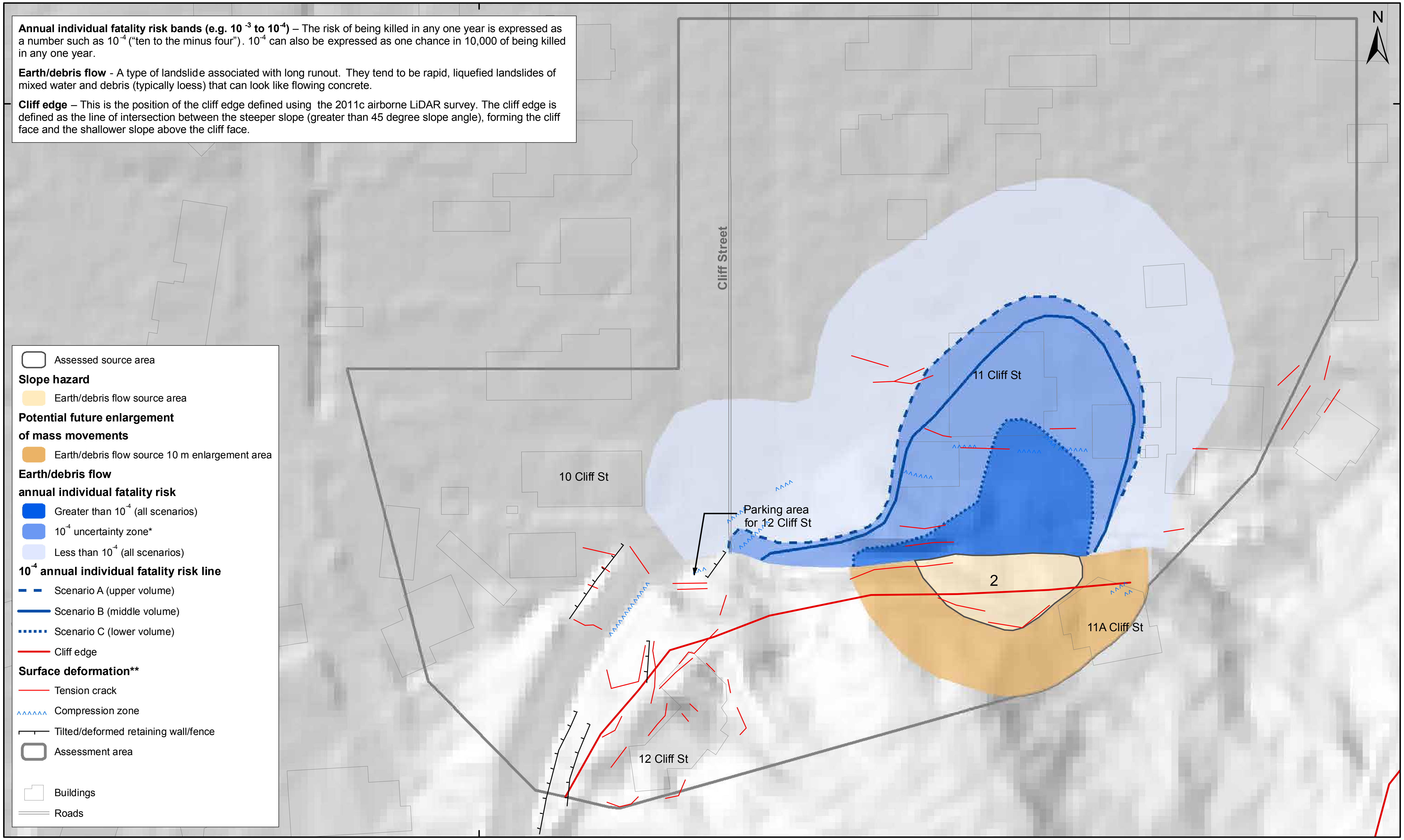
FIGURE 43

Map 2

FINAL

REPORT: CR2014/73

DATE: June 2014



1579400

SCALE BAR: 0 25 50 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: CM, FDP

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EARTH/DEBRIS FLOW ANNUAL INDIVIDUAL FATALITY RISK (Source area 2)

Cliff Street Christchurch

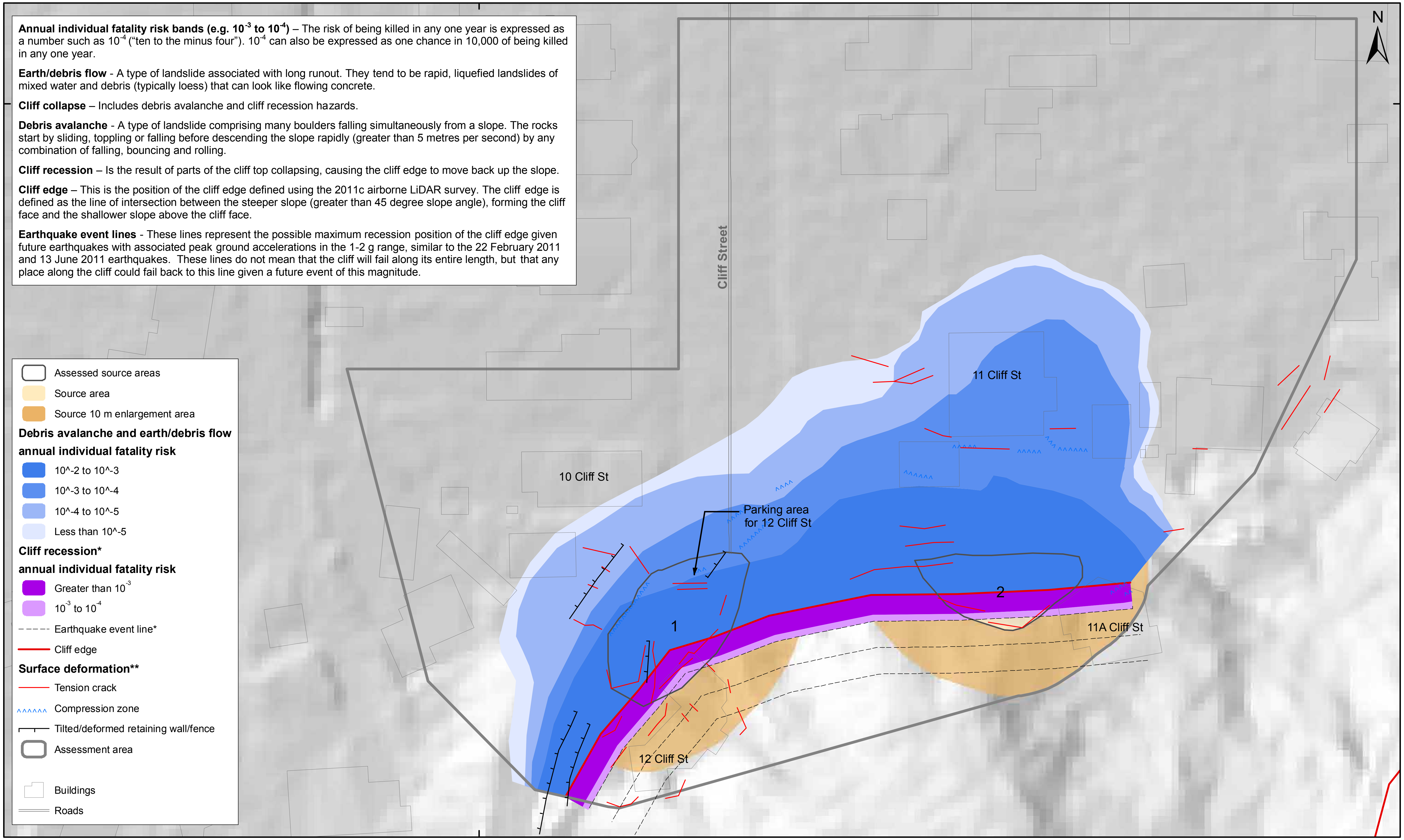
FIGURE 43

Map 3

FINAL

REPORT: CR2014/73

DATE: June 2014



SCALE BAR: 0 25 50 m

EXPLANATION:
* Modified from report CR2012/124
** Taken from report CR2012/317
The results combine the annual individual fatality risk modified from report 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 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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COMBINED CLIFF COLLAPSE AND EARTH/DEBRIS FLOW ANNUAL INDIVIDUAL FATALITY RISK

Cliff Street
Christchurch

FIGURE 43

Map 4

FINAL

REPORT: CR2014/73

DATE: June 2014

5.2.2 Debris avalanches (source area 1)

The risk from debris avalanches from source area 1 was estimated for upper, middle and lower source volumes.

The risk from the debris avalanche, source area 1, adopting the estimated lower, middle and upper source volumes (scenarios A–C respectively) are shown in Figure 43, Map 2. The range of annual individual fatality risks estimated for scenarios A–C (upper to lower source volumes, Table 3), are all less than 10^{-4}. This is because the estimated displacement, and therefore the probability that the source area will fail, and the debris runout to form a debris avalanche, is relatively low.

A 10-m wide strip is added at the crest of the source area, to allow for the future retrogression of the cliff in an up-slope direction, beyond the currently assessed extent. This has been termed a “debris avalanche source 10 m enlargement area”. This area allows for any retrogression/erosion of the source area further back (up slope) from the currently assessed source-area boundary.

It should be noted that the risk from source area 1 is additional to the risk from the smaller cliff collapses assessed in Massey et al. (2012a).

5.2.3 Earth/debris flows (source area 2)

The risk from earth/debris flows originating from the loess slope, adopting the estimated lower, middle and upper source volumes (scenarios A–C respectively) are shown in Figure 43, Map 3.

A 10-m wide strip is added at the crest of the source area, to allow for the future retrogression of the cliff in an up-slope direction, beyond the currently assessed extent. This has been termed an “earth/debris flow source 10 m enlargement area”.

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

The area shown as the “10^{-4} uncertainty zone” represents the area of slope where the risk could be equal to or greater than 10^{-4} in some scenarios, but less than 10^{-4} in other scenarios.

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 assed as being less than 10^{-4} for all scenarios.

5.2.3.1 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 44.

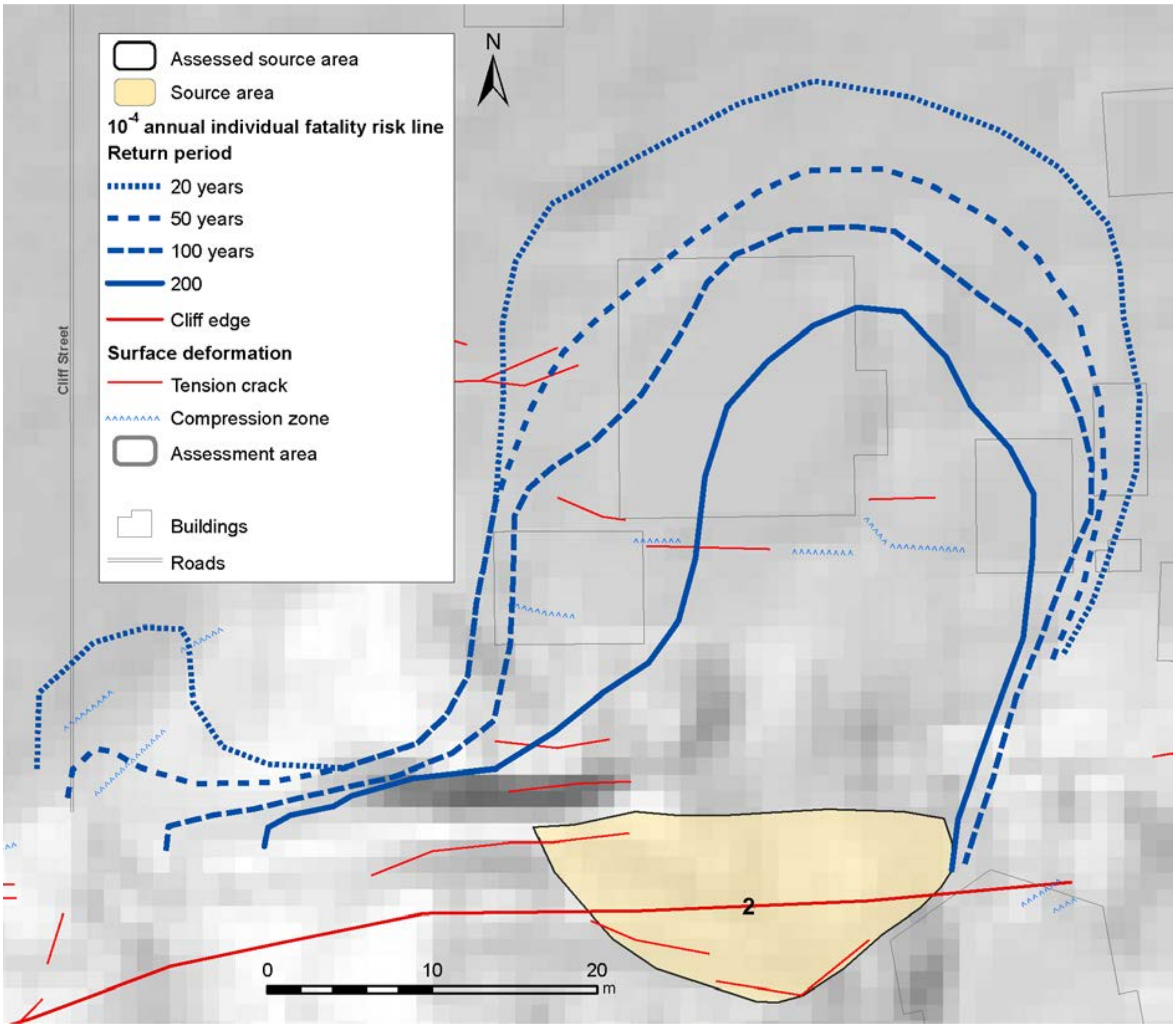

Figure 44 Sensitivity of the risk estimates, upper volume estimates, for triggering event return periods of 20, 50, 100 and 200 years.

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 included within the 10^{-4} annual individual fatality risk contour. Therefore the 50-year return period adopted for the risk estimates shown in Figure 40 is considered reasonable.

5.2.4 Combined risk from slope instability

The total risk from the combined hazards is shown in Map 4 of Figure 43. The results combine the risk presented in Massey et al. (2012a) from cliff collapse of the slope with the annual individual fatality risk estimated adopting scenario B (the middle volume estimates) for both the debris avalanche and earth/debris flow hazards (source areas 1 and 2 respectively) assessed in this report.

The original risk assessment in Massey et al. (2012a) was updated to make it consistent with the input parameters discussed in Massey et al. (2012c) and 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%.

It should be noted that the risks presented in Massey et al. (2012a) were estimated for smaller cliff collapse (debris avalanche) source volumes and different failure mechanisms to the ones assessed for source area 1. Earth/debris flows triggered under static conditions were also not included in the original cliff collapse risk assessment.

6.0 DISCUSSION

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

1. The results from the risk assessment, taking into account the debris avalanche hazard (at the west of the site, source area 1) and the earth/debris flow hazard (at the east of the site, source area 2), have only increased the level of risk at those dwellings that were already within the original cliff collapse risk areas, presented in Massey et al. (2012a). No additional dwellings are within the revised risk zones.
2. The annual frequency of the event that could trigger the earth/debris flow (source area 2) is not known, or indeed whether such events could be triggered. However, a 50-year return period (annual frequency of 0.02) has been assumed after considering the uncertainties associated with the loess shear strength parameters and how these may degrade over time.

6.1 RISK ASSESSMENT SENSITIVITIES AND UNCERTAINTIES

In this section, the sensitivity of the risk model to key uncertainties and reliability of the assessments are discussed.

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

1. The volumes of debris that could be triggered in future events. This was done by comparing the three volume ranges which accounted for variation in the likely source volumes. The three volume ranges also took 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. Debris avalanches at the west of the site (source area 1): There is little difference in the risk between the upper and middle volume estimates, indicating that variations in the source volume have a very limited effect on the risk, as the debris does not tend to runout very far – a function of the granular nature of the rock dominated debris.
    b. Earth/debris flow at the east of the site (source area 2): The main difference in the position of the 10^{-4} risk contour is between the lower volume and the upper volume (there is little difference between the lower and middle volumes), indicating that source volume does have a large effect on the risk. The runout of earth/debris flows is very sensitive to the source volume and the nature of the slope over which the debris passes
2. Changes to the annual frequency of the event that could trigger failure of the source:
    a. Debris avalanches at the west of the site (source area 1): Risk models were run using the 50-year average seismic event annual frequencies (Table 21). Results from the assessment show that risk would reduce by a factor of about two (a factor of 10 is one order of magnitude).

b. Earth/debris flow at the east of the site (source area 2): Risk models were run for event annual frequencies of 0.03, 0.01, 0.005 and 0.002, corresponding to return periods of 30, 100, 200 and 500 years respectively. Results from the assessment show that there is little change between the risk results adopting the 50-year and 100-year return periods. For return periods greater than 100 years the 10^{-4} annual individual fatality risk uncertainty zone reduces incrementally back up slope towards the toe of the rock slope. For a return period of 30 years the 10^{-4} annual individual fatality risk uncertainty zone is further out from the toe of the rock slope increasing the risk at 11 Cliff Street.

The results (Figure 44) show that largest impact on the risk is from the volumes of material that could be generated in an event, and secondly from the annual frequency of the event that could trigger them. These combine for the debris avalanche risk (source area 1) to give slightly less than an order of magnitude uncertainty, in either direction, on the risk estimates. For the earth/debris flow risk (source area 2), these combine to give about an order of magnitude uncertainty, in either direction, on the risk estimates.

It should be noted that these risks are additional to those already estimated from cliff collapse by Massey et al. (2012a), and only marginally increase the risk to the dwellings already identified as being at potentially unacceptable levels of risk in the original estimates.

6.1.1 How reliable are the results?

Potentially significant uncertainties noted and their likely implications for risk are summarised in Table 22. The sensitivity results are reported in Table 22 as “factors” of 2, 3 etc. These factors represent the estimated variability in the risk value for the given issue. Given that the risk is quoted in numbers of 10^{-4} etc., a factor of 10, would relate to an-order-of magnitude variability, where a risk of 10^{-4} could be 10^{-3} or 10^{-5}.

Table 22 Uncertainties and their implications for risk.

Issue	Direction and scale of uncertainty	Implications for risk
Debris avalanches (source area 1)		
a. Choice of whether to use different earthquake event annual frequencies from year 2016 to 50-year average.	Moderate uncertainty between the use of the year 2016 and 50-year average earthquake event frequencies.	Longer term risk is potentially a factor of 2–3 times lower.
b. Volume of debris produced by a source area.	Largest uncertainty between estimated volumes.	A factor of about 4 in either direction.
c. Changes to the probability of failure of each source area from $P_{FAILURE}$ = 0 = displacement of 0.1 m, to $P_{FAILURE}$ = 0 = 0.3 m of displacement.	Small uncertainty in either direction.	About a factor of 1.2 in both directions.
d. Changing the assumed debris height where probability of inundation = 0 from 0.1 m to 0.3 m and 0.1 m.	Small uncertainty in either direction.	Would change modestly by a factor of about 1.1 in either direction.

Issue	Direction and scale of uncertainty	Implications for risk
e. 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.
f. Vulnerability (probability of being killed if inundated by debris).	Assumption of 100% instead of 70% would modestly increase the estimated risk.	Would increase modestly by a factor of about 2. Dwelling construction and age/ability of the person to get out of the way of the debris have a significant effect on the risk.
Earth/debris flows (source area 2)		
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 50-year and 200-year, return periods.	Longer term risk is potentially 2–4 times lower, but shorter term risk could be 3 times higher.
b. Volume of debris produced by a source area.	Largest uncertainty between Scenario A (upper volume) and C (lower volume), and Scenario A and B (middle volumes).	About a factor of 5 between scenarios C and A and C and B. But a factor of 1.1 between scenarios B and A.
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.

7.0 CONCLUSIONS

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

7.1 HAZARD

1. There is the potential for large cliff collapses, larger than those assessed in Massey et al., 2012a) and earth/debris flows to occur at the site:
   a. at the west end of the site, there is potential for cliff collapses of around 1,000–2,500 m^3, largely of rock (corresponding to assessed source area 1); and
   b. on the eastern slope, there is the potential for volumes ranging up to several hundreds of cubic metre of earth/debris flows of mixed soil and weathered rock (corresponding to assessed source area 2).
   c. much of the rock-slope face appears unstable and rocks fall from the slope with no apparent trigger, indicating that parts of the slope face are only marginally stable to unstable, with factors of safety much less than those assessed for the deep-seated failures a) and b) above.
2. The most likely triggers for these newly identified landslide sources are earthquake ground shaking for the cliff collapses and prolonged heavy rainfall for the earth/debris flows.
3. The frequency of landslide events from these sources is difficult to estimate and could be anything from once in a few tens to once in many hundreds of years.

7.2 RISK

The results from the risk assessment, taking into account the debris avalanche hazard (at the west of the site, source area 1) and the earth/debris flow hazard (at the east of the site, source area 2), have only increased the level of risk at those dwellings that were already within the original cliff collapse risk areas, presented by Massey et al. (2012a). No additional dwellings are within the revised risk zones.

7.3 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.
   c. There is currently no precedent data for rates of change of groundwater or water content of loess to provide reliable alert criteria.
2. It should be noted that slope material strengths, and thus factors of safety, may be expected to deteriorate with time, weathering and any further earthquakes.

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 will be regarded as tolerable.
2. Decide how Council will manage risk on land 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);
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 not recommended as it 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

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

a. 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, the damaged storm water systems at the crest of the assessed site, and should if possible be relocated away from this area; and
b. Control surface water seepage by filling the accessible cracks on the slope and providing an impermeable surface cover to minimise water ingress. However, it is not thought that such works alone are sufficient to reduce the risk.

8.3 LONG-TERM ACTIONS

8.3.1 Engineering measures

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;

d. Withdrawal and rezoning of the land for non-residential use; or

e. Any proposed engineering works would require a detailed assessment and 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. It should be noted that the uncertainty relating to the failure mechanisms at source area 1 requires further investigation before engineering solutions can be designed.

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 changes in ground conditions; or

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

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APPENDICES

A1 APPENDIX 1: METHODS OF ASSESSMENT

A1.1 HAZARD ASSESSMENT METHODOLOGY

A1.1.1 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.

A1.1.1.1 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 variable depths within the loess, and to extend to rockhead.

Models were run based on geological cross-sections 1 and 2. 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 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.

A1.1.1.2 Dynamic stability assessment

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 Cliff Street 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 of the loess and rock were derived from drillhole surveys 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 Moa Bone (Redcliffs (Southern Geophysical Ltd., 2013). The shear wave velocity of the loess was estimated from tests carried out by Tonkin and Taylor (2012a).

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 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.

A1.1.1.3 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 Estimation of slope failure volumes

The results of the URS Ltd., engineering geological assessments (Yetton, 2014) and the slope stability modelling carried out by GNS Science identified that there is potential for:

1. Local failures of colluvial loess, loess, fill and rock from the eastern slope, particularly in areas where the bulk loess mass strength is degraded by earthquake-induced cracks.
2. Local large failure of the rock slope (cliff collapse) at the western end of the mass movement area, where displacement appears to be occurring along persistent clay filled discontinuities (cracks/defects in the rock, often referred to as a “joints”).
3. Relatively shallow creep of the loess further up slope away from the edge of the cliff, which are not assessed in this report, as they are not considered to have significant implications for life risk to occupants of nearby properties.

The most likely locations and volumes of potential failures were estimated based on the numerical analyses, current displacement magnitudes inferred from 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 (such as breaks in slope), 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 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 Defender Lane assessment area 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 1870–1945 (a selection of which is presented in Appendix 3);
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.) which incompletely covers the period from 1968 to present (McSaveney et al., 2014).

A1.1.3 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 is based on failure points starting at the cliff crest.
    b. The volume of debris/earth flows and debris avalanches passing a given location within the study area is based on an empirical relationship established from a compilation of run out distances from published international and local (in the Port Hills) earth/debris flows and debris avalanches. Two different empirical models were used based on the dominant type of debris movement.
        - For earth/debris flows, which typically comprise finer-grained material and tend to be fluid, the empirical relationship is based on a data set of over 700 earth/debris flows from New Zealand (including the Port Hills and Banks Peninsula) and overseas, compiled by Massey and Carey (2012).
        - For debris avalanches, which typically comprise coarser grained material and tend to be less fluid and do not run out as far as flows, the empirical relationship is based on data from about 45 debris avalanches that fell from Port Hills slopes during the 2010/11 earthquakes (Massey et al., 2012b).
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). Two different sets of parameters were used depending on whether the failure was assessed to be a flow or avalanche.
        - *Earth/debris flows:* The model was calibrated by "back-analysing" the runout of 5 Port Hills and Banks Peninsula debris flows and the modelled parameters optimised to obtain a good correlation between the modelled versus actual runout.
        - Debris avalanches: The model was calibrated by "back-analysing" the runout of 24 Port Hills debris avalanches that fell during the 2010/11 earthquakes, and the modelled parameters optimised to obtain a good correlation between the modelled versus actual 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.

b. 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 – refer to Section 4.3 for details of the calibration.

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

A1.2 RISK ASSESSMENT

During the 2010/11 Canterbury earthquakes several boulders (about 10) and relatively small volumes of loess (less than 50 m^3) fell from the slopes at Cliff Street. During the earthquakes the slopes also moved, causing cracks to open in the soil and rock mass.

The risk from cliff collapses falling from the slope was originally assessed by Massey et al. (2012a). In addition to the identified cliff collapse risk at the site, further investigation at Cliff Street has identified potential for:

1. Local large (deep-seated) cliff collapse of the rock slope at the western end of the site, which could form a debris avalanche larger than the previously assessed (Massey et al., 2012a) smaller cliff collapses; and
2. Earth/debris flows occurring in loess, colluvial loess, fill and weathered rock from the eastern slope, which could form earth/debris flows (source area 2).

The risk metric assessed is the annual individual fatality risk and this is assessed for dwelling occupants from the assessed cliff collapses and earth/debris flows in initiating from the slopes within the assessment area. Cliff-collapse hazards (comprising debris avalanches and cliff-top recession) within the assessment area were previously assessed by Massey et al. (2012a), and these results are combined with the results in this report, to present combined risk estimates for all of the assessed landslide hazards.

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.

A1.2.1 For debris avalanches (source area 1):

1. Divide the entire study area into a series of 1 m by 1 m grid cells.
2. Consider the possible range of triggering events, in terms of a set of earthquake triggers, and choose a small set of representative earthquake events spanning the range of severity of events from the smallest to the largest.
   - Results from the numerical assessments suggest that it is unlikely that large failure of the rock slope would occur during rainfall events – although smaller rockfalls could still occur, which are included in the original risk assessment for the site (Massey et al., 2012a).
   - Four earthquake peak ground acceleration (PGA) event bands were adopted (0.1–0.4 g, 0.4–1 g, 1–2 g and 2–5 g), as per those in the original risk assessment (Massey et al., 2012a).

- The main source area (source area 1 at the western end of the site) was characterised based on the evidence and assessment collected to date, and 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.

3. Estimate the probability of occurrence (catastrophic failure) of source area 1 within each representative earthquake peak ground acceleration band, as a function of the magnitude of permanent slope displacement estimated from the decoupled results, as follows:
   a. If the estimated displacement of the source is ≤0.1 m then the probability of catastrophic failure was assumed to be zero. Meaning that the source area is unlikely to fail catastrophically if permanent displacements are ≤0.1 m. This was based on measurements of Port Hills slopes that underwent permanent displacement (i.e., cracking) in the 2010/11 Canterbury earthquakes, but where the displacement magnitudes were <0.1 m and where catastrophic failure did not occur.
   b. If the estimated permanent displacement of the source ≥1.0 m then the probability of catastrophic failure was assumed to be 1. Meaning that the source area is likely to fail catastrophically if displacements are ≥1 m.
   c. If the estimated permanent displacements are between 0.1 m and 1 m then the probability of catastrophic failure (P) is assumed to follow a linear interpolation between P = 0 at displacements of 0.1 m, and P = 1, at displacements of 1 m.
4. For each representative event, within each scenario, estimate:
   a. The frequency of the event, the probability of failure and the volume of debris produced in that event, for a given source scenario ($P_{(H)}$).
   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 ($P_{(S:H)}$).
   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 ($V_{(D:T)}$)
5. Multiply 4(a)–(d) for each source area scenario to estimate the annual individual fatality risk at different locations below the slope.
6. These values were then modelled using ArcGIS®. ArcGIS is used to interpolate between the risks 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).
7. 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 (Massey et al., 2012c).

A1.2.2 For earth/debris flows (source area 2):

1. Divide the entire study 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 (source area 2) 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.
    a. 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, was equivalent to a 10–20 year return period rain event, which did not generate any significant earth/debris flows.
    b. 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.
    c. The main source area was characterised based on the evidence and assessment collected to date, and estimates of the likely failure volumes were made.
    d. Three scenarios were considered based on: 1) lower; 2) middle; and 3) upper estimates of the source volume.
    e. Each volume scenario was assessed as having an equal probability of failure in a given event and a 100% probability of failure in the event.
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 ($P_{(S:H)}$).
    c. The probability that a person is present at a given location in their dwelling as the debris moves through it ($P_{(T:S)}$). This probability has been linked to landslide intensity (Du et al., 2013).
    d. The probability that a person is killed if present and inundated by debris ($V_{(D:T)}$).
4. Multiply 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.
5. These values were then modelled using ArcGIS®. ArcGIS is used to interpolate between the risks 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).
6. 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.

A1.2.3 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:

For debris avalanches (source area 1)

1. Probability of inundation $P_{(INUN)}$ = 0 if the maximum height of the debris reaching/passing the grid cell is ≤ 0.1 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 0.1 m but less than 1.0 m, adopting a linear interpolation.

For earth/debris flows (source area 2)

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 difference between the inundation height probabilities adopted for the assessments reflects the dominant movement mechanism and nature of the debris associated with the different hazards. Debris avalanches, sourcing from the rock slopes, comprise multiple boulders of rock, rather than a fine-grained deposits of soil as per the debris derived from the earth/debris flows sourcing from the soil slopes. An earth/debris flow 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. On the other hand, a debris avalanche has significantly more mass than soil and a debris height of 0.3 m or less would still be likely to kill a person if hit by boulders.

A1.2.4 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 risk assessment (contained in Massey et al., 2012a and c) 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 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:

$$P_{(T;S)} = \frac{(0.67)}{(A_F / P_A)}$$ **Equation** 2

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.

A1.2.5 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 a person indoors, it would be unlikely that they 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 are 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. However, for debris avalanches triggered by earthquakes people may be outside in gardens (where two deaths occurred from falling rocks triggered by the 22 February 2011 earthquake).

A1.2.5.1 For debris avalanches (source area 1)

The probability of a person being killed if hit by a boulder is related to the landslide intensity, which in turn is a function of the number and velocity of boulders passing through a given location. In this risk assessment, the probability of a person being hit by N boulders within the debris (should a person be present) has been calculated separately as $P_{(S:H)}$.

Debris velocities derived from RAMMS model outputs are typically >5 m/s for most of the runout areas assessed. However, the velocity rapidly drops to <0.05 m/s in the distal limits of runout over a relatively short distance of several metres. These calculations are similar to field observations made from video footage although some boulders within the distal debris fringe (mainly individual boulders) travelled at higher velocities, i.e., “fly rock”. Fly-rock may occur when moving blocks impact and fracture resulting in high velocity rock fragments being released.

The two-dimensional rockfall modelling (Massey et al., 2014) suggests that boulder velocities in the distal runout zone are still in the range of about 3–5 m/s and not <0.5 m/s as suggested by RAMMS. Such velocities are more consistent with field observations. At boulder velocities of about 5 m/s (18 km/hr), it is unlikely that a person could get out of the way of a boulder (Australian Geomechanics Society, 2007).

Based on these results, a vulnerability factor of 70% has been adopted for this risk assessment as it was the factor adopted by the Canterbury Earthquake Recovery Authority for the previous risk assessments. A constant vulnerability value is thought reasonable as the velocity of the boulders, even in the distal runout zone are still relatively high with people unlikely to be able to get out of the way. The protective effects of buildings have not been taken into account, this is because most people killed by falling boulders during the 22 February 2011 earthquake were outside and therefore not protected by buildings. However, it is noted that buildings do have a sheltering effect as only 45% of buildings hit by boulders were penetrated (Massey et al., 2012c).

A1.2.5.2 For earth/debris flows (source area 2)

For loess earth/debris flows where the debris tends to be very fluid, it is likely that homes (even wooden ones) can 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 is 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 3). 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 A1.1.

Table A1.1 Vulnerability factors for different debris velocities used in the risk assessment.

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

A2 APPENDIX 2: PAST LANDSLIDES 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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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.

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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.

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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.

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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.

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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 !"

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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.

A3 APPENDIX 3: 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 cliff crest derived from the modelling of each synthetic earthquake time history has been plotted in Figure A3.1. Each point on the graph represents the response of the slope crest to a given synthetic free field rock outcrop earthquake input motion (Table A3.1).

The fundamental frequency of the slope varies from 8.5 to 10.6 Hz based on the equation in Bray and Travasarou (2007), where frequency = 1/(4 x H/V_S), and H = slope height of 20–25 m, and V_S = average shear wave velocity for the slope of 850 m/s. The dominant frequency of the input motions is 3.6 Hz and then 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 0.3–0.4 for a slope height of 20 m, and 0.5–0.7 for a slope height of 25 m.

Results from the seismic response assessment suggest that the peak ground acceleration amplification factors (S_T) for Cliff Street (cross-section 1) are about 1.7 (±0.1) – errors at one standard deviation for horizontal motions, and 1.4 (±0.1) for vertical motions – errors at one standard deviation (Table A3.1).

Table A3.1 Results from the two-dimensional site response assessment for cross-section 1, using the synthetic free field rock outcrop motions for the Richmond Hill 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 slope crest – A_{MAX} (g)	Maximum PGA (vertical) at slope crest – A_{MAX} (g)
22 February	0.60	0.68	1.0	0.9
16 April	0.04	0.03	0.1	0.1
13 June	0.33	0.35	0.5	0.5
23 December	0.23	0.11	0.4	0.2

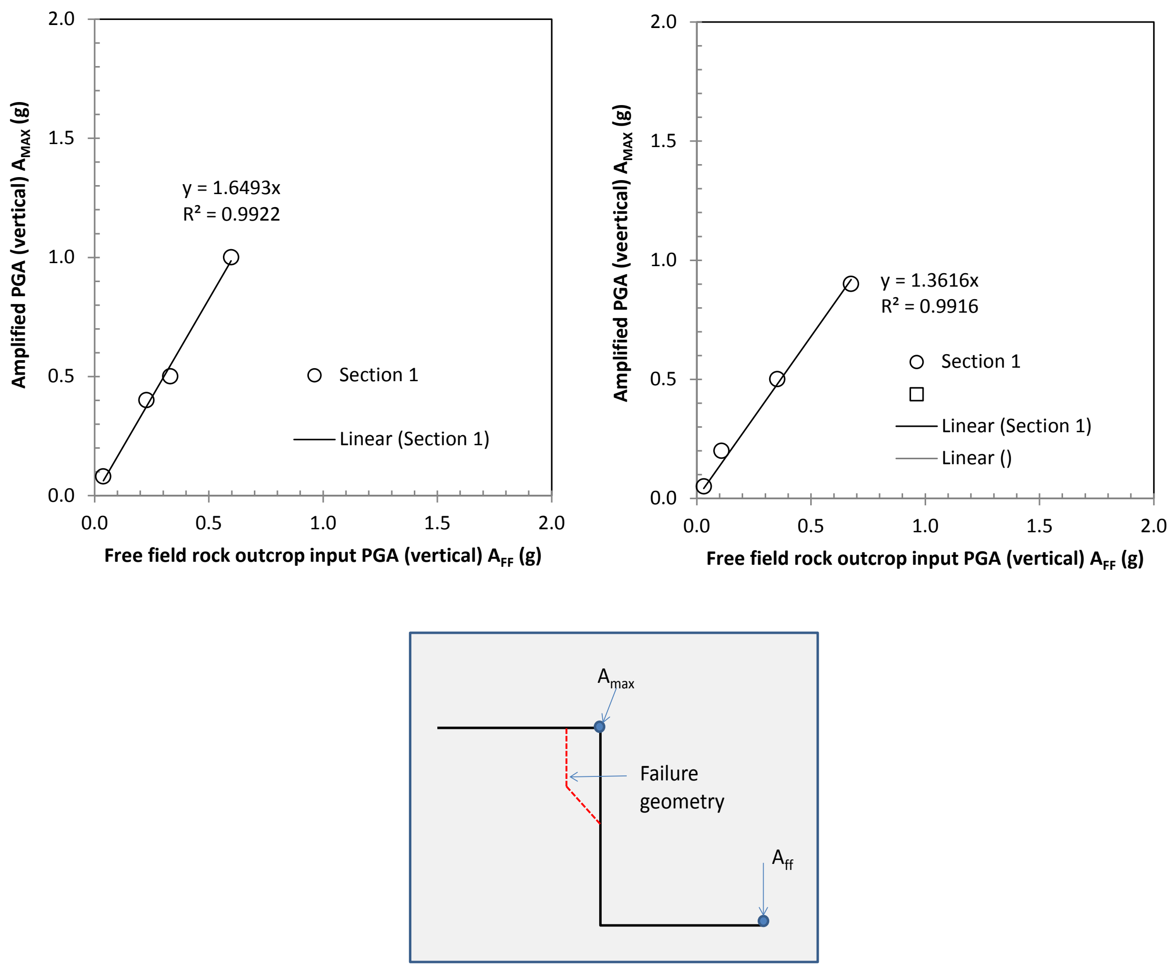


Figure A3.1 Amplification relationship between the synthetic free field rock outcrop input motions (A_{FF}) and the modelled cliff crest maximum accelerations (A_{MAX}) for cross-section 1. A schematic diagram showing the locations of the various recorded accelerations is shown.

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 generally linear. In the range of modelled peak horizontal accelerations, the mean horizontal amplification factor (S_T) is typically in the order of about 1.7 times the input free field peak horizontal acceleration.

The relationship between the modelled vertical and horizontal peak ground accelerations recorded at the slope crest (A_{MAX}) is shown in Figure A3.2. The gradient of the linear fit (cross-section 1) is 0.87 (±0.01) – errors at one standard deviation, with a gradient of 0.9 being the mean plus two standard deviations. The relationship between horizontal and vertical peak ground accelerations appears linear.

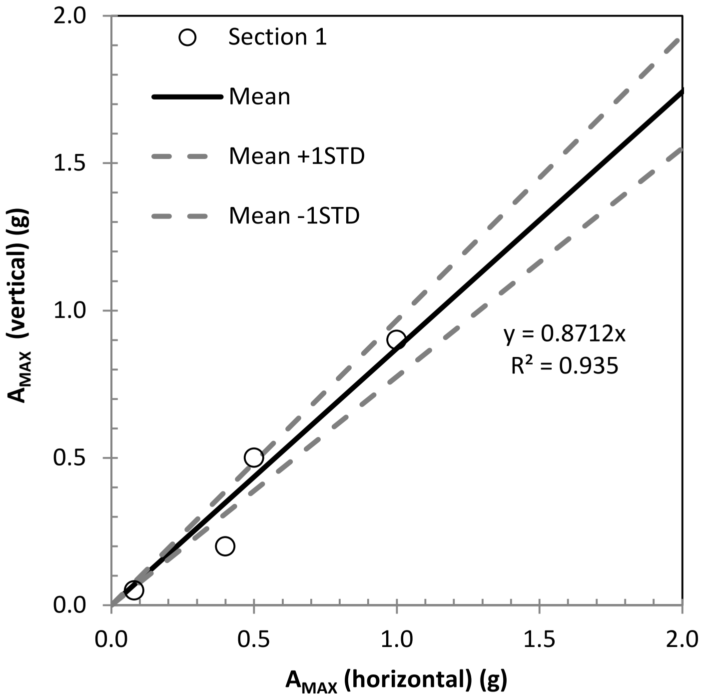


Figure A3.2 Relationship between the modelled horizontal and vertical maximum accelerations modelled at the slope crest (A_{MAX}) for cross-section 1, using the synthetic free field rock outcrop motions for the Cliff Street site by Holden et al. (2014) as inputs to the assessment. The mean and standard deviation trend lines are fitted for A_{MAX} all data. Errors are shown as the mean ± one standard deviation (1 STD).

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.

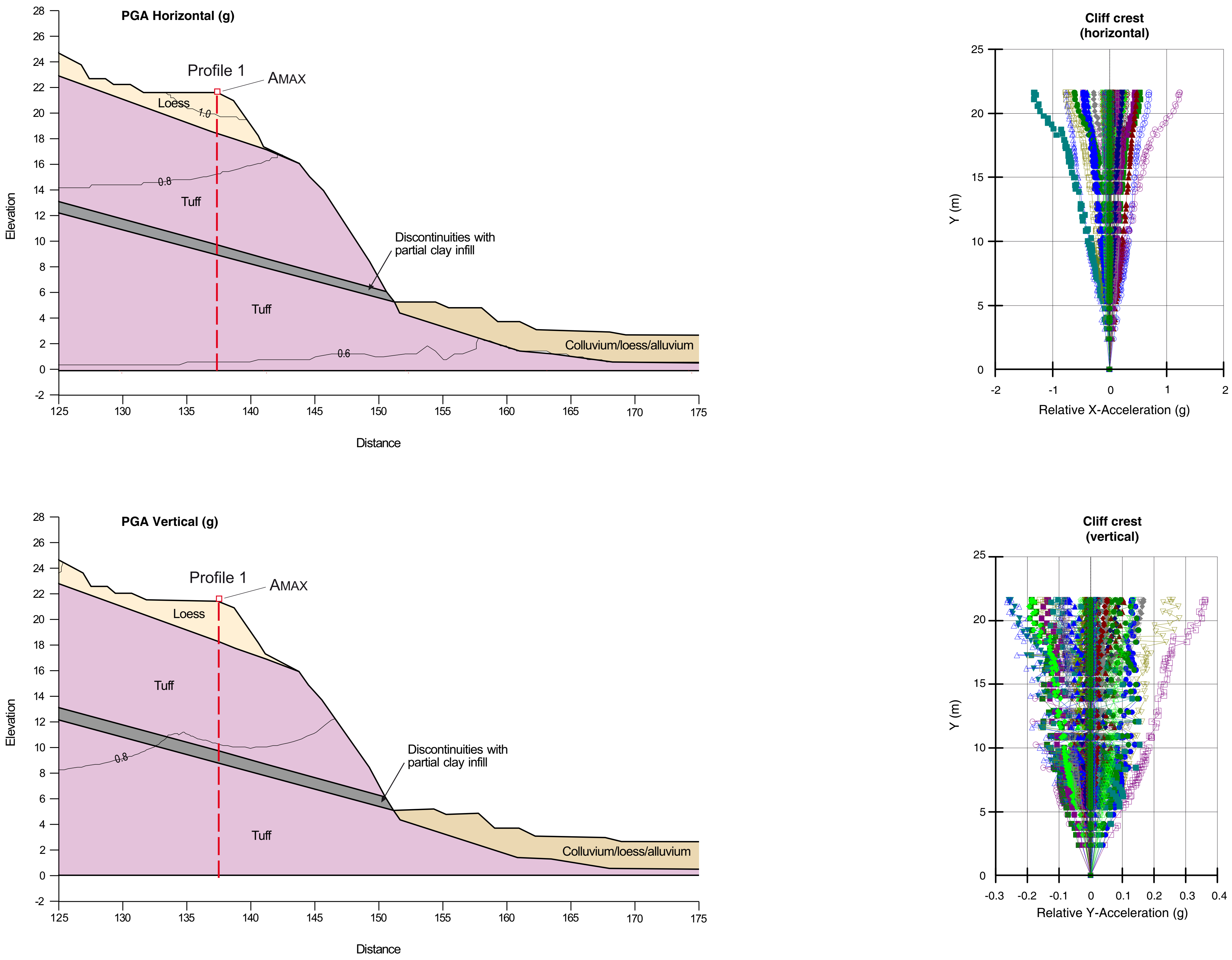
Eurocode 8, Part 5, Annex A recommends:

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 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 M_W 6.9 Loma Prieta earthquake.

Results from the seismic response assessment suggest that the peak ground acceleration amplification factors (S_T) for Cliff Street are about 1.7 times for horizontal motions and 1.4 times for vertical motions.

The model results for cross-section 1 (Figure A3.3) show that the accelerations are mainly amplified within the thicker sequences of loess (further up slope above the slope crest), especially at higher input peak ground accelerations. These results suggest that contrasting material properties, in this case between the underlying rock and the loess, amplifies the accelerations within the loess. These results are similar to those reported by others (e.g., Del Gaudio and Wasowski, 2010) where material impedance contracts have been shown to have a significant effect on the amplification of shaking, which could explain the localised displacements of the surface material mapped and described by Yetton (2014).



Contour values are peak ground acceleration (g).

DRW: PC

CHK: CM

QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 1) to the 22 February 2011, earthquake

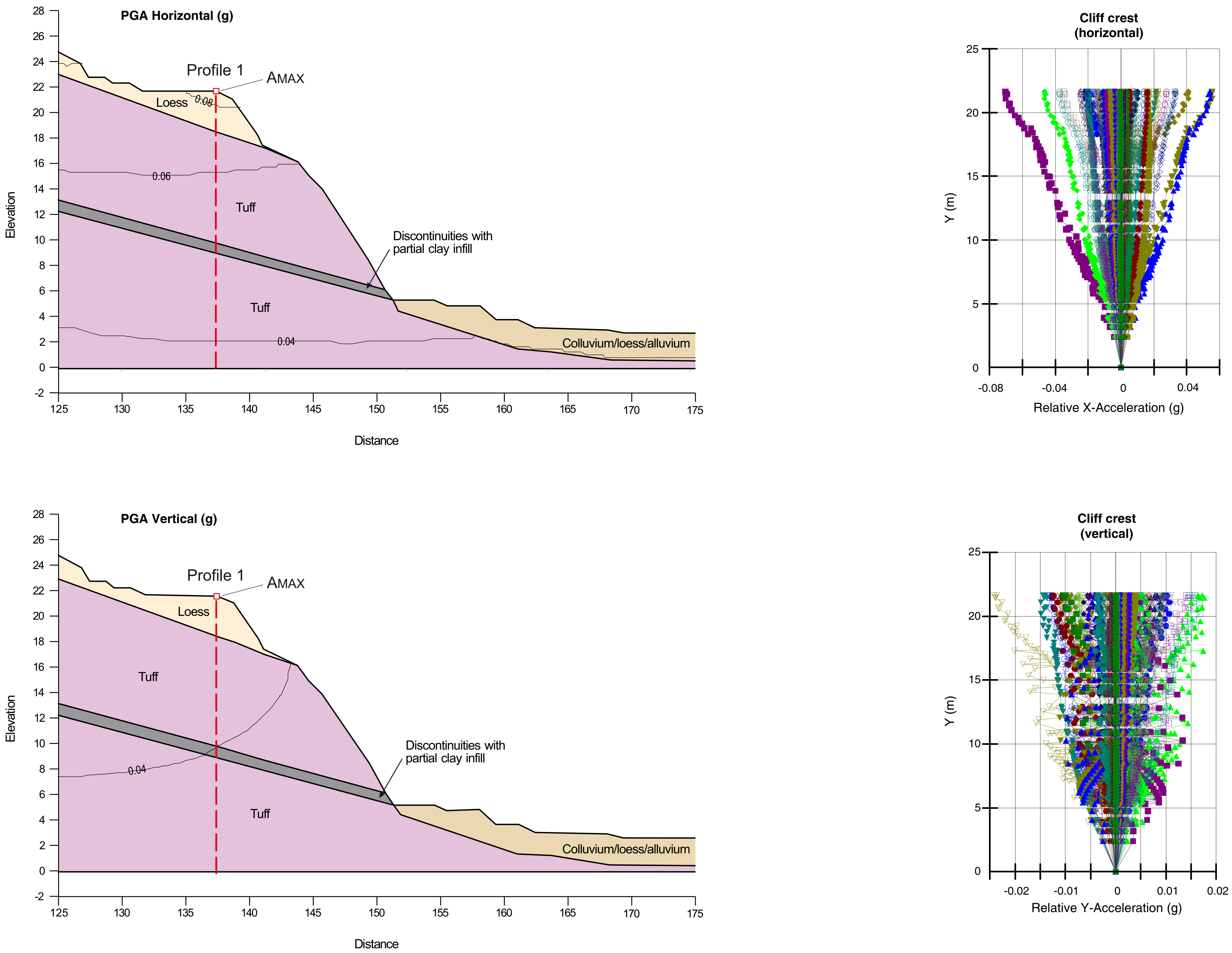
Cliff Street Christchurch

APPENDIX 3
FIGURE A3.3

FINAL

REPORT: CR2014/73

DATE: June 2014



Contour values are peak ground acceleration (g).

DRW: PC

CHK: CM

QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 1) to the 16 April 2011, earthquake

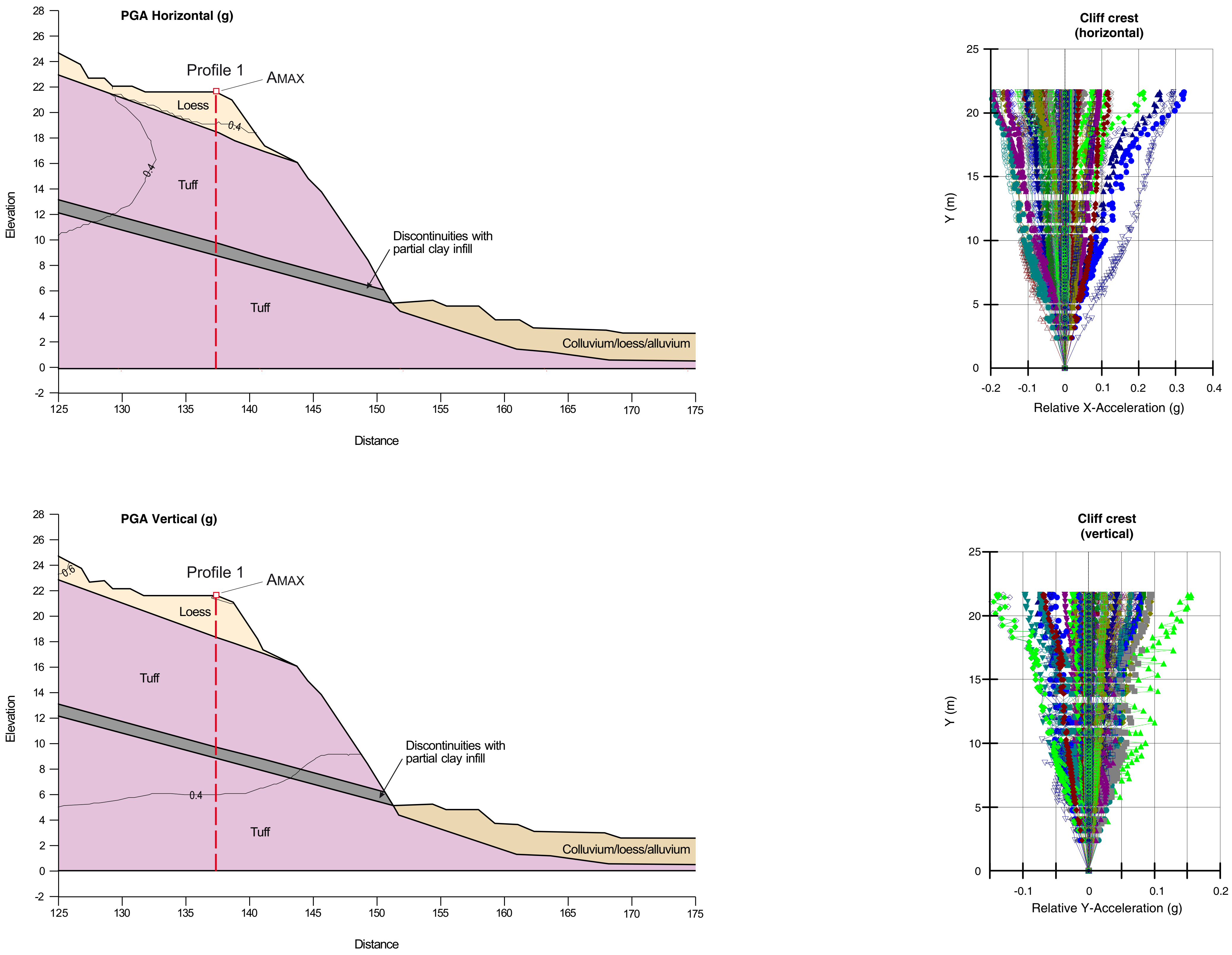
Cliff Street Christchurch

APPENDIX 3
FIGURE A3.4

FINAL

REPORT: CR2014/73

DATE: June 2014



Contour values are peak ground acceleration (g).

DRW: PC

CHK: CM

QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 1) to the 13 June 2011, earthquake

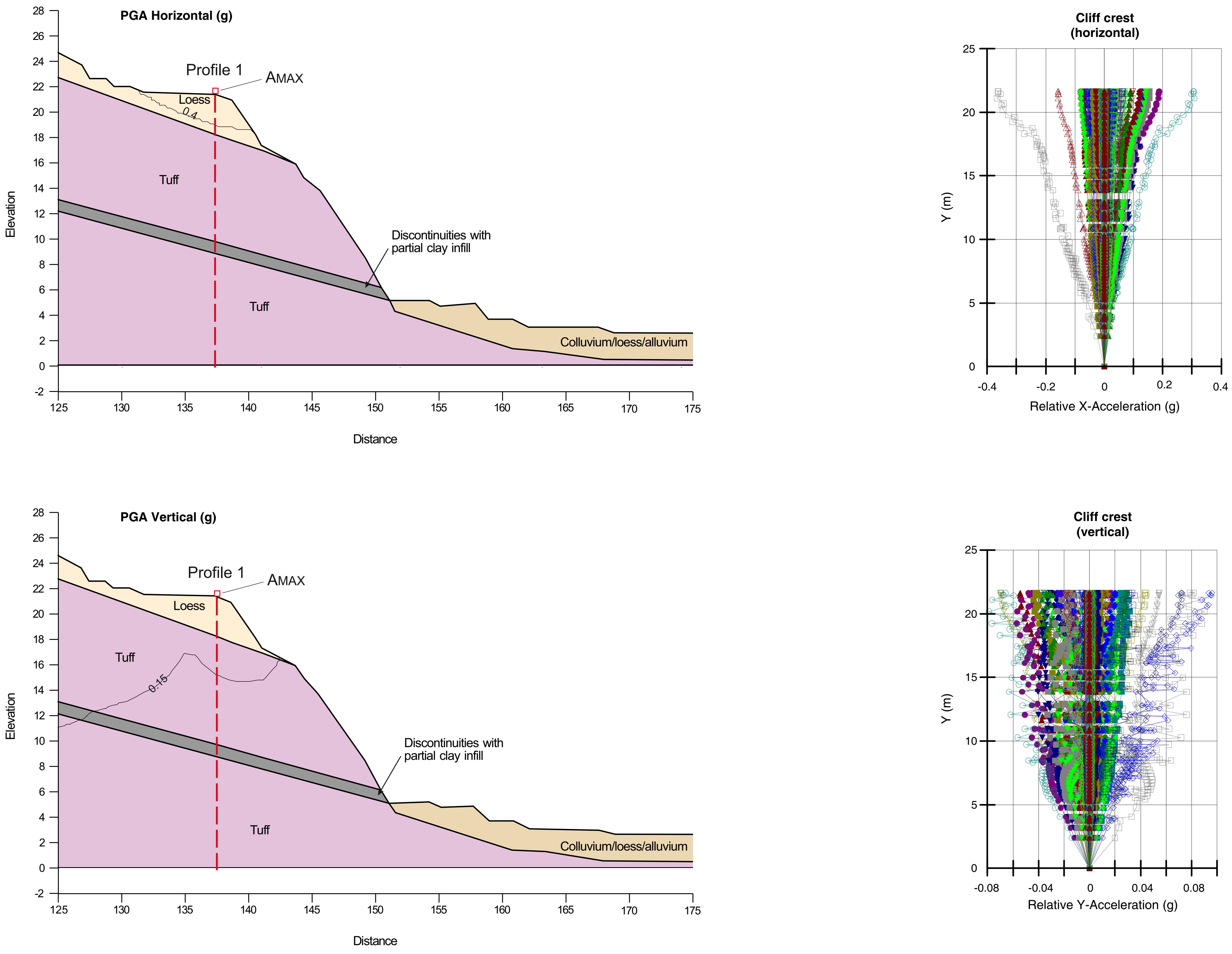
Cliff Street Christchurch

APPENDIX 3
FIGURE A3.5

FINAL

REPORT: CR2014/73

DATE: June 2014



Contour values are peak ground acceleration (g).

DRW: PC

CHK: CM

QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 1) to the 23 December 2011, earthquake

Cliff Street
Christchurch

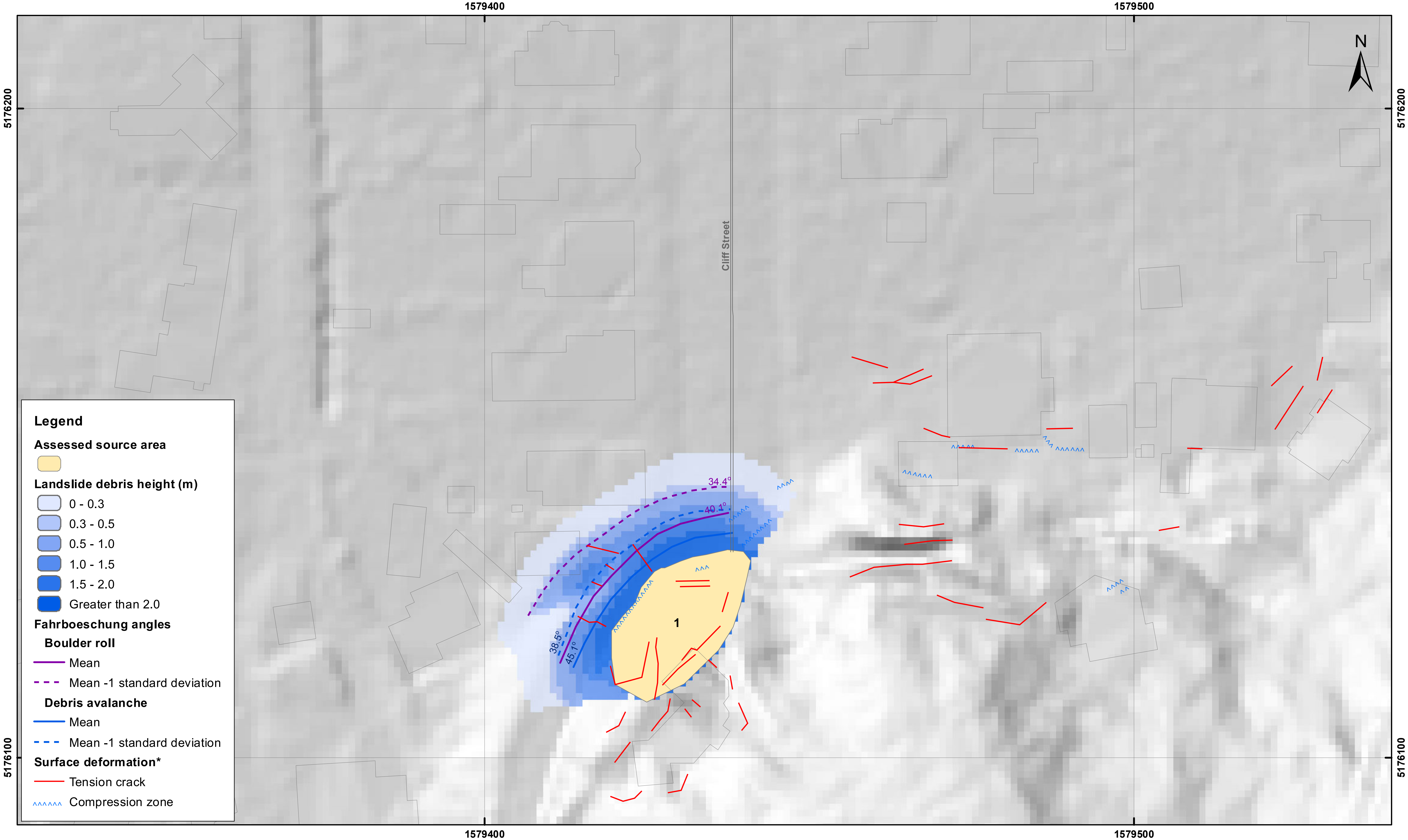
APPENDIX 3
FIGURE A3.6

FINAL

REPORT: CR2014/73

DATE: June 2014

A4 APPENDIX 4: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1 AND 2, ESTIMATED LANDSLIDE RUNOUT HEIGHT



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 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 - Upper Volume (2,300 m^3)

Cliff Street - Port Hills
Christchurch

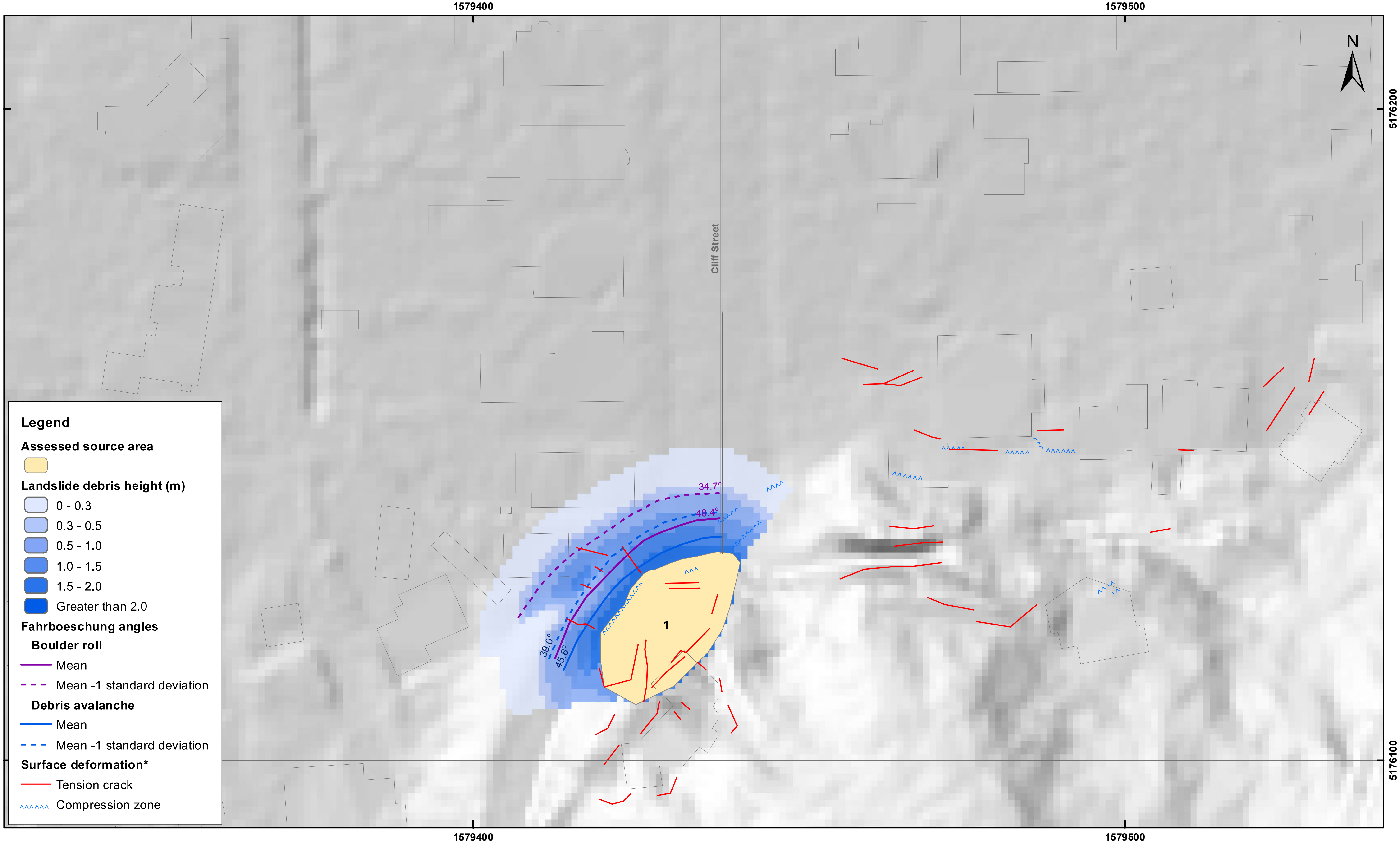
APPENDIX 4

Map 1

FINAL

REPORT: CR2014/73

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 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 (1,800 m^3)

Cliff Street - Port Hills Christchurch

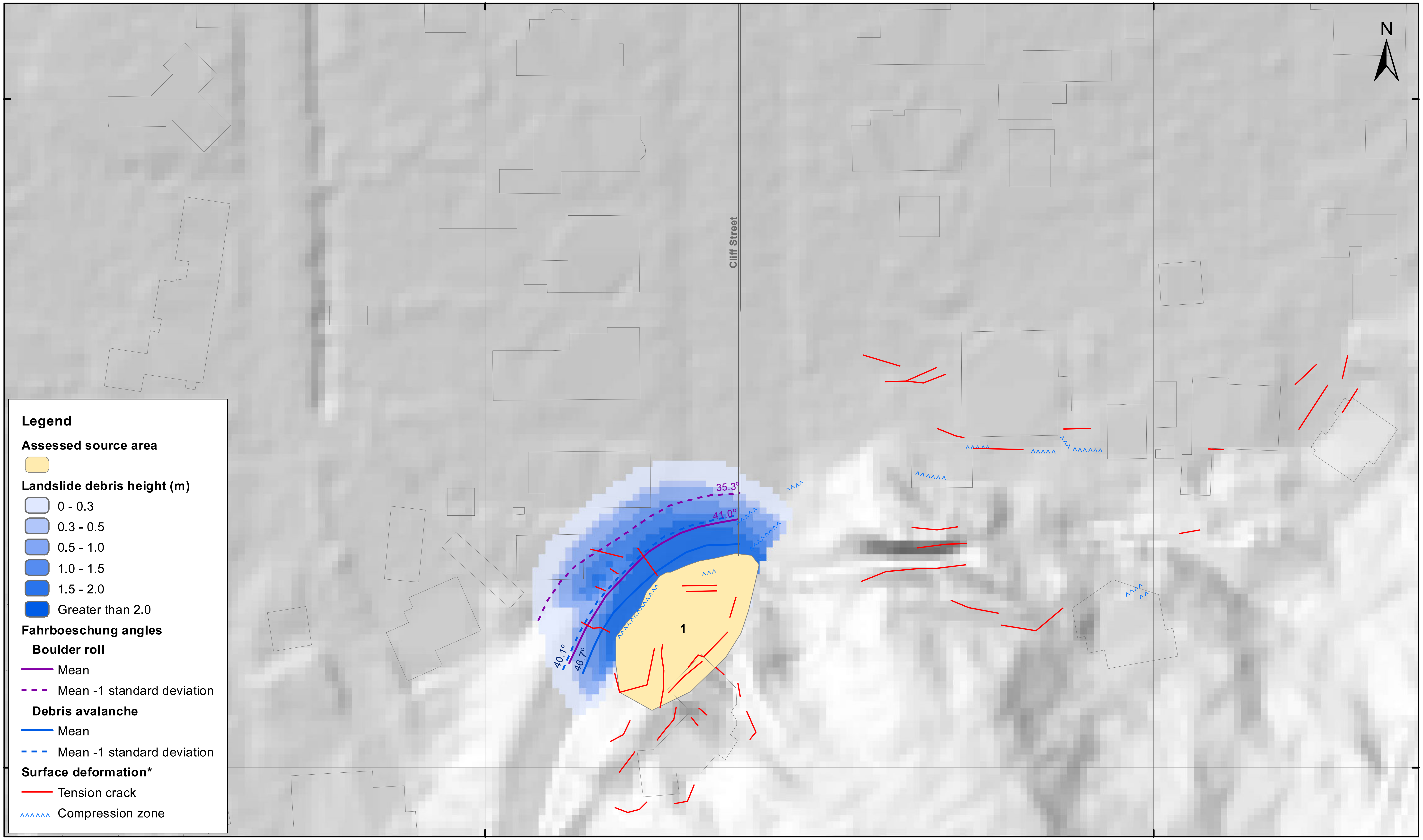
APPENDIX 4

Map 2

FINAL

REPORT: CR2014/73

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 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 (1,100 m^3)

Cliff Street - Port Hills
Christchurch

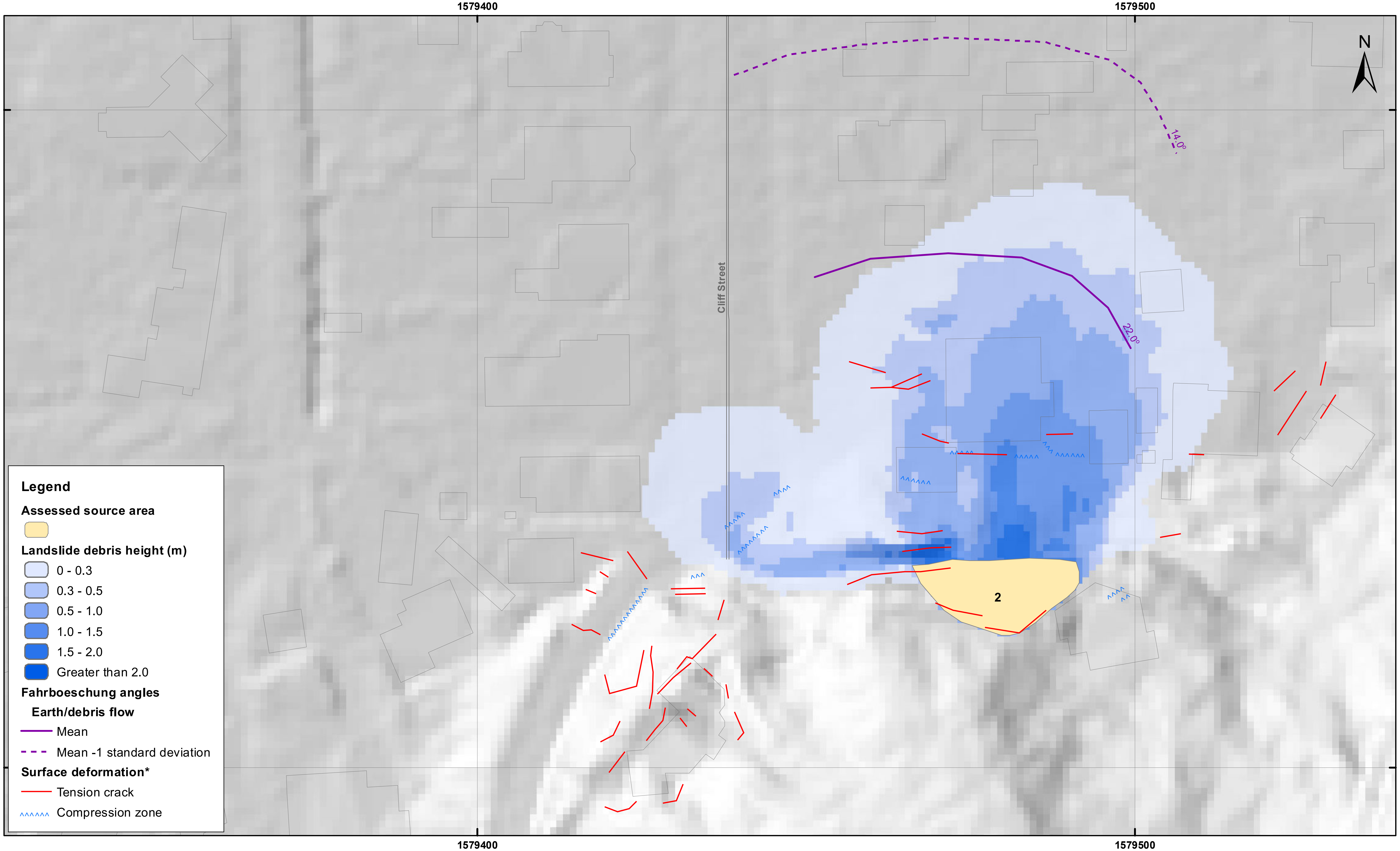
APPENDIX 4

Map 3

FINAL

REPORT: CR2014/73

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 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 (560 m^3)

Cliff Street - Port Hills
Christchurch

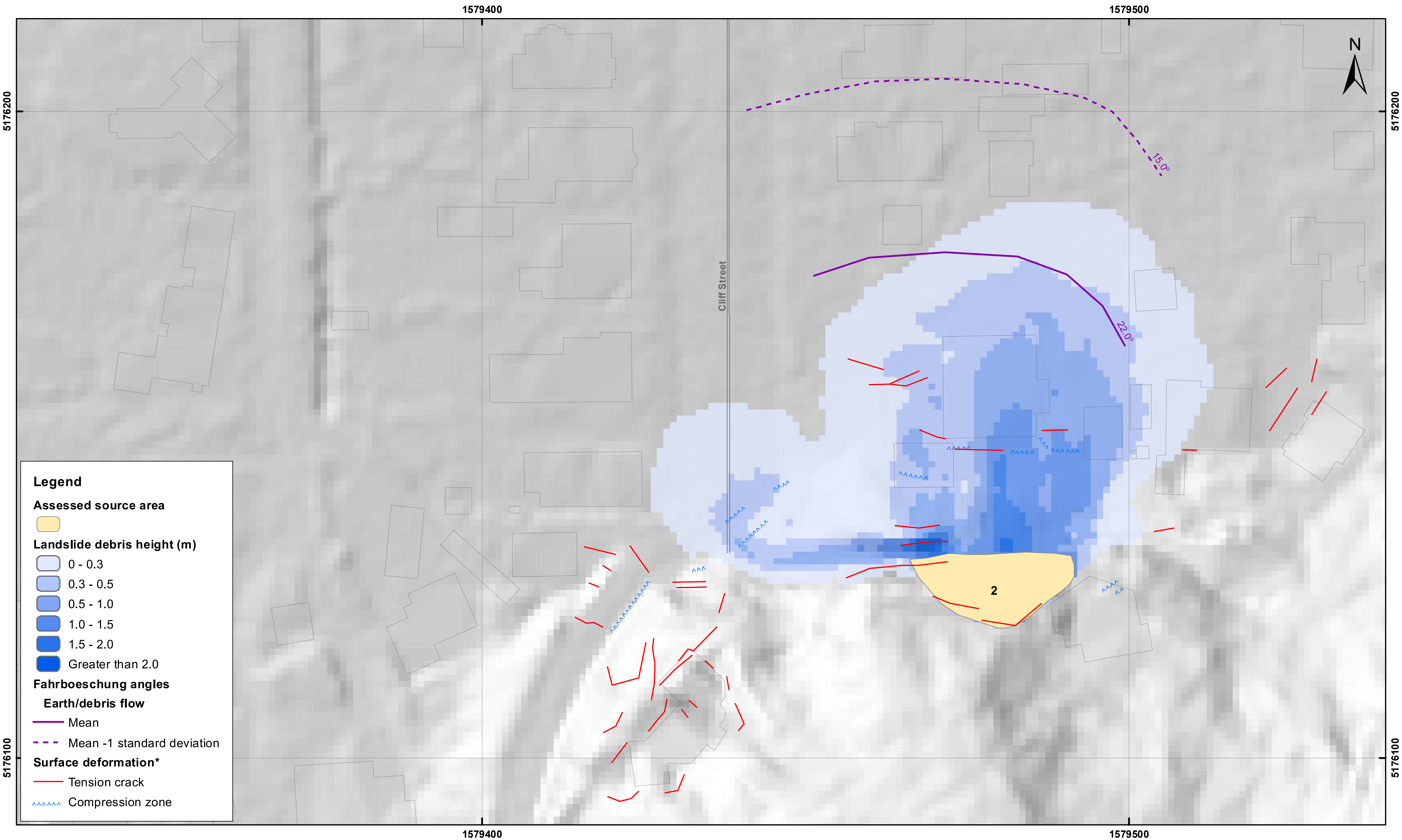
APPENDIX 4

Map 4

FINAL

REPORT: CR2014/73

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 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 - Middle Volume (460 m^3)

Cliff Street - Port Hills
Christchurch

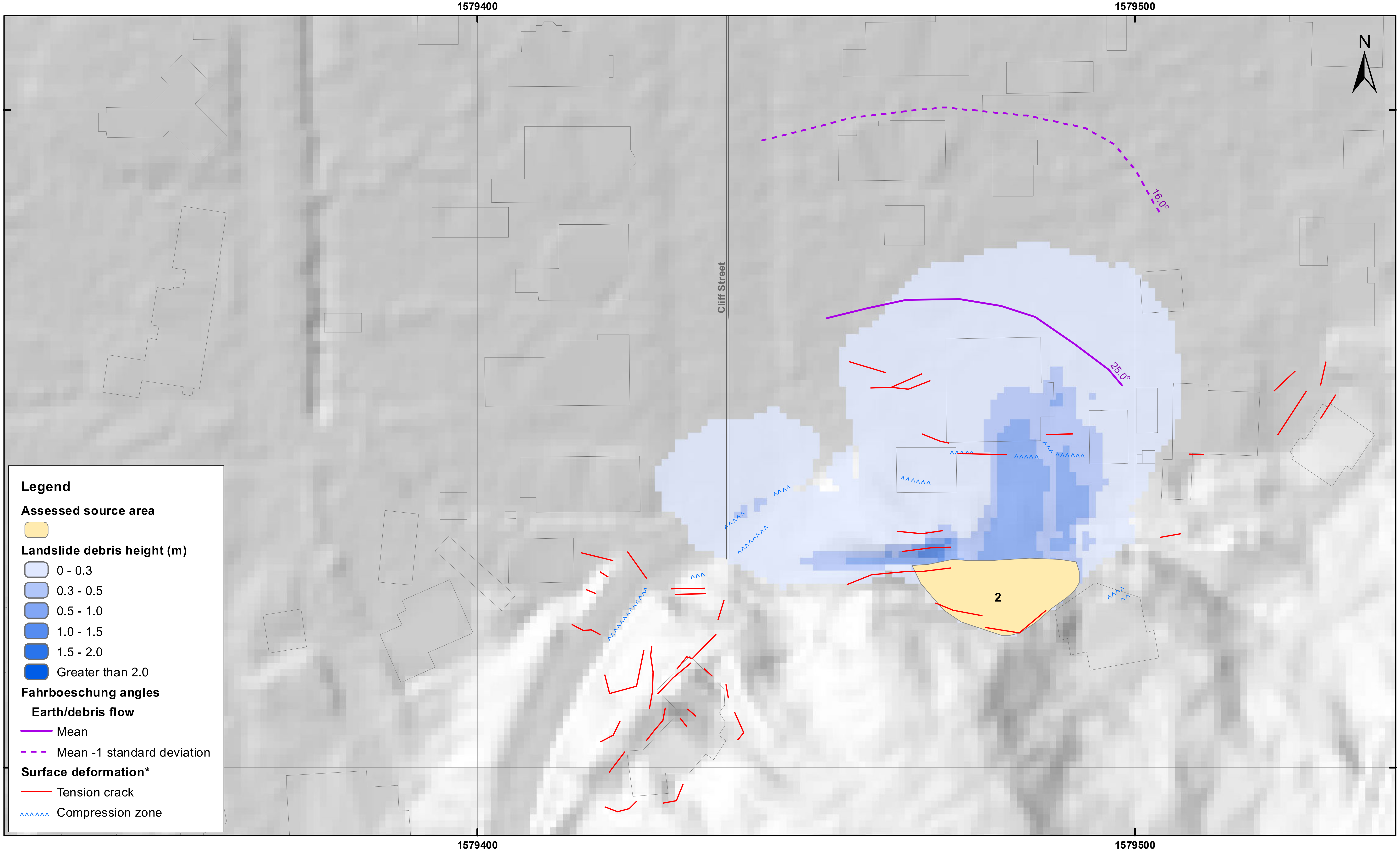
APPENDIX 4

Map 5

FINAL

REPORT:
CR2014/73

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 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 - Lower Volume (180 m^3)

Cliff Street - Port Hills
Christchurch

APPENDIX 4

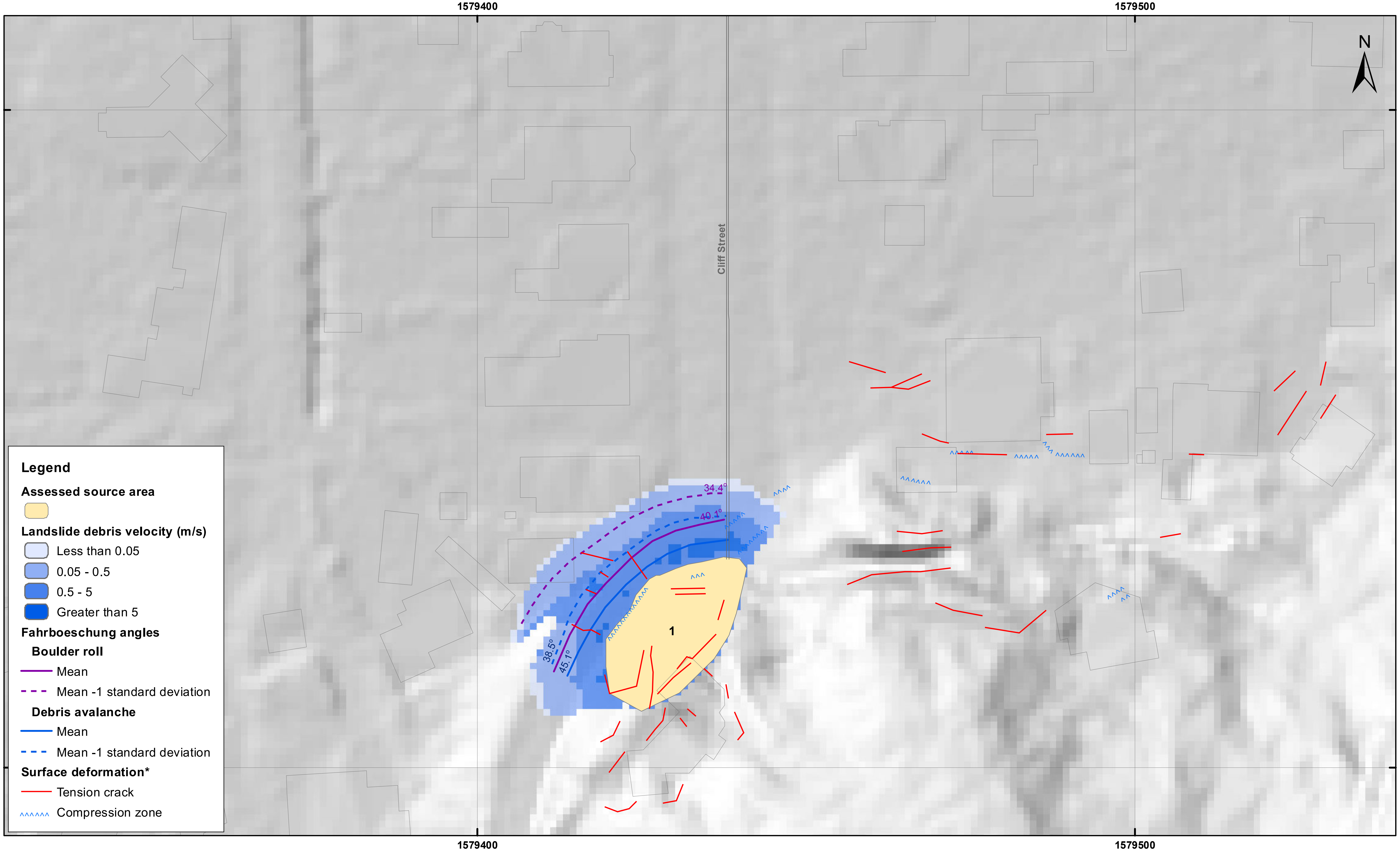
Map 6

FINAL

REPORT: CR2014/73

DATE: June 2014

A5 APPENDIX 5: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1 AND 2, ESTIMATED LANDSLIDE RUNOUT VELOCITY



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 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 1 - Upper Volume (2,300 m^3)

Cliff Street - Port Hills Christchurch

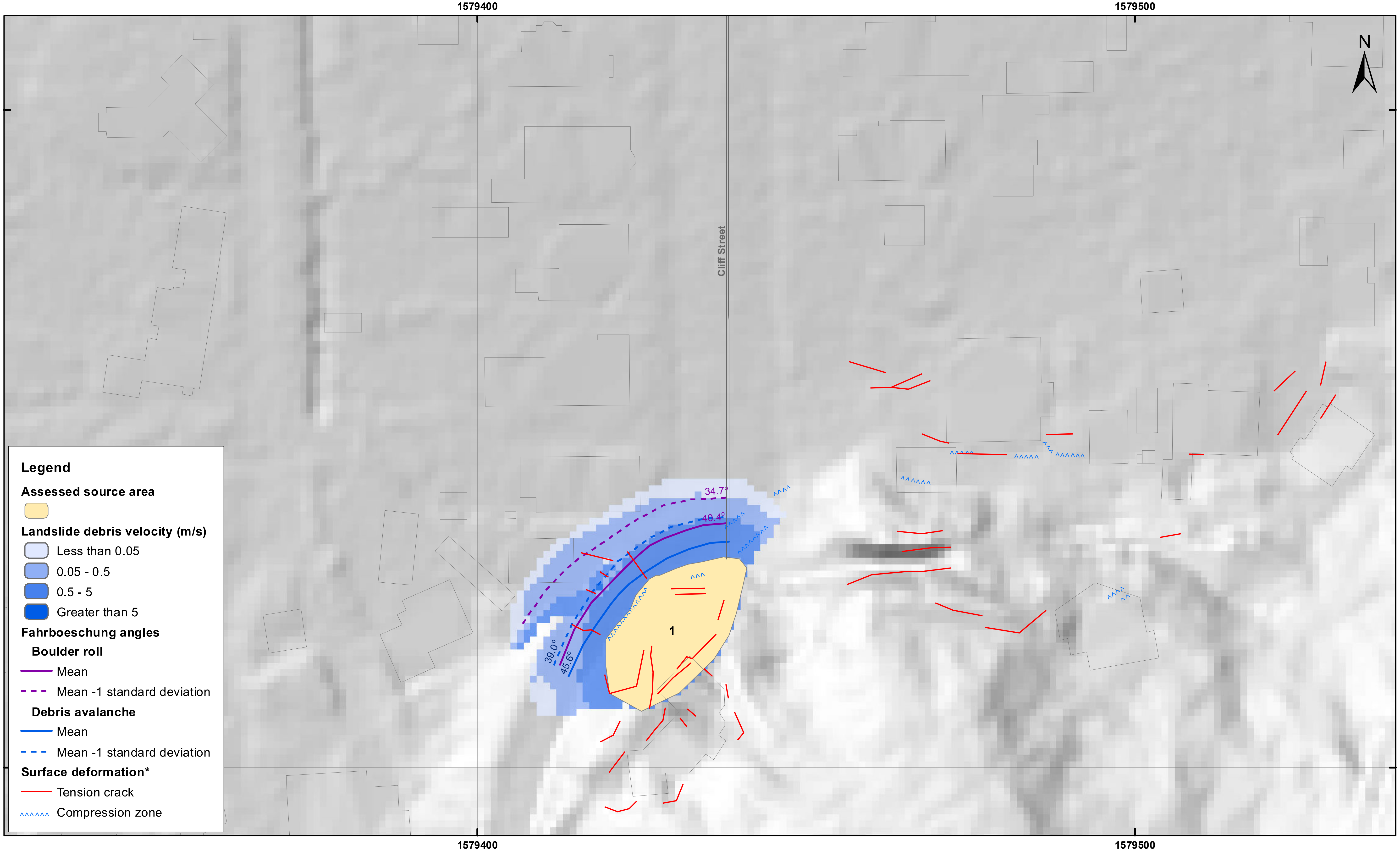
APPENDIX 5

Map 1

FINAL

REPORT: CR2014/73

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 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 1 - Middle Volume (1,800 m³)

Cliff Street - Port Hills
Christchurch

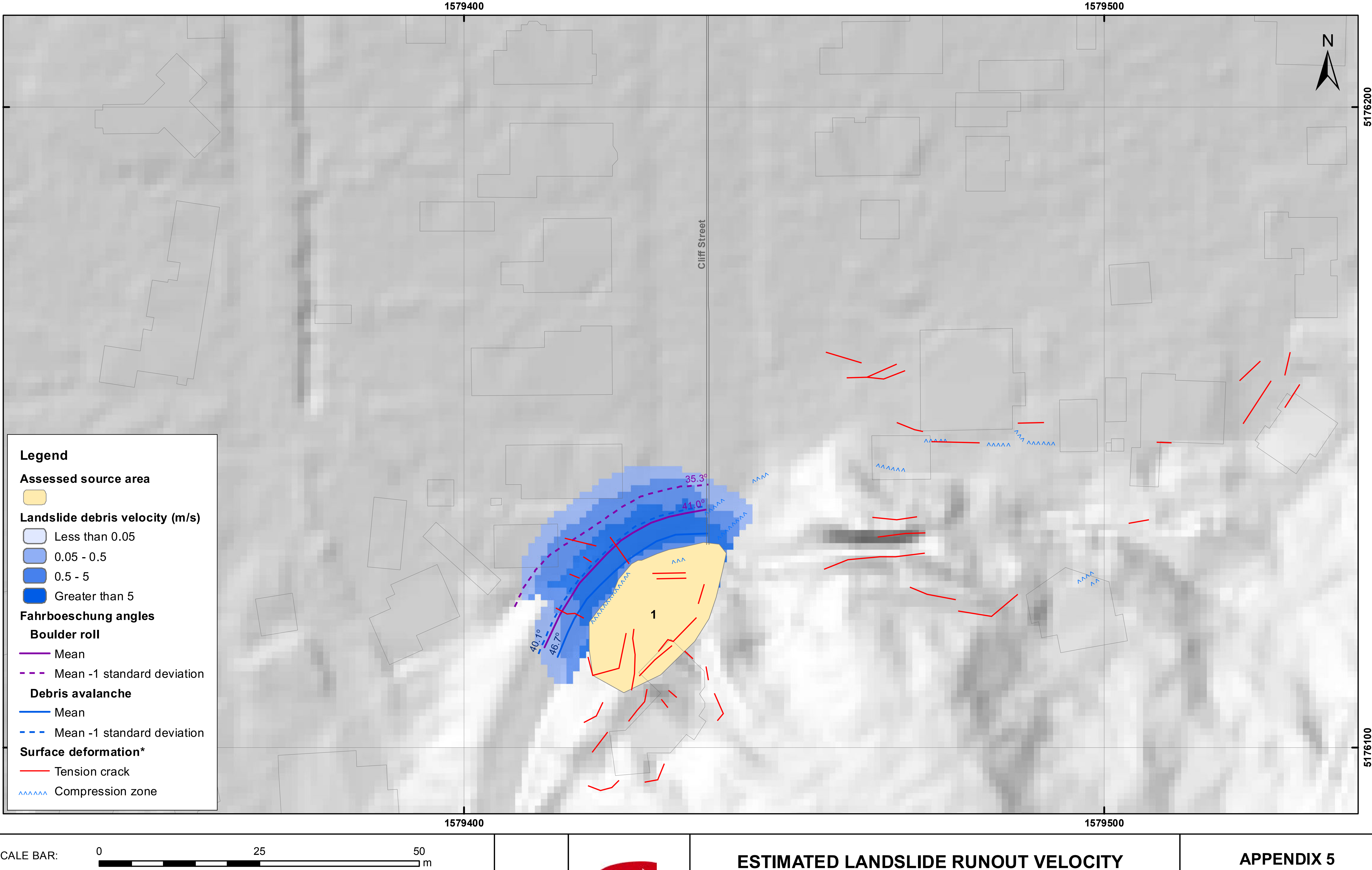
APPENDIX 5

Map 2

FINAL

REPORT: CR2014/73

DATE: June 2014



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 VELOCITY
Source 1 - Lower Volume (1,100 m^3)

Cliff Street - Port Hills
Christchurch

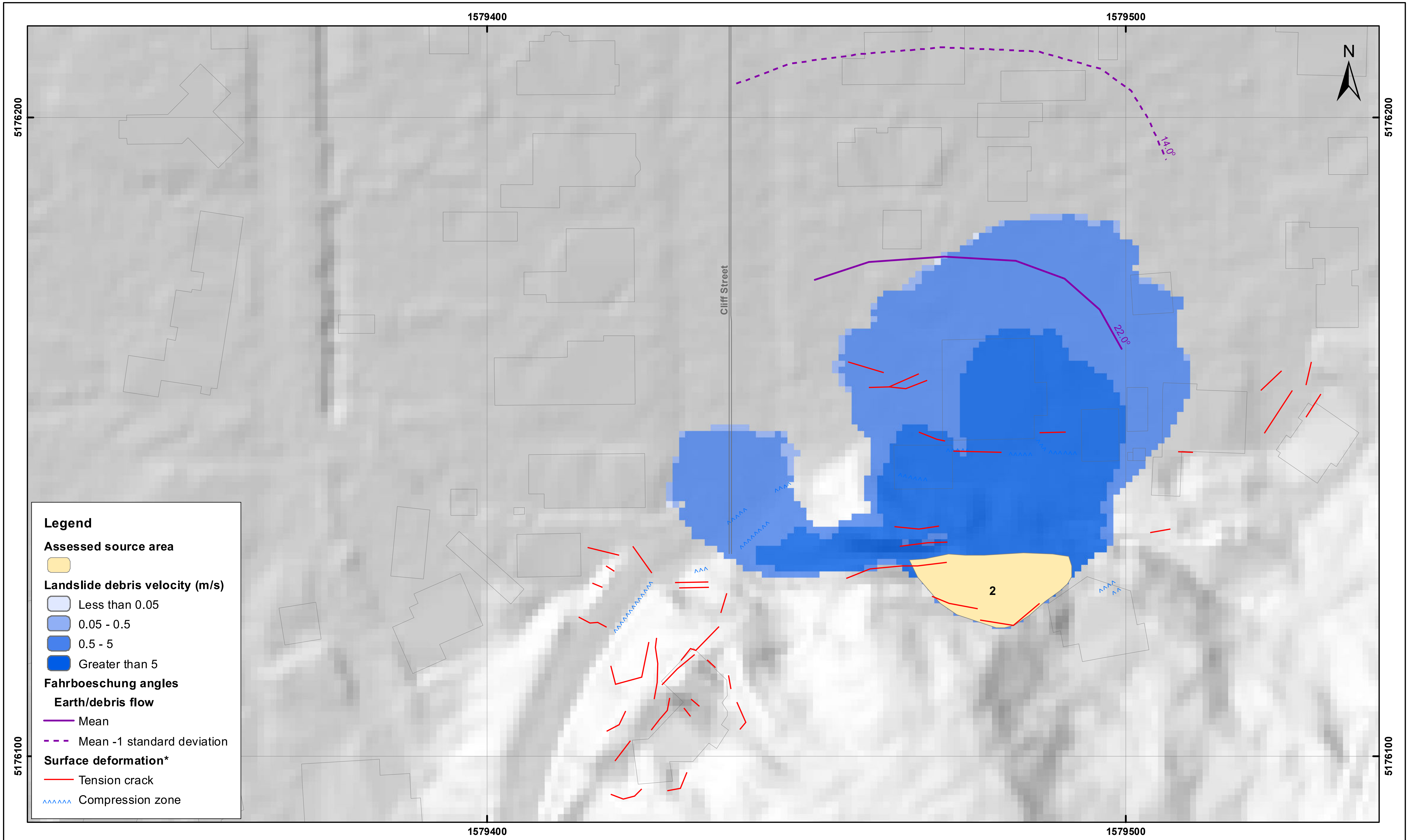
APPENDIX 5

Map 3

FINAL

REPORT: CR2014/73

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 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 - Upper Volume (560 m^3)

Cliff Street - Port Hills Christchurch

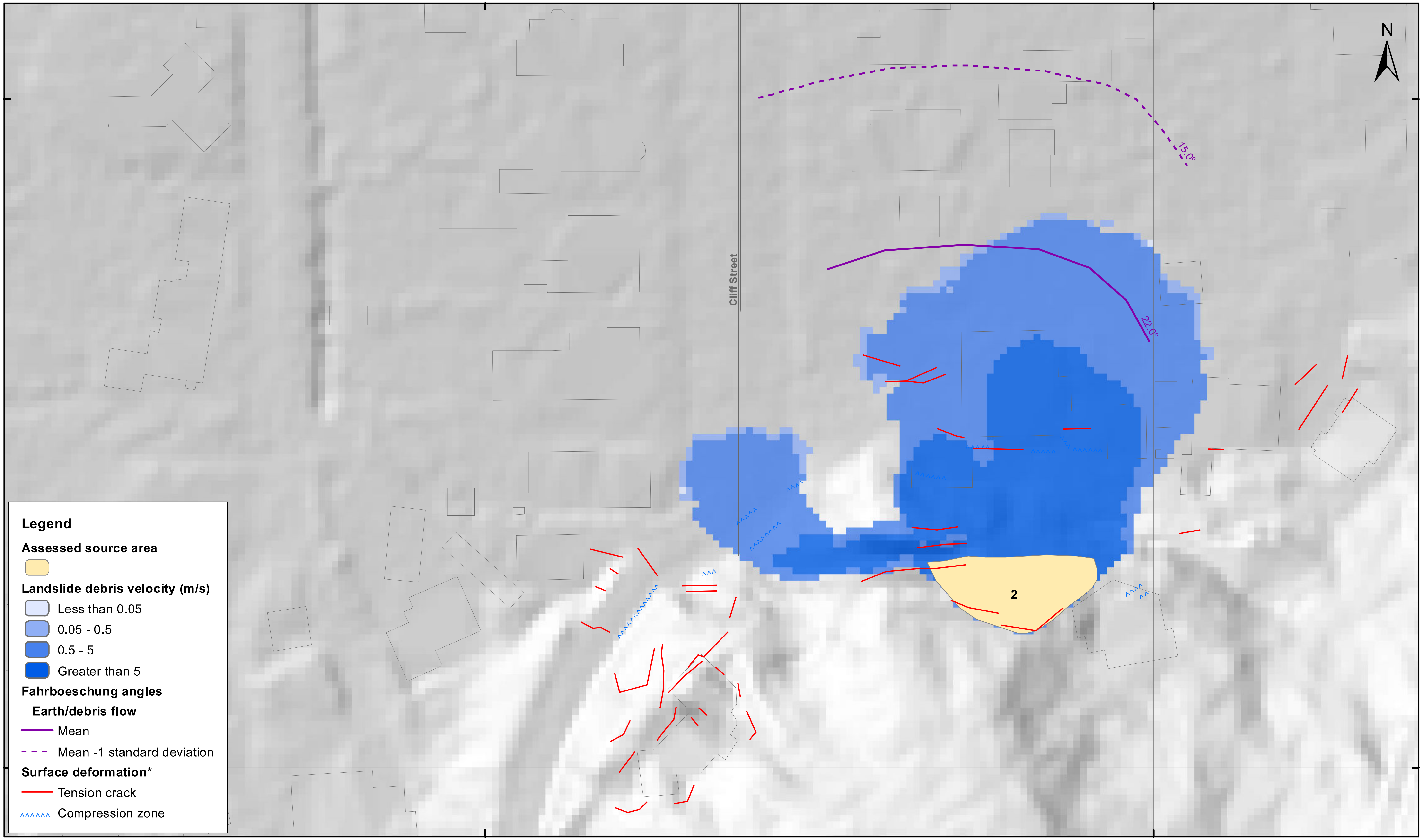
APPENDIX 5

Map 4

FINAL

REPORT: CR2014/73

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 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 - Middle Volume (460 m^3)

Cliff Street - Port Hills
Christchurch

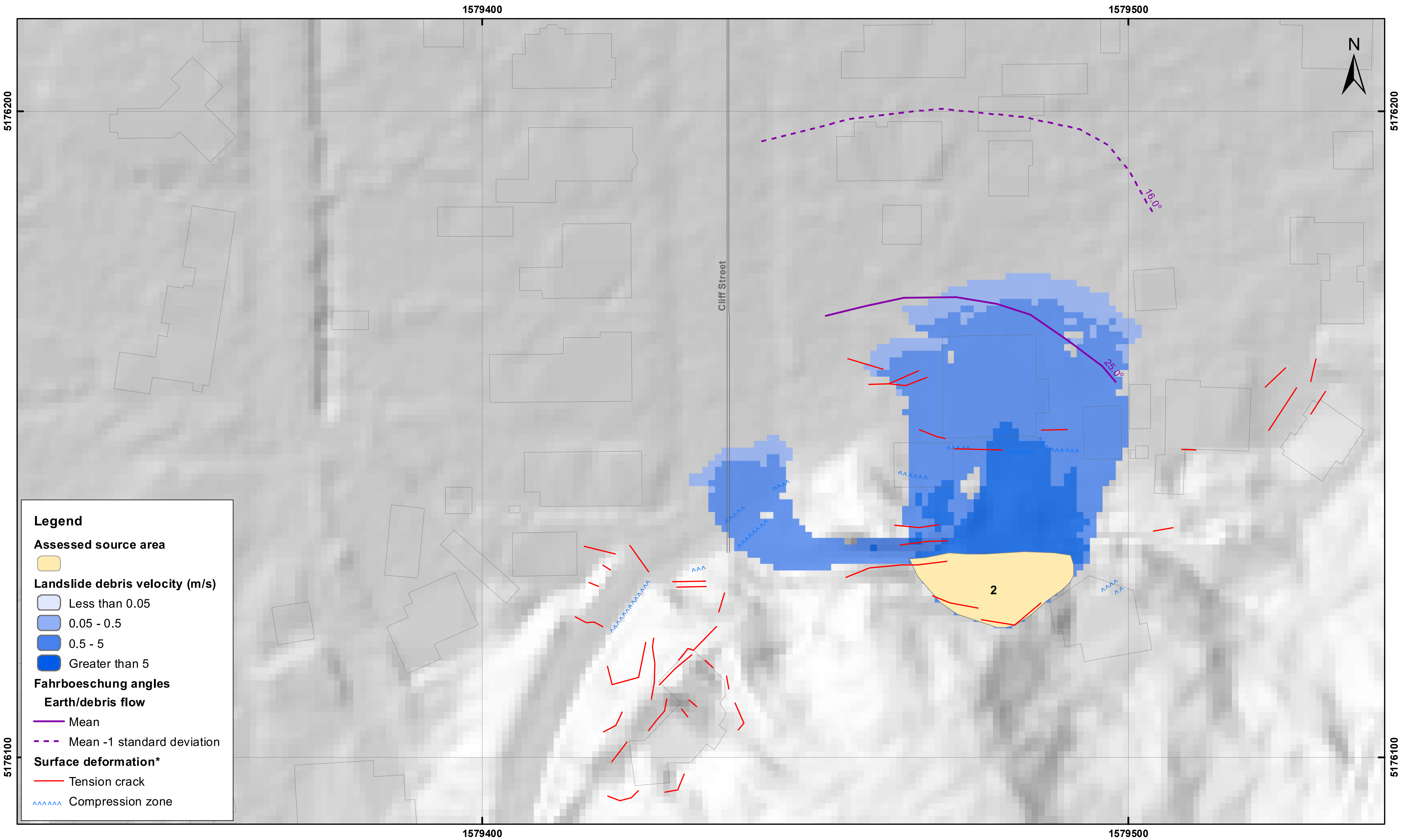
APPENDIX 5

Map 5

FINAL

REPORT: CR2014/73

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 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)

Cliff Street - Port Hills
Christchurch

APPENDIX 5

Map 6

FINAL

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www.gns.cri.nz

Principal Location

1 Fairway Drive
Avalon
PO Box 30368
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4600

Other Locations

Dunedin Research Centre
764 Cumberland Street
Private Bag 1930
Dunedin
New Zealand
T +64-3-477 4050
F +64-3-477 5232

Wairakei Research Centre
114 Karetoto Road
Wairakei
Private Bag 2000, Taupo
New Zealand
T +64-7-374 8211
F +64-7-374 8199

National Isotope Centre
30 Gracefield Road
PO Box 31312
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4657