Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Earth/debris flow risk assessment for Defender Lane

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BIBLIOGRAPHIC REFERENCE

Della Pasqua, F.; Massey, C. I.; Lukovic, B.; Ries, W.; Archibald, G. 2014. Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Earth/Debris flow risk assessment for Defender Lane. GNS Science Consultancy Report 2014/67. 115 p.+ Appendices

REVIEW DETAILS

This report in draft form was independently reviews by T. Taig, TTAC Limited, Dr L. Richards and Dr J. Wartman. Internal GNS Science reviews of drafts were provided by N. Litchfield, M. McSaveney and D. Mieler.

Risk calculations were independently checked by 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 Defender Lane site with numerical slope-stability modelling to assess the risk to people in dwellings within the assessment area.

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

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

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

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

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

ES 2 INVESTIGATION PROCESS AND FINDINGS

Detailed investigations of the site and its history show evidence of past slope failures from the time of European settlement (about 1840 AD) to the present. There is also evidence of several debris fans, dating from pre-European times, present at the site.

The slopes were significantly cracked in the 22 February 2011 earthquakes, and again during the 13 June 2011 earthquakes, though relatively little movement was observed in the other moderate sized earthquakes. Overall ground displacement at the Defender Lane mass movement site through the 2010/11 Canterbury earthquakes is not known as there were no survey markers, installed in the main areas where displacement had been identified, to enable before and after measurements to be made.

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

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

By mapping cracks and relating these to the results of stability assessments, it has been possible to identify 10 potentially significant earth/debris flow source areas, from which landslides of variable volume could occur. The assessed source areas are not the only source areas for landslides within the assessment area; they are representative of the volumes of landslides that could occur from anywhere within the assessment area.

Numerical models have been used to assess the stability of the Defender Lane assessment area. Analyses have considered both:

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

ES2.1 Earth/debris flows

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

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

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

For the 10 assessed source areas, the likely volume of material mobilised during a slope failure event and the frequency of the slope failure triggering event are both uncertain. Nonetheless, the slopes have remained stable during 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 during the March 2014 rain storm, no larger landslides have occurred.

ES2.2 Failure volumes and triggering frequencies

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

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

ES3 CONCLUSIONS

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

ES3.1 Hazard

1. There is potential for volumes ranging from many tens to several hundreds of cubic metres of earth/debris flows (source areas 1–10) of mixed loess and colluvium, which are in addition to the cliff-collapse failures previously assessed (Massey et al., 2012).
2. The most likely triggers for the assessed earth/debris flows sources are prolonged heavy rainfall and strong earthquake shaking (if ground conditions were wet).
3. The frequency of earth/debris flow events from these sources is difficult to estimate. For the assessment, event annual frequencies of once every 20 years to once every 200 years have been assessed.
4. It should be noted that material strengths – and therefore the slope factors of safety – are likely to reduce with time, and the occurrence of future earthquakes. Therefore a conservative approach is warranted to account for this long-term change.

ES3.2 Risk

1. The effect on annual individual fatality risk for residents of the newly-identified sources is to increase the risk levels at dwellings already identified as at particularly high levels of risk from cliff collapse, and to include a several more dwellings that could be at possibly unacceptable levels of risk.
2. The main hazards affecting these dwellings is likely to be a combination of cracking and undercutting (the Class II area and the source areas) as the ground moves beneath the dwelling, and in the runout zone, the impact from debris coming from further upslope.

3. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings in the assessed source area, from future earthquakes is still relatively high. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.2–2 m, depending on the strength of the loess at the time of the earthquake. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the national seismic hazard model.

ES3.3 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. There appears to be reasonable scope for engineering measures to stabilise the slopes (e.g., by removal of loess and installation of drainage measures). Such works would need to be evaluated, designed and implemented by a suitably qualified engineering consultant.

ES4 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

1. 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 sewer main currently runs through the mass movement at the crest of the assessed earth/debris flow source areas, 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.

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 significant land use decisions.

1.0 INTRODUCTION

This report combines recent field information collected from the Defender Lane site with numerical slope-stability modelling to assess the risk to people in dwellings from mass movements at the site.

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.

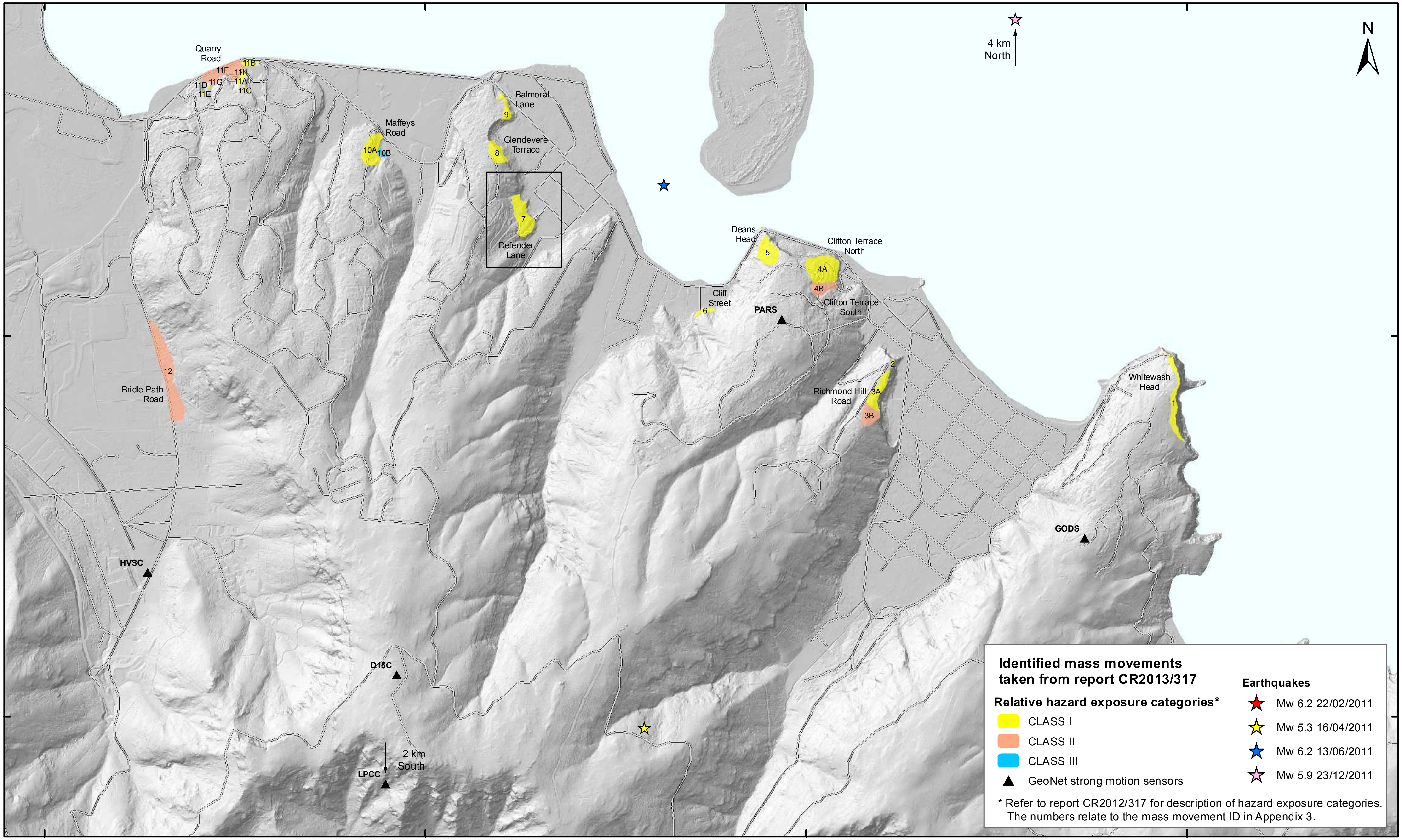
Christchurch City Council contracted GNS Science to carry out detailed investigations of the identified areas of mass movement, in order to assess the nature of the hazard, the frequency of the hazard occurring, and whether the hazard could pose a risk to life, a risk to existing dwellings and/or a risk 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 2 report on the Defender Lane 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

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LOCATION MAP

Port Hills Christchurch

FIGURE 1

Map 1

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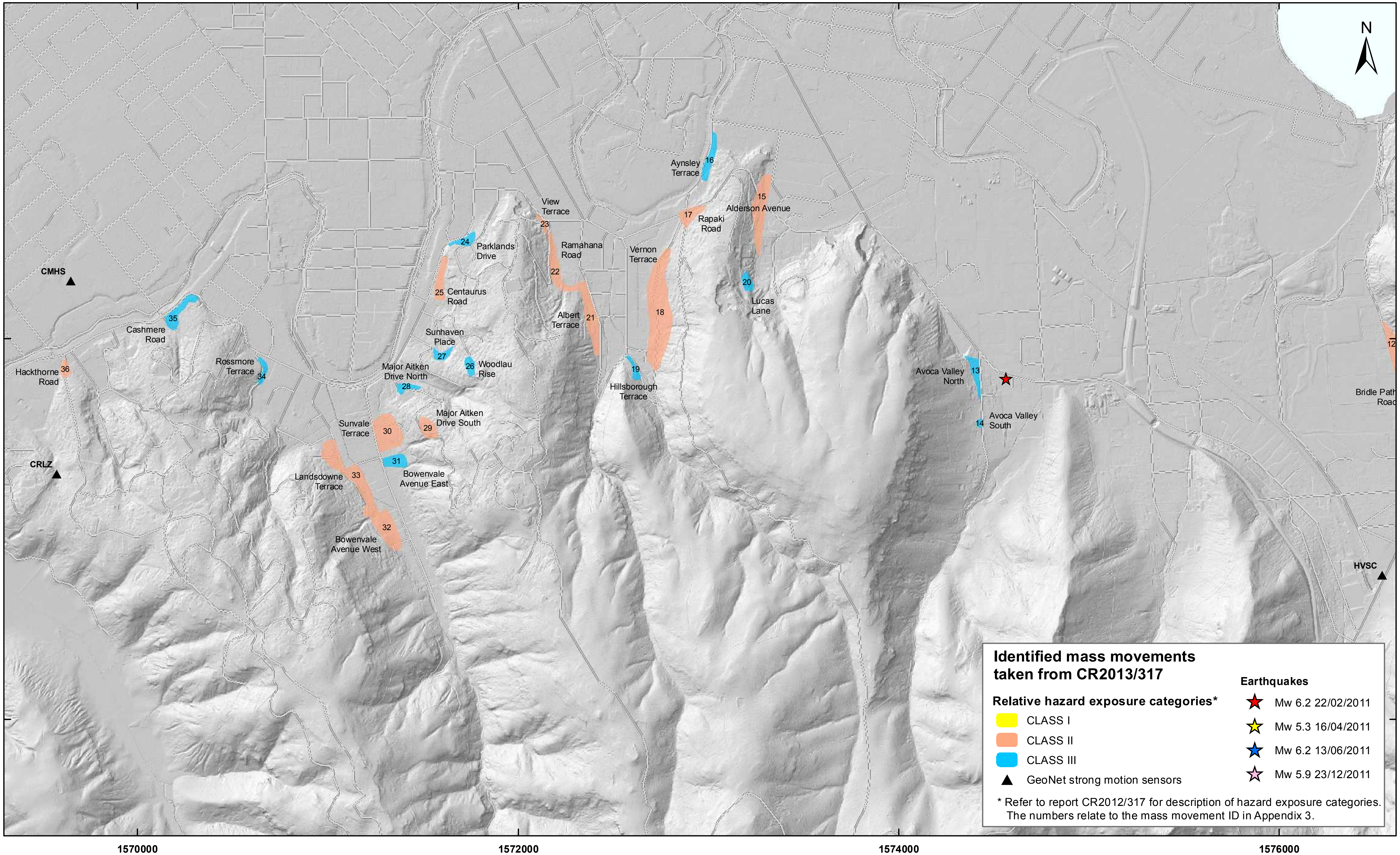
DATE: June 2014



SCALE BAR: 0 0.5 1 km

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

DRW: BL

CHK: CM

LOCATION MAP

Port Hills Christchurch

FIGURE 1

Map 2

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Table 1 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 DEFENDER LANE MASS MOVEMENT

The Defender Lane 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 Defender Lane mass movement 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 boundary of the assessment area.

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/57

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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MASS MOVEMENT LOCATION MAP

Defender Lane
Christchurch

FIGURE 2

FINAL

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1.3 PREVIOUS WORK AT THE DEFENDER LANE SITE

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

1. The risk to life of people in dwellings at the cliff crest from debris avalanches and cliff top recession hazards associated with the steep rock slope (collectively termed cliff collapse) were previously estimated by Massey et al. (2012);
2. Field mapping of the crack distributions was carried out by M. Yetton (Geotech Consulting Ltd.) and GNS Science, and the results are contained in the Stage 1 report (Massey et al., 2013); and
3. Ground investigation and field mapping of the site was carried out by Tonkin and Taylor Ltd, under contract to the Earthquake Commission (Tonkin and Taylor, 2012a). The ground investigations comprised the drilling of four drillholes and three cone penetrometer holes.

Figure 3 Aerial view of the Defender Lane mass movement study area looking west.

1.4 SCOPE OF THIS REPORT

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

1. Estimate the annual individual fatality risk for affected dwelling occupants from the failure of the assessed source areas, 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 CR2013/247LR (Della Pasqua and Massey, 2013).

1.5 REPORT STRUCTURE

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

1.6 METHODS OF ASSESSMENT

The site assessment comprised three stages:

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

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

1.6.1 Engineering geology assessment

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

1. Interpretation of available aerial photographs covering the period 1940–2011, to determine the land use and development history of the site.
2. Surveying of cadastral survey marks within and around the mass movement to determine the magnitudes of slope displacement during the 2010/11 Canterbury earthquakes.
3. Assessment of the results from the surveying of monitoring marks installed on the site by Aurecon NZ Ltd. (under contract to Christchurch City Council), following the 22 February 2011 earthquake. This was undertaken to assess the amount of slope displacement relating to the main earthquakes.
4. Geological and geomorphological field mapping to identify the materials, processes and landforms that have been active within the assessment area.
5. Review of previous ground investigations carried out by Tonkin and Taylor Ltd. (Tonkin and Taylor, 2012a).

6. Establish an engineering geological model of the site and construction of an engineering geological map and five cross-sections, based on the results from the aerial photograph interpretation, surveying, field mapping, and the site investigations. These were used as the basis for the hazard and risk assessments.

1.6.2 Hazard assessment

The hazard assessment method followed three main steps:

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

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

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

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

1.6.3 Estimation of landslide volumes

The results of the engineering geological assessments and the slope stability modelling carried out by GNS Science have been used to define 10 potential landslide sources areas. These are located in areas where the bulk strength of the slope could have been degraded as a result of earthquake-induced cracking. The assessed source areas (shown in Figure 2) do not represent the only potential locations of the source areas that could occur in the assessment area. The assessed source areas are intended to represent the range of potential landslide locations and volumes that could occur in the assessment area.

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

1.6.4 Risk assessment

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

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

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

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

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.

2.0 DATA USED

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

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

Data	Description	Data source	Date	Use in this report
Post-22 February 2011 earthquake digital aerial photographs	Aerial photographs were taken on 24 February 2011 by NZ Aerial Mapping and were orthorectified by GNS Science (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	1940, 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.
Drillhole logs	The logs from cores extracted from holes bored into the cliff top areas covered by this report.	Tonkin and Taylor on behalf of the Earthquake Commission and Aurecon for Christchurch City Council	February 2012 and February 2013	Used in generating the engineering geological models of the cliff interiors.
Downhole shear wave surveys	Downhole shear wave velocity surveys carried out in the Aurecon drillholes.	Southern Geophysical	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.

3.0 SITE ASSESSMENT RESULTS

3.1 SITE HISTORY

3.1.1 Aerial photograph interpretation

Interpretation of pre-2010 aerial photographs and field mapping identified several relict landslide scars (apparent in the 1940 aerial photographs) in the area, with estimated volumes ranging from 300 to 1,500 m^3. Two debris fans (Figure 4) have been identified below the steep rock slope, which pre-date the earliest available aerial photographs. These form prominent breaks in slope, where the slope gradient becomes gradually flatter with distance away from the rock slope. The fans comprise:

- An upper and steeper (45°) slope, composed mainly of boulder-rich colluvium immediately down slope of the rock slope and in some cases mantling the rock slope, and
- A lower less steep (<5°) slope, composed mainly of sandy colluvium (reworked loess) extending away from the gully area towards Taupata Street.

The talus fans are inferred to be the deposits of debris sourcing from the drainage gullies in the loess above the steep rock slope. The debris appears to have accumulated on top of pre-historic former beach surfaces. The likely age of the coastal beach surface on which this material was deposited may be about 3,500–3,700 years before present (McFadgen and Goff, 2005).

Using the 2003 LiDAR survey digital elevation model of these slopes, the historical talus volumes were estimated by projecting the rockslope face at the toe of the slope through the talus to intersect the pre-talus ground surface. The pre-talus ground surface was defined by the ground elevation outside the colluvium fan. Estimated debris volumes are listed in Table 3.

These volumes are indicative only as the estimated debris volumes are likely to represent a mixture of debris with an unknown proportion of dune sand that had accumulated at the toe of the slope prior to the date of LiDAR acquisition. These volumes suggest accumulation rates of 1.7–3.6 m^3/year per fan (a total of 5.4 m^3/year for both fans), assuming that the age of the beach surface is 3,600 years old, and assuming no bulking factor from source to debris. However, it cannot be assumed that the debris in the fans accumulated gradually over time at constant rates. It is possible that the debris was deposited by one or more, much larger, but less frequent events.

Table 3 Estimated historical talus volumes.

Gully area	Pre-fan ground elevation	Debris fan ID	Colluvium fan type	Talus area (m^2)	Talus volume (m^3)	Total volume (m^3)
B	5 m	1	Boulder/Sand	700	6,900	~13,000
		3	Sand	2,200	6,000	
A	5 m	2	Boulder/Sand	350	3,400	~6,300
		4	Sand	1,500	2,900	

In 2008 a rainfall-induced earth/debris flow occurred below No. 24 Egnot Heights, with an approximate volume of about 100 m^3, which ran out many tens of metres downslope resulting in a claim lodged with the Earthquake Commission (N. Traylen, Geotech Ltd., personal communication 2013).

Aerial photographs of the site are available for various dates since 1940, and Google Earth imagery is available for 2003 and 2009. Table 4 summarises the main features noted.

Table 4 Summary of observations from aerial photographs used to assess the site history at the Defender Lane mass movement.

Date/scale of photo	Resolution	Comments
1940 1:10,000 (approx.)	Poor resolution	Eight small areas of bare ground (Areas 1–8 showing as white in Figure 5, are present along the slope facing towards the northeast appear to indicate eroded/unstable loess slope areas. No buildings are present on the slope, and the slope appears vegetated with trees or dense scrub. Several gullies are apparent in the surface of the slope. No buildings appear present immediately below the slope with only some present along 26–28 Taupata Street. Scouring along the gully A and B is apparent in Areas 1 and 7 (Figure 5). There appears to be a cleared track partly along what is now Egnot Heights.
1946 1:5,500 (approx.)	Good resolution	Several small possible landslide scars are apparent in the loess along the cliff crest, e.g., Areas 1, 5, 7 and 8 (Figure 6). Debris from Gully A is clearly visible. This debris has been later excavated for housing development. Scouring along Gullies A and B is visible. No housing development along cliff edge of Defender Lane or beneath the slope face has yet taken place.
1973 1:10,000 (approx.)	Poor resolution	Most of the buildings currently present along the front on Taupata Street appear to have been constructed. Some scouring is apparent along the slope face south of Gully A, Area 12, (Figure 7). Development has occurred along Egnot Heights, Area 13 (Figure 7). No housing development has yet occurred along the slope crest of Defender Lane or beneath the slope face.
1984, 1:6,000 (approx.)	Good resolution	No housing development has yet occurred along the slope crest of Defender Lane or beneath the slope face. Land modification and clearing appears to have commenced beneath the south facing slope of what is now Egnot Heights, Area 21 (Figure 8). Some scouring is apparent on the northern flank of Gully A, Area 20, (Figure 8).
2003	Google Earth Historical Imagery	Houses have now been constructed along the slope crest of Defender Lane. Gully A has been partially backfilled for housing development of 24 Egnot Heights.
4 March 2009	Google Earth Imagery	Most housing developments along the slope crest are now completed. The head of Gully A has now been backfilled for development.
23 October 2009	Google Earth Imagery	Building works at 30B Taupata Street beneath slope. Earth dam works and excavation undercutting talus beneath slope to contain Gully A runoff.
15 February 2009	Google Earth Imagery	Bare ground exposed at slope behind 30B Taupata Street.

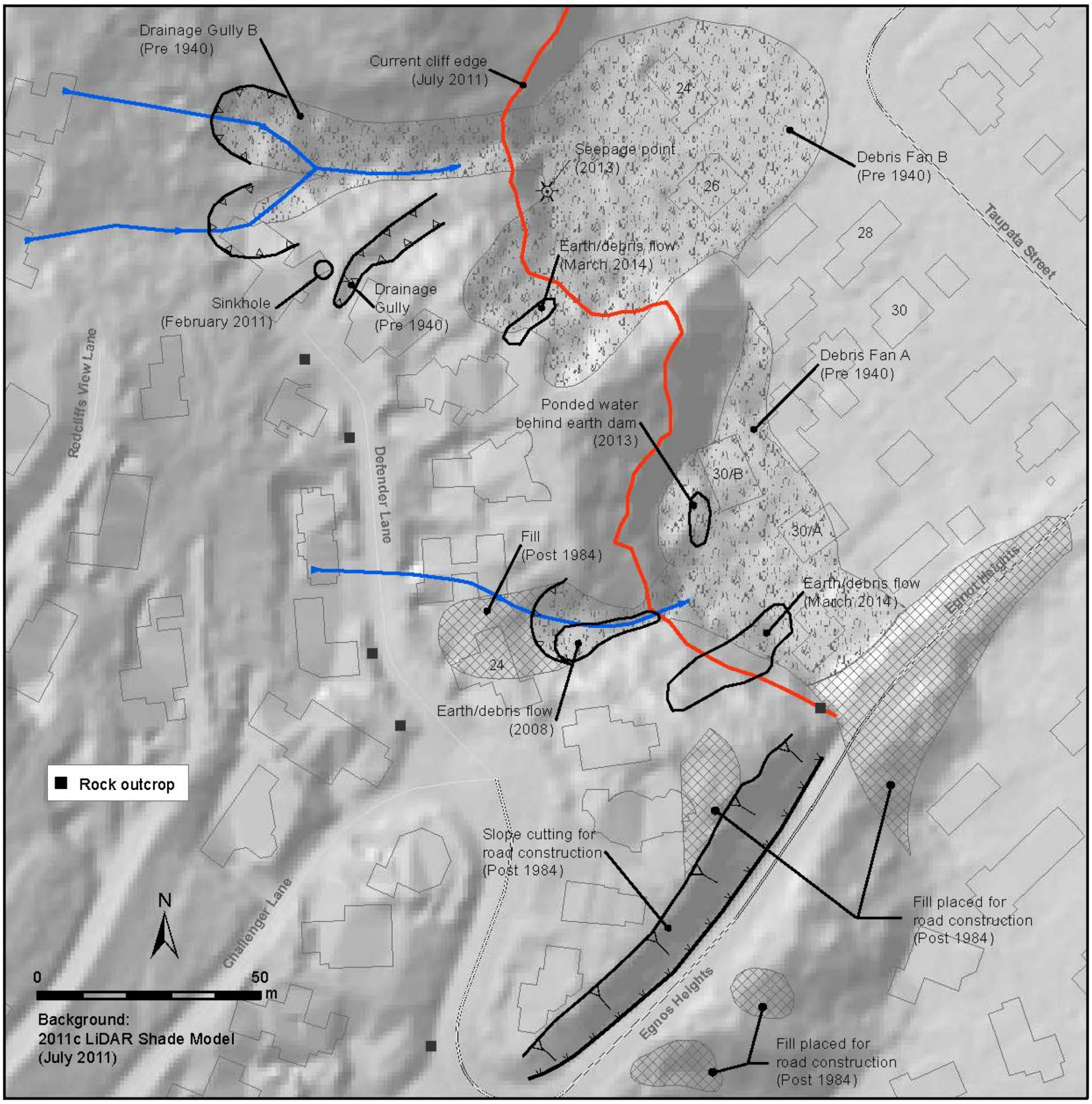


Figure 4 Main features identified at the site from field mapping and the interpretation of historical aerial photographs.

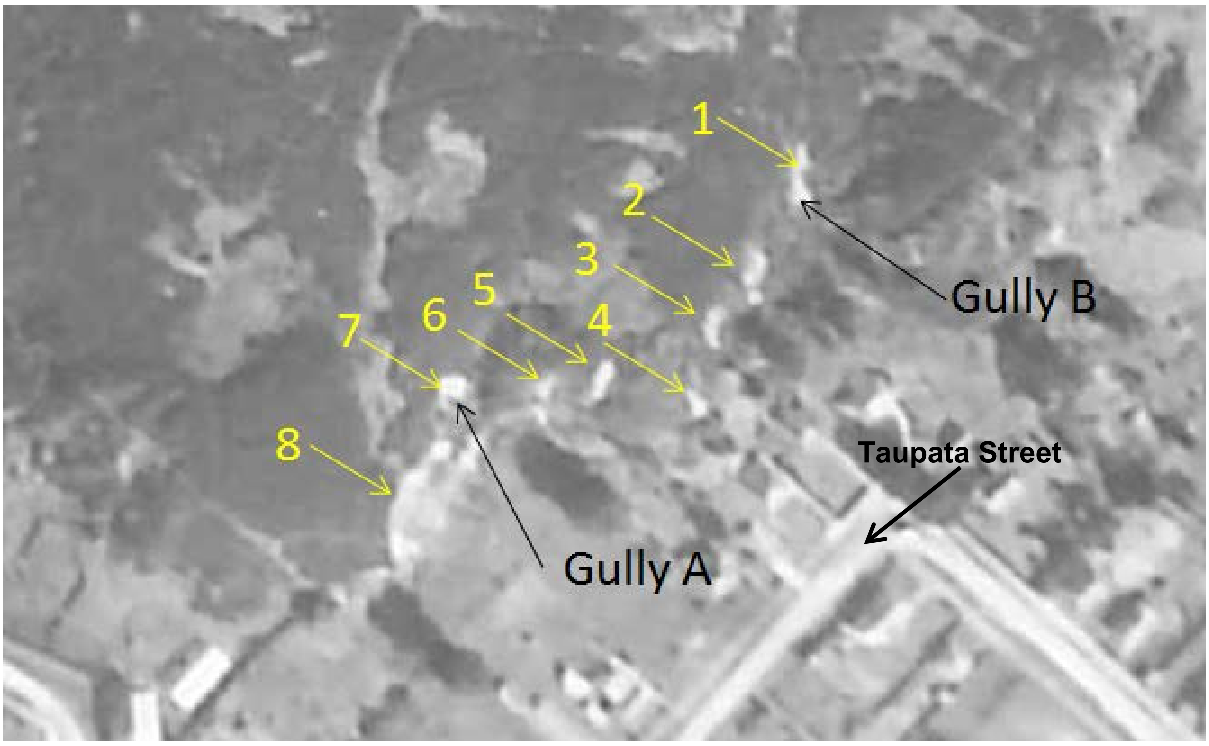


Figure 5 1940 aerial photograph showing gullies A and B and several areas of bare ground/possible landslide scars numbered 1–8.

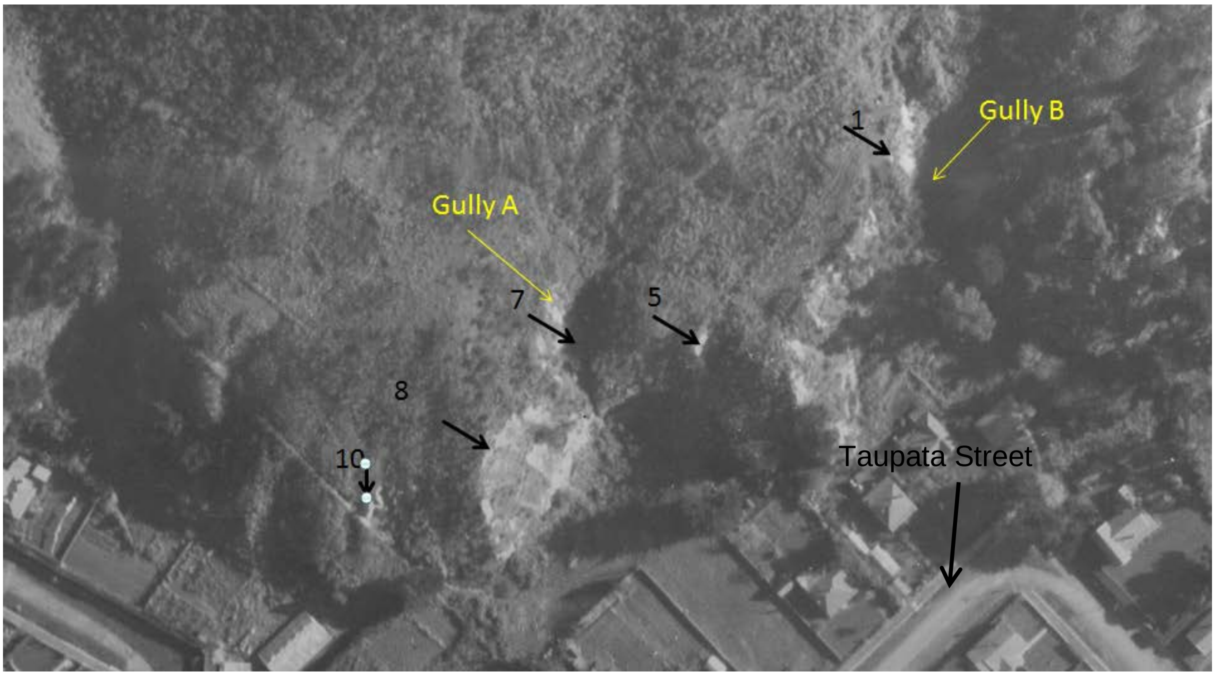


Figure 6 1946 aerial photograph, showing gullies A and B and several areas of bare ground/possible landslide scars (numbered).

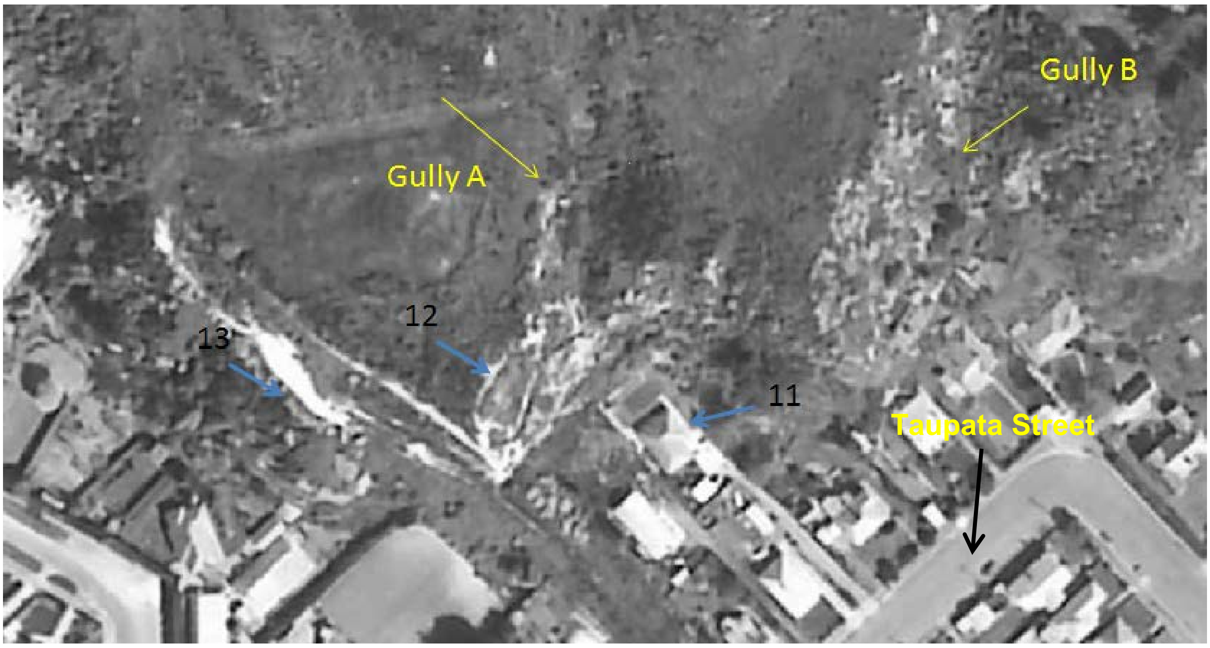


Figure 7 1973 aerial photograph, showing gullies A and B and several areas of bare ground/possible landslide scars (numbered).

Figure 8 1984 aerial photograph, showing gullies A and B and several areas of bare ground/possible landslide scars (numbered).

3.1.2 Before the 2010/11 Canterbury earthquakes

- No large-scale slope deformation has been reported since European settlement (approximately 1840 AD) (Paperspast).
- Geomorphological expressions of several landslide scars, within the assessment area and on the adjacent slopes, are apparent in the 1940 aerial photographs (the date of the earliest available aerial photographs) (Figure 4). These appear to be relatively old and probably pre-date European settlement (1840 AD).
- The debris from these scars appears to have formed two debris fans (A and B, Figure 4) at the slope toe. The combined volume of debris forming each debris fan is about 6,000 m^3 and 13,000 m^3 for fans A and B respectively.

- There is no evidence in the aerial photographs (1940, 1946, 1973, 1975, 1984 and 2011) of past quarrying at the site.
- No cracking was reported or observed following the 4 September 2010 earthquake.

3.1.3 2010/11 Canterbury earthquakes

- *22 February 2011 earthquakes:* The mapped crack distributions (in loess) at the cliff crest (shown on the maps in the Stage 1 report) were mainly generated by the 22 February 2011 earthquake. Horizontal permanent displacement of the slope crest in response to this earthquake has been inferred from analysis of geodetic survey marks and measuring of crack apertures):
  - Available Cadastral survey marks were measured by GNS Science to detect absolute ground movements spanning the earthquake period from before the 22 February 2011 earthquakes to October 2012. The results of this survey are contained in Appendix 2. Vector displacements within the site indicate permanent ground displacements that range up to 160 mm towards the east (e.g., Cadastral mark ID 60, Map 2 Appendix 2). These cadastral survey points however are located on the upper part of the Defender Lane mass movement slope, and away from the slope crest where permanent ground displacements were larger.
  - Horizontal permanent displacement of the slope crest in response to this earthquake (inferred from the measuring of crack apertures) varied between approximately 200–300 mm for the southern spur and approximately 500–600 mm for the northern spur.
  - In addition to the cracking in loess, several cliff collapses and rockfalls occurred from the steep rock slope below Defender Lane. These rockfalls impacted several dwellings at the slope toe, near No. 30A Taupata Street.
- *16 April 2011 earthquake:* No displacement of the cliff top or opening of the mapped cracks was reported or detected by GNS Science.
- *13 June 2011 earthquake:* Horizontal permanent displacement of the cliff crest in response to this earthquake has been inferred from measurements of crack apertures carried out by the Port Hills Geotechnical Group of consultants. These data and observations comprised:
  - Comparison of crack apertures measured pre and post the 13 June 2011 earthquake indicate cumulative displacements ranging from 10 to 30 mm towards the east for both the northern and southern spurs.
  - In addition to the cracking in loess, several cliff collapses and rockfalls occurred from the steep rock slope below Defender Lane. These rockfalls impacted several dwellings at the slope toe, near No. 30A Taupata Street.
- *23 December 2011 earthquake*: No displacement of the cliff top or opening of the mapped cracks was reported or detected by GNS Science. Following the 13 June 2011 earthquake, 11 survey monitoring marks were installed by Aurecon NZ Ltd. along the northern spur crest on the 18 June 2011 (Appendix 2). Measurements of these monitoring marks after the 23 December 2011 earthquake, however, carry excessive errors, which precludes the determination of any permanent ground displacements caused by this earthquake. From the 14 September 2011, an additional 33 monitoring marks were installed by Aurecon NZ Ltd. in and around the Defender Lane mass movement.

3.1.4 After the 2010/11 Canterbury earthquakes

The loess slope at the Defender Lane mass movement has many open cracks and is prone to inflow of surface water. Seepage points have been identified by GNS Science on the cliff face below the loess/rockhead boundary, e.g., behind 26 Taupata Street. However, these appear to have been pre-existing (before the 2010/11 earthquakes), as several residents had constructed "ad-hoc" drainage channels to accommodate the water.

Several small ravelling failures (shallow surface failure of debris <10 m^3 in volume) sourcing from the loess/rockhead boundary have been identified by GNS Science at different times after the 2010/11 earthquakes. For example, in August 2013 a less than 10 m^3 failure of the columnar joined basalt and loess material collapsed into the driveway of 28 Taupata Street.

Results from the assessment of the monitoring mark surveys carried out by Aurecon NZ Ltd. to 26 June 2013, indicates that no detectable movement – outside survey error – of the ground has occurred during this period.

Information provided to GNS Science by the Port Hills Geotechnical Group of Consultants indicates that two small failures occurred in the loess slope below Defender Lane during the 3–6 March 2014 rainstorms (total rainfall 118 mm). An earth/debris flow of approximate volume 10–15 m^3 sourced from the slope below 12 Defender Lane, and a slightly larger earth/debris flow of volume 30–40 m^3 sourced from the slope below 22 Defender Lane (Appendix 2). An erosion hollow (probably formed as a result of the collapse of a local tunnel gully collapse) identified in the garden of 12 Defender Lane after the 22 February 2011 earthquakes, also appeared to have enlarged as a result of the rain.

3.2 SITE INVESTIGATIONS

3.2.1 Geomorphological mapping

The results from the field mapping of slope morphology, interpreted surface materials and their genesis, surface deformation mapping and other relevant information are shown in Figure 9.

The site consists of two east-facing loess spurs, which form a distinct flatter slope overlying volcanic rock. The total slope is about 35 m high and rises above ancient abandoned beach deposits left behind after the sea retreated. The lower part of the slope is formed of volcanic rock and is typically inclined between 60° and 90°, which is overlain by a more gentle loess slope, inclined between 45° and 10°. The two spurs are separated by small gullies, and the lower steeper rock slopes are former coastal cliffs.

3.2.2 Subsurface drilling

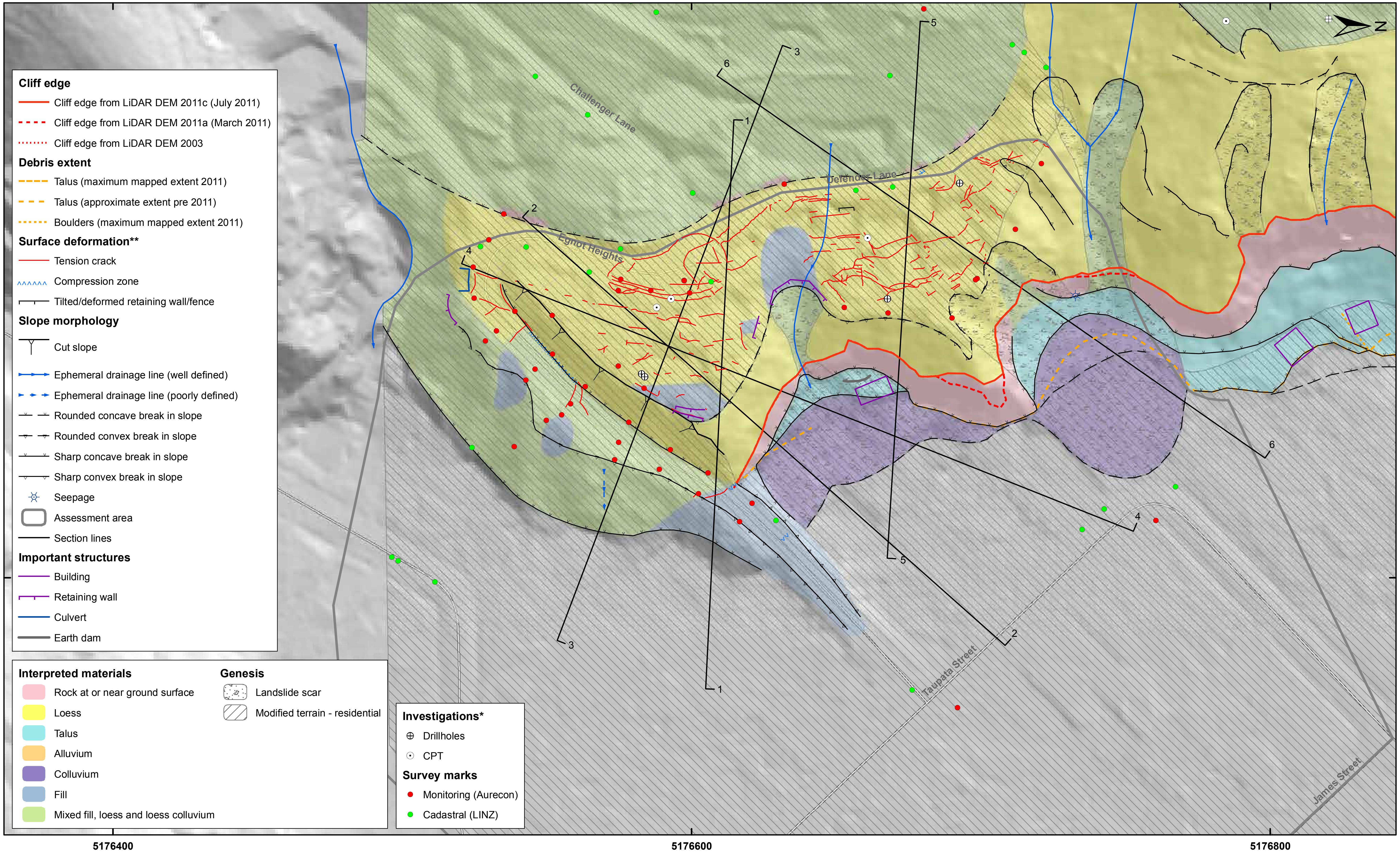
Subsurface information is available from drillholes, drillcores and cone penetrometer testing, and piezometers and inclinometers carried out by Tonkin and Taylor Ltd., for the Earthquake Commission, following the 22 February 2011 earthquake (Tonkin and Taylor, 2012a) (Table 5). The locations are shown on Figure 9.

Generally the Tonkin and Taylor Ltd. drillholes were relatively shallow and did not extend deeper than the upper rock unit. The drillcore logs and core photos are available in Tonkin and Taylor (2012a).

An engineering geological map and cross-section interpretations through the site showing the location and depth of the drillholes are presented in Figures 9 and 10 respectively. Figure 11 presents six cross-sections through the area, and Figure 12 is a map showing rockhead (boundary between the loess/colluvium and underlying rock) levels across the site, inferred from field mapping and the site investigation results.

Table 5 Summary of drilling investigations (from Tonkin and Taylor, 2012a).

Vertical drilholes (HQ-Cored)	Cone penetrometer (CPT) Holes	Installed equipment piezometer	Installed equipment inclinometer
DH-DFR-01	CPT-DFR-01	PZ-DFR-01 (installed in the CPT hole)	INC-DFR-01 (installed in the drillhole)
DH-DFR-02 DH-DFR-02A	CPT-DFR-02	PZ-DFR-02	INC-DFR-02 (installed in the drillhole)
DH-DFR-03		PZ-DFR-03	
	CPT-DFR-04	PZ-DFR-04	



SCALE BAR: 0 50 100 m

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

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

DRW: BL

CHK: FDP, CM

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ENGINEERING GEOLOGY MAP

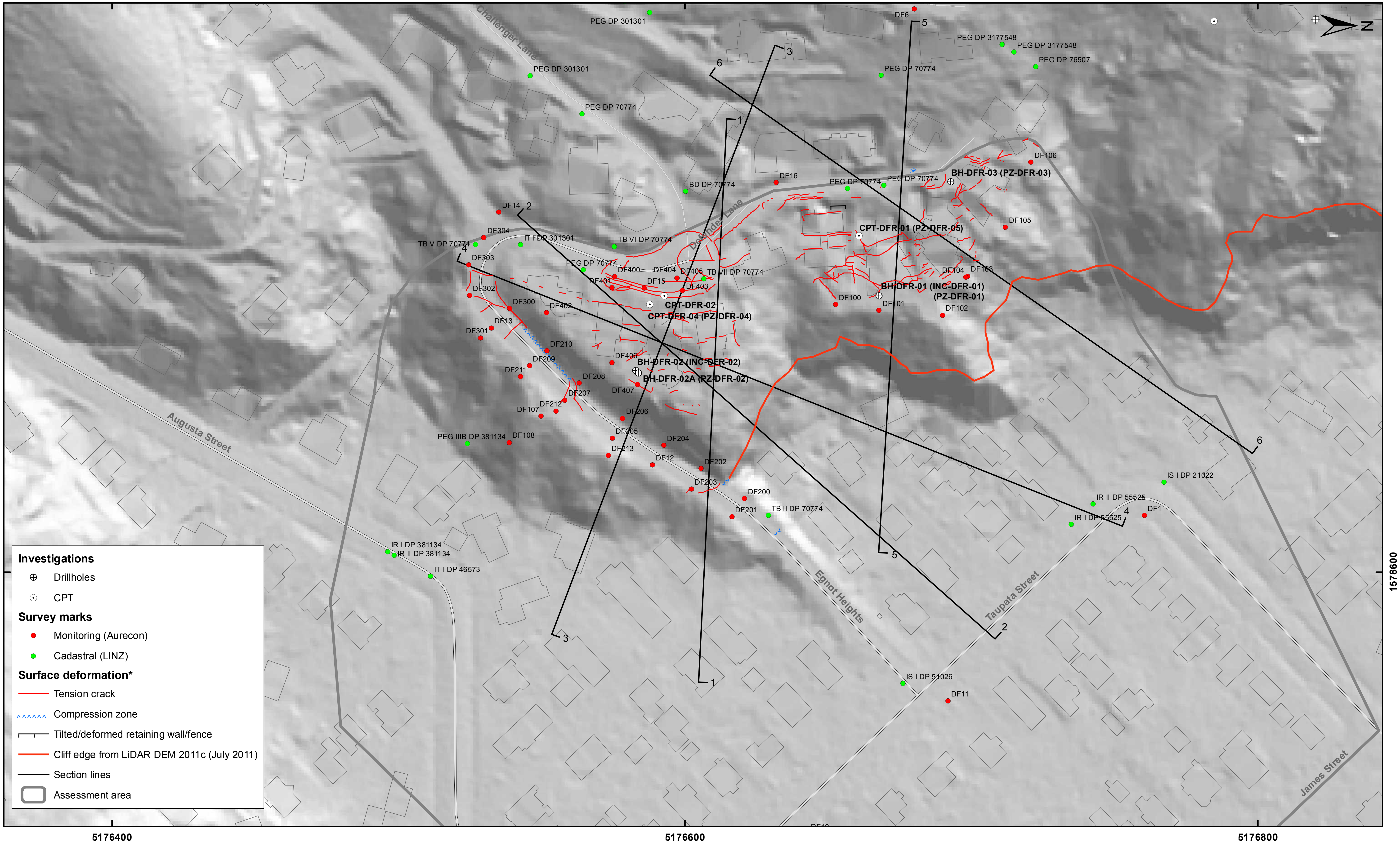
Defender Lane
Christchurch

FIGURE 9

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

*Taken from the report 2012/317

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

DRW: BL

CHK: FDP, CM

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SITE INVESTIGATION MAP

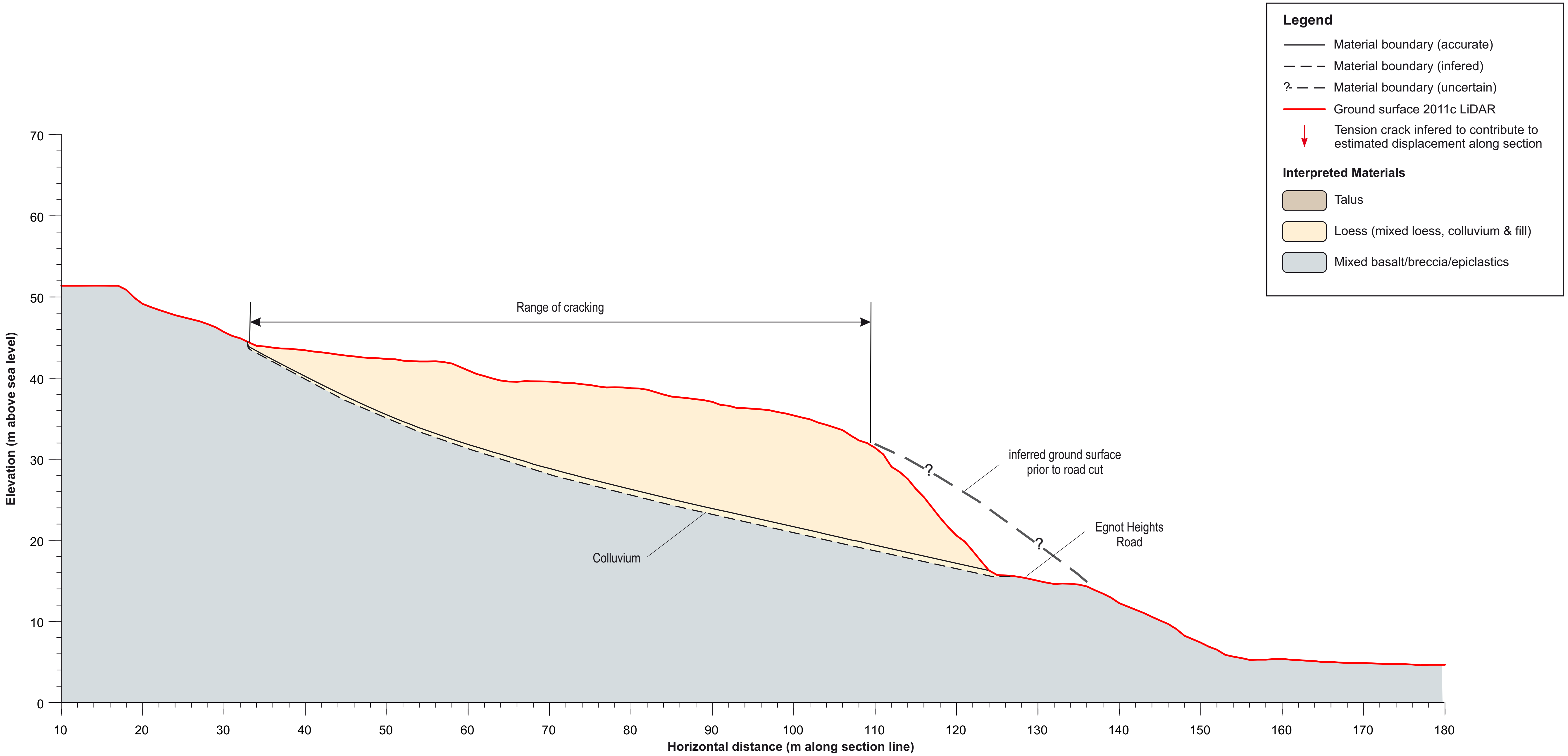
Defender Lane
Christchurch

FIGURE 10

FINAL

REPORT: CR2014/67

DATE: June 2014



Base of loess interpolated from drill hole and outcrop.
Base of colluvium ~1.4m thickness intercepted in hole DH DFR 02

DRW: PC

CHK: CM

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ENGINEERING GEOLOGY CROSS SECTION 1

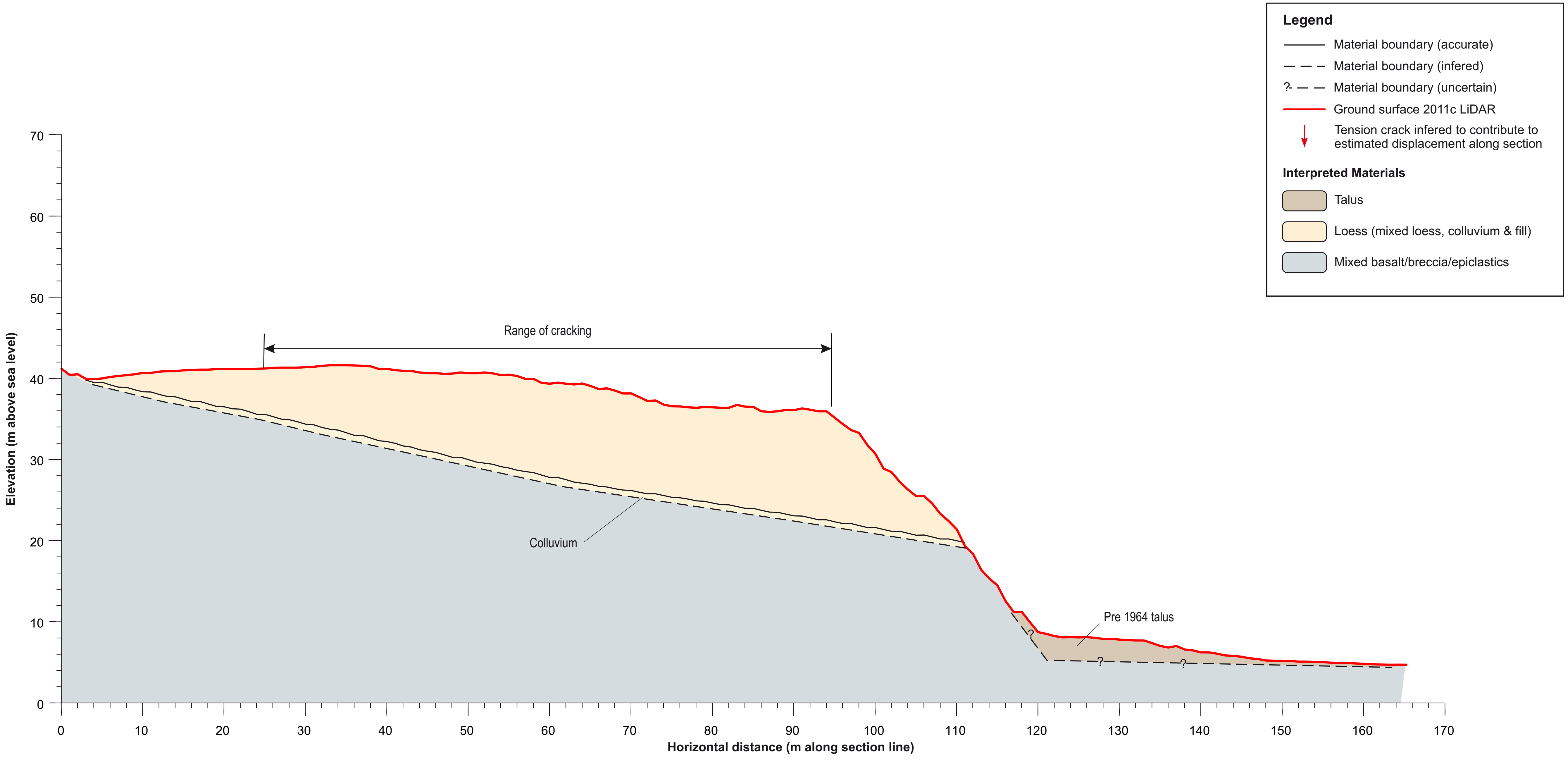
Defender Lane
Christchurch

FIGURE 11

FINAL

REPORT: CR2014/67

DATE: June 2014



Base of loess interpolated from drill hole and outcrop.
Base of colluvium ~1.4m thickness intercepted in hole DH DFR 02

DRW:
PC

CHK:
CM

ENGINEERING GEOLOGY CROSS SECTION 2

FIGURE 11

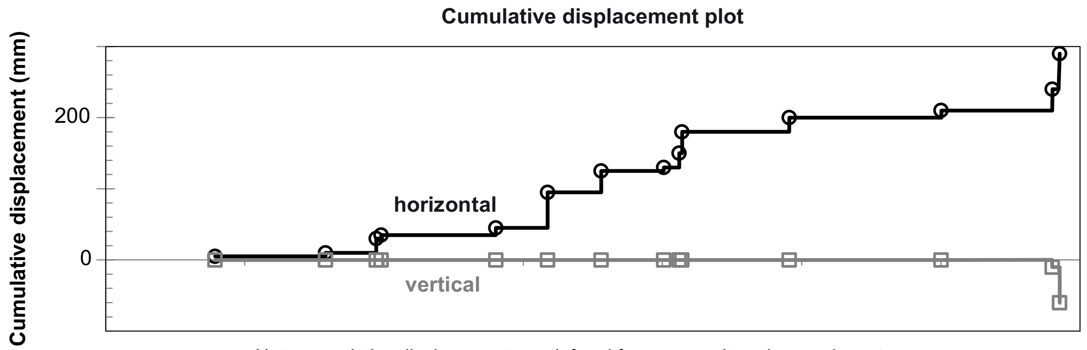
Defender Lane
Christchurch

FINAL

REPORT:
CR2014/67

DATE:
June 2014

Cumulative displacement plot



Note: cumulative displacements are infered from mapped tension crack apetures.

Borehole DH DFR 02 (T&T) summary

	Material type	Weathering	Strength	RQD %
①	Top Soil			
②	Loess		F	
③	Colluvium		F	
④	Basalt breccia	SW	MS	60
⑤	Basalt lava	SW	MS	100
⑥	Basalt lava breccia	MW	W	100
⑦	Basalt lava breccia	SW	W	0
⑧	Basalt lava breccia	MW	W	20
⑨	Basalt lava breccia	MW	W	33

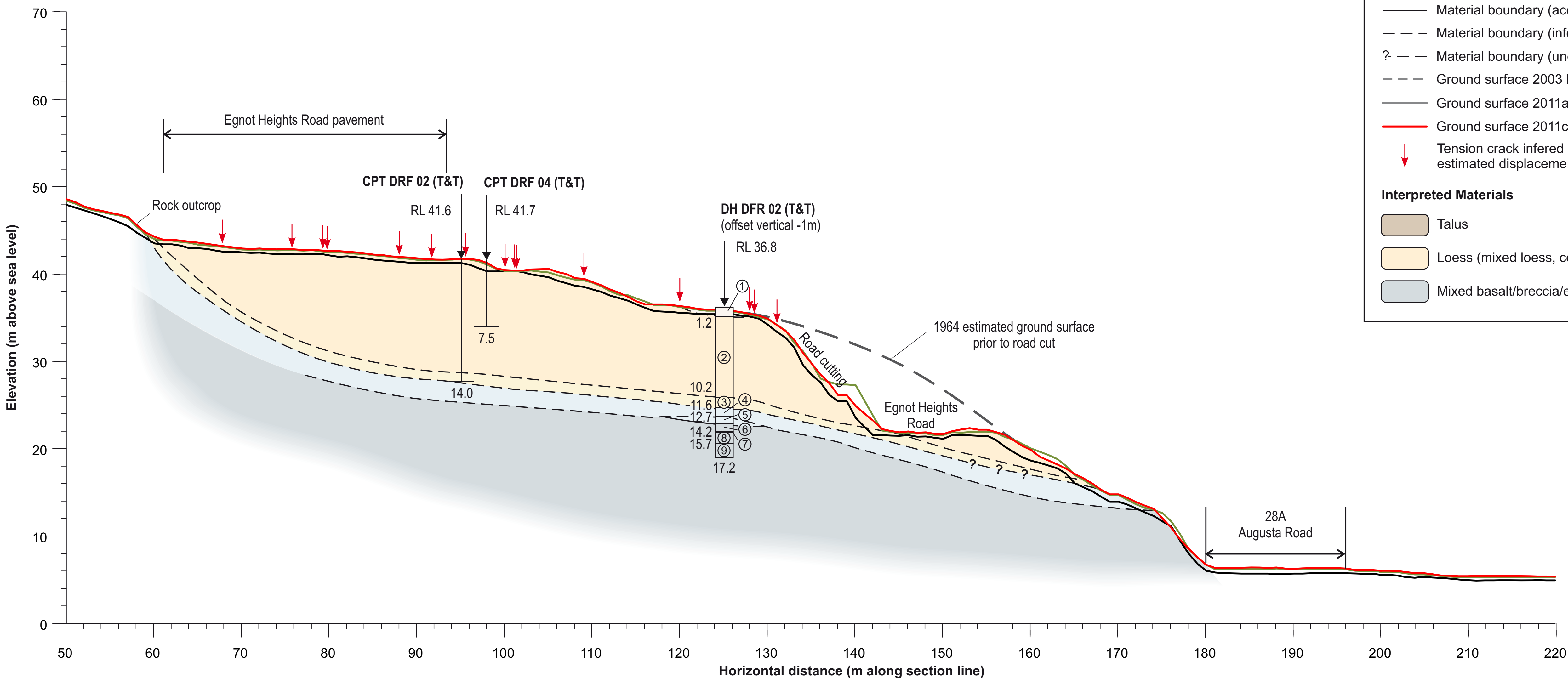
Note: for colour legend refer to interpreted materials

Legend

- Material boundary (accurate)
- Material boundary (infered)
- Material boundary (uncertain)
- Ground surface 2003 LiDAR
- Ground surface 2011a LiDAR
- Ground surface 2011c LiDAR
- Tension crack infered to contribute to estimated displacement along section

Interpreted Materials

- Talus
- Loess (mixed loess, colluvium & fill)
- Mixed basalt/breccia/epiclastics



Base of loess interpolated from drill hole and outcrop.
Base of colluvium ~1.4m thickness intercepted in hole DH DFR 02

DRW:
PC

CHK:
CM

ENGINEERING GEOLOGY CROSS SECTION 3

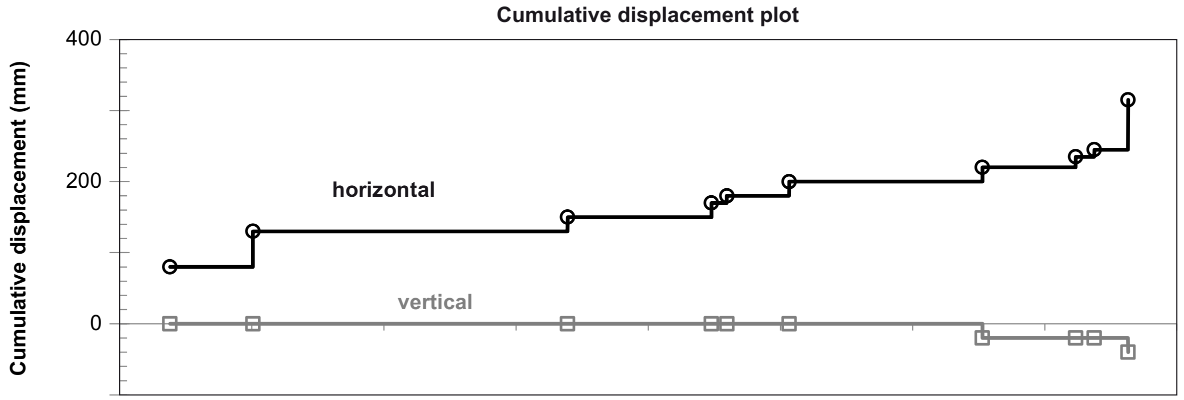
Defender Lane
Christchurch

FIGURE 11

FINAL

REPORT:
CR2014/67

DATE:
June 2014



Note: cumulative displacements are infered from mapped tension crack apetures.

Borehole DH DFR 02 (T&T) summary

	Material type	Weathering	Strength	RQD %
1	Top Soil			
2	Loess		F	
3	Colluvium		F	
4	Basalt breccia	SW	MS	60
5	Basalt lava	SW	MS	100
6	Basalt lava breccia	MW	W	100
7	Basalt lava breccia	SW	W	0
8	Basalt lava breccia	MW	W	20
9	Basalt lava breccia	MW	W	33

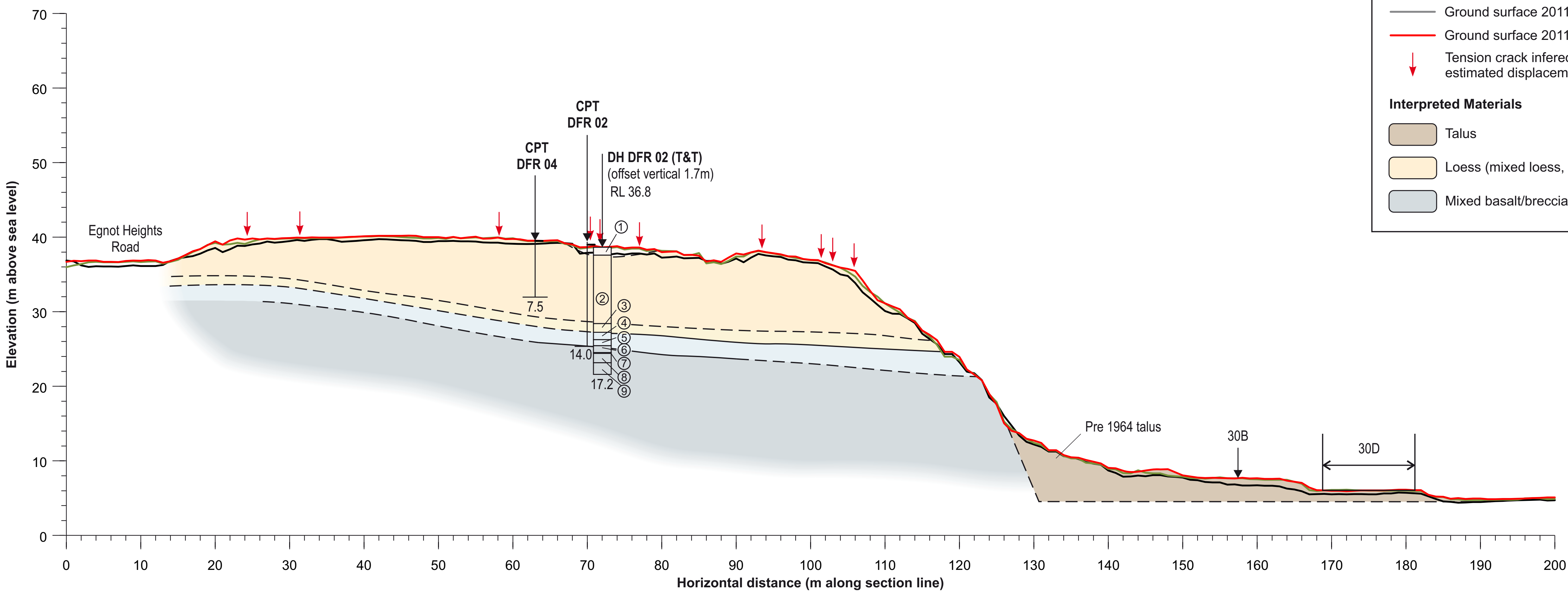
Note: for colour legend refer to interpreted materials

Legend

- Material boundary (accurate)
- Material boundary (infered)
- Material boundary (uncertain)
- Ground surface 2003 LiDAR
- Ground surface 2011a LiDAR
- Ground surface 2011c LiDAR
- Tension crack infered to contribute to estimated displacement along section

Interpreted Materials

- Talus
- Loess (mixed loess, colluvium & fill)
- Mixed basalt/breccia/epiclastics



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

DRW: PC

CHK: CM

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ENGINEERING GEOLOGY CROSS SECTION 4

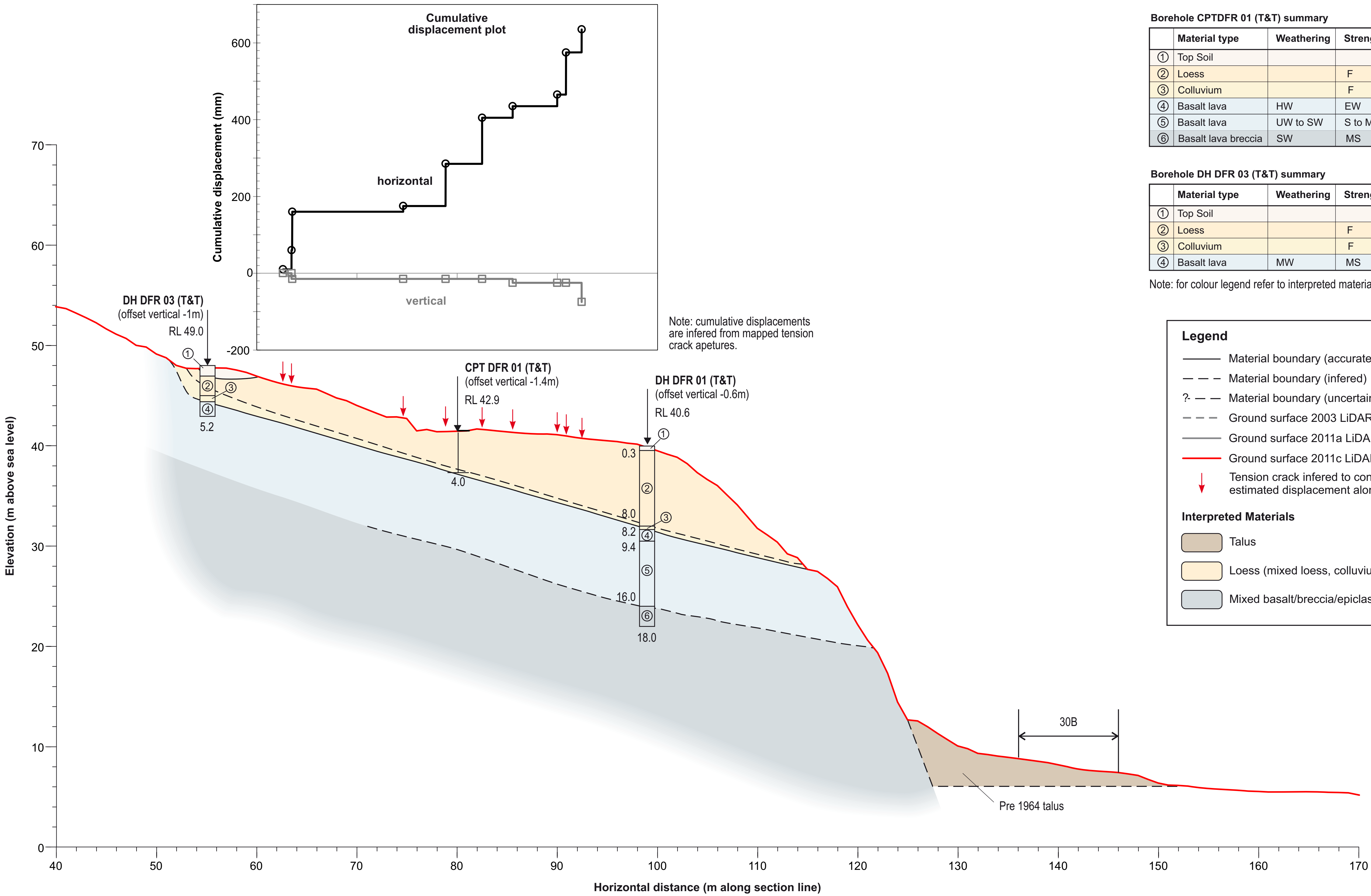
Defender Lane Christchurch

FIGURE 11

FINAL

REPORT: CR2014/67

DATE: June 2014



Borehole CPTDFR 01 (T&T) summary

	Material type	Weathering	Strength	RQD %
1	Top Soil			
2	Loess		F	
3	Colluvium		F	
4	Basalt lava	HW	EW	
5	Basalt lava	UW to SW	S to MS	
6	Basalt lava breccia	SW	MS	

Borehole DH DFR 03 (T&T) summary

	Material type	Weathering	Strength	RQD %
1	Top Soil			
2	Loess		F	
3	Colluvium		F	
4	Basalt lava	MW	MS	

Note: for colour legend refer to interpreted materials

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

DRW: PC

CHK: CM

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ENGINEERING GEOLOGY CROSS SECTION 5

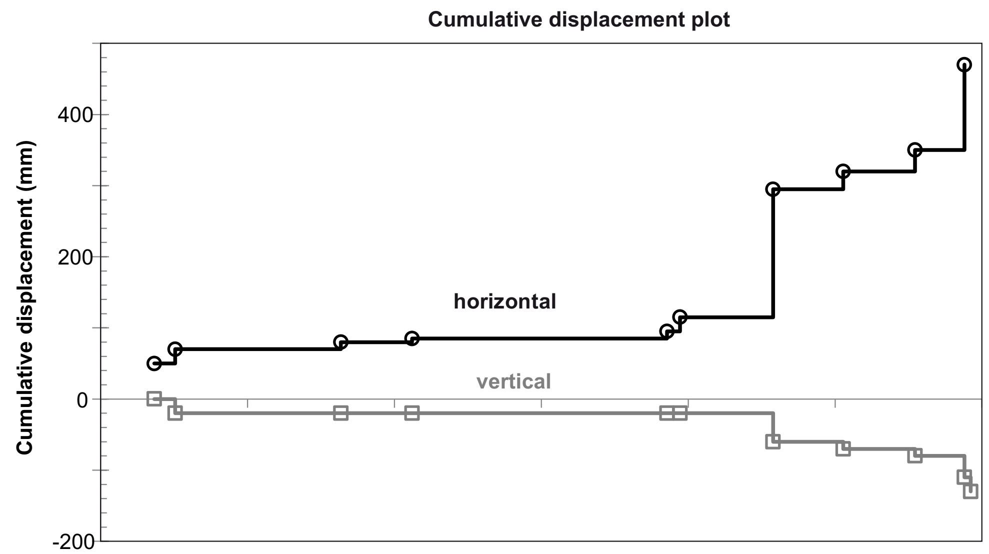
Defender Lane Christchurch

FIGURE 11

FINAL

REPORT: CR2014/67

DATE: June 2014

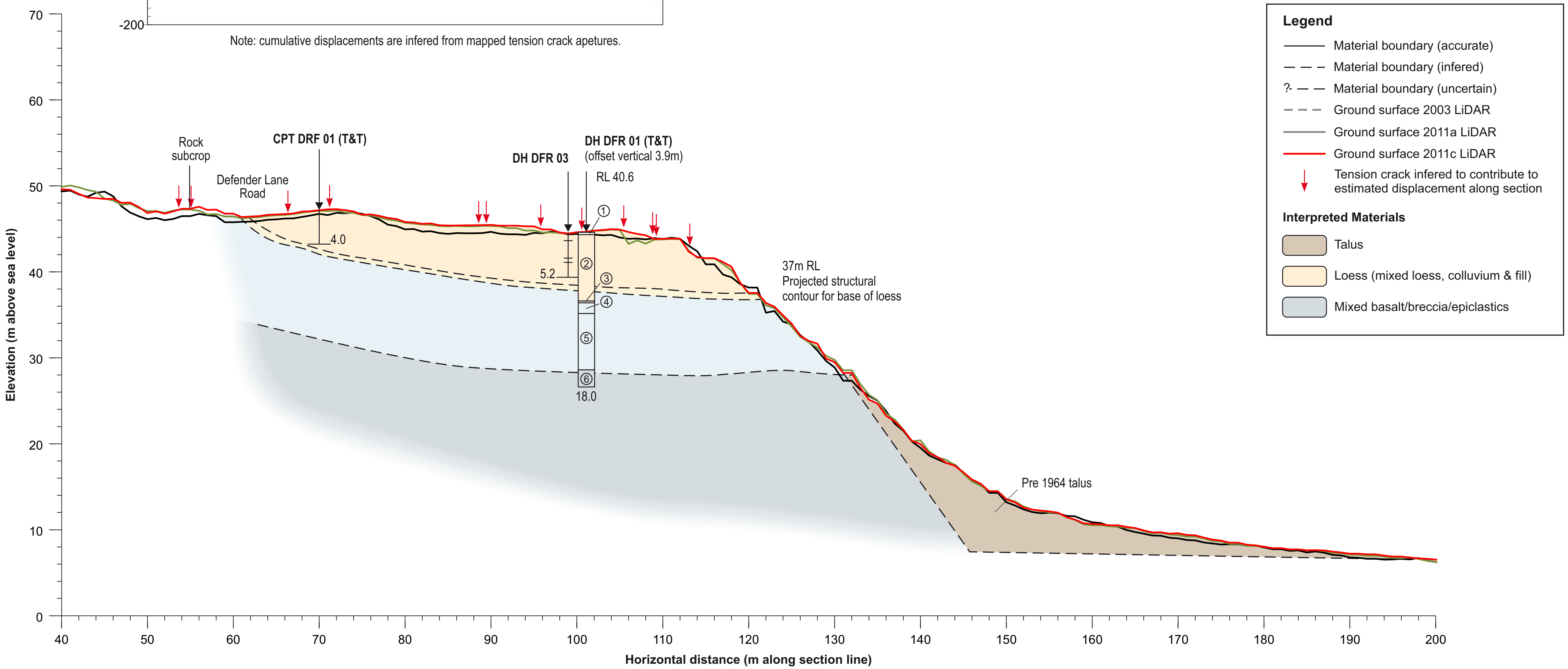


Note: cumulative displacements are infered from mapped tension crack apetures.

Borehole DH DFR 01 (T&T) summary

	Material type	Weathering	Strength	RQD %
1	Top Soil			
2	Loess		F	
3	Colluvium		F	
4	Basalt lava	HW	EW	
5	Basalt lava	UW to SW	S to MS	
6	Basalt lava breccia	SW	MS	

Note: for colour legend refer to interpreted materials



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

DRW: PC

CHK: CM

ENGINEERING GEOLOGY CROSS SECTION 6

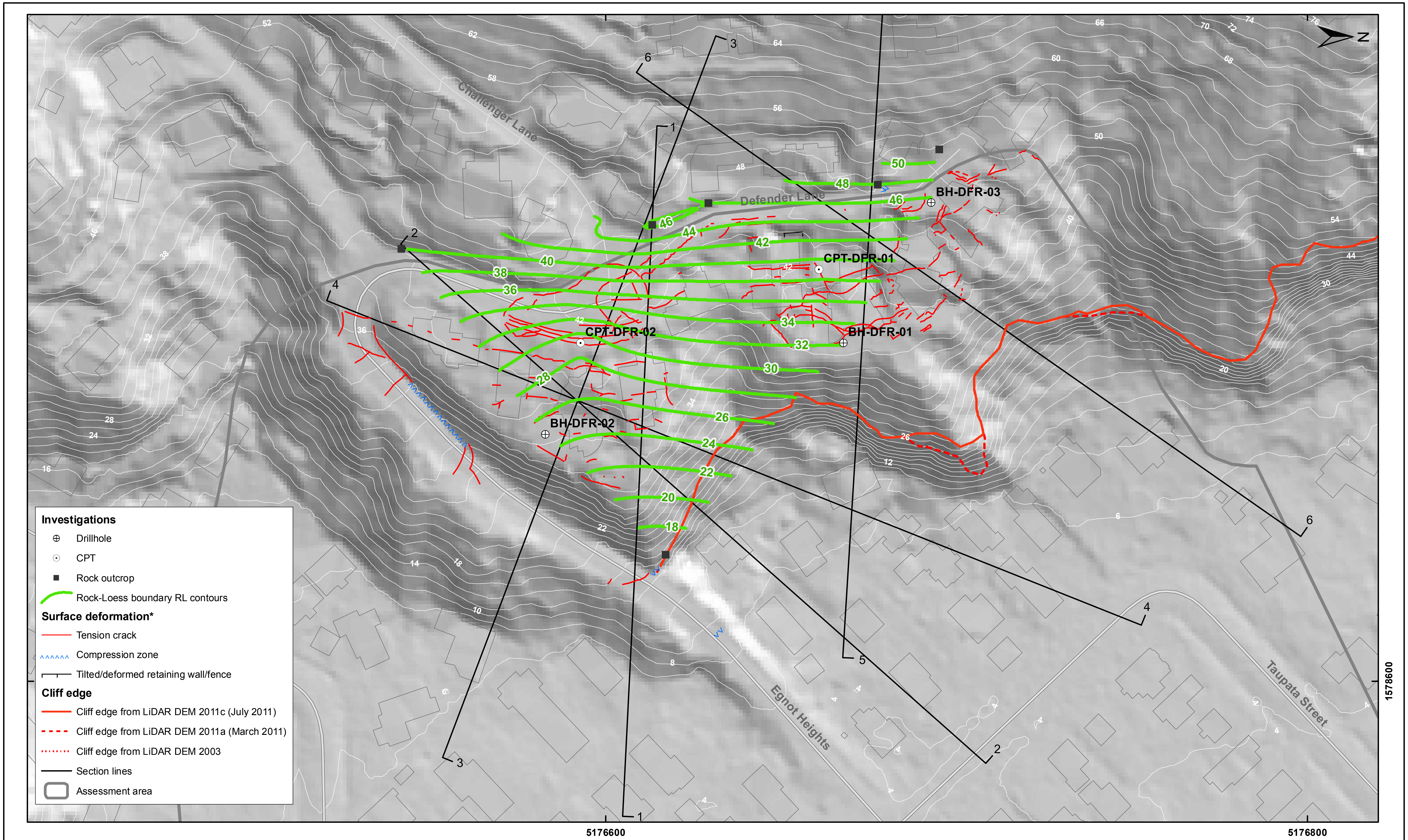
Defender Lane Christchurch

FIGURE 11

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report 2012/317

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

DRW: BL

CHK: FDP, CM

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INTERPOLATED ROCK- LOESS BOUNDARY

Defender Lane
Christchurch

FIGURE 12

FINAL

REPORT: CR2014/67

DATE: June 2014

3.2.3 Surface movement

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

1. Cadastral survey marks (details held by Land Information New Zealand) i.e., property boundaries and roads footpaths etc.; and
2. Monitoring survey marks installed by Aurecon NZ Ltd. for Christchurch City Council, to monitor surface displacement.
3. Mapping of crack apertures at the slope crest carried out by M. Yetton (Geotechnics Ltd.).

The survey datasets (1 and 2) adopt reference control marks that are outside the area of movement, but still within the local area. Therefore, any regional offsets caused by tectonic displacements are removed from the data.

3.2.3.1 Surveyed slope displacements

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

Post-earthquake displacements were inferred from cadastral surveys (Appendix 2) and range up to approximately 160 mm. However, the location of the survey marks is limited to sites away from the slope edge where the largest permanent displacements, inferred from measuring crack apertures, were recorded.

The monitoring survey marks installed by Aurecon NZ Ltd. were installed in October 2011 after the 22 February, 16 April and 13 June 2011 earthquakes. The results from the surveying of the monitoring marks after the 23 December 2011 earthquake, however, have excessive systematic errors in them, which precludes the identification of any permanent ground displacements (Appendix 2).

3.2.3.2 Inferred slope displacement from crack apertures

Displacements of the slope crest during the 22 February and 13 June 2011 earthquakes are mainly inferred from measured tension-crack apertures, following the method described in Massey et al. (2013). Crack apertures – relative displacements across cracks in both the horizontal and vertical directions – were measured with a tape measure at locations that were thought to best represent the overall displacement across the crack. Cracks with apertures of less than 5 mm were generally ignored, and so the inferred total displacements of the cliff edge are slightly underestimated.

The mapped crack distributions (in loess) at the cliff crest (shown on the maps in the Stage 1 report) were mainly generated by the 22 February 2011 earthquake. Only minor cracking was identified by Port Hills Geotechnical Group consultants after 13 June 2011 earthquakes and so it is inferred that the displacements recorded across crack apertures are mainly in response to the 22 February 2011 earthquakes. Displacement of the cliff crest in response to this earthquake have been estimated for each cross-section, by adding together the mapped crack apertures (openings) along the cross-sections (Figure 11, cross-sections 3–6). Total inferred displacements along each cross-section are summarised in Table 6.

Table 6 Measured cumulative crack apertures which formed during the 22 February and 13 June 2011 earthquakes, measured by the Port Hills Geotechnical Group. Displacements are obtained from field mapping of tension crack apertures along survey lines. Errors are nominally estimated as being ±5 mm.

	Vertical component		Horizontal component		Resultant vector		Apparent dip of loess/rock interface (º)
Cross-section	**All surveyed (mm)**	**Near the slope crest (mm)**	**All surveyed (mm)**	**With vertical component only (mm)**	**Magnitude (mm)**	**Dip (º)**	
Southern Spur							
3	60	60	290	80	100	37	10
4	40	40	315	115	122	19	3
Northern Spur							
5	75	60	635	230	238	15	15
6	130	110	500	385	400	16	4

There are no test pits available at the site to determine the depth of the cracks or if the surface cracks observed in the loess extend down into bedrock. However, field mapping of exposures of loess at the Richmond Hill cliff crest show that cracks in the bedrock do not necessarily extend upwards into the loess, and that surface cracks in loess do not propagate into bedrock (Massey et al., 2014). Some cracks were also mapped along the side of the road (Egnot Heights and Defender Lane), which contain vertical components. However as these crack apertures were measured on tilted bitumen slabs, their relationship to surface ground movement is not known.

In general two distinct crack patterns were identified in the loess (and fill) at the cliff crest:

- Set 1 indicates mainly extensional (horizontal) displacements across cracks – occurring well back from the slope crest – and are inferred to be a function of shallow inelastic response of the loess (and fill) above rock head during shaking. However, in some locations there are cracks with >100 mm of vertical displacement recorded, these are thought to be related to localised collapse of the ground or to some other mechanism.
- Set 2 indicates both horizontal and vertical displacement – these cracks are thought to relate to deeper-seated deformation in the loess. These patterns are shown in Figure 13.

3.2.4 Sub-surface movement (Inclinometers)

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

Inclinometer tubes were installed in drillholes DH-DFR-01 (INCL-DFR-01) and DH-DFR-02 (INCL-DFR-02) sites (Figure 9 and Table 5) at the slope crest between July and October 2011. The inclinometer displacements are monitored at 0.5 m intervals and the inclinometer accuracy is quoted as ±6 mm over 25 m of tubing (Slope Indicator, 2005). Initial readings were conducted on 15 July 2011 (INCL-DFR-01) and 21 October 2011 (INCL-DFR-02) (Tonkin and Taylor, 2012a).

From the four epochs of survey presented by Tonkin and Taylor (2012a) there appears to be no sub-surface displacements of the inclinometer tubes outside survey error, within the period spanning the initial reading to the survey on 13 December 2012. However, two Geotechnics surveys are not shown on the inclinometer graphs reported by Tonkin and Taylor (2012a), namely: 20 June 2012 and 6 August 2012 surveys. The 6 August 2012 reading has a slight deflection in the monitoring tube installed in INCL-DFR-02 at the 8–10 m depth range, the deflection infers movement towards bearing 123°. This represents only one round of survey. Further surveys have shown no deflection outside survey error at this depth. The depth of the deflection in the omitted 6 August 2012 reading is coincident with the loess-rock (rockhead) interface, and the direction of movement is down dip of this surface and out of the slope towards Egnot Heights. From drillcore samples, the depth range coincides with the base of the loess above its contact with the thin layer of colluvial loess at 10.2–11.6 m. The omitted reading also follows a period when the piezometric head levels in June 2012 recorded in piezometer PZ-DFR-02 rose 6 m, above the piezometer tip (Table 8).

INCL-DFR-01 readings show a sinusoidal deflection between 8 and 12 m depth below ground level within the volcanic breccia. This shape of is typically associated with non-encapsulation of the inclinometer tube by the surrounding material during installation, implying that this is an artefact of the installation and not related to landside movement mechanisms. However, this cannot be verified as no inclinometer construction details are available in Tonkin and Taylor (2012a).

3.2.5 Groundwater

No consistent or systematic groundwater records are available for the period of monitoring, and the temporal resolution of the monitoring is inadequate to establish a link between pore pressures (water levels) and rainfall. Simultaneous measurements of all piezometers are unfortunately not available and so a relationship between piezometers cannot be established.

The only available groundwater data is from piezometers PZ-DFR-01, 02, 03 and 04, which were installed by Tonkin and Taylor (2012a) as shown in Figure 9 and Table 5. Manual recording of water levels within the piezometers (piezometric head levels) was reported by Tonkin and Taylor (2012a) starting in July 2011 and ending in July 2012. Manual recording means that the water levels in the piezometer are recorded only when a person goes to site and measures them, no automatic instruments were installed by Tonkin and Taylor Ltd.

A compilation of available piezometer construction details, the materials in which they have been installed and the measured piezometric head levels are shown in (Table 8).

There are only three rounds of measurements reported by Tonkin and Taylor (2012a), where water levels were recorded above the end of the drillholes, as shown in Table 8. A maximum daily rainfall (24-hourly rainfall recorded 9 am–9 am) occurred on 13 August 2012 and totalled 61 mm. The water levels within the piezometers were not recorded by Tonkin and Taylor (2012a) for this period. There are no records of water loss/gain of circulation or presence of perched groundwater from the drilling log records (Tonkin and Taylor, 2012a).

- The results from the limited piezometric head level records indicate piezometer PZ-DFR-01 was installed with the tip within the loess colluvium. The two measurement records show that piezometric head levels were 1.2 and 3.2 m above the base of the loess/colluvium.
- Piezometer PZ-DRF-02 was installed with the tip approximately 8 m below the Loess/colluvium base.
- Piezometer PZ-DFR-03 also appears to have been installed with the tip 1.5 m below the loess base of the loess/colluvium.
- Piezometer PZ-DFR-04 appears to have been installed with the tip 0.9 m below the base of the loess.
- Piezometer PZ-DFR-05 appears to be installed with its tip at the base of the loess, without apparent allowance for bentonite seal and pack sand.

The limited results show that for many of the piezometers the recorded piezometric head levels are below or at the loess/colluvium interface with the rock. The limited monitoring of water levels in these drillholes means they are of little use for this study.

Results from piezometer PZ-DFR-01 however, show that there may be perched groundwater water within the loess/colluvium, overlying rockhead during periods of rainfall. This is consistent with field observations of seepage points along the rock slope face, below the location of the piezometer. Site observations made during mapping of the slope by GNS Science, suggest the volcanic rock mass underlying the loess/colluvium is cracked and dilated in parts and that the permeability of these materials may be higher than the overlying loess/colluvium, suggesting that it may be acting as an under-drain. This is consistent with the seepage points and ad-hoc constructed drainage channels (by residents), that are apparent at the base of the rock slope.

Table 7 Summary of inclinometer measurements (Tonkin and Taylor, 2012a).

Inclinometer	Host drillhole	Bedded lithology	First-reading date	Subsequent reading dates	Source	Displacement record	Remarks
INCL-DFR-01	DH-DFR-01	Volcanic breccia	15 July 2011	12/10/2011 24/12/2012 1/6/2012 20/6/2012 6/8/2012 13/12/2012 13/03/2013 18/06/2013 13/03/2014	Tonkin and Taylor (2012a) Tonkin and Taylor (2012a) Tonkin and Taylor (2012a) Geotechnics Ltd. additional data Geotechnics (2014)	Anomalous Anomalous Anomalous Anomalous Anomalous Anomalous	Inadequate installation due to non-encapsulation.
INCL-DFR-02	DH-DFR-02	Volcanic Breccia	21 October 2011	20/12/2011 3/2/2012 1/6/2012 20/6/2012 6/8/2012 13/12/2012 13/03/2013 18/06/2013 13/03/2014	Tonkin and Taylor (2012a) Tonkin and Taylor (2012a) Tonkin and Taylor (2012a) Geotechnics Ltd. additional data Geotechnics (2014)	Within error Within error Within error Within error 4 mm deflection No deflection noted	6 m piezometric head level recorded in June 2012.

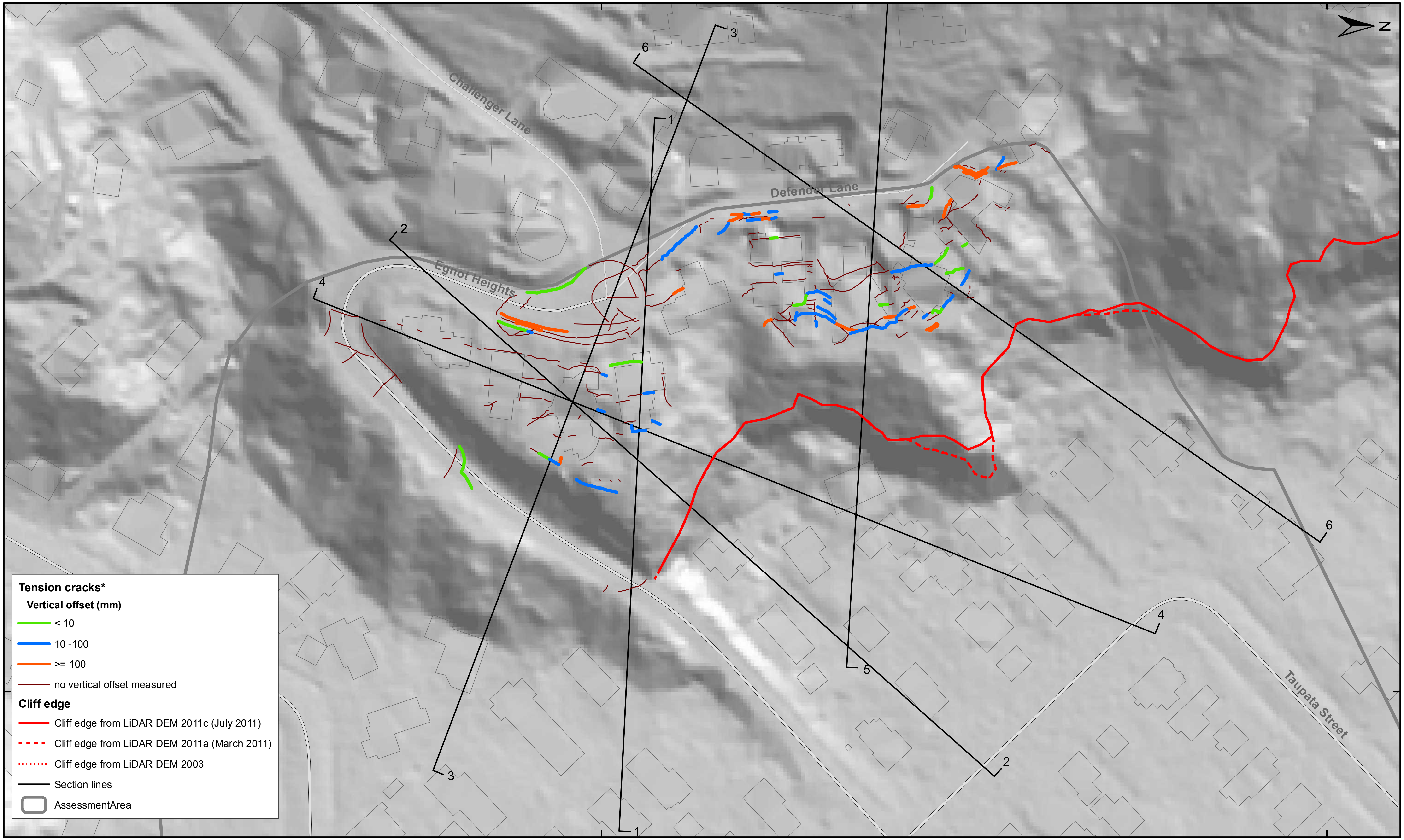
Table 8 Summary of piezometer measurements (Tonkin and Taylor, 2012a).

Piezometer ID	Location	Loess/ colluvium thickness (m)	Loess/ colluvium base (m AMSL[1])	Collar elevation (m AMSL)	Piezo depth (m)	Piezo[3] depth (m AMSL)	Level of piezometer tip above loess/ colluvium base (m)	Piezo Screen (m)	Reported piezometric head level changes and date recorded		
									Aug 2011	March 2012	June 2012
PZ-DFR-01	Slope Crest	8.2	32.4	40.6	8	32.6	0.2	2	~1.2m (33.8 m AMSL i.e., ~1.4 m above base of loess/colluvium)	~3m (35.6 m AMSL i.e., ~3.2 m above base of loess/ colluvium)	No measurement
PZ-DFR-02	Slope Crest	11.6	25.3	36.9	~17.4*	19.5	-7.9 (below loess)	Unknown[2]	No measurement	0 m	~6 m (22.5 m AMSL i.e., ~2.8m below base of loess/ colluvium)
PZ-DFR-03	Defender Lane	3.7	45.3	49	~5.2*	43.8	-1.5 (below loess)	Unknown[2]	No measurement	No measurement.	~2 m (45.8 m AMSL i.e., ~0.5 m above base of loess/colluvium)
PZ-DFR-04	Egnot Heights	~7	34.7	41.7	10.9*	33.8	-0.9 (below loess)	Unknown[2]	No measurement	0 m	No measurement
PZ-DFR-05	Defender Lane	~4	38.9	42.9	4*	38.9	-0 (at base of loess)	Unknown[2]	No measurement	0 m	No measurement

[1] Represents meters above mean sea level adopting the 2011c LiDAR survey data.

[2] The depth of the piezometer screen is unknown.

[3] EOH refers to the End of Hole.



SCALE BAR: 0 50 m

EXPLANATION:

* Taken from report 2012/317

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

DRW: BL

CHK: FDP, CM

GNS SCIENCE TE PŪ AO

MEASURED TENSION CRACK VERTICAL OFFSET

Defender Lane
Christchurch

FIGURE 13

FINAL

REPORT: CR2014/67

DATE: June 2014

3.3 ENGINEERING GEOLOGICAL MODEL

3.3.1 Slope materials

Slope materials interpreted from the logging of drillcores (Tonkin and Taylor, 2012a) and mapping of the cliff face are summarised in Figure 14–Figure 16. The boundaries and locations of the materials interpreted from the drillholes, drillcores and field mapping are shown on the engineering geology map and cross-sections in Figure 9 and Figure 11 respectively.

3.3.1.1 Fill

The slope crest has been backfilled for construction of residential homes over old gullies or reclaimed cliff edges. For example, fill material was used for the purpose of backfilling the retaining wall at 24 Egnot Heights where the earth/debris flow occurred in 2008. However, no records of the fill composition and compaction process carried out are available.

3.3.1.2 Talus

The pre-earthquake talus at the toe of the rock slope contains meter-sized boulders derived from the columnar jointed cliff face. Much of the finer sandy material within the talus, and extending past the limit of talus, appears to have sourced from the upper loess/colluvium slope, and deposited as a gently rising (about 5° in angle) debris fan. It is not known whether the fan has accumulated in response to a few infrequent large events, or by the gradual accumulation of debris from more frequent but smaller events

3.3.1.3 Loess

The loess mantling the cliff in the assessment area is similar to other areas of the Port Hills. It is a relatively cohesive silt dominated soil with only minor clay mineral content. Its strength is largely controlled by the soil moisture content and this has been well studied, e.g., Bell et al. (1986), Bell and Trangmar (1987), McDowell (1989), Goldwater, (1990), Yetton (1992) and Carey et al. (2014).

The loess in the main zone of cracking at the slope crest appears to be unsaturated and relatively firm where exposed. It forms recessive slopes above the underlying volcanic breccia of approximately 40°. The loess is highly hygroscopic and when exposed to water (rain) it quickly disintegrates into muddy silt.

The measured thickness of loess inferred from drillhole records and field mapping varies up to 14 m (e.g., CPT-DFR-02). The loess is thicker to the south (Egnot Heights) and it pinches out against bedrock towards the west.

3.3.1.4 Colluvium

A layer of loess (mixed) colluvium was intersected in drillholes above the bedrock. It consists of sandy-clayey loess silt with minor angular gravels. Lava clasts may also be present. Seepage along the cliff face suggests that this interface carries groundwater runoff, particularly for example behind 26 Taupata Street where a series of ad-hoc channels have been constructed (by residents). The presence of groundwater seepage/runoff is also illustrated by attempts to construct a precarious earth dam behind 30B Taupata Street. The thickness of the colluvium inferred from drillhole records varies up to 1.4 m (e.g., DH-DFR-02).

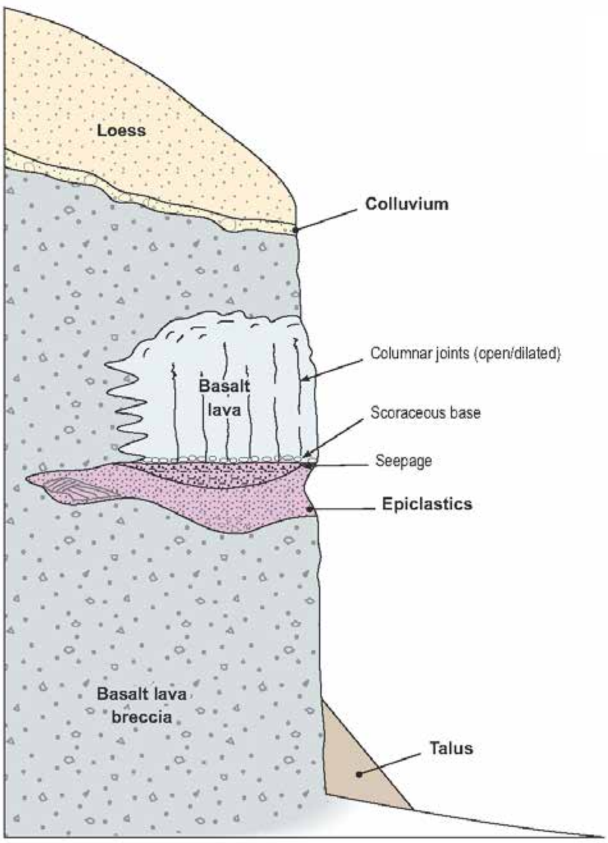
3.3.1.5 Volcanic rock

The cliff face comprises shallow (flat to locally 25°), easterly-dipping, highly variable (both laterally and vertically) basaltic lava flows with blocky irregular basal columnar-joints. Joint spacings are highly variable from less than a meter to many meters. A variable auto-brecciated scoria/lava (basalt lava breccia) forms the lower most exposed unit at the base of the cliff line. The schematic cross-section (Figure 14) shows where the scoria and lava flows units are separated by an epiclastic layer.

A detailed engineering geological description of the main geological units forming the cliffs is provided by Massey et al. (2014).

3.3.1.6 Epiclastics

This unit underlies conformably the columnar jointed basalt flow. It forms reddish-brown poorly sorted polymictic sands and angular gravels with weakly developed cross bedding structures. In some locations this unit appears to comprise fine-grained silts and clays. These materials are highly variable both laterally and vertically. Good exposures occur along the cliff face behind 30B Taupata Street where the paleo-depressions are infilled by columnar jointed lava flows (Figure 15 and Figure 16).



Unit	Description
Fill	Mixture of topsoil, overburden, loess and other materials in backfilled areas for housing development
Loess	Clayey silt with variable sand content. Stiff to soft and none to low plasticity.
Colluvium	Sandy-clayey loess silt with minor angular gravel. Lava clasts may be present. Firm to soft. Low plasticity.
Basalt lava	Un-weathered to slightly weathered vesicular basalt. Strong. Irregularly columnar jointed basal layer grading into scoraceous agglutinate lava towards the top of flow.
Basalt lava breccia	Auto brecciated lava with variably sized domains of coherent lava. Slightly weathered red-brown volcanic breccia. Angular clasts in scoriaceous matrix.
Epiclastics	Redish-brown poorly sorted polymictic sands and gravels with weakly developed cross bedding structures. Paleo-depressions infilled by lava flows.
Talus	Boulder and Sand colluvium derived predominantly from gullies

Figure 14 Schematic lithological summary of the materials at the Defender Lane mass movement.

Figure 15 Key lithologic units at the Defender Lane mass movement.

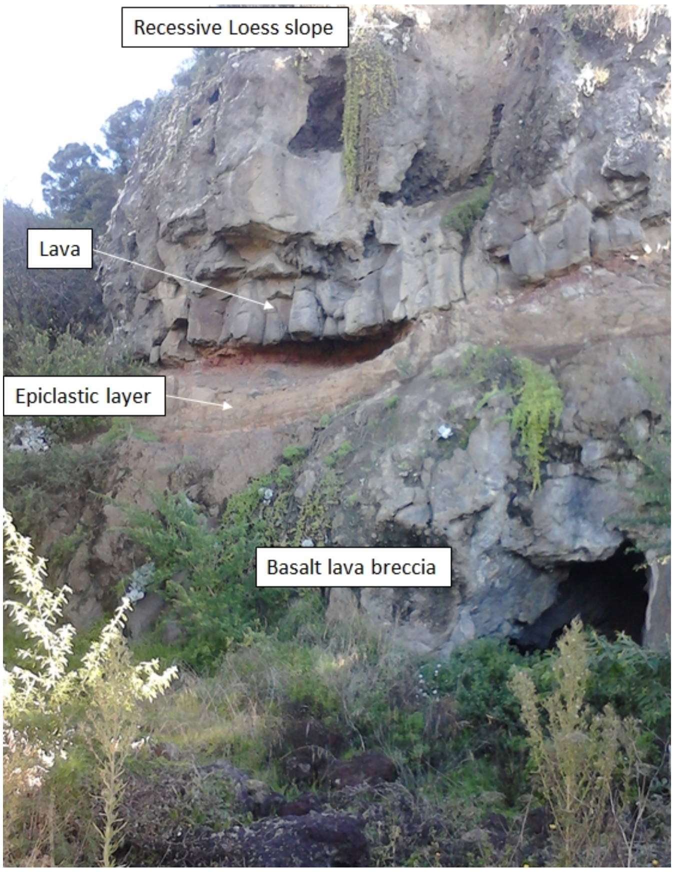


Figure 16 Inset from Figure 15 showing typical cliff section.

3.3.1.7 Loess-rock surface boundary

The surface boundary between the base of the loess and the underlying volcanic rock (rockhead) was interpolated using: drillhole intersections, cone penetrometer test depths of refusal and field mapped rock outcrop exposures as control points. The resulting rockhead surface (Figure 12) shows an overall dip of about 16° towards the east, which is approximately coincident with the slope aspect. Rockhead dips out of the slope at an angle of about 15° in the northern spur and about 5° in the southern spur. It should be noted that there is an apparent parallel trend between the overall strike of cracks and the strike of rockhead.

The interpolated rockhead surface on Figure 12 was used to define the loess and colluvium elevation shown in cross-sections (Figure 11).

3.3.2 Geotechnical test results

Strength parameters have been assigned to the materials forming the slope based on the results from in-house (GNS Science) laboratory tests (drillcore samples, fallen boulders and loess block samples collected in the field) and the published results of testing of similar materials in the Port Hills by others.

3.3.2.1 Loess

Material parameters adopted for the loess in the assessment area are shown in Table 10. These are based on: 1) descriptions of the drillcore materials; 2) Port Hills soil strength test results reported by Carey et al. (2014) and other published data; and 3) numerical slope stability back-analysis.

In situ water contents

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

The *in situ* water content of the loess block samples varies mostly between 6 and 10%, with two samples in the 3–5% range (Carey et al., 2014). The samples used for testing were all taken from free-draining slopes exposed to the weather, and were sampled between January and February 2013, and January and February 2014, near the end of summer. The *in situ* water contents are therefore thought to represent the lower end of the range (Figure 17). The samples were taken from an east-facing slope formed in loess. Even if the samples were collected in winter, the water contents of the loess at this accessible site would still not be representative of the water content of the loess deeper in the slope, as the outside face of the slope is free draining.

The water contents of the loess in drillhole samples were all substantially higher than those for the block samples. The difference may reflect the sampling method, where drilling includes using water as a flush, and block sampling does not (Table 8).

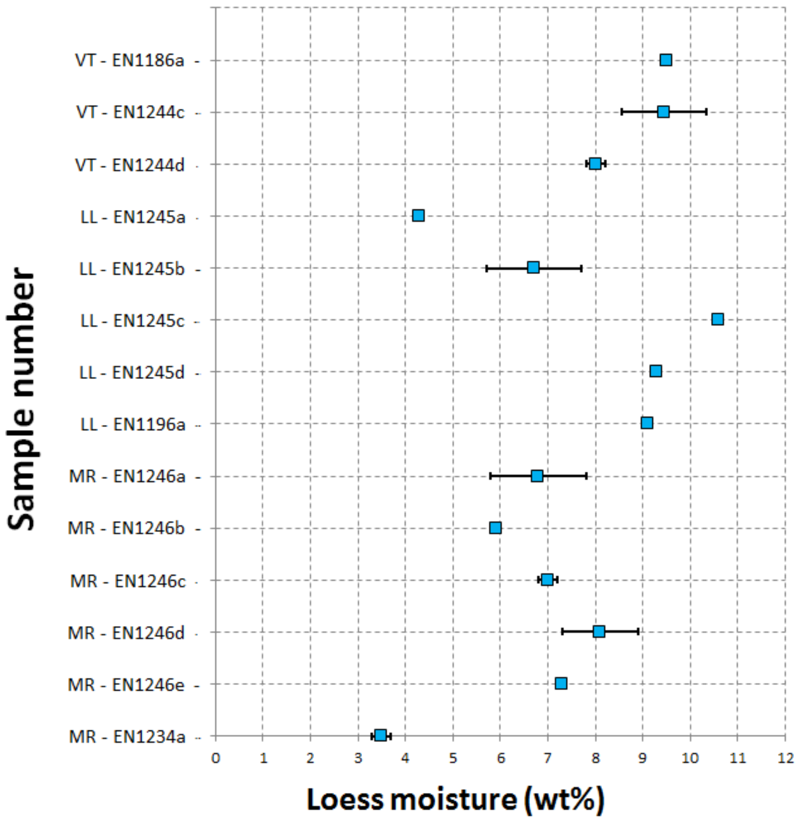


Figure 17 In-ground moisture (water, wt%) content of collected loess samples.

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 9 and Table 10 and plotted in Figure 18. 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 9 are considered to be more representative of the bulk residual strength parameters for the loess slope rather than peak strength parameters.

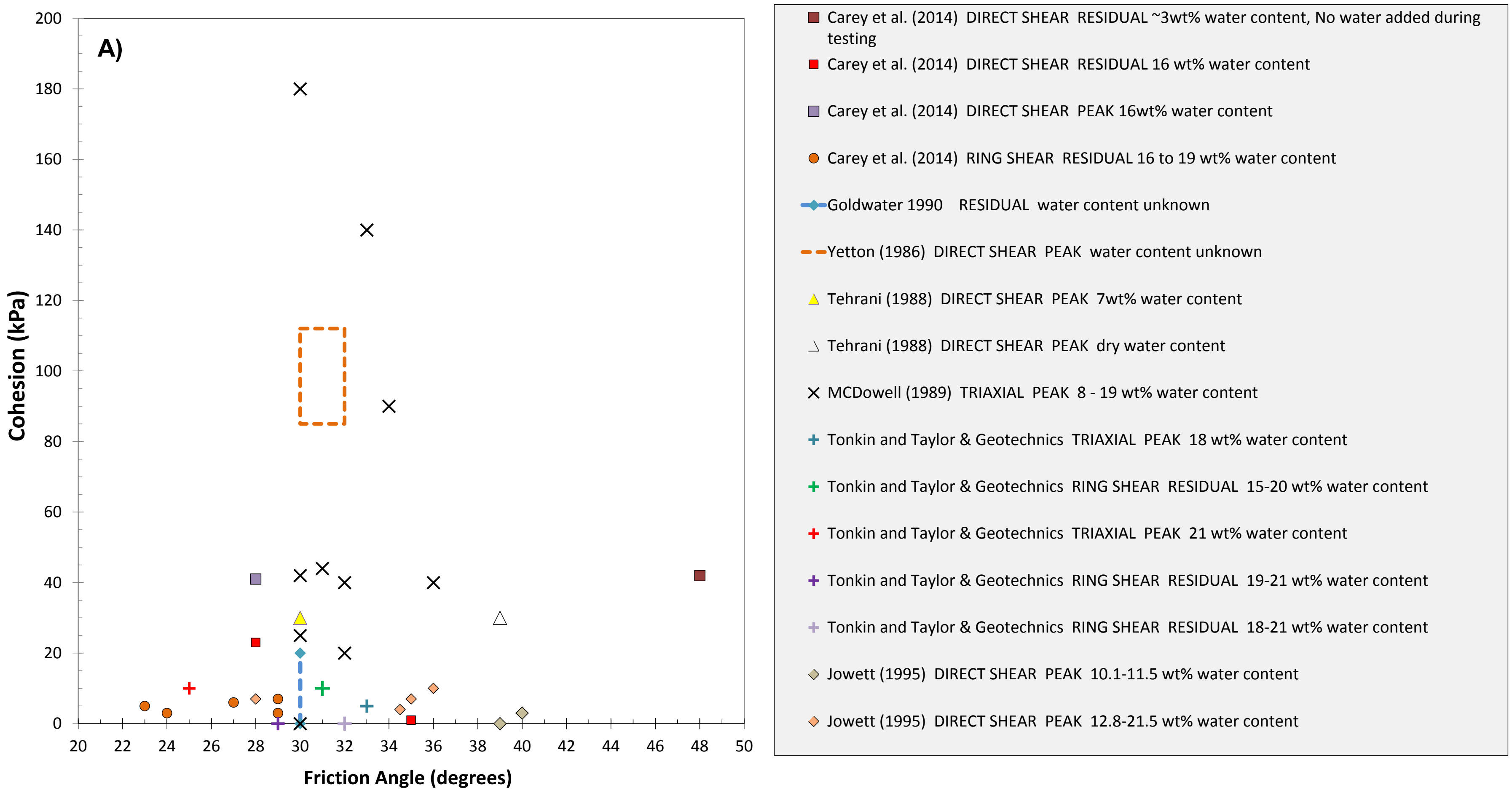
Effect of moisture content on loess shear strength

Comparison can be made with shear strength results from other published Port Hills investigations (Table 10) by plotting them alongside the results of the GNS Science testing (Figure 18).

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 to >15 wt% leading to a reduction in the cohesion and increased susceptibility to failure. The data plotted in Figure 18 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)
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
Friction Angle (degrees)
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

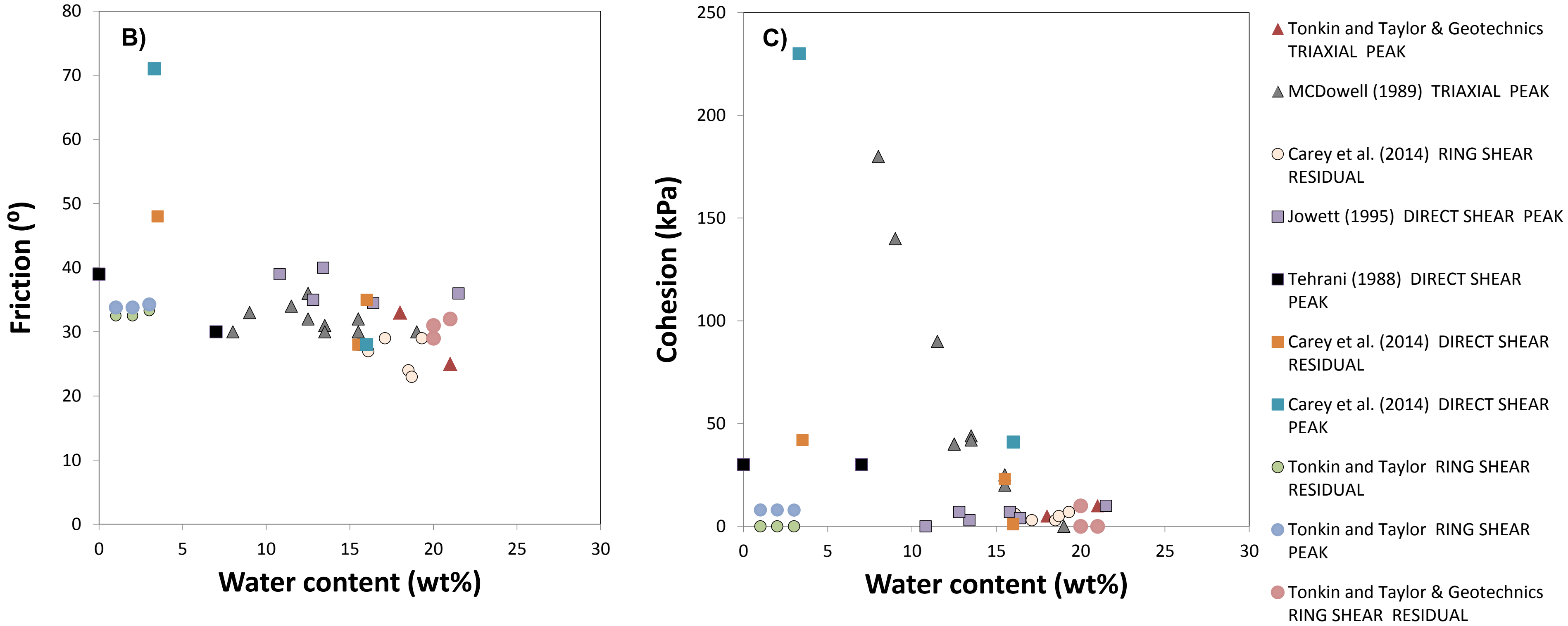


Figure 18 Loess residual shear strength results (from Table 9 and Table 10). A) Cohesion and friction laboratory results plotted for loess. B) Loess residual cohesion plotted against water content. C) Loess residual friction plotted against water content.

Table 9 Shear strength test results (from Carey et al., 2014).

Site	Sample	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	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	Ring Shear-C	Block Sample		16.1				6	27	EN1195b
		Ring Shear-G	Block Sample		17.9				0	37	EN1195c
Richmond Hill	EN1196	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	Ring Shear-G	Drillcore	17.1	17.9				20	35	EN1230b
Maffeys Road[2]	EN1243	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 10 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 (2012b) for EQC
Clifton Terrace (residual)	31–32	0–15	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 (2012a) 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)

Loess compressive strength tests

A summary of unconfined compressive strength tests carried out GNS Science on samples of loess from the Port Hills are shown in Table 11. Unconfined compressive strength tests were carried out to constrain the range in loess Young's modulus value for finite element numerical models.

Figure 19 shows the range in Young's modulus and compressive strength with sample water content. With exception of sample EN1254a (which is clay dominated), 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 19–Figure 21, for the different sites tested.

The results show that the Young's modulus and unconfined compressive strength of the loess is sensitive to water content, but not as sensitive as the cohesion.

Table 11 Unconfined compressive strength test results carried out by GNS Science on block samples.

Location	Lab number	Water content (%)	Dry density (t/m^3)	Saturation ratio (%)	Compressive strength (MPa)	Axial tangent modulus (Young's modulus) (MPa)
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
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 12 Other loess Young's modulus tests results.

Location	Water content (%)	Dry density (t/m^3)	Compressive strength (MPa)	Axial tangent modulus (Young's modulus) (MPa)	Data source
Ahuriri loess	2.4	1.8	1.73	44.4	Jowett (1995)
Timaru loess	No data	No data	1.71	46.3	Jowett (1995)

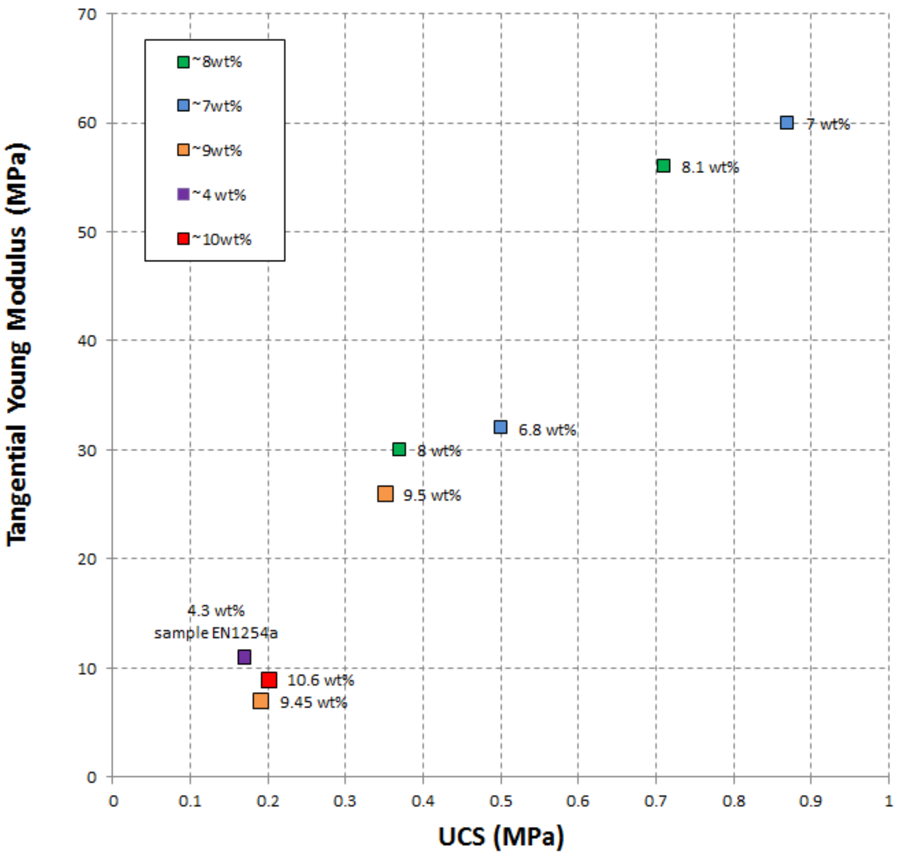


Figure 19 Loess Young's modulus versus the unconfined compressive strength and moisture content (wt%).

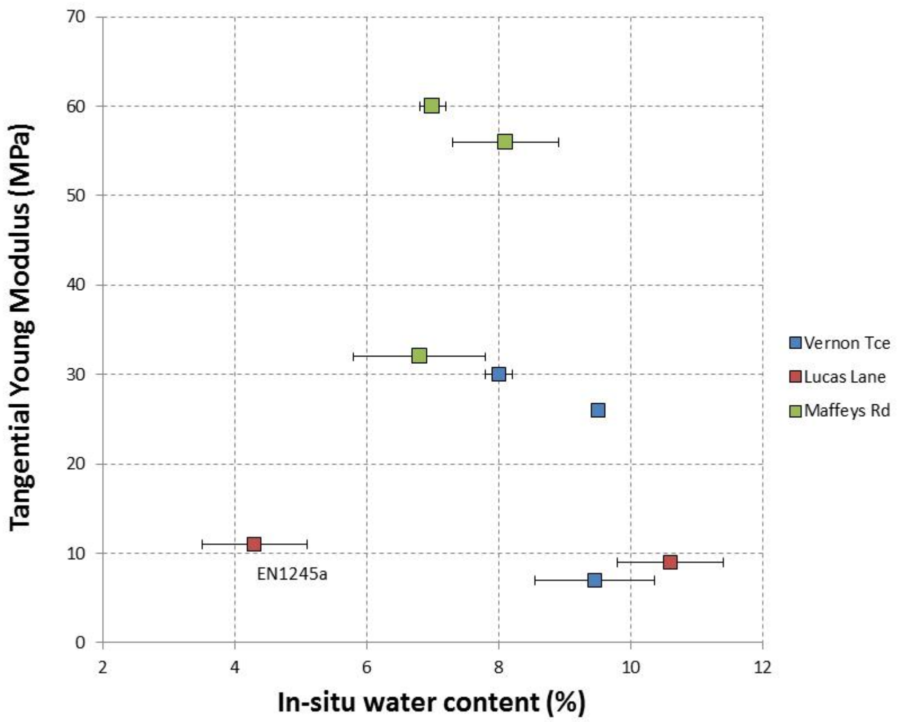


Figure 20 Loess Young's modulus versus water content (moisture content).

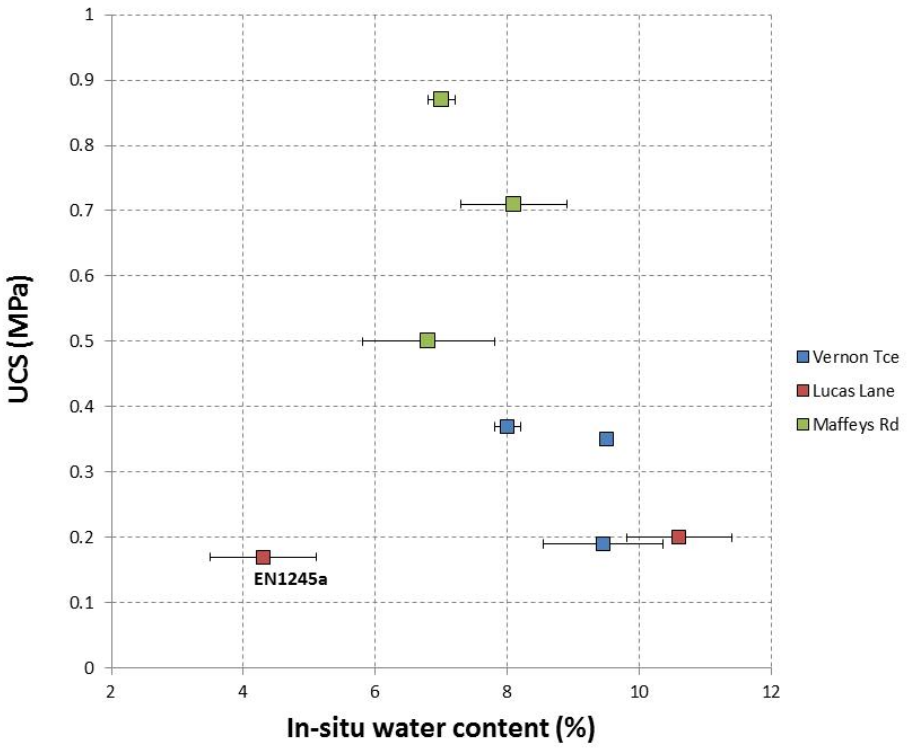


Figure 21 Loess compressive strength versus water content.

3.3.3 Adopted parameters for numerical models

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

3.3.3.1 Loess shear strength from back analysis

The bulk value of cohesion and friction for the loess slope was assessed by numerical back-analysis of slope stability for two geological cross-sections (cross-sections 3 and 5). Figure 22 shows an example of the back-analysed friction and cohesion values that yield different factors of safety (each data point represents a modelled slide surface). Factors of safety greater than 1 are shown as diamonds and factors of safety less than 1 are shown as crosses. If a slope has a static factor of safety of 1, then the slope is assessed as being unstable. The shear strength values obtained from the laboratory testing of loess samples are shown for comparison. The results show that:

- As previously established in Section 3.3.2.2, the shear strength of the loess, particularly the cohesion, is very sensitive to changes in water content. Figure 22 shows that the factor of the safety of the slope is fundamentally linked to the shear strength of the loess. 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 1.1 to less than 1.
- The range of the laboratory shear strengths typically plot below the line representing a factor of safety of 1, and therefore represent the lower end of the strength range considered to be reasonable.
- The laboratory testing was carried out at higher water contents than those measured from block samples, which were sampled at the driest period of the year. However, It is feasible that water contents could increase in response to increased rainfall and infiltration

- 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, the results form the back-analysis suggests that failure of the loess is likely.

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 2013). These data are shown as a black triangle on Figure 22.

The sensitivity of the loess slope at the Defender Lane mass movement to changing water contents was addressed in this study by adopting a range of strength parameters for the stability assessments (friction (ϕ): 25–35° and cohesion (c): 5–30 kPa).

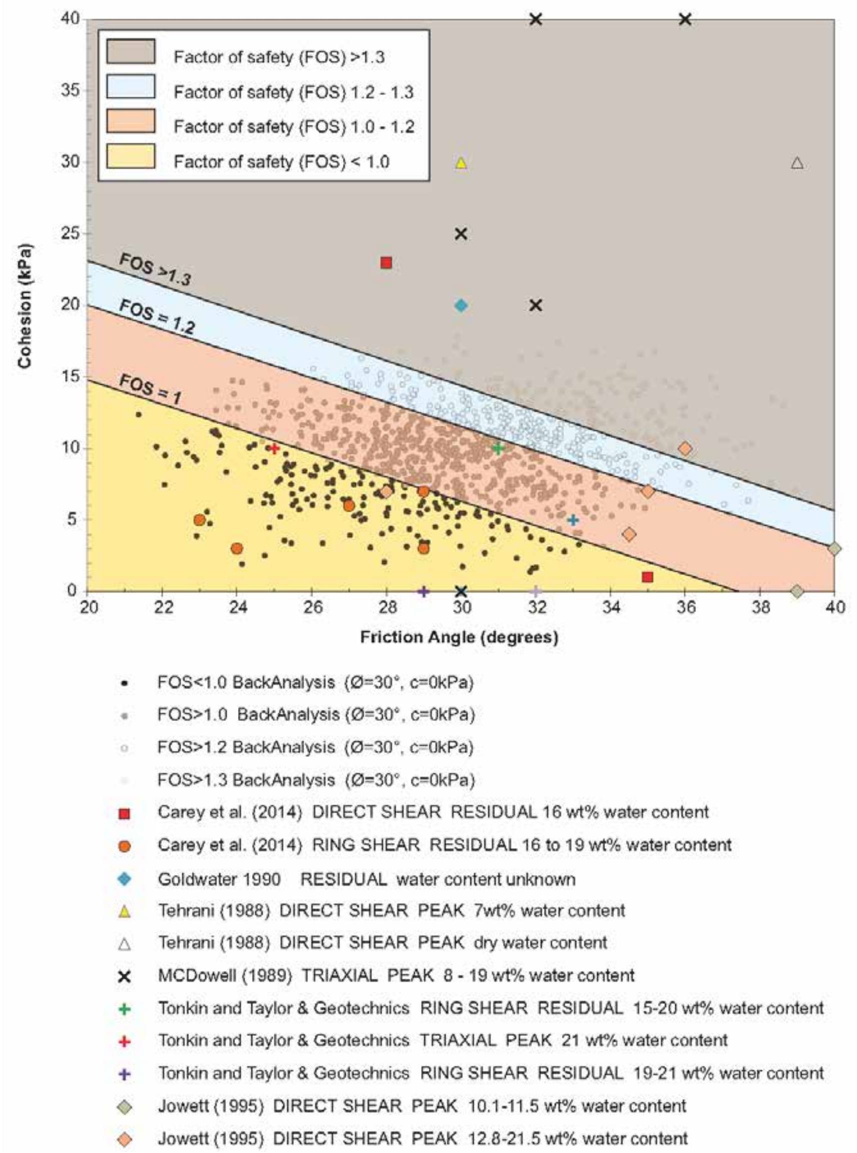


Figure 22 Sensitivity assessment of the loess slope bulk shear strength parameters of friction and cohesion from limit equilibrium modelling. Datapoints shown as diamonds represent modelled slide surfaces with factors of safety >1. Data points shown as crosses represent modelled slide surfaces with a factors of safety <1. The larger circles represent the shear strength of the samples tested in the laboratory.

3.3.3.2 Other loess parameters

A Young's modulus of 30 MPa was adopted based on laboratory testing results from the loess (Table 11 and Table 12). 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 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 13.

Table 13 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)	

3.3.3.3 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, 2013) based on nearby survey available from Redcliff drillholes DH-MB-02, and results from the dynamic probing carried out by Tonkin and Taylor Ltd. for the Earthquake Commission at Clifton Terrace (Tonkin and Taylor, 2012b). The results from the dynamic probing are summarised in Figure 23. The mean shear wave velocity is 306 m/s (±93 m/s at one standard deviation) and the mode is 222 m/s.

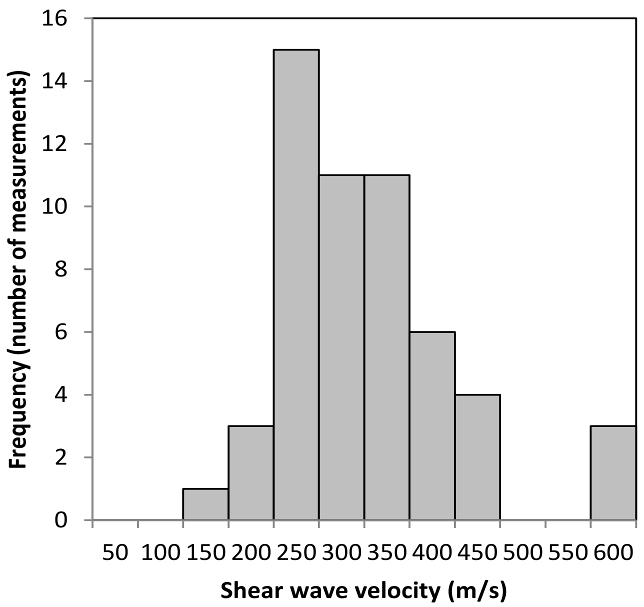


Figure 23 Loess shear wave velocity results from dynamic probing reported by Tonkin and Taylor (2012b) for loess at Clifton Terrace.

The corresponding shear wave velocity for the loess intersection (0.6–2.7m) measured in the drillhole MB02 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 Argentinean loess as a function of normal stress and moisture content (Table 14) 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:

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

Where ρ is the density of the loess 1,700 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 14 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 (2013)
Port Hills loess from Clifton Terrace dynamic probing	126–582	Tonkin and Taylor (2012b)
Loess Moisture content ~16 wt%	280–300	Rinaldi et al. (2001)
Loess Moisture content 6.4 wt%	300–320	Rinaldi et al. (2001)

3.3.3.4 Colluvium

Material parameters adopted for the colluvium layer underlying the loess material in the assessment area are based on: 1) descriptions of the drillcore materials; and 2) Port Hills soil strength test results reported by Carey et al. (2014), and other published results.

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

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

Shear wave velocity surveys carried out by Tonkin and Taylor (2012b) show there is little difference between the shear-wave velocity of the loess and the colluvium at Clifton Terrace. Therefore, the shear wave velocities for loess have been adopted for the colluvium material.

3.3.3.5 Volcanic bedrock

Rock strength and shear modulus parameters for the underlying volcanic basalt lava and breccia unit are shown in Table 15 and are based on laboratory testing of samples taken from the Moa Bone drillcores and from down-hole shear wave velocity surveys of the same drillholes. However, as this assessment addresses earth/debris flow hazard and risk derived from failure of the loess slope above the volcanic rock units, the rock strength parameters are not critical for slope stability modelling.

Table 15 Bulk (mass) geotechnical material parameters derived from testing and field surveys and used for the modelling.

Unit	Cohesion c (kPa)	Friction ϕ (°)	Tensile strength (kPa)	Unit weight (KN/m^3)	Young's modulus (MPa)	Poisson's ratio	Shear modulus G_S (MPa)
Loess and colluvium	5–35	25–35	10	17	30	0.3	160–280
Underlying mixed volcanic lava and breccia	200	37	60	20	1,100	0.1	820

3.3.4 Rainfall and groundwater response

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

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

McSaveney et al. (2014) examined a list of Earthquake Commission claims for landslide damage for the period 1997–2010, and a Geotechnical Consulting Ltd. landslide investigations list, which covers the period 1992–2009. Any duplicate records for the period 1997–2009 contained in the data sets were removed. These records, though incomplete with respect to all of the landslides that occurred over those intervals, may be approximately complete with respect to the episodes of rain associated with landslide occurrences that damaged homes and urban properties (Figure 24).

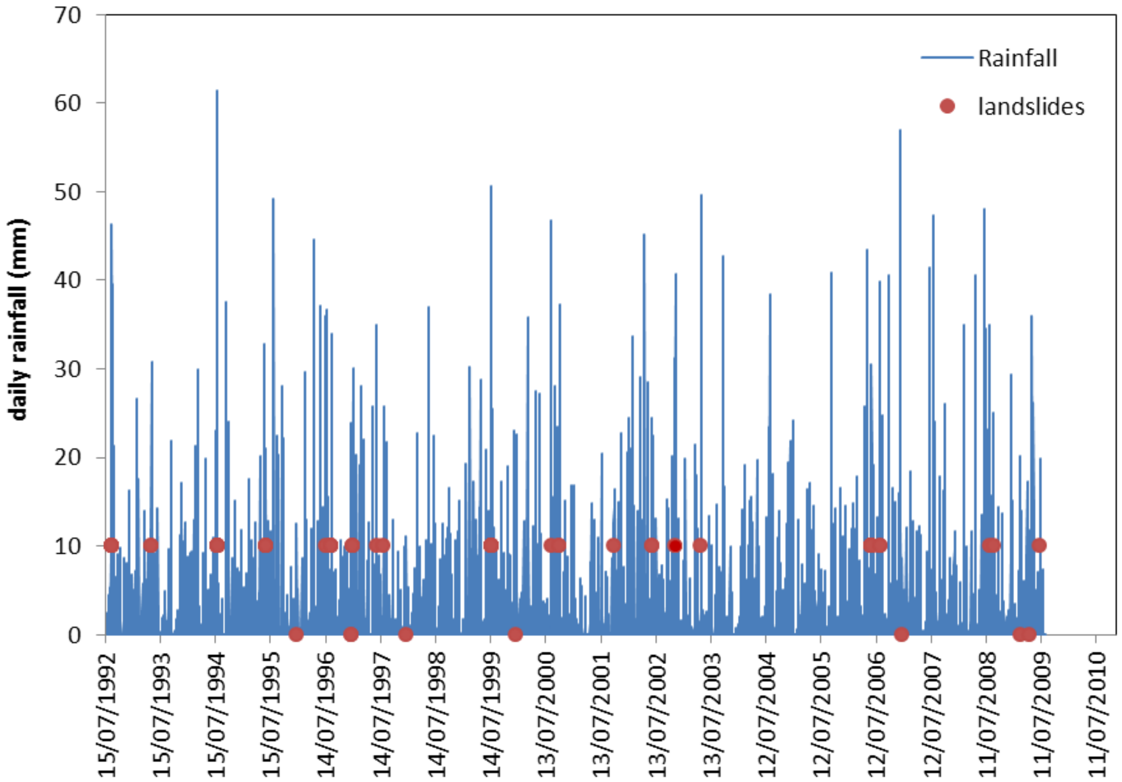


Figure 24 Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills. Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills investigated by Geotechnical Consulting Ltd, or listed by the Earthquake Commission as causing damage to homes. Landslides without rain are plotted at 0 mm, all others are plotted at 10 mm of rain (the minimum rainfall for triggered landslides).

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

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

Following the 2010/11 Canterbury earthquakes there have been two notable rainfall events (Table 16):

- 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 Defender lane). 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. During this storm, two small (less than 50 m^3) earth/debris flows were triggered. No further landslides or movement of the slopes within the assessment area have been reported to GNS Science.

The frequency of high-intensity rainfalls in Christchurch has been well studied (e.g., Griffiths et al., 2009, Figure 25; 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 16. 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 16 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)

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.

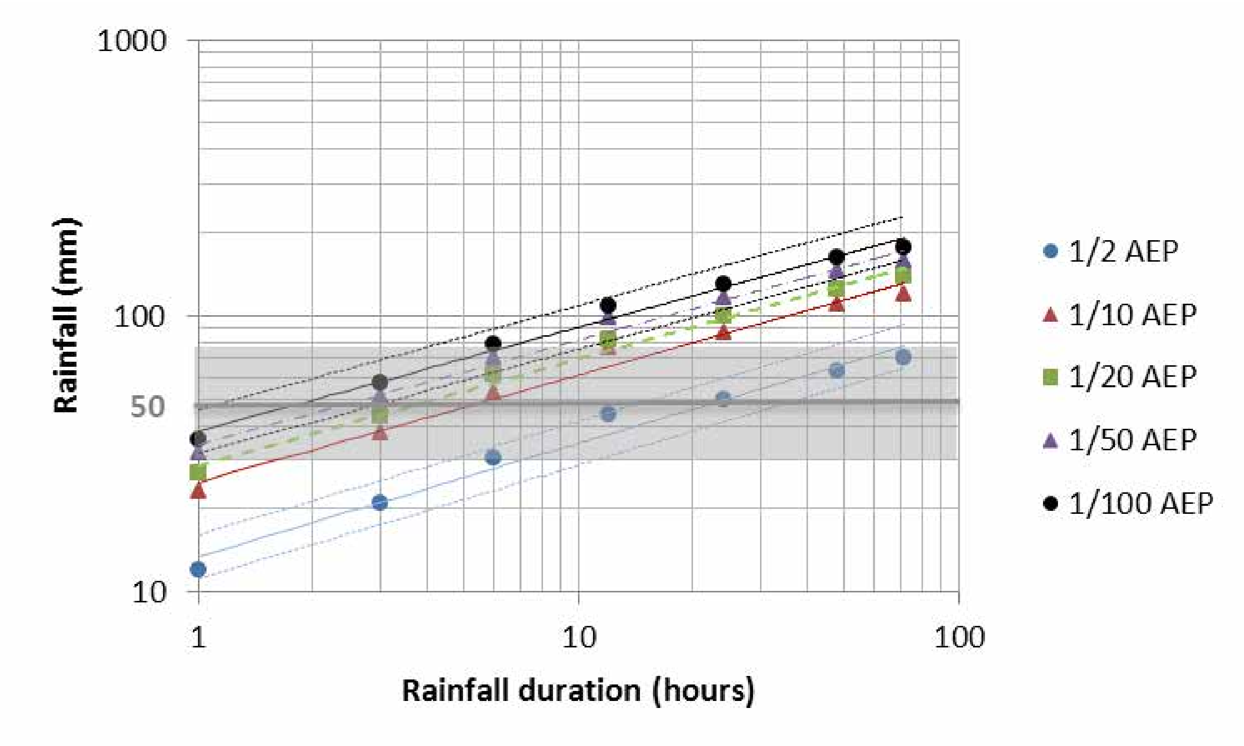


Figure 25 Rainfall depth-duration-return period relations estimated for Christchurch Gardens by Griffiths et al. (2009) using recorded rainfall data. Error limits of 20% are shown by dotted lines for the 1/2 and 1/100 AEP curves. Shaded area covers the range of 30–75 mm of rainfall over which the expected number of soil landslides in the Port Hills rises from very few to many. Rockfalls can occur without rain, but the probability of rockfalls occurring increases with increasing intensity of rainfall.

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.

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. However, at present it is not possible to link rainfall to groundwater levels at the site, due to the lack of high resolution monitoring data and the uncertainties surrounding the installations of the existing standpipe piezometers.

3.4 SLOPE FAILURE MECHANISMS

3.4.1 Landslide types affecting the site

The main landslide types at this site during the 2010/11 Canterbury earthquakes were:

- Disrupted rockfalls and small debris avalanches (adopting the scheme of Keefer, 1984) and cliff-top recession (collectively termed cliff collapse), derived predominantly from the columnar jointed basalt layer and other perched boulders in the slope, as well as thin slabs of loess falling of the slope face (Figure 26). The life risk associated with these hazards was quantified by Massey et al. (2012).
- Coherent soil block slide (Keefer, 1984), where the where the basal slide surface appears to be within the loess/colluvium overlying bedrock. In general, the vector displacements inferred from survey marks and crack apertures are consistent with displacement of the mass sub-parallel to the dip of rock head.

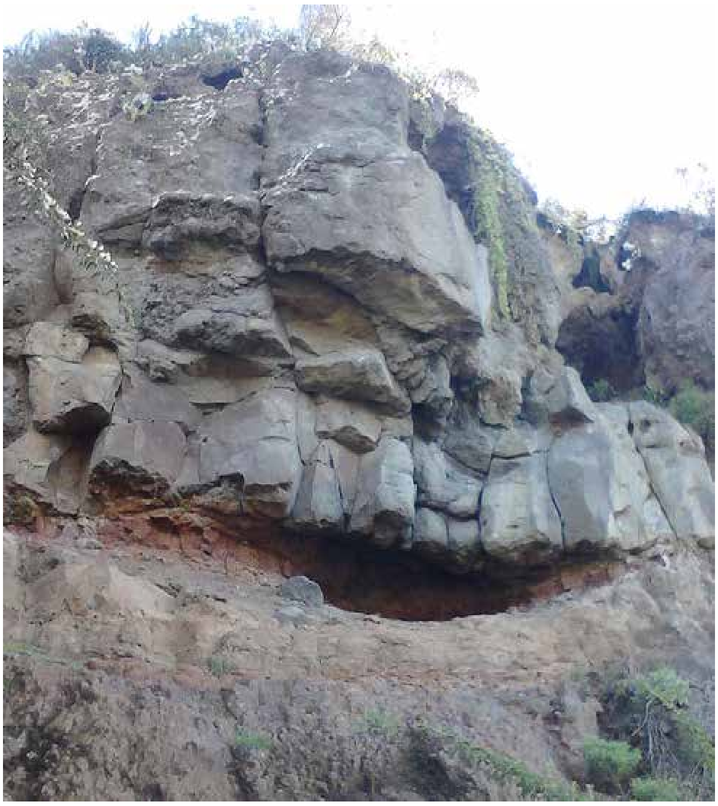


Figure 26 Columnar jointed blocks of basalt perched on the slope behind 28 Taupata Street.

Based on the aerial photograph interpretation, field mapping, ground investigation and monitoring (carried out by Tonkin and Taylor Ltd for the Earthquake Commission; Tonkin and Taylor, 2012a), laboratory testing and site observations of the impacts of the 2010/11 Canterbury earthquakes, engineering geological models of the mass movement have been developed (Figure 27). These models have formed the basis for numerical modelling of the stability of the slopes within the identified mass movement boundary.

Based on these results, it is possible that landslides, occurring from the steep loess slopes, within the assessment area, could develop into more mobile earth/debris flows. This is because:

1. The slope is currently cracked allowing surface water to infiltrate the slope more readily.
2. The shear strength of the loess and colluvium will reduce with increasing water contents and the slope will be subjected to increased pore-water pressures within the slope mass and in open tension cracks. Broken services within the mass movement could also be contributing water to the area.
3. There are several large relict landslide scars and associated debris fans present in the assessment area, suggesting large slope failures have occurred in the area. It is not known whether the debris forming the fans originates from a few infrequent but large landslides, or from the accumulation of debris from many smaller landslides.
4. There are several small and discrete areas of fill along Defender Lane (Figure 27), that appear to have cracked in response to the 2010/11 earthquakes. It is possible that these areas could form potential earth/debris flow source areas. However, given the current lack of subsurface information, it is not possible to currently assess whether such failures are possible in a future event. It is recommended that these areas are investigated further as part of an assessment of potential engineering mitigation measures.

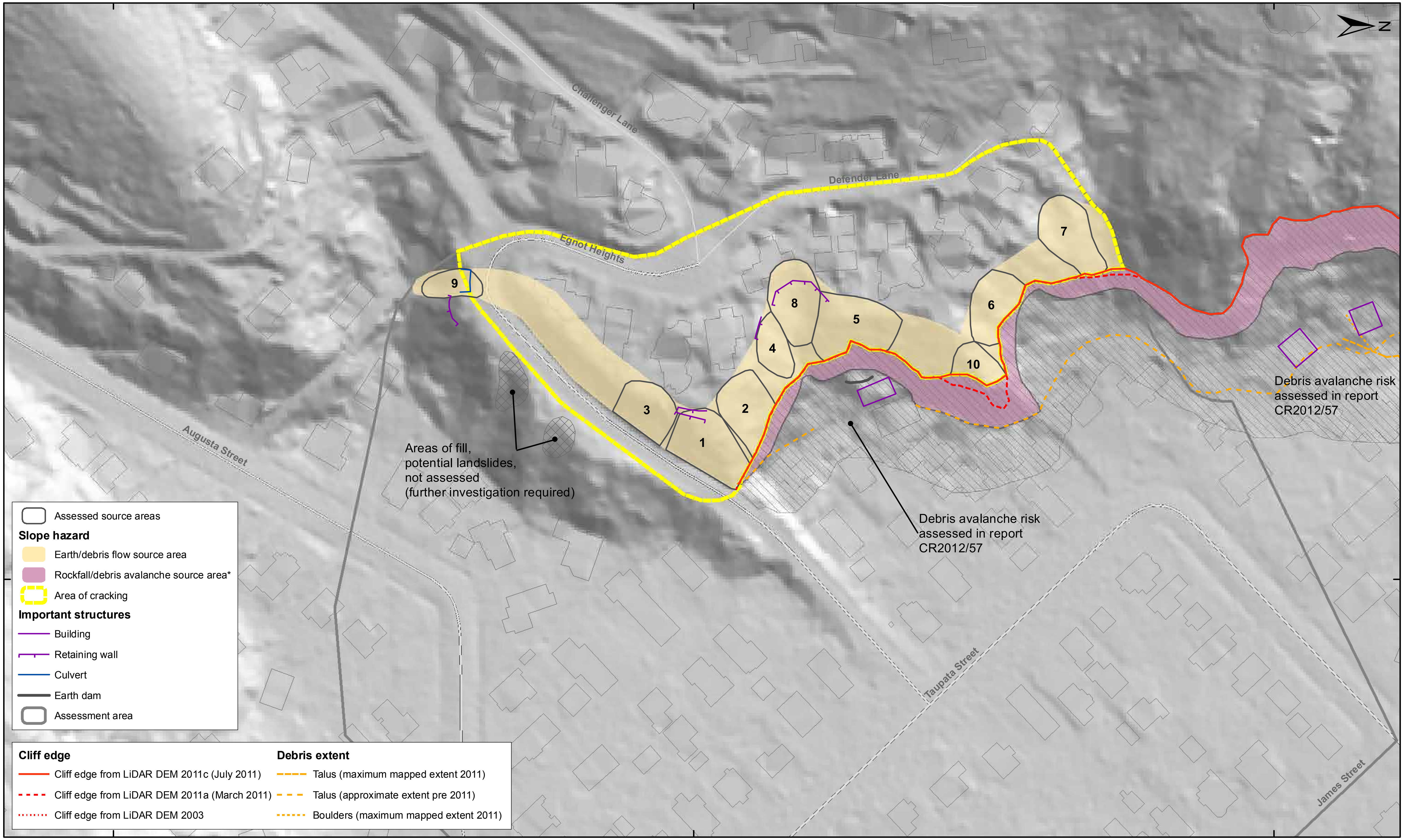
3.4.2 Failure mechanisms adopted for modelling

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

- Mechanism 1: Shallow failure of loess, at the lower part of the slope, forming several discrete landslide source areas (1–10); and
- Mechanism 2: Deeper block-slide failure of the loess through the underlying colluvium layer, forming a coherent slide. A colluvium layer was intercepted in all drillholes within the main area of cracking. For the purpose of the model, the colluvium layer is considered to extend beneath the loess, above the volcanic lava sequences, over most of the site.

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

The main additional hazard affecting the site (additional to cliff collapse hazards) is from earth/debris flows. The results from the site investigations have been used to define 10 main source areas within the assessment area. These source areas are thought to represent the location, shape and extent of the likely earth/debris flows that could occur in the assessment area.



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/57

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: FDP, CM

GNS SCIENCE TE PŪ AO

ENGINEERING GEOLOGY MODEL

Defender Lane Christchurch

FIGURE 27

FINAL

REPORT: CR2014/67

DATE: June 2014

4.0 HAZARD ASSESSMENT

4.1 SLOPE STABILITY – STATIC CONDITIONS

Five slope cross-sections were assessed (cross-sections 1–3, 5 and 6 in Figure 11). Geotechnical material strength parameters used in the modelling are from Table 15, and models using variable loess shear strength parameters were run to assess the sensitivity of the slope – along a given cross-section – to failure.

4.1.1 Failure mechanism 1

Results from assessment of representative cross-sections are shown in Table 17, and examples of limit equilibrium model and finite element model outputs are shown in Figure 28–Figure 30. These results are shown for loess shear strength parameters of friction (ϕ) of 30 and 35° and cohesion (c) of 10 and 30 kPa.

Results from the limit equilibrium and finite element modelling (Table 17) 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 from the finite element model assessment. 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 31 where a relatively small reduction in cohesion from 10 to 5 kPa results in a drop in the factor of safety to below one for most cross-sections assessed.

Table 17 Mechanism 1. Example results from the static limit equilibrium model slope stability assessment, adopting loess shear strength parameters of (friction (ϕ) of 30 and 35° and cohesion (c) of 10 and 30 kPa.

Cross-section	**Mechanism**	**Minimum FoS[1]** Loess friction 30°, cohesion 10 kPa	**Minimum FoS[1]** Loess friction 35°, cohesion 30 kPa	**Slide surface**	**Slice depth[2] (m)**	**Slice length[3] (m)**
Southern spur						
1	1	1.0	1.7	MIN	2	20
				MID	4	23
				MAX	8	27
2	1	1.0	1.7	MIN	2	12
				MID	3	21
				MAX	5	26
3	1	1.1	2.0	MIN	2	7
				MID	3	16
				MAX	5	17
Northern spur						
5	1	1.3	2.2	MIN	2	8
				MID	3.5	14
				MAX	5.5	19
6	1	1.2	2.3	MIN	1	6
				MID	2	13
				MAX	3	16

1 FoS is the factor of safety for the limit equilibrium method adopting the circular path search.

2 Estimated depth (perpendicular to slide surface) and length (crown to toe of failure) of failure based on the slide surface geometry using loess shear strength parameters of friction (ϕ) of 30° and cohesion (c) of 10 kPa.

3 Downslope length of semi-circular failure mass using loess shear strength parameters of friction (ϕ) of 30° and cohesion (c) of 10 kPa.

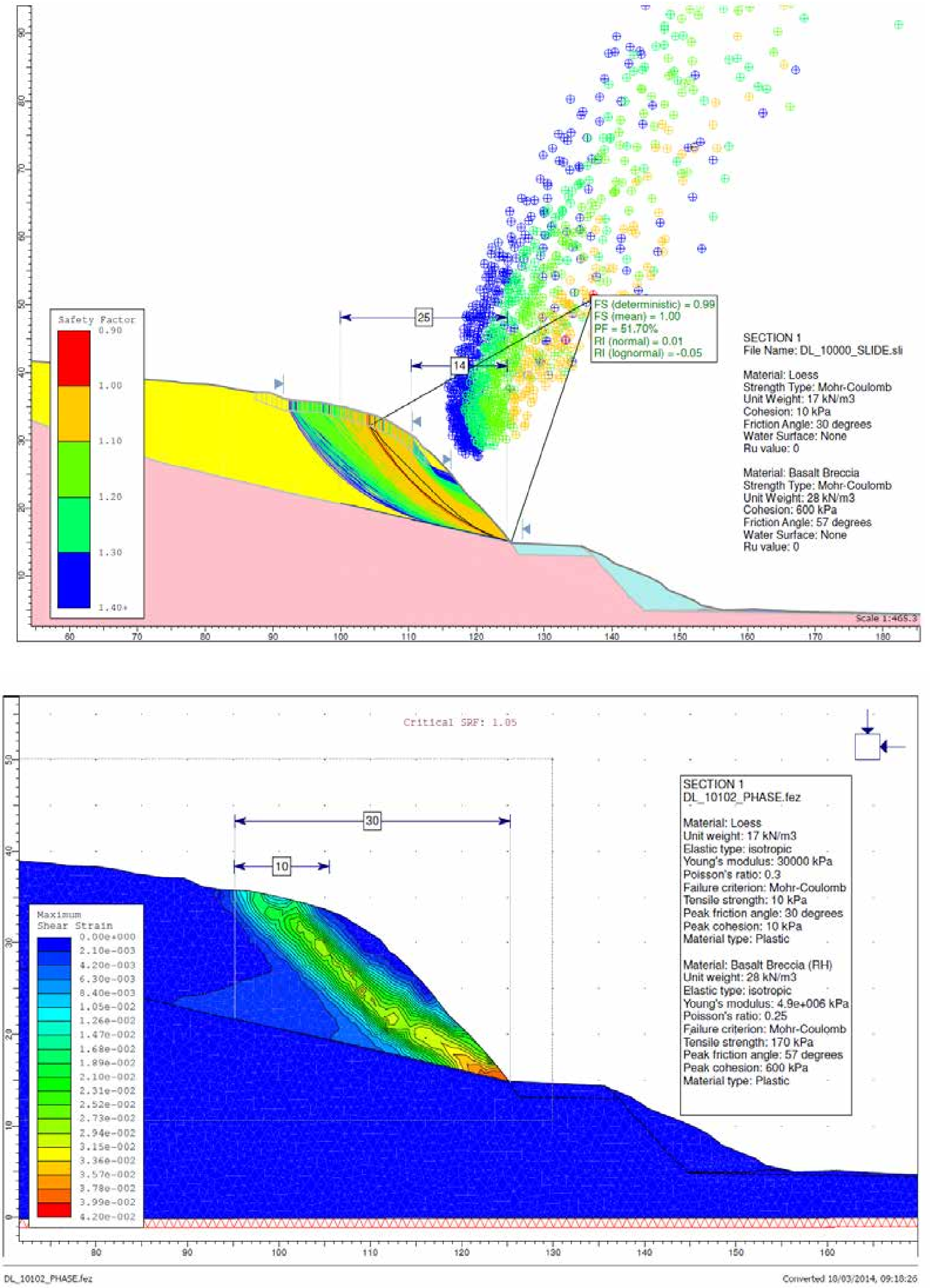


Figure 28 Mechanism 1. Limit equilibrium and finite element modelling assessment results for cross-section 1, adopting friction (ϕ) = 30°, and cohesion (c) = 10 kPa.

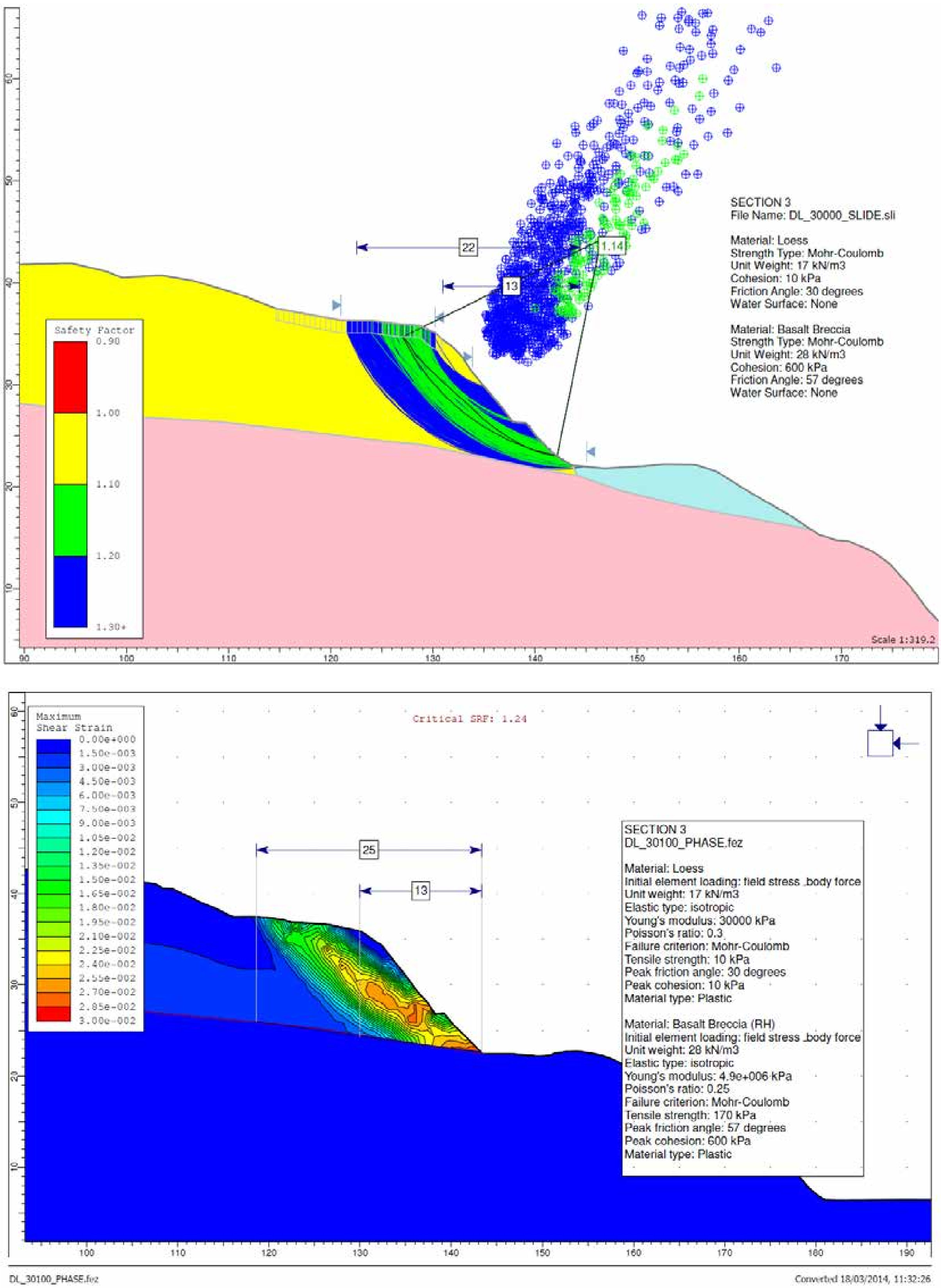


Figure 29 Mechanism 1. Limit equilibrium and finite element modelling assessment results for cross-section 3, adopting friction (ϕ) = 30°, and cohesion (c) = 10 kPa.

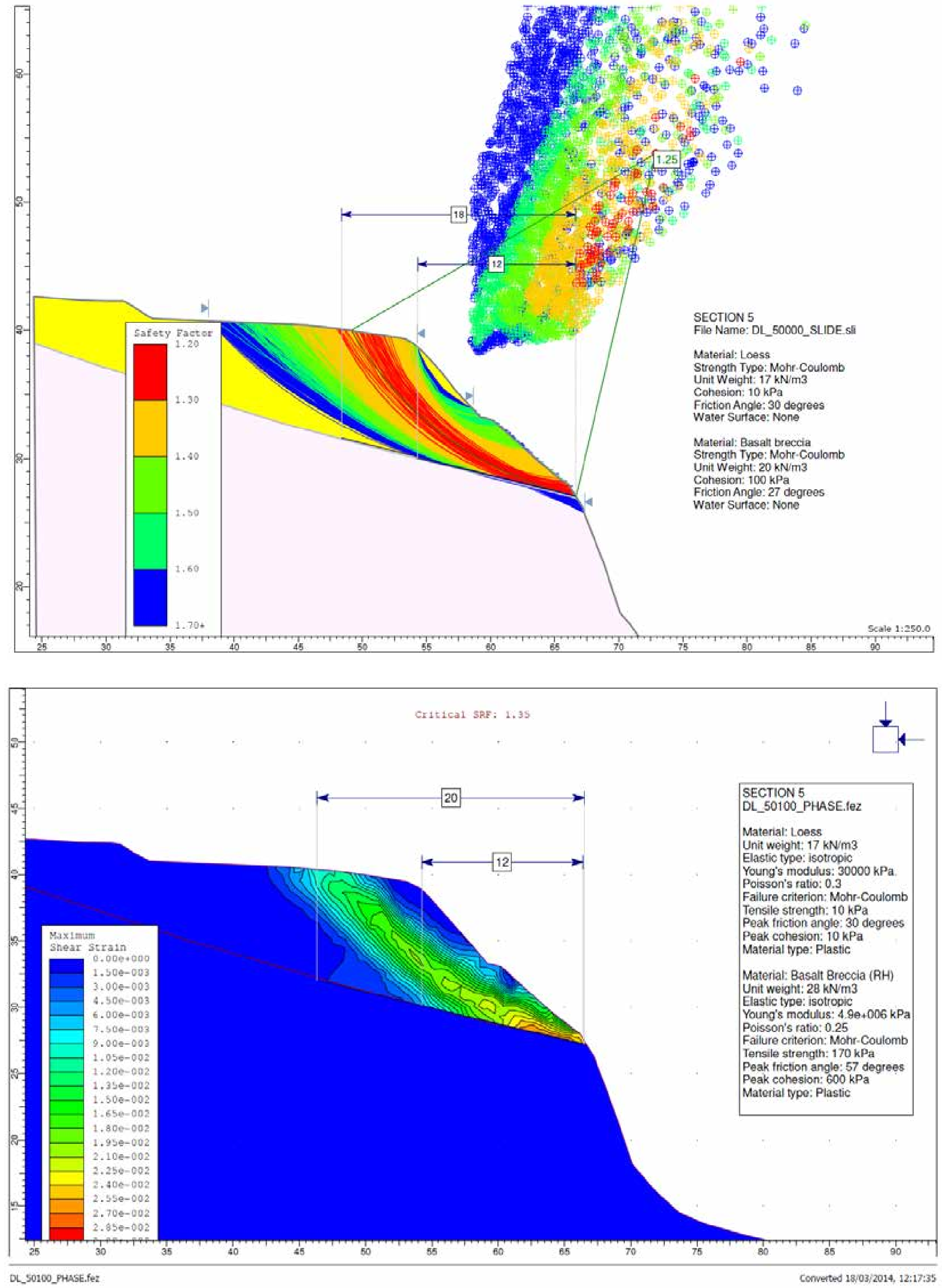


Figure 30 Mechanism 1. Limit equilibrium modelling and finite element modelling assessment results for cross-section 5, adopting friction (ϕ) = 30°, and cohesion (c) = 10 kPa.

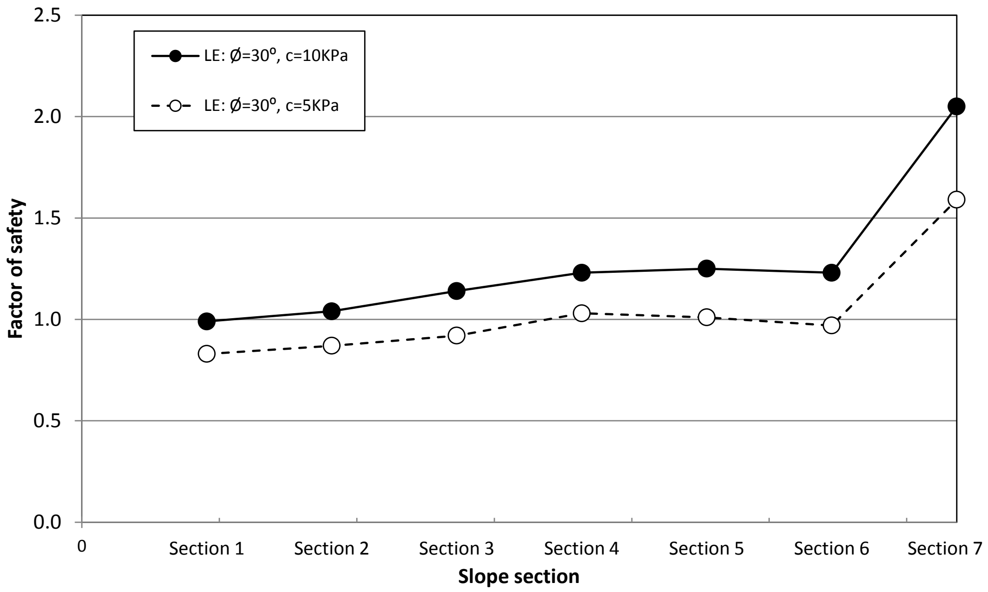


Figure 31 Effect on factor of safety output with reduction in cohesion (c) from 10 to 5 kPa (based on the limit equilibrium method for failure mechanism 1).

If a slope has a static factor of safety of one, then the slope is assessed as being marginally stable. Slopes relating to structures designed for civil engineering purposes are typically designed to achieve a long-term factor of safety of 1.5 under drained conditions, as set out in the New Zealand Building Code. Results from the stability assessment indicate that if the loess shear strength parameters that are typically used in the Port Hills (M. Yetton, Geotech Consulting Ltd., personal communication 2013) are adopted (friction (ϕ) = 30° and cohesion (c) = 10 kPa), then the factors of safety of the assessed cross-sections 1, 2 3, 5 and 6 are generally less than 1.3. The exception is cross-section 7, where the factor of safety is greater than 2. Results from the stability assessment, adopting loess shear strength parameters of friction (ϕ) of 35° and cohesion (c) of 30 kPa, indicate the factors of safety for the sections assessed range from 1.7 to 2.3.

If the lower shear strength parameters are representative of the *in situ* loess within the Defender Lane assessment area, then it is likely that failure of the assessed slide surfaces could occur under static (rain) conditions.

4.1.2 Failure mechanism 2

Failure mechanism 2 was assessed adopting a range of values for the colluvium for cross-section 5, which has the steepest loess/rock (rockhead) interface, which dips out of the slope at an angle of 15°. The results are shown in Table 18, and example stability models are shown in Figure 32.

The results show:

- The factor of safety for cross-section 5 is sensitive to the presence of the weaker colluvium layer.
- The strength of the colluvium is not known, and this failure mechanism is only credible if the colluvium is weaker than the overlying loess.

- If loess parameters of friction (ϕ) of 30° and cohesion (c) of 10 kPa are adopted – which are the loess parameters adopted in the assessment of mechanism 1 (Table 17) – the factor of safety for mechanism 2 is higher, than that for mechanism 1.
- The cracks, with vertical movement components, which formed during the 2010/11 Canterbury earthquakes, are more consistent with relatively shallow failures of the slope crest, similar to those simulated for mechanism 1, and not deeper-seated failures like those simulated for mechanism 2.
- Failure mechanism 2 is still credible, if groundwater levels (and therefore pore pressures) were to rise by about 1.5–3 m above rockhead, assuming the colluvium has similar strength parameters to the loess (friction ϕ of 30° and cohesion (c) of 10 kPa).

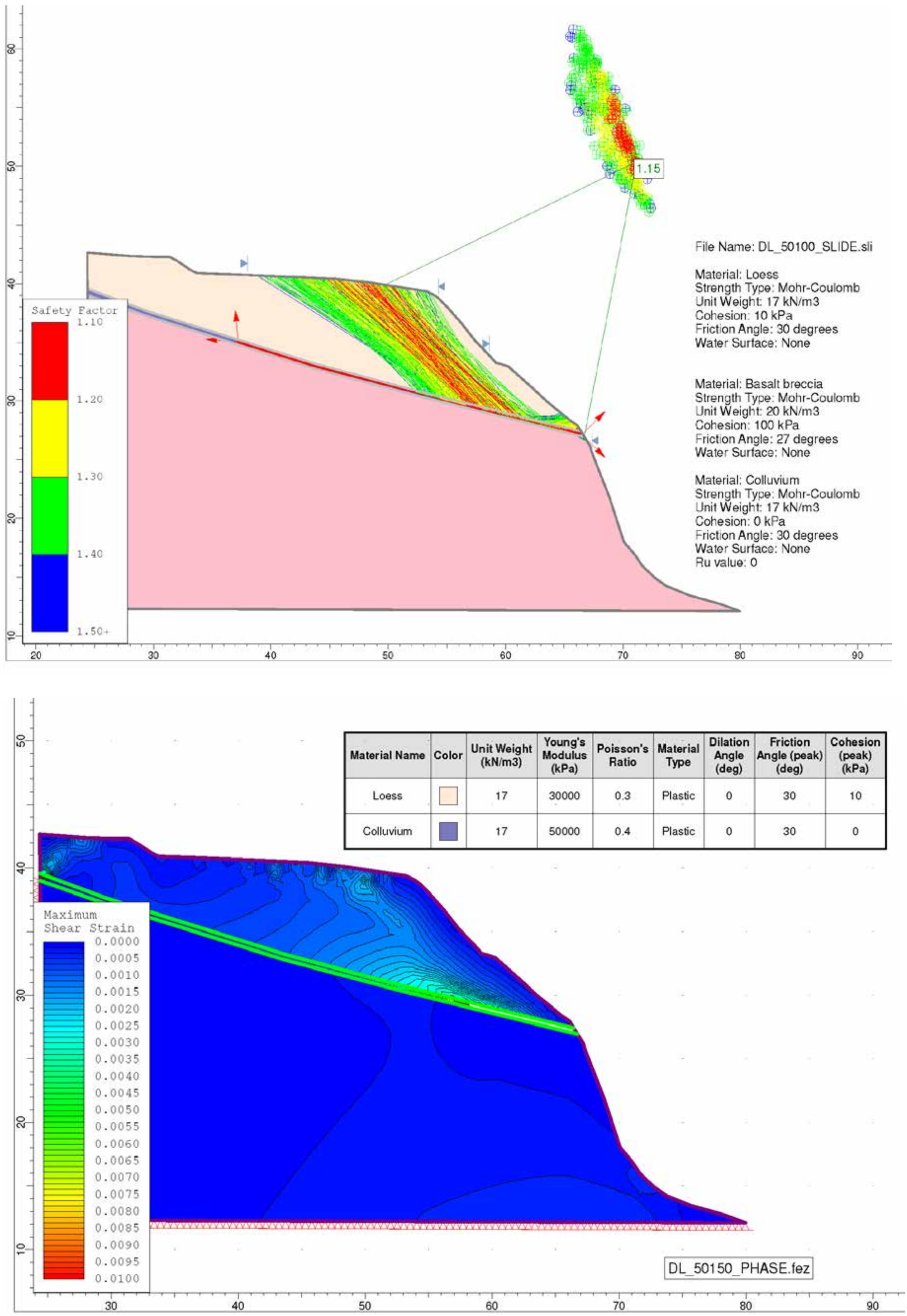
Table 18 Mechanism 2. Example results from slope stability assessment of cross-section 5.

Failure mechanism	LOESS	Colluvium layer	FoS[1] SLIDE	Search surface	SRF[2] PHASE2
	Cohesion (kPa) / Friction (°)	Cohesion (kPa) / Friction (°)			
Mechanism 2	10 / 30	10 / 30	1.3	Block	1.1
Mechanism 2	10 / 30	15 / 21	1.3	Block	1.1
Mechanism 2	10 / 30	0 / 30	1.2	Block	1.0

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

2 The finite element model was also used for comparison. Where the slope has been assessed using the finite element model, the stability of the slope is assessed in terms of the stress reduction factor (SRF).

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 et al., 1999).



Material Name	Color	Unit Weight (kN/m3)	Young's Modulus (kPa)	Poisson's Ratio	Material Type	Dilation Angle (deg)	Friction Angle (peak) (deg)	Cohesion (peak) (kPa)
Loess		17	30000	0.3	Plastic	0	30	10
Colluvium		17	50000	0.4	Plastic	0	30	0

Figure 32 Mechanism 2. Limit equilibrium modelling and finite element modelling assessment results for cross-section 5, adopting friction (ϕ) = 30°, and cohesion (c) = 0 kPa for the colluvium.

4.1.3 Sensitivity to groundwater

4.1.3.1 Mechanism 1

For failure mechanism 1, the adopted loess shear strength parameters (Table 15) are based on experimental tests carried out under water contents ranging from 16–19% to 7–10% wt, and therefore represent the upper end of the range of *in situ* water contents measured in samples. The effects of strength reduction of the loess, by water infiltration into the slope, were modelled by adopting a range of loess shear strength parameters. Therefore the range of parameters adopted take into account that the material could have increased moisture – and therefore lower shear strength – during periods of wet weather typical of most winters. The results from the sensitivity analysis (Figure 32) show that the factor of safety is most sensitive to changing cohesion.

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

Results from analyses of cross-section 5 show that the influence of increasing groundwater at the base has little influence on the factor of safety as the shapes of the simulated slide surfaces are relatively shallow. However, the factor of safety reduces to below 1 by including water filled tension cracks to a depth of about 5 m below ground level.

4.1.3.2 Mechanism 2

The sensitivity of the factor of safety to changes in transient ground water (pore pressure) for mechanism 2 has also been simulated as per mechanism 1. Results show that the factor of safety is sensitive to increasing groundwater levels above rockhead. For the range of shear strength parameters adopted for modelling of cross-section 5, the groundwater levels would need to rise by about 1.5–3 m above rockhead for the factor of safety to drop below 1. The factor of safety reduces to about 1 by including water filled tension cracks to a depth of about 5 m below ground level.

4.2 DYNAMIC CONDITIONS – BACK-ANALYSIS OF SLOPE DEFORMATION

Dynamic stability assessment comprised: 1) back-analysing the performance of the slope during the 2010/11 Canterbury earthquakes to calibrate the models and check that the resultant displacements are consistent with the displacements recorded during the earthquakes; and 2) using the calibrated models to forecast the likely magnitudes of future displacements under given levels of peak ground acceleration. A large displacement implies that the displacing mass could break down with the potential to form a large failure.

4.2.1 Amplification of ground shaking

The first stage of the assessment was to calculate the maximum acceleration at the cliff crest (A_{MAX}) to quantify any amplification effects. Results from the dynamic site response assessment are contained in Appendix 4. This was done for cross-sections 3 (southern spur) and 5 (northern spur), as they best represent the shape and geology of the overall slope, and the largest permanent displacements occurred in these areas during the 2010/11 Canterbury earthquakes.

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 cliff crest (A_{MAX}) is approximately linear. Over the range of modelled peak horizontal accelerations the amplification factor is typically about 2.8 times the input free field peak acceleration (Appendix 4). The input peak accelerations are those derived from the synthetic free field rock outcrop earthquake time acceleration histories (Holden et al., 2014).

The results from cross-sections 3 and 5, showing the response of the slope during the 22 February 2011 earthquake (Appendix 4), suggest that the impedance contrasts between the materials contribute most to the amplification of shaking, but that the peak horizontal accelerations (for all modelled earthquakes) concentrate around the convex break in slope, defined as A_{MAX}. These findings are similar to those reported by others (e.g., Del Gaudio and Wasowski, 2010), where material impedance contrasts have been shown to have a significant effect on the amplification of shaking. In experimental data, as the slope displaces during an earthquake, the slide surface can "base isolate" the mass above, resulting in lower levels of shaking and displacement. Therefore, the reported amplification factors are near the upper bound of published topographic amplification factors. Assessment of this is outside the scope of this report.

4.2.2 Back-analysis of permanent slope deformation

Earthquake-induced permanent displacements were calculated adopting the decoupled method (Makdisi and Seed, 1978) using 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}). Failure mechanisms 1 and 2 were adopted for all assessments. Permanent displacements are estimated along the slide surface, where the displacing mass is treated as rigid-plastic body and no internal plastic deformation of the mass is accounted for. Also, the mass accrues no displacement at accelerations below the yield acceleration.

The 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 Defender Lane mass movement were recorded during these events. Variable loess material strength parameters were used; these are listed in Table 19. The results were then compared to the recorded coseismic permanent slope displacements for each earthquake, for each cross-section (Figure 33 and Figure 34).

Table 19 Loess material strength parameters used for modelling permanent coseismic displacements for cross-sections 3 and 5. Coseismic displacements are inferred from survey records and field mapping of cracks.

Cross-section	Earthquake	Loess friction (ϕ) (degrees)	Loess cohesion (c) (kPa)	Total inferred coseismic displacement (m)	Inferred coseismic displacement with vertical only (m)
3	22 February 2011	30 35 35 35	10 25 30 40	0.2–0.3	0.1–0.2
5	22 February 2011	30 35 35 35	10 25 30 40	0.5–0.6	0.2–0.4
3	13 June 2011	30 35	10 30	0.01–0.02	0.01–0.02
5	13 June 2011	30 35	10 30	0.01–0.02	0.01–0.02

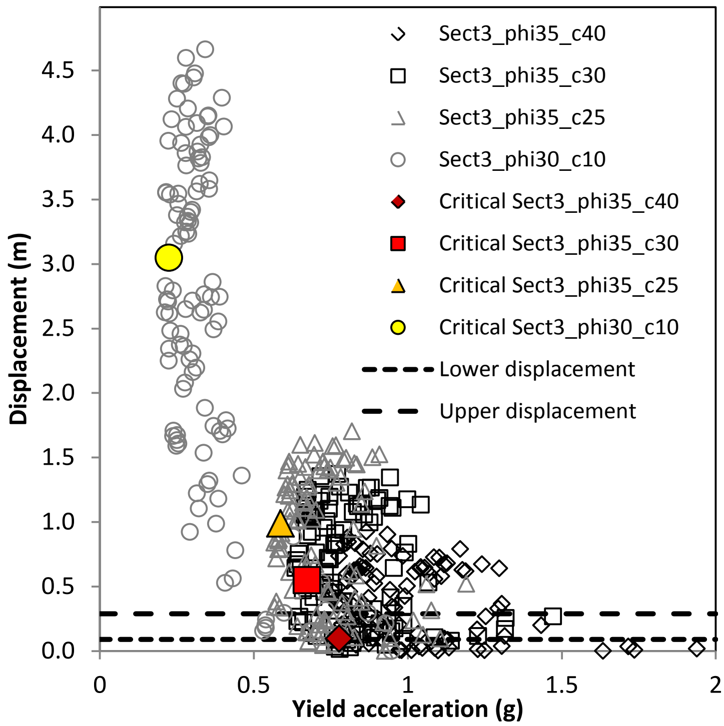


Figure 33 Modelled Slope/W decoupled displacements for cross-section 3 Defender Lane, adopting variable estimates of the loess material strength, for failure mechanism 1. Each data point represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. Critical slide surfaces are those with the lowest yield accelerations for the given material strength parameters. The dashed lines represent the total recorded coseismic displacement (Upper displacement) and displacement with horizontal and vertical components only (Lower displacement) for the cross-section.

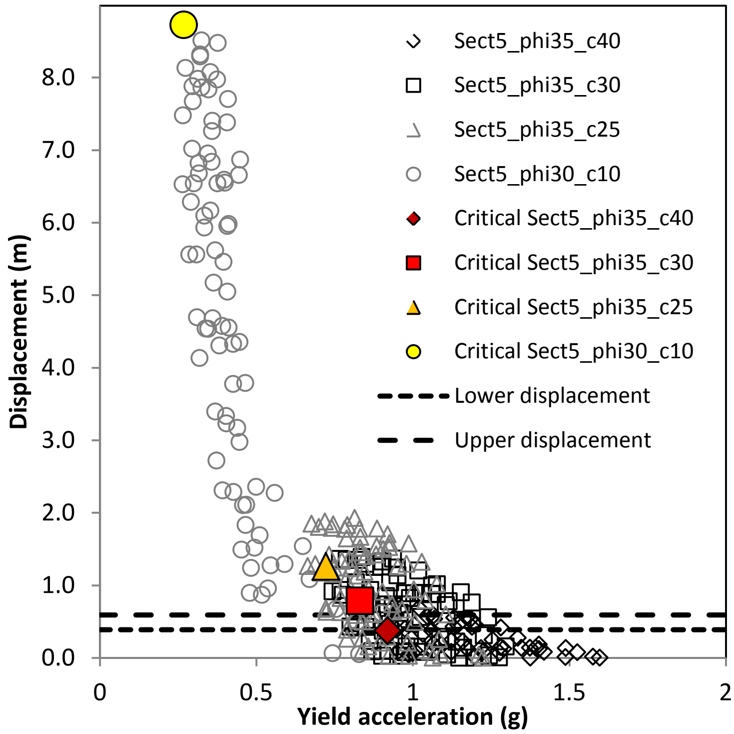


Figure 34 Modelled Slope/W decoupled displacements for cross-section 5 Defender Lane, adopting variable estimates of the loess material strength, for failure mechanism 1. Each data point represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. Critical slide surfaces are those with the lowest yield accelerations for the given material strength parameters. The dashed lines represent the total recorded coseismic displacement (Upper displacement) and displacement with horizontal and vertical components only (Lower displacement) for the cross-section.

Results from the assessment show that for failure mechanism 1, the modelled slide surfaces have lower yield accelerations and larger calculated permanent displacements, than those modelled adopting failure mechanism 2. For failure mechanism 1, the "best" correlation between the inferred permanent coseismic displacements and modelled displacements is for modelled slide surfaces adopting loess material strength parameters of friction (ϕ) of 35° and cohesion of 30–40 kPa. The range of recorded displacements shown on Figure 33 and Figure 34 are those inferred: 1) only from crack apertures of the slope crest where both horizontal and vertical displacements had been recorded for the 22 February 2011 earthquake, referred to as "Lower displacement"; and 2) from all mapped and measured crack apertures, referred to as "Upper displacement".

Results from the assessment adopting the 13 June 2011 synthetic earthquake motions, and loess material strength parameters of friction (ϕ) = 30° and 35° and cohesion (c) = 10 and 30 kPa, are shown in Figure 34. The loess parameters of friction (ϕ) = 30° and cohesion (c) = 10 kPa were adopted as lower strength estimates as they represent the "typical" loess strength parameters used by engineering consultants working in the Port Hills. The loess parameters of friction (ϕ) = 35° and cohesion (c) = 30 kPa were adopted as "average" parameters as these gave the best correlation between recorded and modelled permanent coseismic slope displacements in response to the 22 February 2011 earthquake.

Results show that the magnitude of the modelled permanent displacements adopting the "average" loess strength parameters are within the range of recorded displacements. However, estimated permanent displacements adopting the "lower" strength estimates suggest displacements should have been much larger than those actually recorded, indicating the parameters are not representative of the loess at the time of the earthquake. The locations of the slope where recorded permanent displacements were both horizontal and vertical correspond well with the location of the modelled slide masses with the lowest yield accelerations (Figure 35).

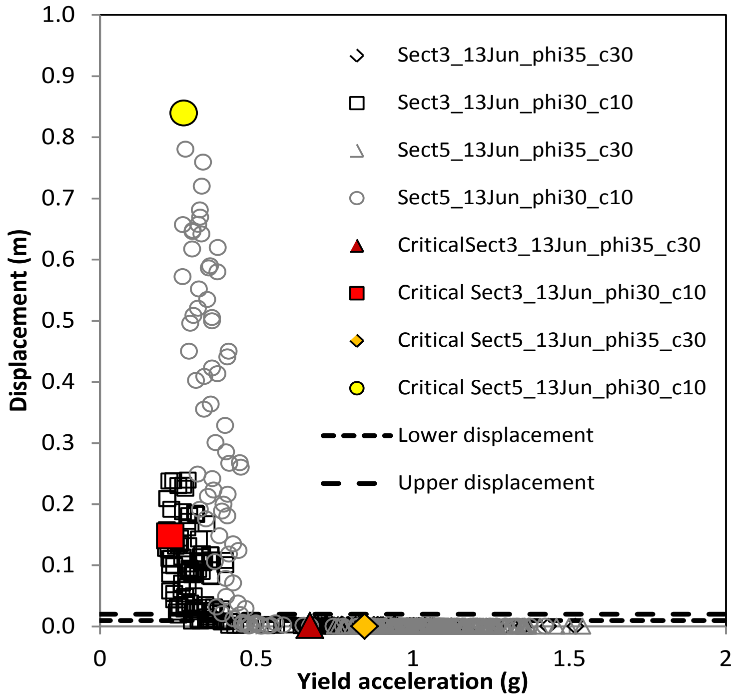


Figure 35 Modelled Slope/W decoupled displacements for cross-sections 3 and 5 Defender Lane, adopting variable estimates of the loess material strength, for failure mechanism 1. Each data point represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 13 June 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. Critical slide surfaces are those with the lowest yield accelerations for the given material strength parameters. The dashed lines represent the total recorded coseismic displacement (Upper displacement) and displacement with horizontal and vertical components only (Lower displacement) for the cross-section.

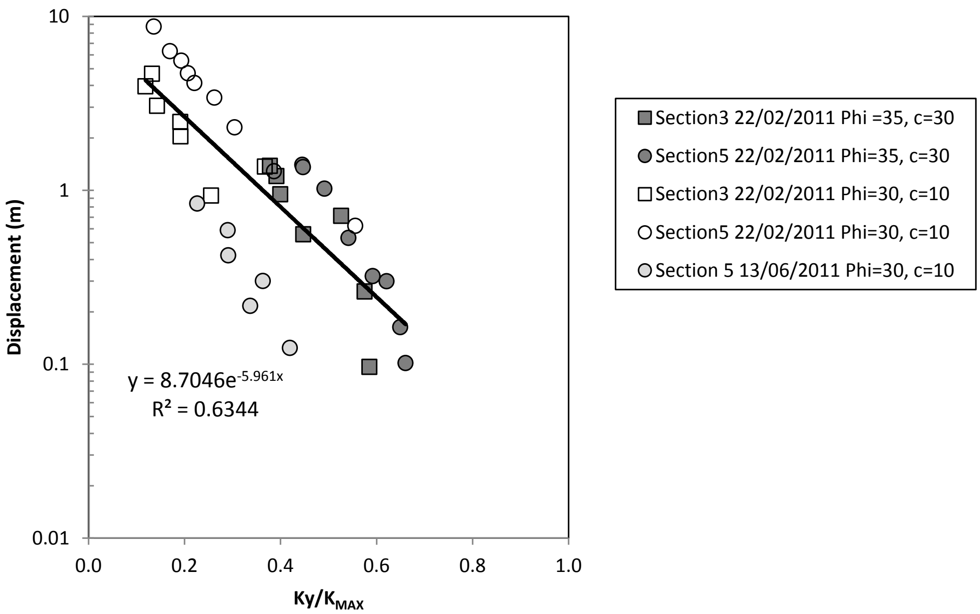
For these assessments the displacements measured from survey marks and crack apertures are assumed to represent the coseismic permanent displacement of the slope during the 22 February and 13 June 2011 earthquakes. The estimated displacements of the slope for the 16 April and 23 December 2011 earthquakes, adopting the "lower" and "average" loess strength estimates, were 0 m, where actual recorded permanent displacements of the slope crest were also 0 m.

Permanent displacements estimated using the average loess parameters of friction (ϕ) of 35° and cohesion (c) of 30 kPa, gave the best comparison with inferred permanent displacements.

4.2.3 Forecast modelling of permanent slope displacement

Permanent displacements, from the decoupled assessment results from the 22 February and 13 June 2011 modelled earthquakes, were then calculated for a range of slide-surface geometries (failure mechanism 1 only) with different ratios of yield acceleration (*Ky*) to the maximum average acceleration of the failure mass (K_{MAX}). The average maximum acceleration (K_{MAX}) was calculated for each selected slide surface. About 10 slide surfaces were chosen to represent the results from each earthquake input motion, adopting the "lower" and "average" estimates of the loess material strength parameters. Results are shown in Figure 36, and the cross-sections showing the slide-surface geometries from the 22 February 2011 earthquake for cross-sections 3 and 5 are shown in Figure 37 and Figure 38.

The selected slide-surface geometries shown in Figure 37 and Figure 38 show that their shapes are consistent with the results from the field mapping of cracks, where the location of the modelled slide masses with the lowest yield accelerations correspond to the locations where vertical and horizontal permanent displacements were recorded.



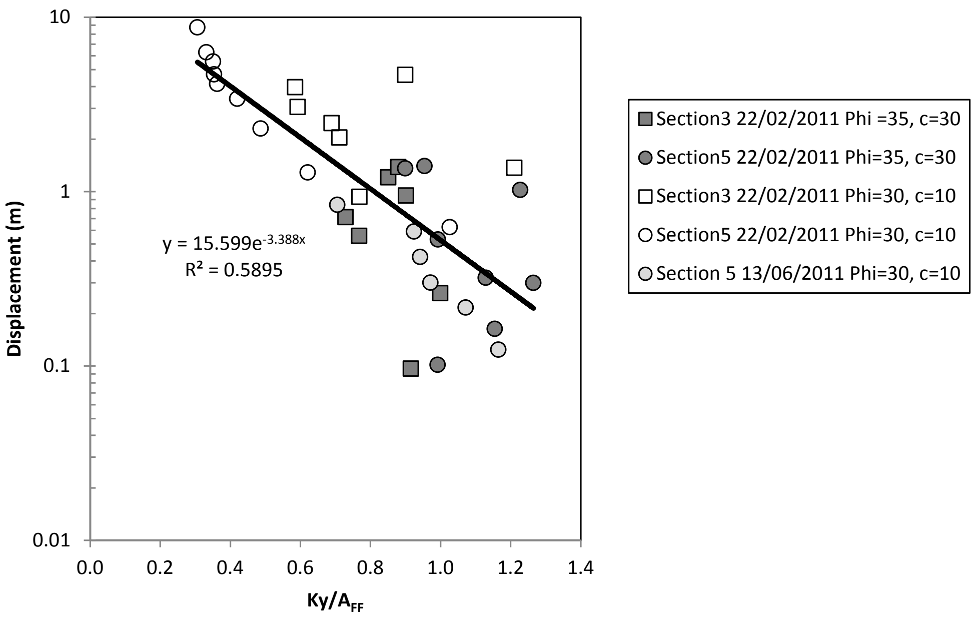


Figure 36 Cross-sections 3 and 5, decoupled Slope/W displacements calculated for different ratios of yield acceleration to maximum average acceleration of the mass (*Ky/K$_{MAX}$*) and the maximum acceleration of the input motion (*Ky/A$_{FF}$*), for selected slide-surface geometries. The results shown adopt the "average" loess strength parameters of friction (ϕ) = 35° and cohesion (c) = 30 kPa; and the "lower" estimates of ϕ = 30° and c = 10 kPa. Synthetic rock outcrop time acceleration histories for the 22 February and 13 June 2011 earthquakes were used as inputs for the assessment. (N = 39). The black dashed lines are exponential trend line fitted to the semi–log data. The formula and the coefficient of determination (R^2) for the trend lines are shown.

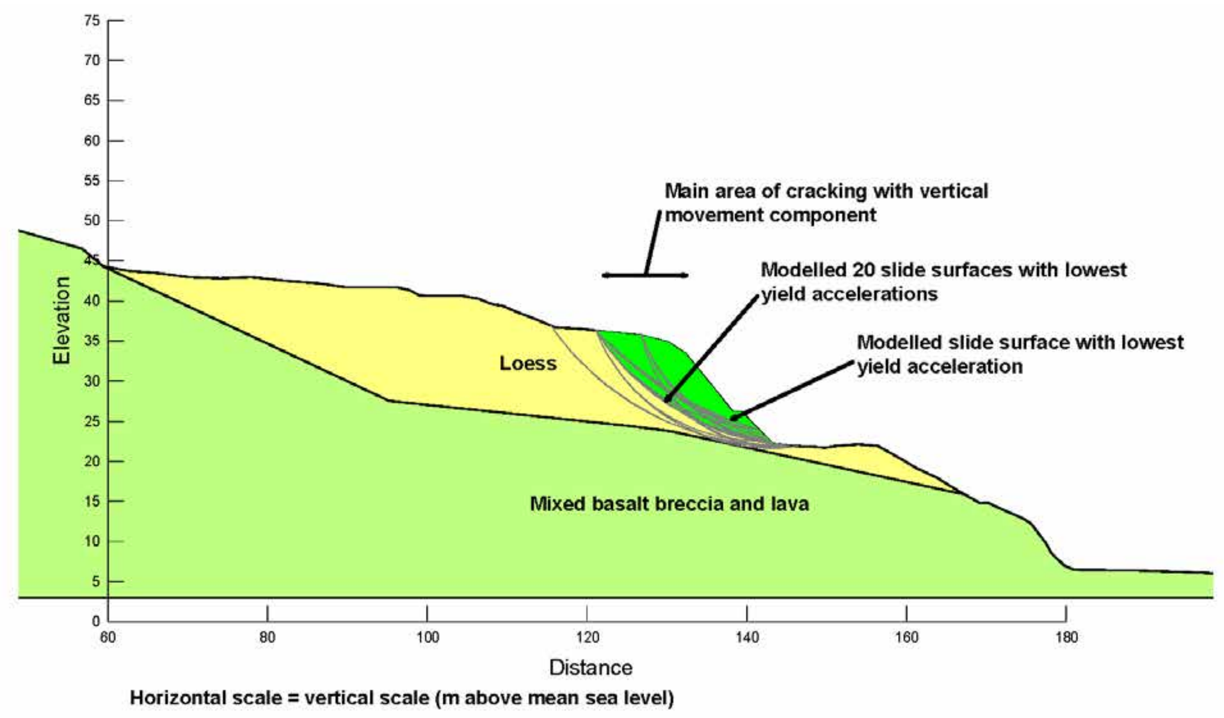


Figure 37 Cross-section 3 seismic slope stability assessment for the 22 February 2011 earthquake (loess parameters of friction (ϕ) of 35 and cohesion (c) of 30 kPa).

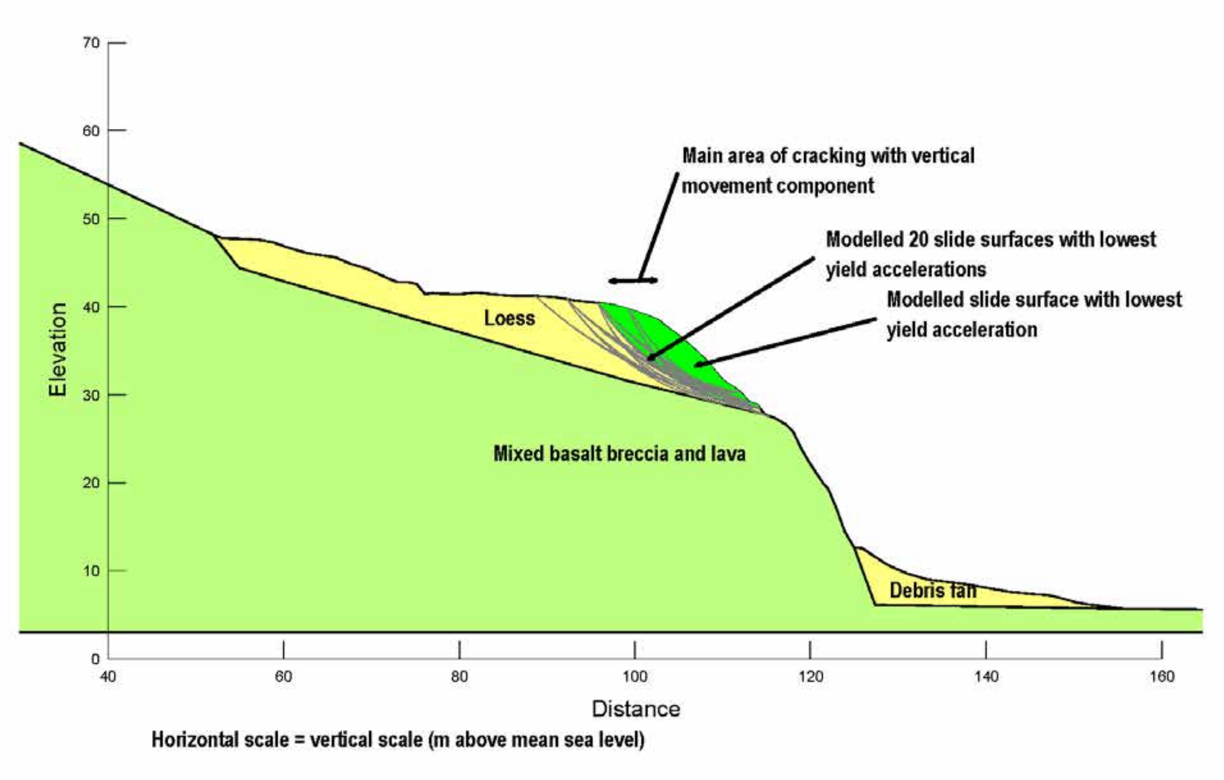


Figure 38 Cross-section 5 seismic slope stability assessment for the 22 February 2011 earthquake (loess parameters of friction (ϕ) of 35 and cohesion (c) of 30 kPa).

The results show that between Ky/K_{MAX} values of 0.2–0.8 and Ky/A_{FF} values of 0.2–1.2, the data are well fitted to a straight line (exponential trend line) in semi-log space. The coefficient of determination (R^2) is 0.7 for Ky/K_{MAX} and 0.6 for Ky/A_{FF}, and includes all of the plotted data (N = 39). The gradients of the fitted lines are different, with the Ky/K_{MAX} line having a 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 poorer coefficient of determination for ratios of Ky/A_{FF} is also not unusual as Newmark (1965) displacements are highly sensitive to the high frequency components of the input motions, which can vary from event to event. By comparison, K_{MAX} "filters" the higher frequency components, and thus is less sensitive to the input motion characteristics. For ratios of Ky/A_{FF} in Figure 36, the estimated magnitudes of displacement are consistent with those reported by Jibson (2007), where these data plot between the ranges for earthquakes of M6.5–7.5 as reported by Makdisi and Seed (1978) and plotted by Jibson (2007).

The peak ground acceleration of the input motion (A_{FF}) does not take into account amplification effects caused by the slope geometry, and at this site, the material contrasts within the slope, between the loess/volcanic colluvium and the underlying rock (Appendix 4). From the data in Figure 36, the mean ratio of K_{MAX} to A_{FF} for cross-section 1 is 2.1 (mean plus one standard deviation), meaning that K_{MAX} is 2.1 times greater than the peak horizontal ground acceleration of the input motion, if assuming the mean plus one standard deviation of the mean.

The results show that the modelled slide surfaces concentrate behind the slope crest (Figure 37 and Figure 38), with very few extending further back up slope. As expected, those shallow slide surfaces nearest the cliff edge have lower factors of safety, and therefore lower yield accelerations (Ky), compared to those deeper slide surfaces further back from the cliff edge. The assessment using the 22 February 2011 synthetic earthquake shows that even when the lower loess strength parameters are adopted, displacement of the entire loess mass (corresponding to the main area of mapped cracks) is not likely to occur as a result of landslide processes. It is therefore possible that many of the mapped cracks, up-slope and away from the slope crest – where only horizontal displacements were recorded – formed in response to the inelastic behaviour of the loess during shaking, rather than large-scale landslide processes.

4.2.3.1 Forecast modelling results

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

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

The relationship between the yield acceleration and the maximum average acceleration (from Figure 36) has been used to determine the likely range of displacements of a given failure mass with an adopted yield acceleration (Ky) at given levels of peak free field horizontal ground accelerations (A_{FF}) and the equivalent maximum average ground acceleration *(K_{MAX})*. The results are shown in Table 20. Conservative yield accelerations have been adopted to take into account the possibility that the current shear strength of the materials has degraded as a result of the past movement.

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

Table 20 Forecast modelling results from the dynamic slope stability assessment for cross-sections 3 and 5 – for selected slide surfaces through loess, adopting failure mechanism 1. Estimated displacements are rounded to the nearest 0.1 m.

Free field peak ground acceleration (A_{FF})[1] (g)					0.2	0.5	0.7	1.0
Adopted K_{MAX} to A_{FF} ratio (mean + 1 standard deviation)					2.1			
Equivalent K_{MAX}					0.4	1.0	1.4	2.0
Cross-section	**Loess material strength[2]**		**Yield acceleration (g)**		**Estimated displacement (m)**			
	Friction ϕ (°)	**Cohesion kPa**						
3 and 5	30	10	Lower	0.2	0.3	2.4	3.5	4.6
3 and 5	35	30	Average	0.6	0	0.2	0.6	1.4

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

2 Loess material strength parameters adopted for the assessment represent the lower and average estimates of their strength.

4.3 SUMMARY OF RESULTS FROM THE STATIC AND DYNAMIC STABILITY ASSESSMENTS

1. Under current conditions it is possible that failure of the trial slide surfaces would occur under either static or dynamic conditions. It should be noted that material strengths – and therefore the slope factors of safety – are likely to reduce with time, and the occurrence of future earthquakes.

2. The strength of the loess has been shown to be sensitive to changes in water content. The earthquake-induced cracks in the slope and broken services caused mainly by the 22 February and 13 June 2011 earthquakes, now allow surface water to more easily enter the soil mass, possibly changing the water content of the loess over time. The time frame over which such changes could occur is not known.

3. It is therefore reasonable to assume that the slopes at the Defender Lane mass movement are now more susceptible (vulnerable) to future earthquakes and other triggering events in particular rain.

4. Under static and dynamic conditions the trial slide surfaces with the lowest factors of safety and those with the lowest yield accelerations (*Ky*), are those nearest the slope crest (and are relatively shallow), and not those further back from the crest, which represent large failure of the entire loess mass.

5. Seismic site response assessment suggests that the peak ground amplification factors between the peak synthetic rock outcrop free field accelerations (A_{FF}) and the modelled peak accelerations at the cliff crest (A_{MAX}) for the Defender Lane mass movement are about 2.8–3 for both horizontal and vertical motions. Results show that the accelerations are mainly amplified within the loess, especially at higher input peak ground accelerations. These results suggest that contrasting material properties, in this case between the underlying rock and the loess, amplify the accelerations within the loess.

6. Estimated dynamic slope displacements (from the decoupled method) behind the slope crest using the “lower” loess material strength parameters (friction (ϕ) = 30° and cohesion (c) = 10 kPa) are too large when compared to the inferred displacements from crack apertures (with vertical components). Permanent displacements estimated using the average loess parameters (friction (ϕ) = 35° and cohesion (c) = 30 kPa), gave the best comparison with the recorded displacements. This is expected as the main earthquake (22 February 2011) occurred at the end of summer when the loess water content would be at its seasonally lowest.
7. Measurements and observations made of the performance of loess slopes during the 2010/11 Canterbury earthquakes indicated that permanent coseismic displacements of typically less than 1 m occurred, which triggered small – volumes of debris typically less than 100 m^3 – earth/debris flows.
8. Under longer duration earthquake shaking or wetter site conditions, coseismic permanent slope displacements could be larger and therefore volumes of any resultant earth/debris flows could also be larger. If the 22 February 2011 earthquake had occurred at the end of winter, it is likely that larger displacements of the slope would have occurred, which may have resulted in earth/debris flows.
9. However, given the current cracked nature of the slopes, and the increased susceptibility of the loess to strength loss caused by increasing moisture contents, the main additional landslide type at the Defender Lane mass movement is rainfall-induced earth/debris flows sourcing from the loess. This is additional to: 1) slope cracking; and 2) cliff-collapse hazards (from the steep rock slopes refer to Massey et al., 2012) hazards.

4.4 POTENTIAL DEBRIS SOURCE VOLUME ESTIMATION

The likely locations and volumes of potential sources (1–10), adopting failure mechanism 1, have been estimated based on:

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

Three possible failure volume estimates – a 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 39) and estimated volumes are shown in Table 21. Source areas 1–10 represent the range (volume and locations) of potential earth/debris flows that could occur at the site.

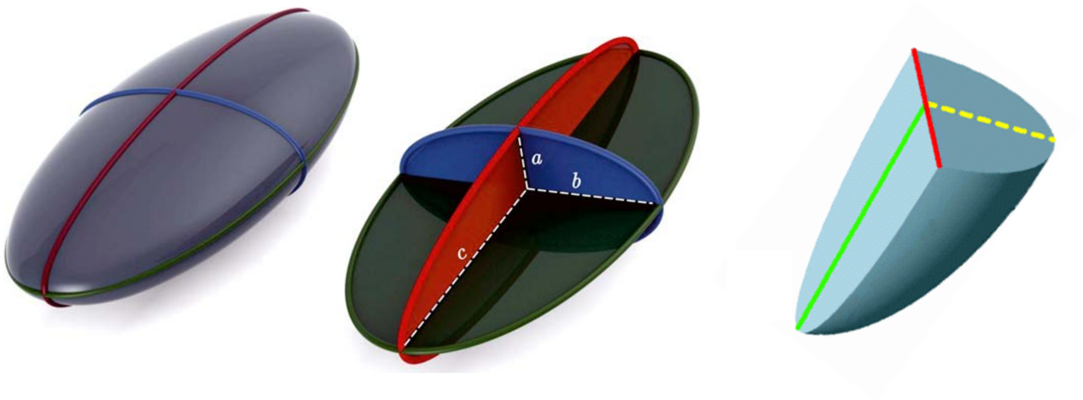


Figure 39 Estimation of landslide volume assuming a quarter-ellipsoid shape.

Table 21 Example earth/debris flow source volumes (the first digit in the number is significant) and the mean minus one standard deviation Fahrboeschung angles.

Source ID	Volume type	Estimated volume (m^3)	Fahrboeschung angle (°)
1	LOWER	80	26.4
	MID	290	23.5
	UPPER	1130	20.6
2	LOWER	780	24.6
	MID	660	21.7
	UPPER	1770	19.8
3	LOWER	40	28.4
	MID	200	24.3
	UPPER	580	22.0
4	LOWER	90	26.2
	MID	280	23.6
	UPPER	1020	20.8
5	LOWER	130	25.3
	MID	620	21.8
	UPPER	1650	19.9
6	LOWER	70	26.7
	MID	330	23.2
	UPPER	650	21.8
7	LOWER	40	28.3
	MID	270	23.6
	UPPER	650	21.8
8	LOWER	25	29.4
	MID	230	24.1
	UPPER	540	22.1
9	LOWER	20	29.4
	MID	80	26.5
	UPPER	280	23.8
10	LOWER	30	28.8
	MID	110	25.7
	UPPER	190	24.5

The credibility of these potential failure volumes has been evaluated by comparing them against estimated volumes of individual landslides (mainly earth/debris flows) 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 40 and the data are well modelled by a log normal distribution, adopting the area depth relationships of Larsen et al. (2010). The estimated range of source volumes shown in Table 21 (about 50–2000 m^3) are well within the range of other loess landslides in the Port Hills mapped by Townsend and Rosser (2012).

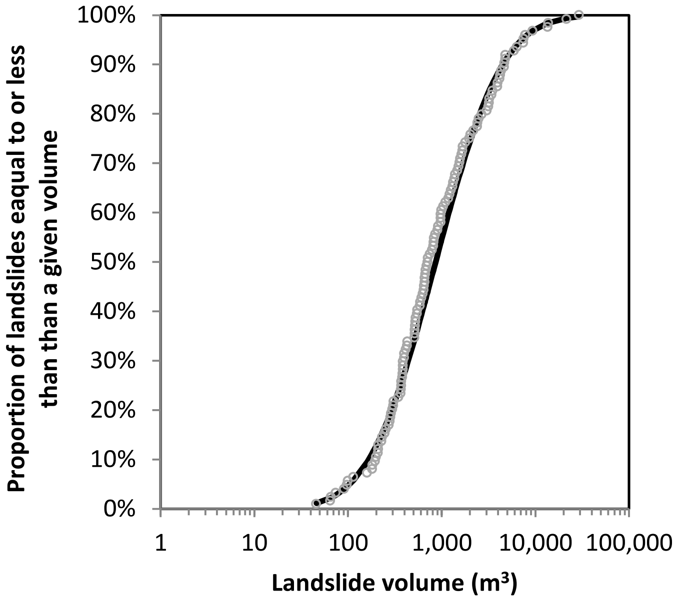


Figure 40 Estimation of landslide volumes in the Port Hills loess from Townsend and Rosser (2012) adopting the area depth relationships of Larsen et al. (2010).

4.5 POTENTIAL DEBRIS RUNOUT

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

4.5.1 Empirical data

For each source the fahrboeschung angles have been defined applying the method described in Massey and Carey (2012) as shown in Figure 41, using published debris flow records reflecting different types of slope shape that could affect the debris runout.

It is noted, as detailed in 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 mean and mean minus one standard deviation fahrboeschung angles for each potential source area (1–10) and each volume (lower, middle and upper estimates), are plotted in Figure 42.

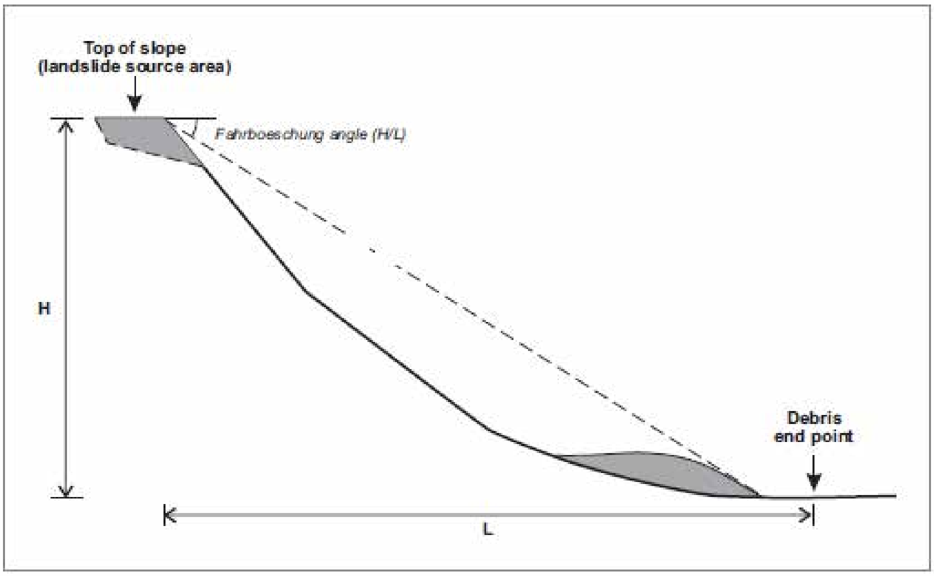


Figure 41 Fahrboeschung angle method, after Massey and Carey (2012).

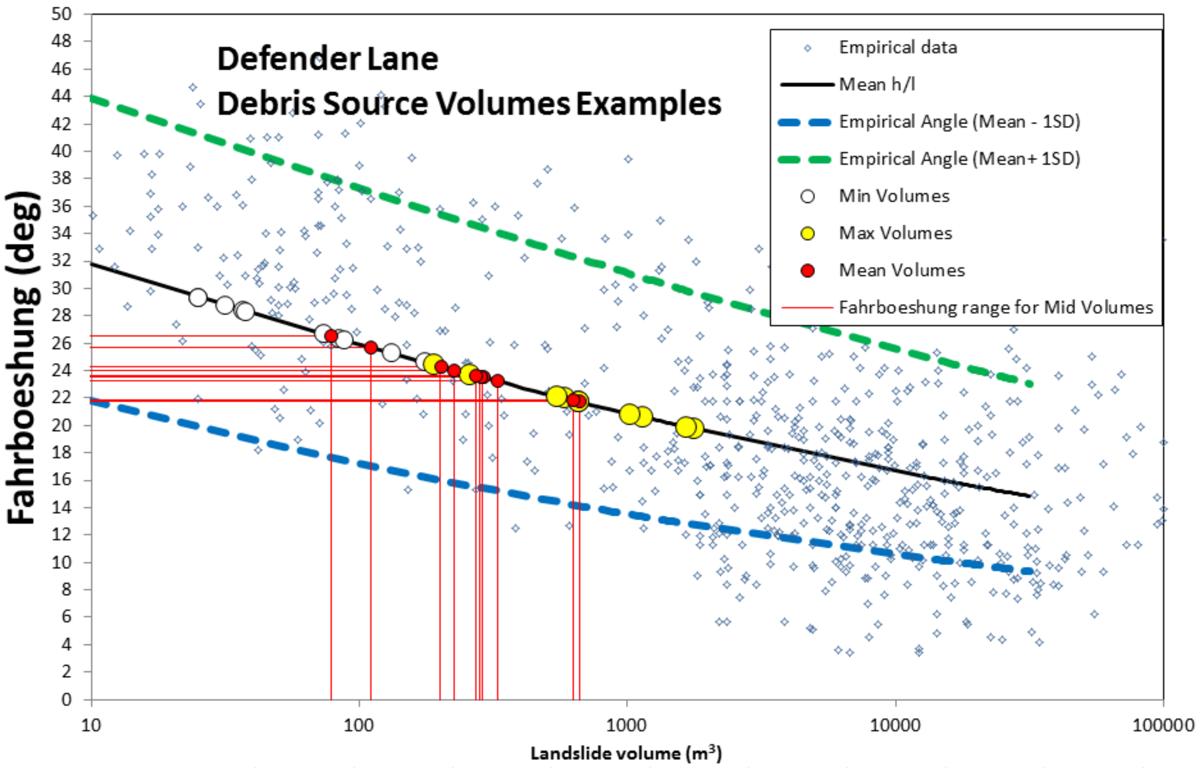


Figure 42 Estimation of Fahrboeschung angles for volumes shown in based on empirical runout data presented by Massey and Carey (2012).

4.5.2 Numerical modelling

RAMMS software takes into account the slope geometry of the site when modelling debris runout. The RAMMS model parameters were calculated from the back-analysis of four Port Hills debris flows. The modelled parameters mu (µ) and xi were optimised to obtain a good correlation between the modelled versus actual runout and deposited debris heights.

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). However, to the best of our knowledge there are no direct physical means of deriving these parameters from field measurements. The model, with calibrated input parameters (μ = 0.06 (7°) and x_i = 200 m/s^2), was used to estimate the likely velocity and depth of the debris at given locations down the slope for a series of failure volumes estimated from the engineering geological model. The μ and x_i values are comparable to results from other assessments compiled by Andres (2010) for debris flows (Figure 43).

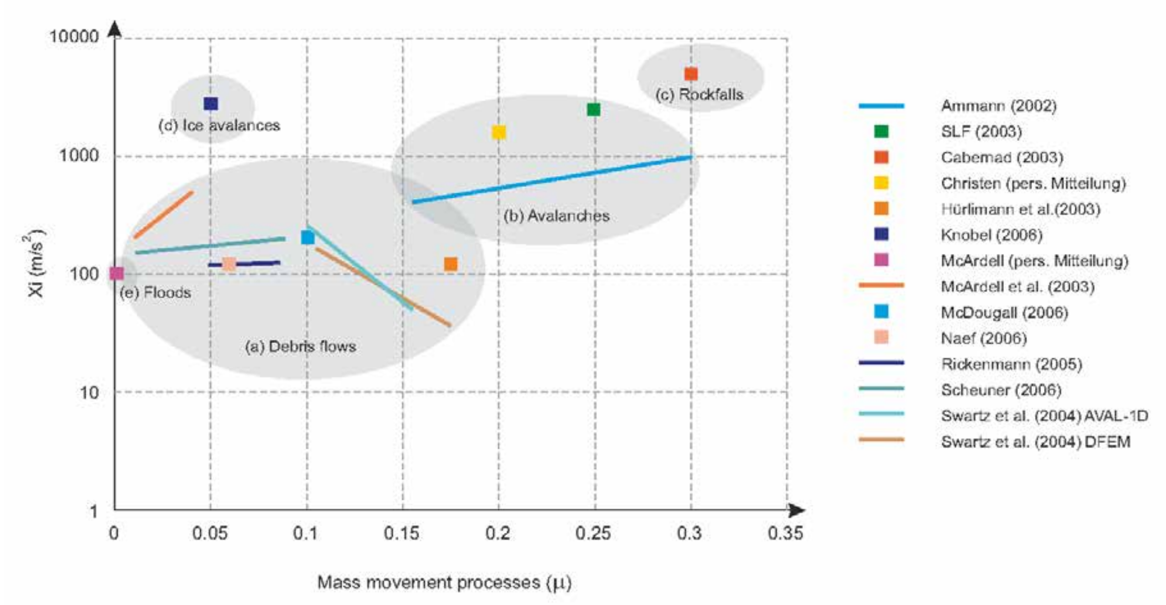


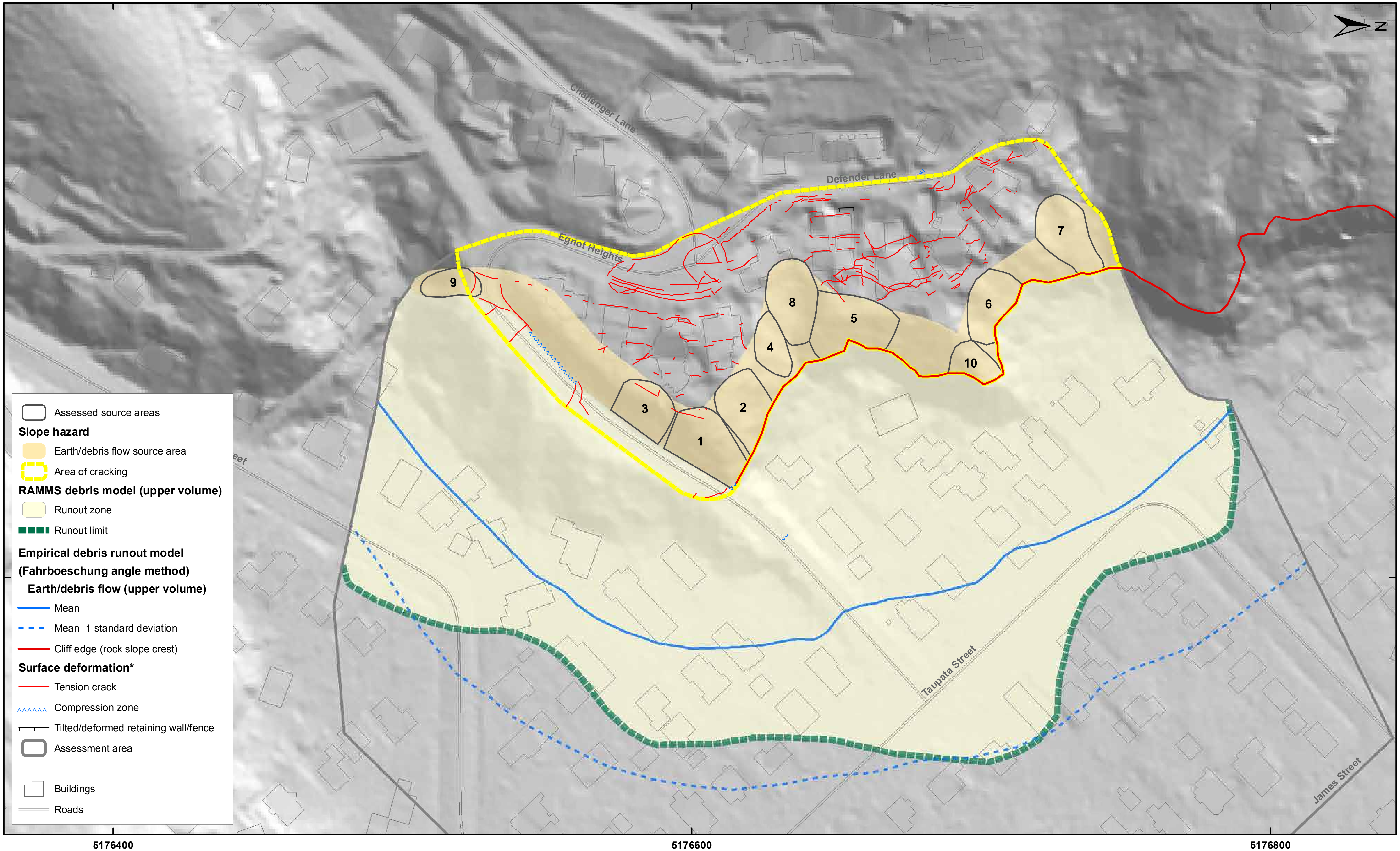
Figure 43 Range of parameters for different mass movement processes: **a**) debris flows, **b**) snow avalanches, **c**) rockfalls, **d**) ice avalanches, **e**) debris floods. Modified from Andres (2010).

4.5.3 Forecast runout modelling

A hazard map (Figure 44, Map 1) presents the empirical and numerical runout limits from all of the modelled source areas (1–10) for the upper volume source estimates.

The mean and mean minus one standard deviation fahrboeschung angles for each potential source area (1–10), are shown in Figure 44. The empirically derived fahrboeschung runout lines are based on combining the runout lines from each of the source areas for the upper source volume estimates only. The estimated runout distances from RAMMS are shown in Appendix 5 (debris height) and Appendix 6 (debris velocity), for source areas 1–10 using the middle source volume estimates, along with the corresponding mean and mean minus one standard deviation fahrboeschung angles.

Figure 44, Map 2, shows a hazard map for debris avalanches (cliff collapse hazard), modified from Massey et al. (2012), which illustrates the fahrboeschung angles and runout limits of debris avalanches. Note that the runout limits assessed for the earth/debris flow hazards (Figure 44, Map 1) are much further out from the toe of the slope, than the runout limit of the debris avalanches. This reflects the more fluid and mobile nature of the earth/debris flows.



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

DRW: BL

CHK: FDP, CM

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EARTH/DEBRIS FLOW HAZARD MAP

Defender Lane
Christchurch

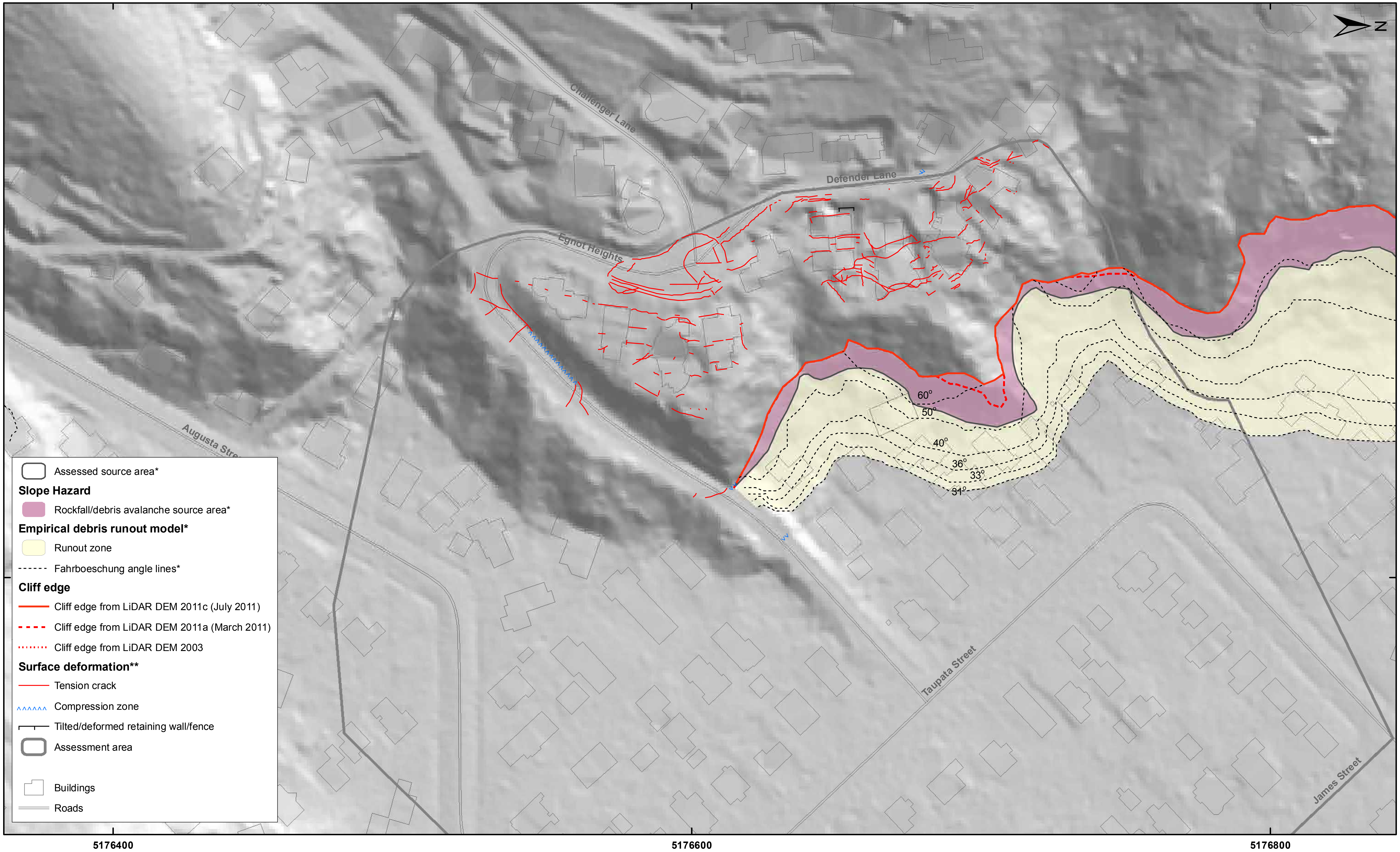
FIGURE 44

Map 1

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Hazard and risk assessed in report CR2012/57
** 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: FDP, CM

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CLIFF COLLAPSE HAZARD MAP

Defender Lane
Christchurch

FIGURE 44

Map 2

FINAL

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5.0 RISK ASSESSMENT RESULTS

5.1 TRIGGERING EVENT FREQUENCIES

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

5.1.1 Frequency of earthquake triggers

For earthquake triggers, the frequency of a given free-field peak ground acceleration occurring is obtained from the New Zealand National Seismic Hazard Model (Table 22) (Stirling et al., 2012). The increased level of seismicity in the Christchurch region is incorporated in a modified form of the 2010 version of the National Seismic Hazard Model (Gerstenberger et al., 2011).

5.1.1.1 Peak ground acceleration and permanent slope displacement

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

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

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

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

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

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

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

Permanent slope displacements during the 2010/11 Canterbury earthquakes estimated from crack apertures were about 0.3–0.6 m (for the 22 February 2011 earthquakes), and the slope did not fail catastrophically (i.e., with the debris running out as an earth/debris flow). This did not mean the slope was “safe” however, as the dwellings located in the assessed source areas still suffered significant damage.

The estimated magnitude of permanent slope displacement, in the assessed sources and area of cracking (Figure 44), in a future earthquake, was based on the decoupled assessment results for failure mechanism 1. The permanent displacement 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 36). Different levels of peak ground acceleration were adopted, and each multiplied by the site-specific ratio of K_{MAX} to A_{FF} (assuming the mean plus one standard deviation) to estimate the equivalent maximum average acceleration of the mass (K_{MAX}) for the given value of A_{FF}. For example, an A_{FF} of 0.2 g would have an equivalent K_{MAX} of 0.3 g, assuming a ratio of 2.1.

Table 22 The annual frequency of a given peak ground acceleration occurring on rock (Site Class B) for different years adopting the 2012 probabilistic seismic hazard model (PSHM) for Christchurch (Gerstenberger et al., 2011). Note: these are free-field rock outcrop peak ground accelerations (equivalent to A_{FF}).

Free field peak horizontal ground accelerations (A_{FF})[1] (g)	**0.2**	**0.5**	**0.7**	**1.0**
Year 2016 annual frequency of event (from PSHM)	0.090	0.0157	0.0059	0.00164
Year 2016 return period (years)	11	64	169	610
Next 50-year average annual frequency of event (from the PSHM)	0.042	0.0072	0.0027	0.00076
Next 50-year average return period (years)	24	139	370	1316
Adopted K_{MAX}[2] to A_{FF} ratio	2.1 (mean + 1 STD)			
Equivalent K_{MAX} for the given A_{FF}	0.4	1.0	1.4	2.0
Estimated displacements (m) for Ky of 0.2 (lower loess strength)	**0.3**	**2.4**	**3.5**	**4.6**
Estimated displacements (m) for Ky of 0.6 (upper loess strength)	**0**	**0.2**	**0.6**	**1.4**

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

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

The relationship in Figure 36 is based on past performance of the slope, however, the slope moved about 0.6 m during the 2010/11 Canterbury earthquakes. It is now possible that catastrophic failure of the slope could occur at further permanent slope displacements of less than 0.6 m. At free-field peak ground accelerations of greater than 1.0 g the amount of permanent displacement could lead to catastrophic failure and runout of debris, as performance of the slope at these levels of ground acceleration is unknown. At these ratios of Ky/K_{MAX} (less than 0.1) the magnitude of displacement tends to increase rapidly with only relatively small changes in the Ky/K_{MAX} ratio.

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

It should be noted that the displacements at different ratios of Ky/K_{MAX} were calculated using the synthetic earthquake acceleration time histories for the 22 February and 13 June 2011 earthquakes, which were both near-field earthquakes of short duration, but high amplitude. The calculated displacements in Figure 36 represent displacements in response to these earthquakes (adopting failure mechanism 1). Earthquakes of longer duration may affect the site in different ways, for example the response of the loess (at higher water contents representative of winter conditions) may be non-linear, and could lead to larger permanent displacements. Conversely, the peak amplitudes relating to longer duration earthquakes from more distant sources are likely to be lower and may not trigger displacement of the slope.

It should also be noted that parts of the slope have already moved more than 0.6 m during the 2010/11 Canterbury earthquakes, and it is not known how much more movement the slope can take, before failing catastrophically.

5.1.1.2 Deaggregation of the National Seismic Hazard Model

The seismic performance of the slope in future earthquakes was inferred from assessing its performance in past earthquakes, mainly the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, using the relationship established between peak ground acceleration and the amount of permanent slope displacement. These earthquakes varied in magnitude between M5.2 and M6.3, and were “near-field” i.e., their epicentres were very close, within 5 km, of the Defender Lane 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 interfaces; 2) mapped active faults; and 3) unknown or “background” earthquakes. For the risk assessment it is important to deaggregate the national seismic hazard model to assess which earthquake sources contribute the most to it.

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

5.1.2 Frequency of rainfall triggers

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

- There is evidence of historical and prehistoric earth/debris flows at the site;
- The 5 March 2014 rainstorm in Lyttelton (130 mm) triggered several large earth/debris flows in the Port Hills. 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, which triggered only a few small (less than 50 m^3) earth/debris flows;
- 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 no larger landslides occurred during the 10–20 year return period event, the water content of the loess and colluvium at the time of this event was likely to have been seasonally low as the storm occurred at the end of a dry summer.

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

5.1.3 Overall triggering event frequency

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

For rainfall (static) triggers:

- For the risk assessment, various return periods of 20, 50, 100 and 200 years for the triggering event were assumed, and the sensitivity of the risk estimates to these return periods assessed.
- Failures of the slope could occur from anywhere within the identified source areas and could vary greatly in volume. The assessed source areas (1–10) represent the geometries and volumes of the earth/debris flows that could potentially fail in the assessment area.

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

5.2 DWELLING OCCUPANT RISK

The results from the risk assessment are shown in Figure 45 as the annual individual fatality risk. The map shows the annual individual fatality risk estimated for earth/debris flows sourcing from the loess slope adopting the estimated lower, middle and upper source volumes. The risks presented should be added to those estimated by Massey et al. (2012) for cliff collapse hazards associated with the steep rock slopes.

5.2.1 Earth/debris flows

The risk from earth/debris flows from source areas 1–10, adopting the estimated lower, middle and upper source volumes, and a return period of 50 years for the triggering event, are shown in Figure 45.

Based on the results from the static and dynamic stability assessments of sections 1–3, 5 and 6, the potential earth/debris flow source areas are located along the crest of the steeper slope. The risk associated with the source areas is inferred to be the same as the risk in the runout zone immediately below the assessed source areas, which is shown as 10^{-4} or greater in Figure 45.

A 10 m wide strip has been added to the crest of the identified source areas, where the source areas could potentially in the future retrogress in an up-slope direction, beyond the currently assessed extent of the sources. This has been termed an “earth/debris flow source 10 m enlargement area”.

Two annual individual fatality risk lines representing the position of the 10^{-4} risk contour are shown on the map for the middle and upper estimates of source volume, adopting the 50-year return period. The range of annual individual fatality risks estimated for the lower source volumes are less than 10^{-4} for all modelled source areas and therefore no line can be shown for the lower source volumes.

The area shown as the “10^{-4} uncertainty zone” represents the area of slope where the risk could be greater than 10^{-4} for the upper and middle source volumes but less than 10^{-4} for the lower source volume estimates. This 10^{-4} uncertainty zone corresponds to the approximate extent of the mapped debris fans from past earth/debris flows.

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

All of the results shown in Figure 45 are predicated on the event that could trigger failure of the sources, having an annual frequency of 0.02 (return period of 50 years), and that each source area has a 10% probability of occurring in that event.

5.2.2 Other variables adopted for the risk assessment

Other variables used in the risk assessment were discussed at a workshop with Christchurch City Council on 18 March 2014. Based on the results from the workshop the risk estimates presented in Figure 45 adopt the following main variables:

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

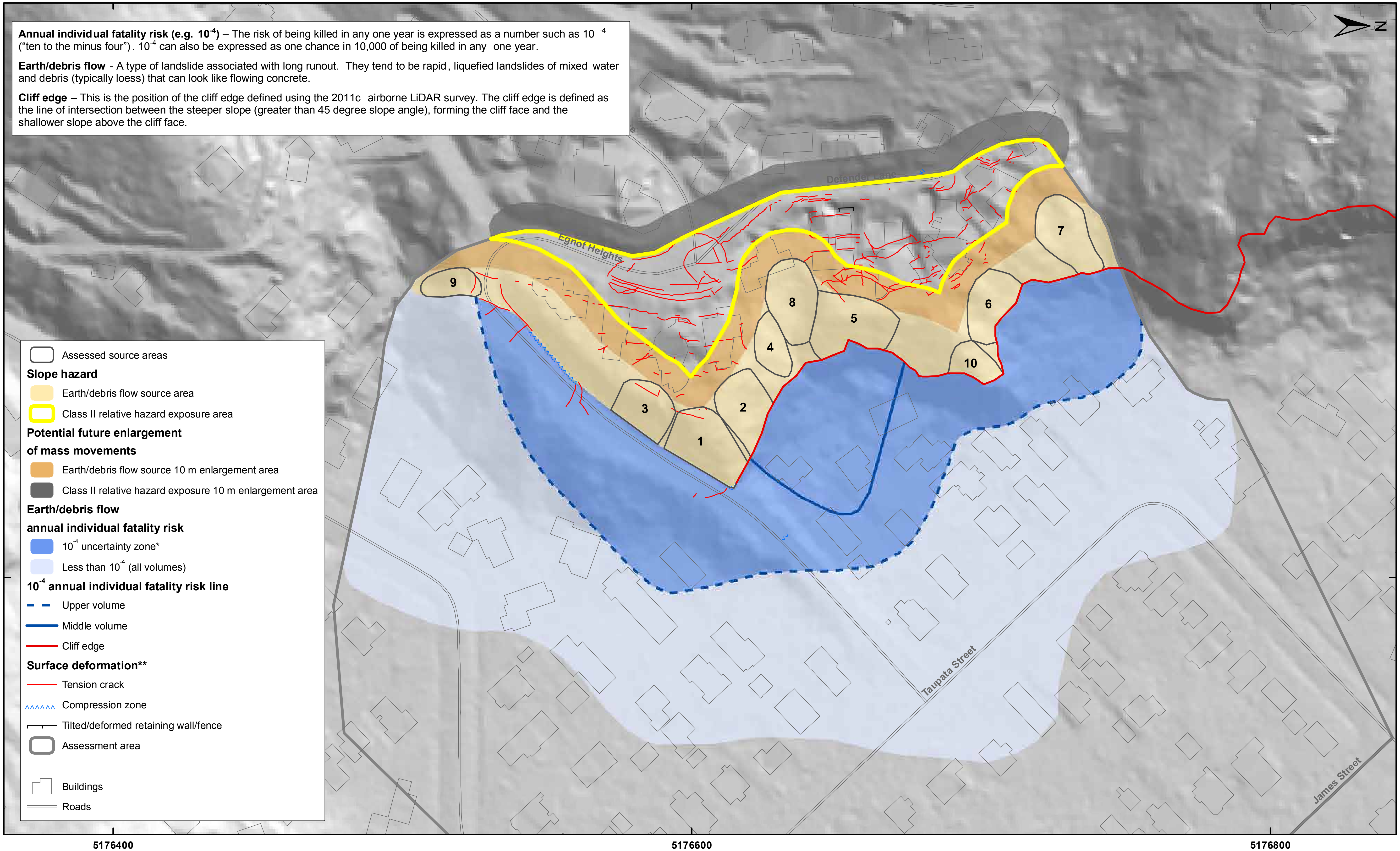
5.3 SLUMPING AND CRACKING

The area of slope between the earth/debris flow source 10 m enlargement area and the mapped extent of cracking that was highlighted in the Stage 1 report as a Class I area, has now been re-assessed as being in a Class II area. A Class II area is defined in the Stage 1 report (Massey et al., 2013) as:

- Coherent slides and slumps of predominantly loess with associated cumulative inferred displacement of the mass of greater than 0.3 m, where dwellings and critical infrastructure is present within the moving mass. It is possible that renewed movement may severely impact critical infrastructure and dwellings. The level of disruption to critical infrastructure and dwellings is likely to be a function of where they are within the feature. The most hazardous places are the mainly extensional and compressional areas. Given the magnitudes of displacement it is unlikely that damage to dwellings would pose an immediate life risk to their occupants.

A 10 m wide area has been added to the inferred boundary of the Class II hazard exposure area, where the area of slumping and cracking could potentially in the future enlarge in an up-slope or lateral direction beyond the currently recognised boundary. This has been termed a "Class II relative hazard exposure 10 m enlargement area".

The estimated magnitudes of displacement of the Class II area in a future earthquake are dependent on the nature of earthquake shaking and the critical yield acceleration of the slope, which is a function of the bulk shear strength of the loess, at the time of the earthquake. Estimated permanent coseismic displacements of cross-sections 3 and 5 are shown for various values of peak free field ground accelerations (A_{FF}) in Table 22, for the area behind the slope crest as shown in Figure 37 and Figure 38.



SCALE BAR: 0 50 100 m

EXPLANATION:

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

DRW: BL

CHK: FDP, CM

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EARTH/DEBRIS FLOW ANNUAL INDIVIDUAL FATALITY RISK

Defender Lane Christchurch

FIGURE 45

FINAL

REPORT: CR2014/67

DATE: June 2014

5.3.1 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 and 100 years for the triggering event. The results are plotted for in Figure 46. The risks adopting the 200 year triggering event frequency are all less than 10^{-4} and cannot therefore be plotted in Figure 46.

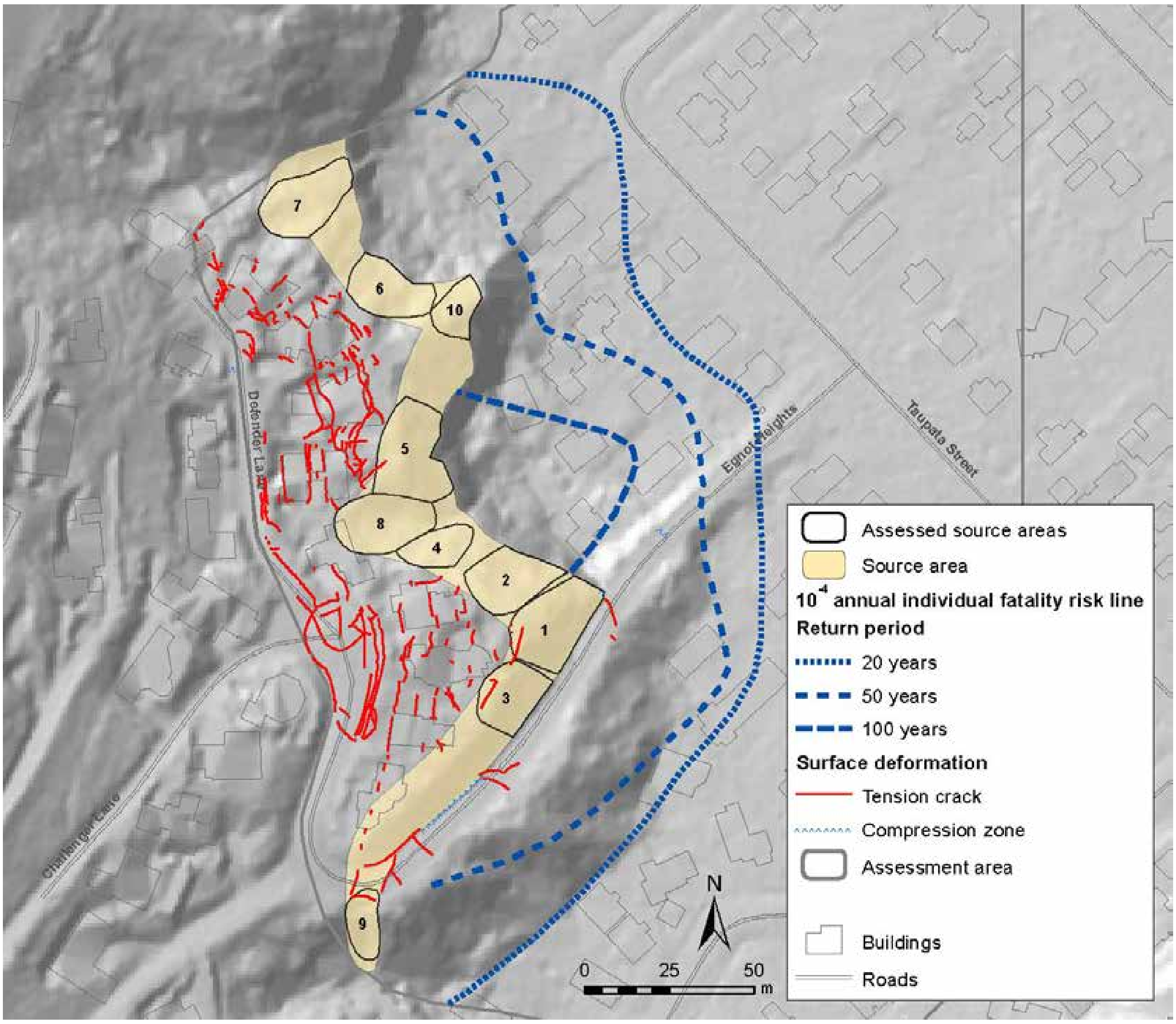

Figure 46 Sensitivity of the risk estimates, upper volume estimates, for triggering event return periods of 20, 50, 100 and 200 years.

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 20–100 year return periods, the changing risk has little impact on the numbers of dwellings within the 10^{-4} annual individual fatality risk contour. Therefore the 50-year return period adopted for the risk estimates shown in Figure 45 is considered reasonable.

6.0 DISCUSSION

6.1 RISK ASSESSMENT

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

Earth/debris flow hazards:

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

Cliff-collapse hazards:

1. Cliff collapses are a credible hazard affecting the site and could occur from anywhere along the cliff.
2. The risks associated with cliff-collapse hazards (debris avalanches and cliff-top recession) were previously assessed in Massey et al., 2012), and have not been reassessed in this report.

6.1.1 Annual frequency of the event

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

The area has already undergone more than 0.3 m of permanent slope displacement, during the 2010/11 Canterbury earthquakes and this displacement may have reduced the shear strength of critical materials in the slope, making the slope more susceptible to future earthquakes. In addition, there may be an unknown amount of further displacement that the slopes may be able to undergo before failing catastrophically (i.e., where the magnitude of displacement causes the failure mass to break down to become a mobile failure). At the current time there is no practical means for estimating the numerical value of the “degraded strength”, of the slope.

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

- Loess shear strength is critically dependent on water content, and the volcanic colluvium present at the site appears to be, in part, reworked loess.
- At high water contents the range of shear strengths, derived from ring-shear testing, could feasibly represent the strength of the loess in the slope.

- Under such conditions, results from the numerical slope stability back-analysis indicate that failure of the slope is likely.
- It is more likely that failure would occur through specific zones within the loess, e.g., through more permeable zones where water contents are likely to increase more readily, or above permeability boundaries such as soil fragipans.
- Pore pressure above rockhead and within the colluvium and loess, as well as pore water pressures within the open cracks would also reduce the slope factor of safety.
- Given the now-cracked nature of the slopes, it is feasible that water contents of the loess and colluvium could increase in response to rainfall, as water can more readily enter the slope via the cracks and broken services.
- Recent, historic and pre-historic landslides have occurred at the site. The debris fans at the slope toe suggest a debris accumulation rate of about 5.4 m^3/year (calculated from 19,300 m^3 of debris overlying a surface about 3,600 years old). Based on the assumption that the debris fans were derived from landslide debris, it can be inferred that:
  - Assuming an accumulation rate of 5.4 m^3/year, the return period of the mean lower, middle and upper earth/debris flow source volumes, estimated in this report, would be 24, 57 and 157 years respectively, corresponding to annual frequencies of 0.04, 0.018 and 0.006. This assumes that all of the accumulated debris comes from a single source in an event. These are irrespective of whether or not the earthquake–induced cracks have increased the vulnerability of the slope to landslides.
  - It is not known whether the accumulated debris forming the fans is from one large event or from the coalescing of debris from many smaller events. The volume of loess in the fan may also be a lower estimate of the total volume deposited, as loess is easily erodible.
- Based on the field evidence it is reasonable to assume that the return period of the event that could trigger failure of the estimated source volumes is somewhere between >10 years and 200 years; 10 years being the return period of the March 2014 rain event and 200 years being the return period of the mean upper source volume based on the pre-historic debris accumulation rates.

6.2 RISK ASSESSMENT SENSITIVITIES AND UNCERTAINTIES

In this section the sensitivity of the risk model to key uncertainties and reliability of the assessments is identified.

The sensitivity of the estimated risk has been assessed to the following changes:

1. Changes to the volumes of earth/debris flows that could be triggered in future events. This was done using the three volume ranges to account for any variation in the likely source volumes. The three volume ranges also takes into account the variability of the debris runout velocities and inundation heights, as large volume s of debris tend to travel further down slope at higher velocities.
2. Changes to the annual frequency of the event that could trigger failure of the sources: Risk models were run adopting event annual frequencies of 0.05, 0.02, 0.01, 0.005 and 0.002, corresponding to return periods of 20, 50, 100, 200 and 500 years respectively.

a. Results from the assessment show that there is little change between the risk results adopting the 50-year and 100-year return periods. The risk at the dwellings in the 10^{-4} annual individual fatality risk uncertainty zone decreases by a factor of about two (for a return period from 50 to 100 years).

b. 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, reducing the numbers of dwellings within the zone. The risk at the dwellings in the 10^{-4} annual individual fatality risk uncertainty zone decreases by a factor of about four (for a return period from 50 to 200 years).

c. For return periods of less than 50 years the 10^{-4} annual individual fatality risk uncertainty zone increases and the risk at the dwellings increases by a factor of 2 to 3 (for a return period from 50 to 20 years).

3. Changes to the probability of failure of each source area: Risk models were run assuming that each source area has a 100% probability of failure in 50 years rather than a 10% probability of failure in 50-years. As expected, the results from this assessment show that the risk increases by approximately an order of magnitude. This would result in several more dwellings being included in the 10^{-4} annual individual fatality risk uncertainty zone.

4. Figure 47 shows the probability of depth as a function of debris height and velocity. The risk assessment may not adequately take into account the sheltering effect of buildings. Variable vulnerabilities have been adopted linked to debris velocity. However, the vulnerability of a person in a dwelling is related to the nature of the structure, for which there is no data available for use in the risk assessment.

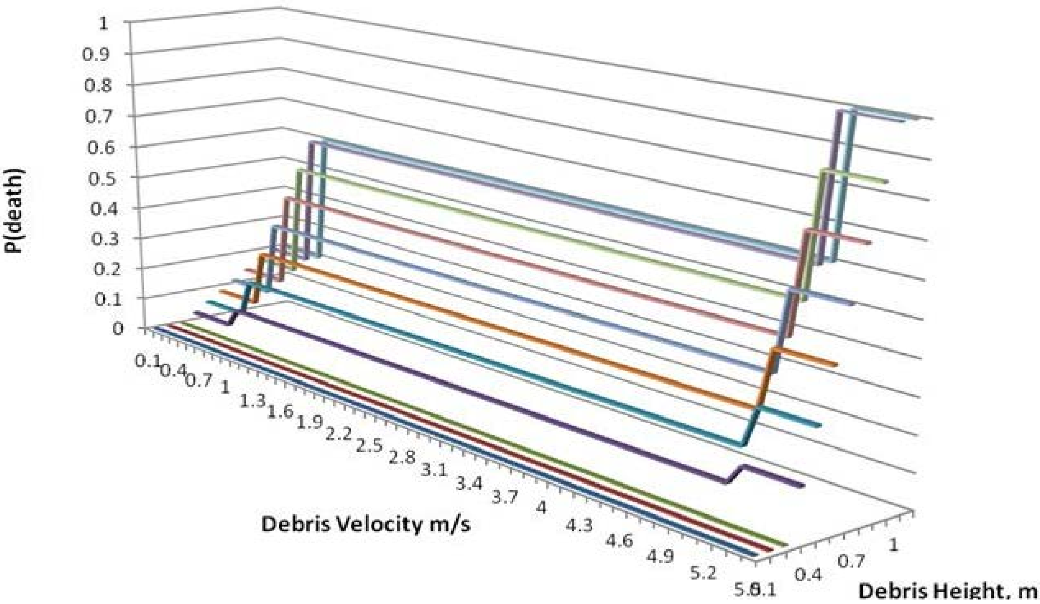

Figure 47 Probability of death as a function of debris velocity and height.

The results (Figure 45 and Figure 46) 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 to give more than an order of magnitude uncertainty, in either direction, on the risk estimates.

It is therefore possible that the dwellings within the 10^{-4} uncertainty zone are at levels of risk that are less than 10^{-5}, but conversely could be as high as 10^{-3}.

6.2.1 How reliable are the results?

Potentially significant uncertainties noted and their likely implications for risk are summarised in Table 23. The sensitivity results are reported in Table 23 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 23 Uncertainties and their implications for risk.

Issue	Direction and scale of uncertainty	Implications for risk
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 and the 50-year and 20-year return periods. But larger uncertainty between the 50 year and 200 year plus return periods.	Longer term risk is potentially 2–4 times lower in the distal runout zone. Shorter term risk is potentially 2–3 times greater in the distal runout zone.
b. Volume of debris produced by a source area in an event.	Large uncertainty in either direction.	About an order of magnitude uncertainty (factor of 10) in both directions.
c. Volume of debris passing a given distance down the slope.	Derived from RAMMS modelling. Uncertainties are difficult to estimate, but are taken into account by assuming three different failure volumes per source	Approximately a factor of less than 5 in each direction.
d. Groundwater distributions, water content of the loess and transient water pressures in cracks.	These are largely unknown (or undefined) and have a major impact on the stability of the loess. Potentially quite large and mainly in the upward direction.	Unknown
e. Changes to the probability of failure of each source area from 10% in 50 years to 100% in 50 years.	Largest uncertainty in the upward direction if many source areas were to fail during the same event.	About an order of magnitude uncertainty (factor of 10) in the upward direction.
f. Future earthquake ground motions and the response of the slope.	Large uncertainty and mainly in the upward direction. Displacement magnitudes estimated in this report use the 2010/11 earthquake time histories, future earthquakes could be longer duration and/or lower amplitude, and occur in winter when water contents are high.	Unknown
g. Changing the assumed debris height where probability of inundation = 0 from 0.3 m–0.5 m and 0.1 m.	Small uncertainty in either direction.	Would change modestly by a factor of about 1.3 in either direction.
h. 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 1.5.
i. 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 potential for volumes ranging from many tens to several hundreds of cubic metres of earth/debris flows (source areas 1–10) of mixed loess and colluvium, which are in addition to the cliff-collapse failures previously assessed (Massey et al., 2012).
2. The most likely triggers for the assessed earth/debris flows sources are prolonged heavy rainfall and strong earthquake shaking (if ground conditions were wet).
3. The frequency of earth/debris flow events from these sources is difficult to estimate. For the assessment, event annual frequencies of once every 20 years to once every 200 years have been assessed.
4. It should be noted that material strengths – and therefore the slope factors of safety – are likely to reduce with time, and the occurrence of future earthquakes. Therefore a conservative approach is warranted to account for this long-term change.

7.2 RISK

1. The effect on annual individual fatality risk for residents of the newly-identified sources is to increase the risk levels at dwellings already identified as at particularly high levels of risk from cliff collapse, and to include a several more dwellings that could be at possibly unacceptable levels of risk.
2. The main hazards affecting these dwellings is likely to be a combination of cracking and undercutting (the Class II area and the source areas) as the ground moves beneath the dwelling, and in the runout zone, the impact from debris coming from further upslope.
3. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings in the assessed source area, from future earthquakes is still relatively high. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.2–2 m, depending on the strength of the loess at the time of the earthquake. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the national seismic hazard model.

7.3 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. There appears to be reasonable scope for engineering measures to stabilise the slopes (e.g., by removal of loess and installation of drainage measures). Such works would need to be evaluated, designed and implemented by a suitably qualified engineering consultant.

8.0 RECOMMENDATIONS

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

8.1 POLICY AND PLANNING

1. Decide what levels of life risk to dwelling occupants 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

1. 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 sewer main currently runs through the mass movement at the crest of the assessed earth/debris flow source areas, 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.

2. Reduce water ingress into the slopes, where safe and practicable to do so, by:
    a. Identifying and relocating any 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 surface drainage. In particular, the sewer main currently runs through the mass movement at the crest of the assessed earth/debris flow source areas, and should if possible be relocated away from this area; and
    b. Filling in the identified accessible cracks near the cliff crest and provide an impermeable surface cover to minimise water ingress.

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.

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 significant 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.

A1.1.2 Static slope stability

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

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

Models were run based on geological cross-sections 1–3, 4 and 5. The critical slide surface was determined based on the lowest calculated factor of safety. Sensitivity analyses were run assuming a range of geotechnical material strength parameters based on the estimates of their strength to test model sensitivity. These were derived from in-house laboratory testing on samples of materials taken from the site, and samples of similar materials taken from other sites in the Port Hills and published information on similar materials. Strength parameters were also assessed by back-analysis in the limit equilibrium and dynamic analyses.

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

A1.1.3 Dynamic stability assessment (decoupled method)

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

The dynamic (earthquake) stability of the slope was assessed with reference to procedures outlined in Eurocode 8 (EN-1998-5, 2004) Part 5. For the Defender Lane 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, 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, 2013). The shear wave velocity of the loess was estimated from tests carried out by Tonkin and Taylor (2012b).

2. Displacement assessment steps:

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

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

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

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

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

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

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

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

A1.1.4 Forecasting permanent slope displacements

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

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

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

A1.1.5 Estimation of slope failure volumes

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

Three failure volumes (upper, middle and lower) were estimated for each potential source area to represent a range of source volumes. The credibility of these potential failure volumes was evaluated by comparing them against: 1) the volumes of relict failures recognised in the geomorphology near the site and elsewhere in the Port Hills; 2) historically recorded failures; and 3) the volumes of material lost from the 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 between 1870 and 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).

A1.1.6 Debris runout modelling

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

1. Empirical fahrboeschung method:
    a. The fahrboeschung model is based on a relationship between topographical factors and the measured lengths of runout of debris (Corominas, 1996). The fahrboeschung[1] (often referred to as the "travel angle") method (Keylock and Domaas, 1999) uses the slope of a straight line between the top of the source area (the crown) and the furthest point of travel of the debris. The analysis adopts the slope crest as the crown of each potential source area.
    b. The volume of earth/debris passing a given location within the assessment area is based on an empirical relationship established from a compilation of runout distances from published international and local (in the Port Hills) earth/debris flows. For earth/debris flows, which tend to be very fluid ("soupy" to "porridge-like" in consistency), the empirical relationship is based on a data set of over 700 earth/debris flows from New Zealand (including the Port Hills and Banks Peninsular) and overseas, compiled by Massey and Carey (2012).
2. Numerical methods:
    a. Numerical modelling of landslide runout was carried out using the RAMMS® debris-flow software. This software, developed by the Snow and Avalanche Research Institute based in Davos, Switzerland, simulates the runout of debris flows and snow and rock avalanches across complex terrain. The module is used worldwide for landslide runout analysis and uses a two-parameter Voellmy rheological model to describe the frictional behaviour of the debris (RAMMS, 2011). The physical model of RAMMS Debris Flow uses the Voellmy friction law. This model divides the frictional resistance into two parts: a dry-Coulomb type friction (coefficient μ) that scales with the normal stress and a velocity-squared drag or viscous-turbulent friction (coefficient xi). However, to the best of our knowledge there is no direct physical means of deriving these parameters from field measurements, other than back-analysis of past earth/debris flows in similar materials and terrain.
    b. RAMMS software takes into account the slope geometry of the site when modelling debris runout. The model was calibrated by "back-analysing" the runout of four Port Hills and Banks Peninsula earth/debris flows and the modelled parameters optimised to obtain a good correlation between the modelled versus actual runout.
    c. 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.
    d. The RAMMS modelling uses a "bare earth" topographic model, and so the runout impedance of buildings and larger trees was not considered.

[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.

A1.2 RISK ASSESSMENT

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

A1.2.1 Fatality risk for dwelling occupants

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

1. Divide the entire assessment area into a series of 1 m by 1 m grid cells.
2. Consider the possible range of triggering events from non-earthquake triggers (mainly rain). The annual frequency of the event (rainfall) that could trigger failure of any of the identified source areas (1–10) is difficult to estimate. The variation of risk across the slope has, therefore been assessed using a range of event frequencies and earth/debris flow source volumes:
   - It has been assumed that the return period of the event (mainly rainfall) that could trigger failure of the assessed source area is unlikely to be less than 10–20 years (event annual frequency of 0.1–0.05), as the rainfall recorded in the Port Hills 3–5 March 2014 (which did not cause substantial failures), was equivalent to a 10–20 year return period rain event.
   - Event annual frequencies ($P_{(H)}$) of 0.05, 0.02, 0.01, and 0.005 corresponding to return periods of 20, 50, 100 and 200 years, were used for the assessment.
   - The main source area was characterised based on the geological evidence and assessment collected to date, from which estimates of the likely failure volumes were made.
   - Three scenarios were considered based on: 1) lower; 2) middle; and 3) upper estimates of the source volume.
   - Each source volume scenario was assessed as having an equal probability of failure in a given event, of a given annual frequency.
3. For each representative event, and for each scenario, estimate:
   a. The frequency of the event and the volume of debris, for a given source scenario, produced in that event ($P_{(H)}$).
   b. The height of the debris reaching/passing a given grid cell and the probability of a person at that location being inundated (buried) by the debris ($P_{(S:H)}$). This is discussed in a later section.
   c. The probability that a person is present at a given location in their dwelling as the debris moves through it ($P_{(T:S)}$).
   d. The probability that a person is killed if present and inundated by debris ($V_{(D:T)}$). In some risk assessments the vulnerability has been linked to landslide intensity, which is a combination of the landslide velocity and the volume of debris (e.g., Du et al., 2013). For this assessment a variable vulnerability has been adopted based on the velocity of the debris.

4. Combine 3(a)–(d) for each source area scenario to estimate the annual individual fatality risk at different locations below the slope at different event annual frequencies.
5. These values were then modelled using ArcGIS®. ArcGIS is used to interpolate between the risk calculated at given grid cells so as to produce contours of equal risk. A single contour was presented for each scenario (lower, middle and upper source volumes) for each event annual frequency, representing the estimated risk of 10^{-4} (ten to the minus four, or 1 chance in 10,000 of dying per year).
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.

Probability of inundation

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

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

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

Probability of a person being present

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

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

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

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

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

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

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

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

Equation 3

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

Probability of the person being killed if inundated by debris

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

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

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

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

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

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

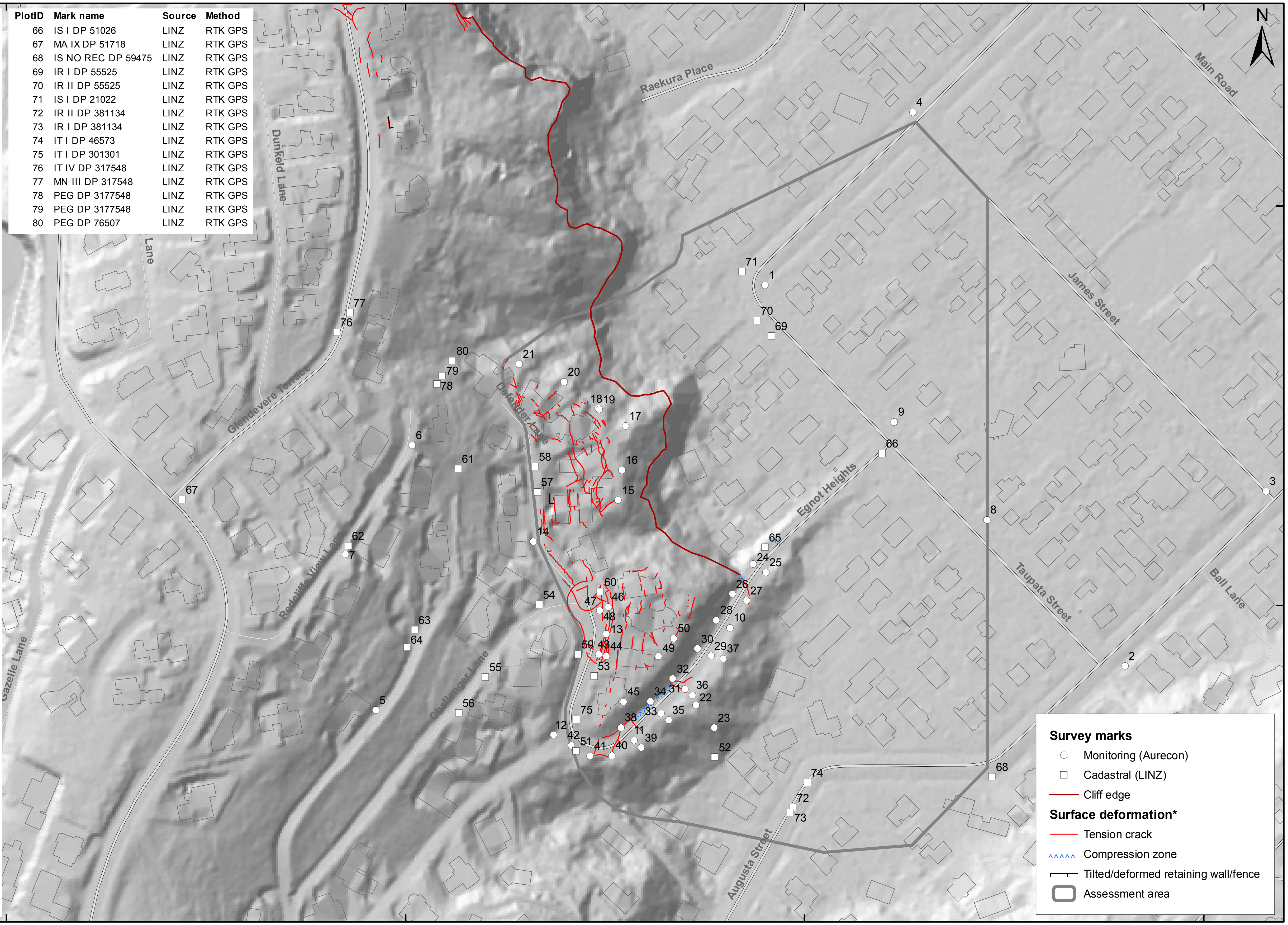
For the risk assessment, the velocity ranges given in Australian Geomechanics Society (2007) were used, and these were linked to the vulnerabilities reported by Finlay et al. (1999) and Du et al. (2013), as no specific information on how Zealand buildings perform when impacted by debris was available (Table A1.1). 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	Building collapse or building inundated with debris, death almost certain.	1
0.5– 5	Inundated building with debris, but person not buried.	0.6
0.05–0.5	Building is hit but the person not buried and escape possible.	0.2
<0.05	Debris strikes building only.	0

A2 APPENDIX 2: RESULTS FROM THE SURVEY OF CADASTRAL SURVEY MARKS

PlotID	Mark name	Source	Method
1	DF1	Aurecon	TS
2	DF2	Aurecon	TS
3	DF3	Aurecon	TS
4	DF4	Aurecon	TS
5	DF5	Aurecon	TS
6	DF6	Aurecon	TS
7	DF7	Aurecon	TS
8	DF10	Aurecon	TS
9	DF11	Aurecon	TS
10	DF12	Aurecon	TS
11	DF13	Aurecon	TS
12	DF14	Aurecon	TS
13	DF15	Aurecon	TS
14	DF16	Aurecon	TS
15	DF100	Aurecon	TS
16	DF101	Aurecon	TS
17	DF102	Aurecon	TS
18	DF103	Aurecon	TS
19	DF104	Aurecon	TS
20	DF105	Aurecon	TS
21	DF106	Aurecon	TS
22	DF107	Aurecon	TS
23	DF108	Aurecon	TS
24	DF200	Aurecon	TS
25	DF201	Aurecon	TS
26	DF202	Aurecon	TS
27	DF203	Aurecon	TS
28	DF204	Aurecon	TS
29	DF205	Aurecon	TS
30	DF206	Aurecon	TS
31	DF207	Aurecon	TS
32	DF208	Aurecon	TS
33	DF209	Aurecon	TS
34	DF210	Aurecon	TS
35	DF211	Aurecon	TS
36	DF212	Aurecon	TS
37	DF213	Aurecon	TS
38	DF300	Aurecon	TS
39	DF301	Aurecon	TS
40	DF302	Aurecon	TS
41	DF303	Aurecon	TS
42	DF304	Aurecon	TS
43	DF400	Aurecon	TS
44	DF401	Aurecon	TS
45	DF402	Aurecon	TS
46	DF403	Aurecon	TS
47	DF404	Aurecon	TS
48	DF405	Aurecon	TS
49	DF406	Aurecon	TS
50	DF407	Aurecon	TS
51	TB V DP 70774	LINZ	RTK GPS
52	PEG IIIB DP 381134	LINZ	RTK GPS
53	PEG DP 70774	LINZ	RTK GPS
54	BD DP 70774	LINZ	RTK GPS
55	PEG DP 70774	LINZ	RTK GPS
56	PEG DP 301301	LINZ	RTK GPS
57	PEG DP 70774	LINZ	RTK GPS
58	PEG DP 70774	LINZ	RTK GPS
59	TB VI DP 70774	LINZ	RTK GPS
60	TB VII DP 70774	LINZ	RTK GPS
61	PEG DP 70774	LINZ	RTK GPS
62	TB II DP 76507	LINZ	RTK GPS
63	PEG DP 301301	LINZ	RTK GPS
64	PEG DP 301301	LINZ	RTK GPS
65	TB II DP 70774	LINZ	RTK GPS
66	IS I DP 51026	LINZ	RTK GPS
67	MA IX DP 51718	LINZ	RTK GPS
68	IS NO REC DP 59475	LINZ	RTK GPS
69	IR I DP 55525	LINZ	RTK GPS
70	IR II DP 55525	LINZ	RTK GPS
71	IS I DP 21022	LINZ	RTK GPS
72	IR II DP 381134	LINZ	RTK GPS
73	IR I DP 381134	LINZ	RTK GPS
74	IT I DP 46573	LINZ	RTK GPS
75	IT I DP 301301	LINZ	RTK GPS
76	IT IV DP 317548	LINZ	RTK GPS
77	MN III DP 317548	LINZ	RTK GPS
78	PEG DP 3177548	LINZ	RTK GPS
79	PEG DP 3177548	LINZ	RTK GPS
80	PEG DP 76507	LINZ	RTK GPS



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

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

DRW: BL

CHK: GA, CM, FDP

GNS SCIENCE TE PŪ AO

SURVEY MARKS Index Map

Defender Lane Christchurch

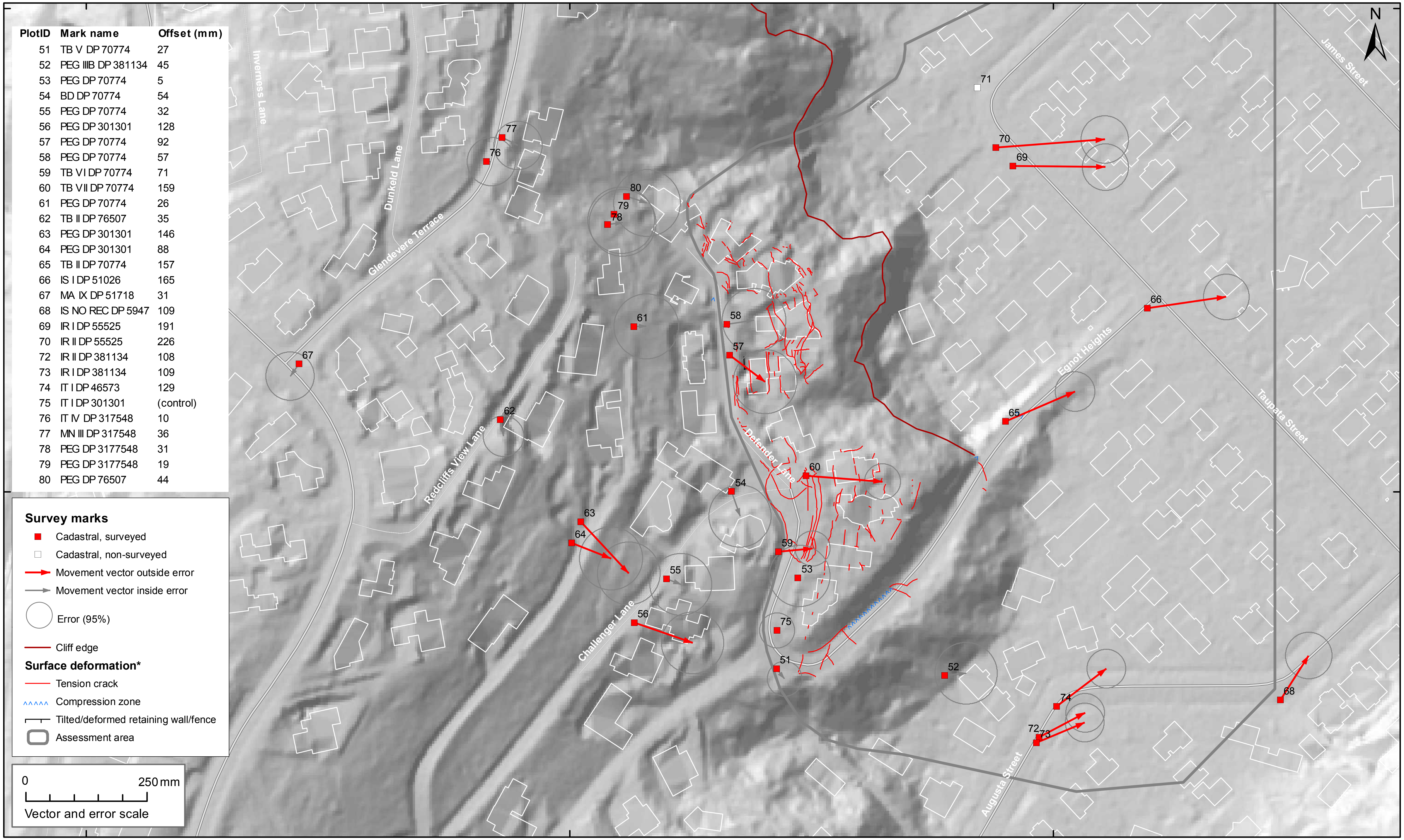
APPENDIX 2

Map 1

FINAL

REPORT: CR2014/67

DATE: June 2014



PlotID	Mark name	Offset (mm)
51	TB V DP 70774	27
52	PEG IIIB DP 381134	45
53	PEG DP 70774	5
54	BD DP 70774	54
55	PEG DP 70774	32
56	PEG DP 301301	128
57	PEG DP 70774	92
58	PEG DP 70774	57
59	TB VI DP 70774	71
60	TB VII DP 70774	159
61	PEG DP 70774	26
62	TB II DP 76507	35
63	PEG DP 301301	146
64	PEG DP 301301	88
65	TB II DP 70774	157
66	IS I DP 51026	165
67	MA IX DP 51718	31
68	IS NO REC DP 5947	109
69	IR I DP 55525	191
70	IR II DP 55525	226
72	IR II DP 381134	108
73	IR I DP 381134	109
74	IT I DP 46573	129
75	IT I DP 301301	(control)
76	IT IV DP 317548	10
77	MN III DP 317548	36
78	PEG DP 3177548	31
79	PEG DP 3177548	19
80	PEG DP 76507	44

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

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

DRW: BL

CHK: GA, CM, FDP

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

Defender Lane
Christchurch

APPENDIX 2

Map 2

FINAL

REPORT: CR2014/67

DATE: June 2014

PlotID	Mark name	Offset (mm)
1	DF1	11
4	DF4	12
5	DF5	11
6	DF6	(control)
7	DF7	5
15	DF100	14
16	DF101	14
17	DF102	12
18	DF103	12
19	DF104	21
20	DF105	15
21	DF106	17

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

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

DRW: BL

CHK: GA, CM, FDP

GNS SCIENCE TE PŪ AO

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

Defender Lane
Christchurch

APPENDIX 2

Map 3

FINAL

REPORT: CR2014/67

DATE: June 2014

PlotID	Mark name	Rate (mm/yr)	StartDate	EndDate
1	DF1	5	18/07/2011	26/06/2013
4	DF4	6	18/07/2011	26/06/2013
5	DF5	19	18/07/2011	26/06/2013
6	DF6	(control)	18/07/2011	26/06/2013
7	DF7	4	18/07/2011	26/06/2013
15	DF100	10	18/07/2011	26/06/2013
16	DF101	1	18/07/2011	26/06/2013
17	DF102	2	18/07/2011	26/06/2013
18	DF103	2	18/07/2011	26/06/2013
19	DF104	5	18/07/2011	26/06/2013
20	DF105	9	18/07/2011	26/06/2013
21	DF106	4	18/07/2011	26/06/2013
8	DF10	3	14/09/2012	26/06/2013
9	DF11	2	14/09/2012	26/06/2013
10	DF12	1	14/09/2012	26/06/2013
11	DF13	1	14/09/2012	26/06/2013
12	DF14	2	14/09/2012	26/06/2013
14	DF16	9	14/09/2012	26/06/2013
24	DF200	2	14/09/2012	26/06/2013
25	DF201	4	14/09/2012	26/06/2013
26	DF202	2	14/09/2012	26/06/2013
27	DF203	2	14/09/2012	26/06/2013
28	DF204	2	14/09/2012	26/06/2013
29	DF205	6	14/09/2012	26/06/2013
30	DF206	5	14/09/2012	26/06/2013
31	DF207	2	14/09/2012	26/06/2013
32	DF208	2	14/09/2012	26/06/2013
33	DF209	3	14/09/2012	26/06/2013
34	DF210	1	14/09/2012	26/06/2013
35	DF211	2	14/09/2012	26/06/2013
36	DF212	3	14/09/2012	26/06/2013
37	DF213	1	14/09/2012	26/06/2013
38	DF300	2	14/09/2012	26/06/2013
39	DF301	2	14/09/2012	26/06/2013
40	DF302	2	14/09/2012	26/06/2013
41	DF303	3	14/09/2012	26/06/2013
42	DF304	3	14/09/2012	26/06/2013
43	DF400	3	14/09/2012	26/06/2013
44	DF401	2	14/09/2012	26/06/2013
45	DF402	7	14/09/2012	26/06/2013
46	DF403	3	14/09/2012	26/06/2013
47	DF404	3	14/09/2012	26/06/2013
48	DF405	5	14/09/2012	26/06/2013
49	DF406	7	14/09/2012	26/06/2013
50	DF407	5	14/09/2012	26/06/2013

SCALE BAR: 0 50 100 m

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

DRW: BL

CHK: GA, CM, FDP

GNS SCIENCE TE PŪ AO

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

Defender Lane
Christchurch

APPENDIX 2

Map 4

FINAL

REPORT: CR2014/67

DATE: June 2014

A3 APPENDIX 3: 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.

A4 APPENDIX 4: RESULTS FROM THE TWO-DIMENSIONAL SITE RESPONSE ASSESSMENT FOR CROSS-SECTIONS 3 AND 5

The results from the two-dimensional site response modelling are shown for cross-sections 3 and 5. The maximum acceleration (A_{MAX}) at the cliff crest derived from the modelling of each synthetic earthquake time history has been plotted in Figure A4.1. Each point on the graph represents the response of the slope crest to a given synthetic free field rock outcrop earthquake input motion (Table A4.1).

The fundamental frequency of the slope varies from 4 to 7.5 Hz based on the equation in Bray (2007), where frequency = 1/(4 x H/V_S), and H = slope height of 30 m, and V_S = average shear wave velocity for the slope of between 480 and 900 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 0.8–0.9.

Results from the seismic response assessment suggest that the peak ground acceleration amplification factors (S_T) for Defender Lane (all data for cross-sections 3 and 5) are about 2.8 (±0.1) – errors at one standard deviation for horizontal motions, and 3.2 (±0.1) for vertical motions – errors at one standard deviation (Figure A4.1).

Table A4.1 Results from the two-dimensional site response assessment for cross-sections 3 and 5 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) cross-section 3 / cross-section 5**	**Maximum PGA (vertical) at slope crest – A_{MAX} (g) cross-section 3 / cross-section 5**
22 February	0.88	0.66	2.40 / 2.50	1.90 / 2.20
16 April	0.05	0.02	0.11 / 0.11	0.06 / 0.07
13 June	0.38	0.27	1.00 / 0.90	1.10 / 0.85
23 December	0.16	0.13	0.55 / 0.55	0.45 / 0.35

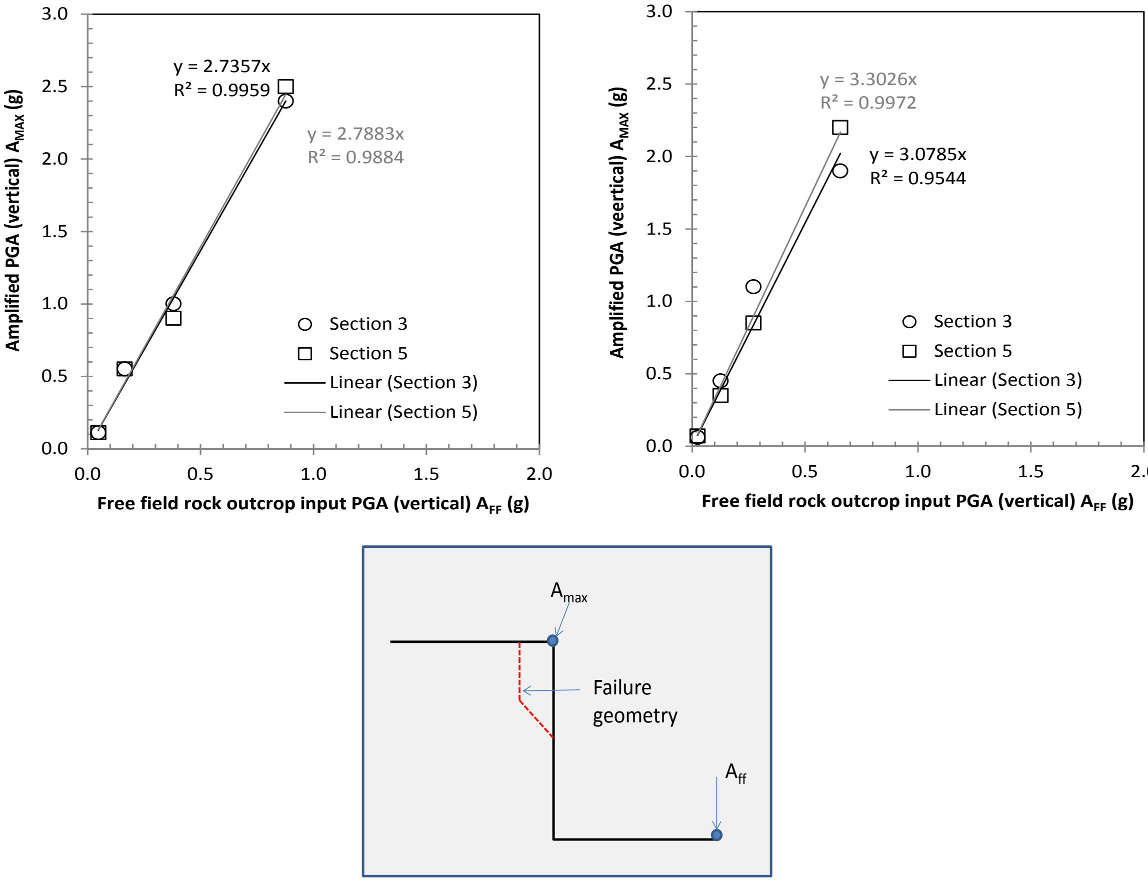


Figure A4.1 Amplification relationship between the synthetic free field rock outcrop input motions (A_{FF}) and the modelled cliff crest maximum accelerations (A_{MAX}) for cross-sections 3 and 5. A schematic diagram showing the locations of the various recorded accelerations is shown.

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 horizontal amplification factor (S_T) is typically in the order of about 2.8 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 A4.2. The gradient of the linear fit (all data, cross-sections 3 and 5) is 0.86 (±0.03) – errors at one standard deviation, with a gradient of 0.92 being the mean plus two standard deviations. The relationship between horizontal and vertical peak ground accelerations appears linear to a horizontal peak ground acceleration of 1 g, but then appears to become non-linear at higher horizontal peak ground accelerations.

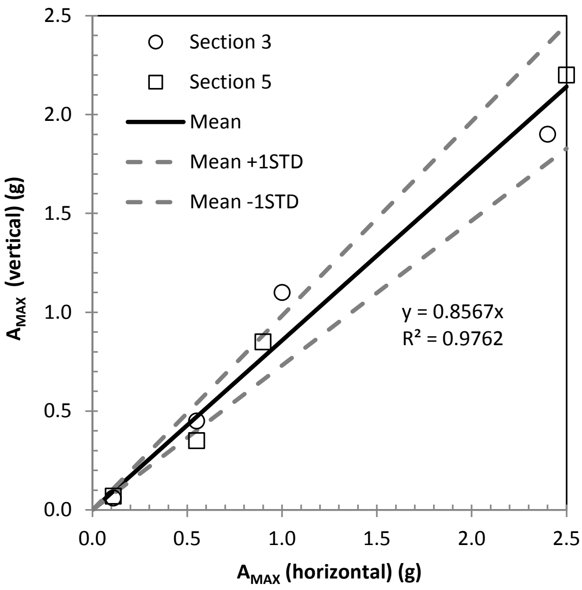


Figure A4.2 Relationship between the modelled horizontal and vertical maximum accelerations modelled at the slope crest (A_{MAX}) for cross-sections 3 and 5, using the synthetic free field rock outcrop motions for the Defender lane site by Holden et al. (2014) as inputs to the assessment. The mean and standard deviation trend lines are fitted for A_{MAX} all data. Errors are shown as the mean ± one standard deviation (1 STD).

Cross-sections 3 and 5 comprise about 10 m of loess overlying mixed basalt lava and breccia, where the mean shear wave velocities of the materials change from 600–1,200 m/s in the basalt breccia and lava, to 200–400 m/s in the loess (Figure A4.3). The results suggest that the impedance contrasts between the materials contribute most to the amplification of shaking, but that the peak horizontal accelerations (for all modelled earthquakes) concentrate around the convex break in slope, defined as A_{MAX}.

These results are similar to those reported by others (e.g., Del Gaudio and Wasowski, 2010), where material impedance contrasts have been shown to have a significant effect on the amplification of shaking. In experimental data, as the slope displaces during an earthquake, the slide surface can “base isolate” the mass above, resulting in lower levels of shaking and displacement. Therefore, the reported amplification factors are near the upper bound of published topographic amplification factors. Assessment of this is outside the scope of this report.

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:

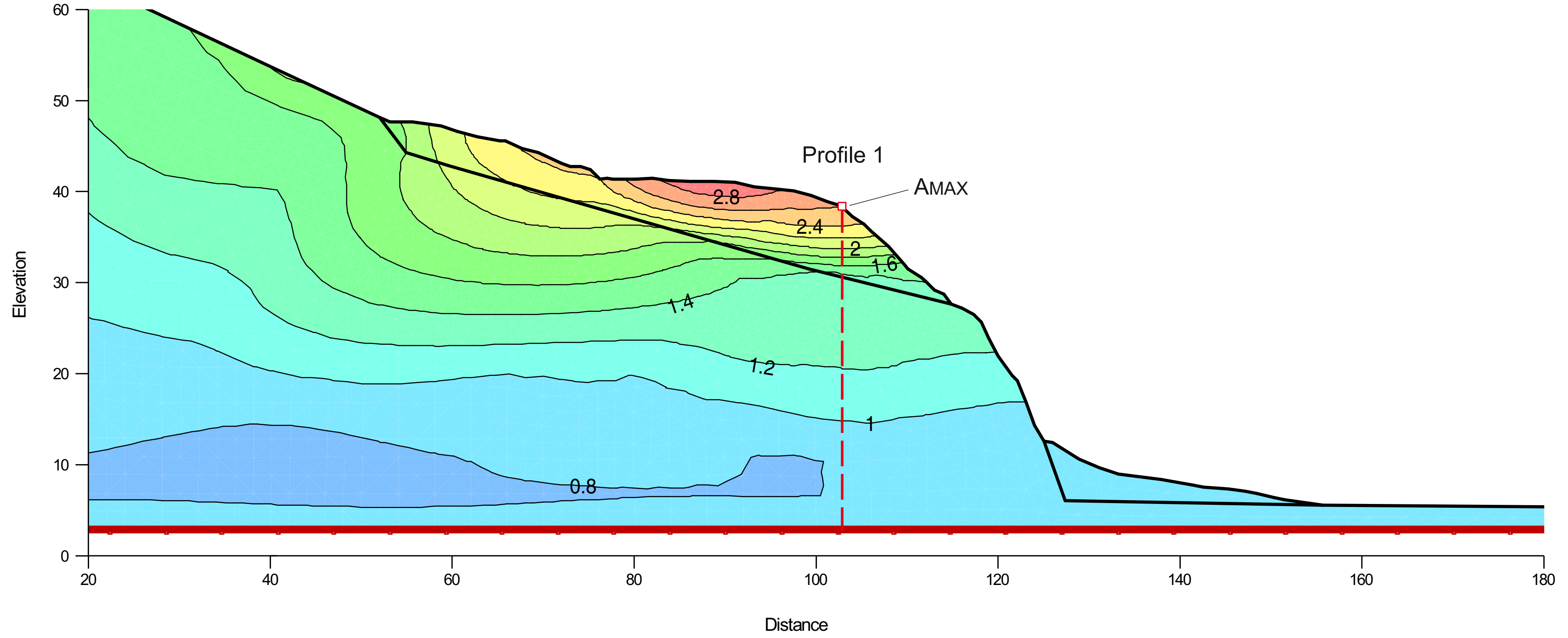
1. Isolated cliffs and slopes. A value $S_T \geq 1.2$ should be used for sites near the top edge;
2. Ridges with crest width significantly less than the base width. A value $S_T \geq 1.4$ should be used near the top of the slopes for average slope angles greater than 30° and a value $S_T > 1.2$ should 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%;

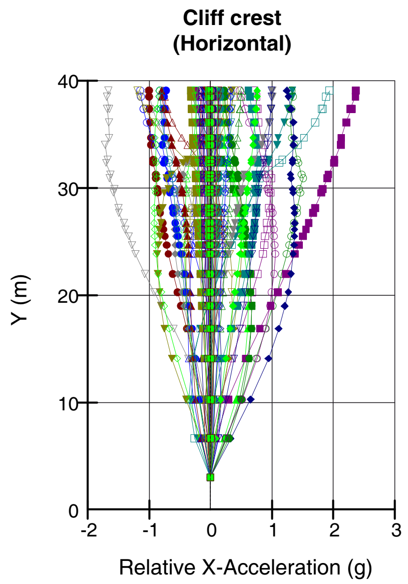
4. Spatial variation of amplification factor. The value of S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base.

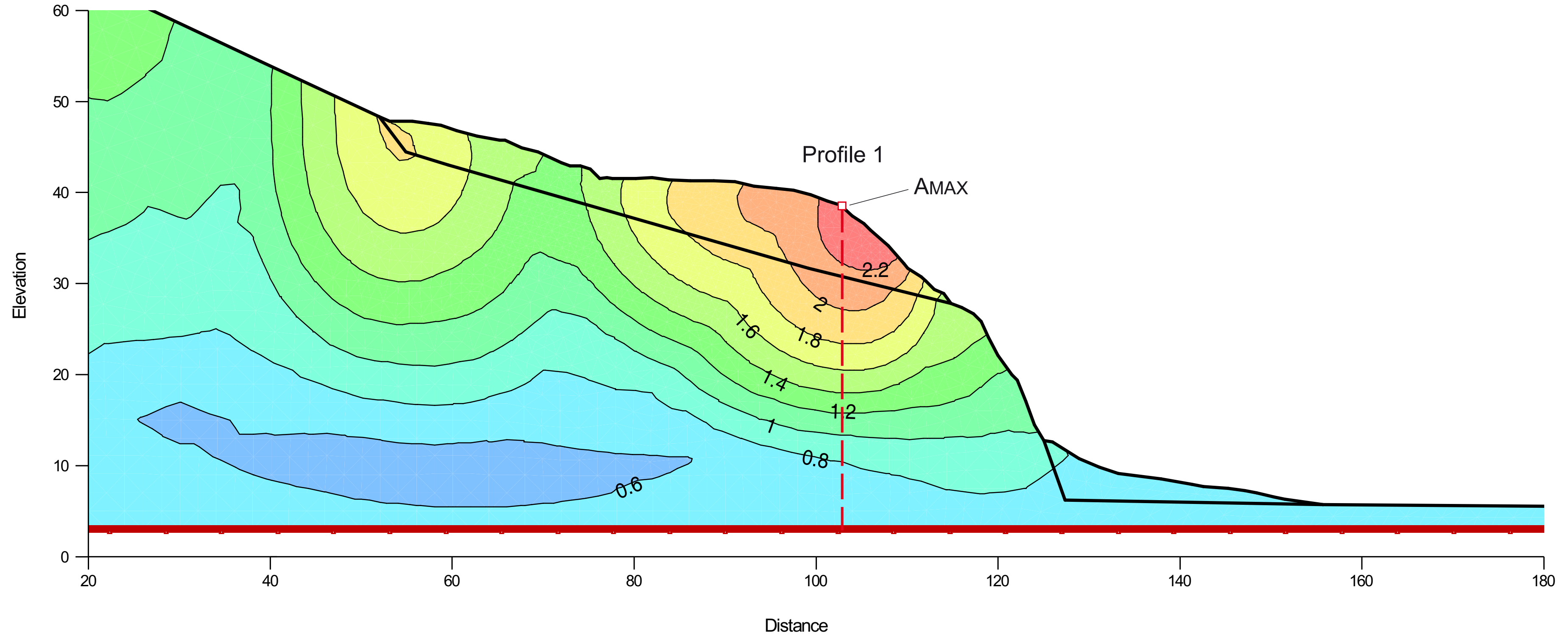
5. These amplification factors should in preference be applied when the slopes belong to two-dimensional topographic irregularities, such as long ridges and cliffs of height greater than about 30 m.

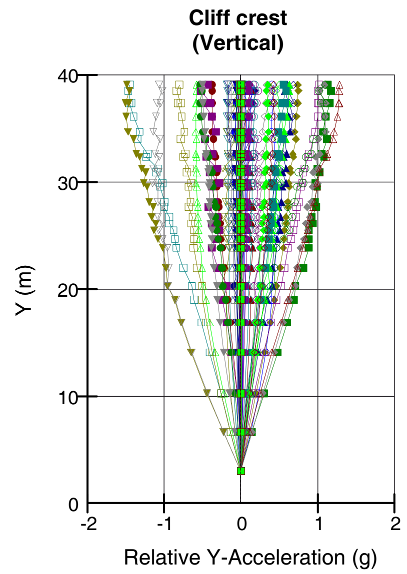
Ashford and Sitar (2002) recommend an S_T of 1.5 be applied to the maximum free-field acceleration behind the crest based on their assessment of slopes in homogenous materials, typically >60° to near vertical and of heights (toe to crest) of typically >30 m. This factor is based on the assessment of slopes that failed during the 1989 Loma Prieta M_W 6.9 earthquake.

Results from the seismic response assessment suggest that the horizontal peak ground acceleration amplification factors (S_T) for Defender Lane are about 2.8 times greater than the free field input motions. These are larger than those values reported by Ashford and Sitar (2002), and are in part a function of the impedance contrasts within the slope, which are not reported to occur in the slopes assessed by Ashford and Sitar (2002). These higher factors may also be a function of the site to earthquake source distances. In the case of Defender Lane, the site is within 5 km of the epicentres of the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, making them all “near-field” earthquakes.









DRW: PC

CHK: CM

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QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 5) to the 22 February 2011, earthquake

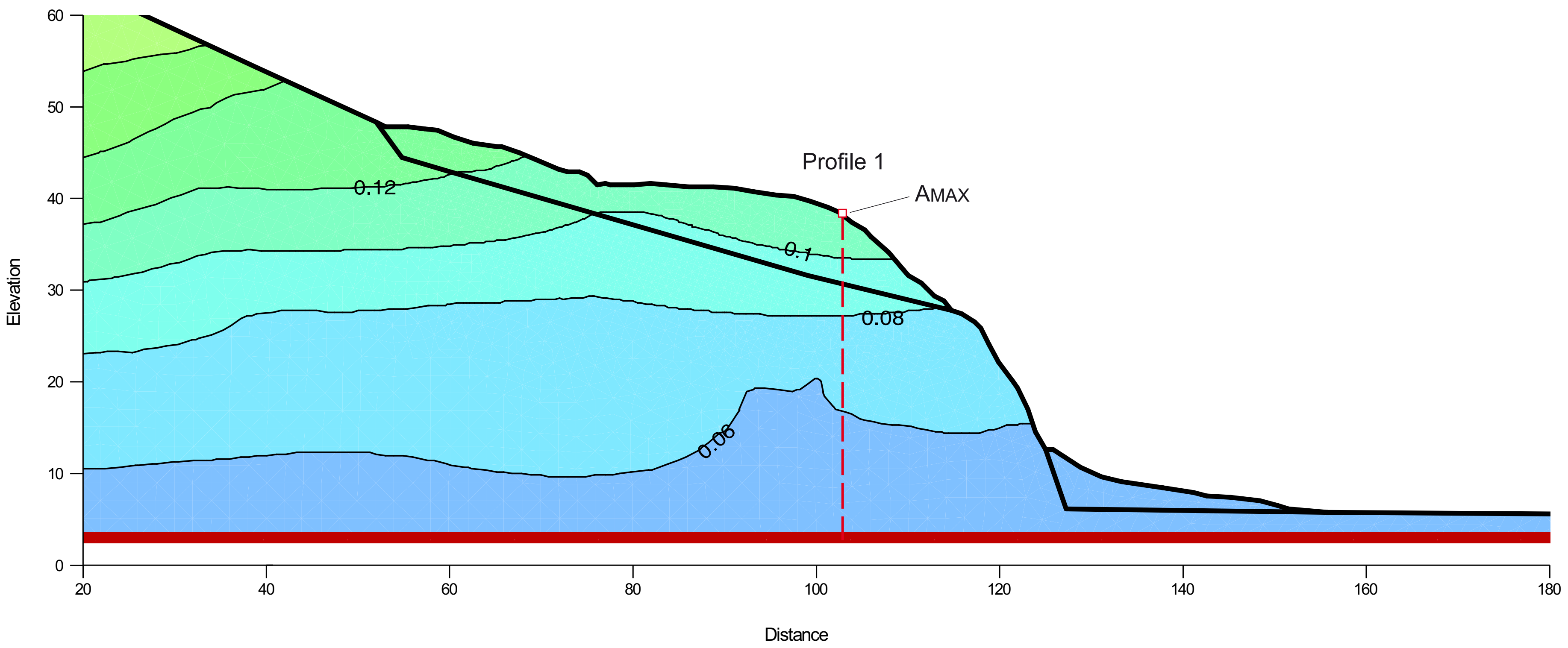
Defender Lane Christchurch

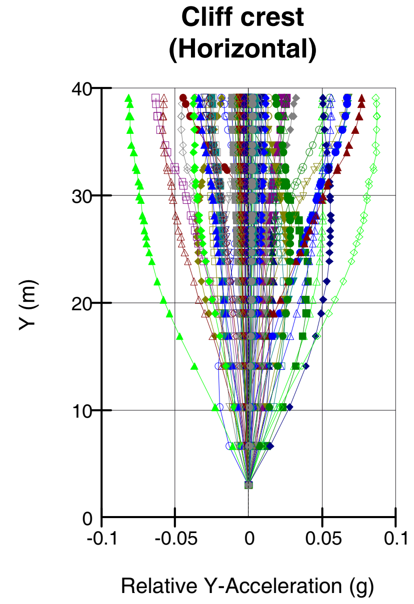
Figure A4.3a

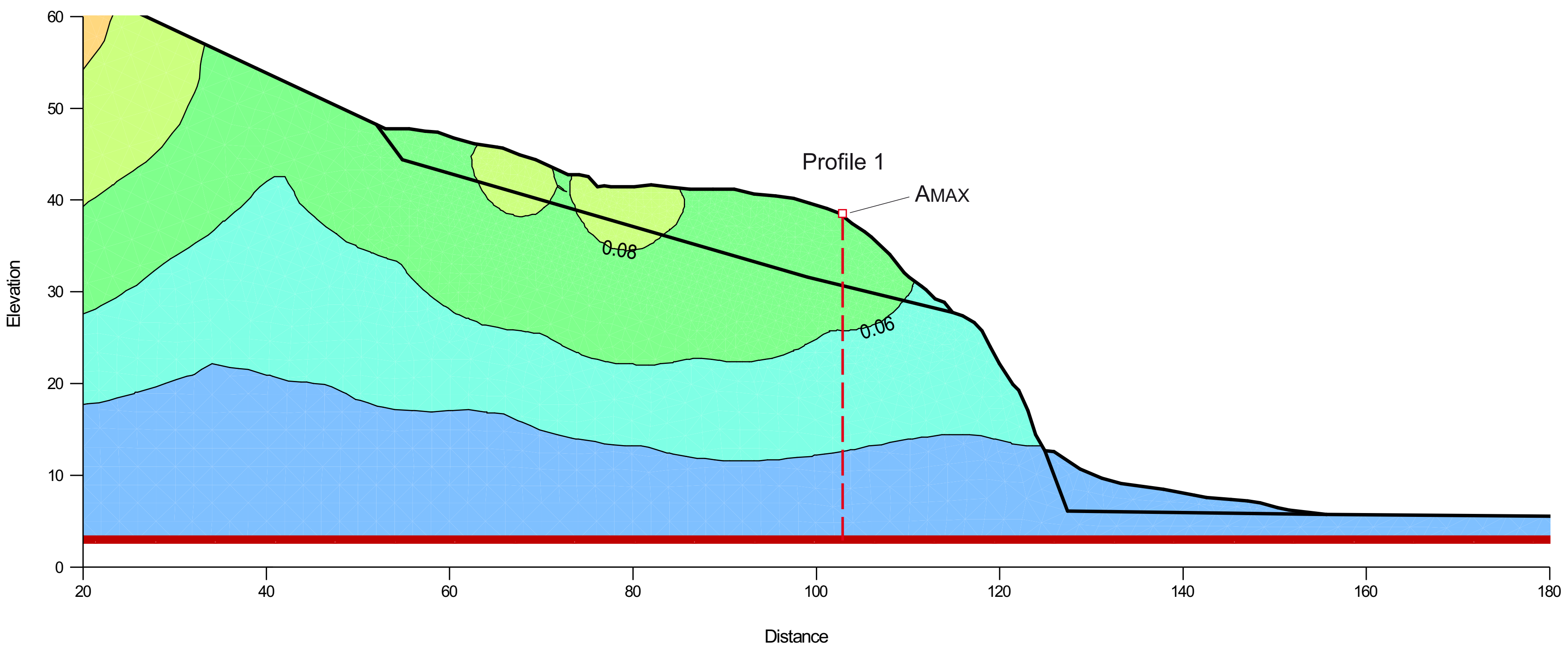
FINAL

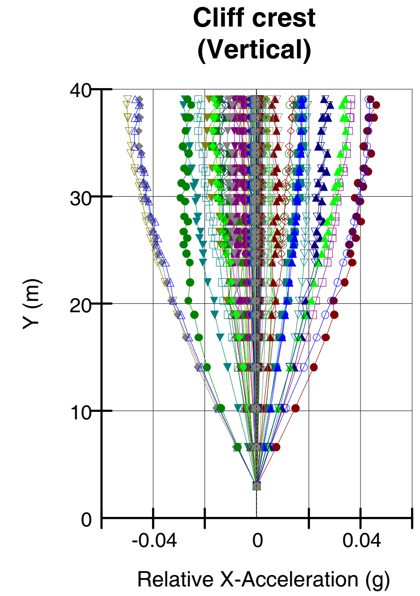
REPORT: CR2014/67

DATE: Jun 2014









DRW: PC

CHK: CM

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QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 5) to the 16 April 2011, earthquake

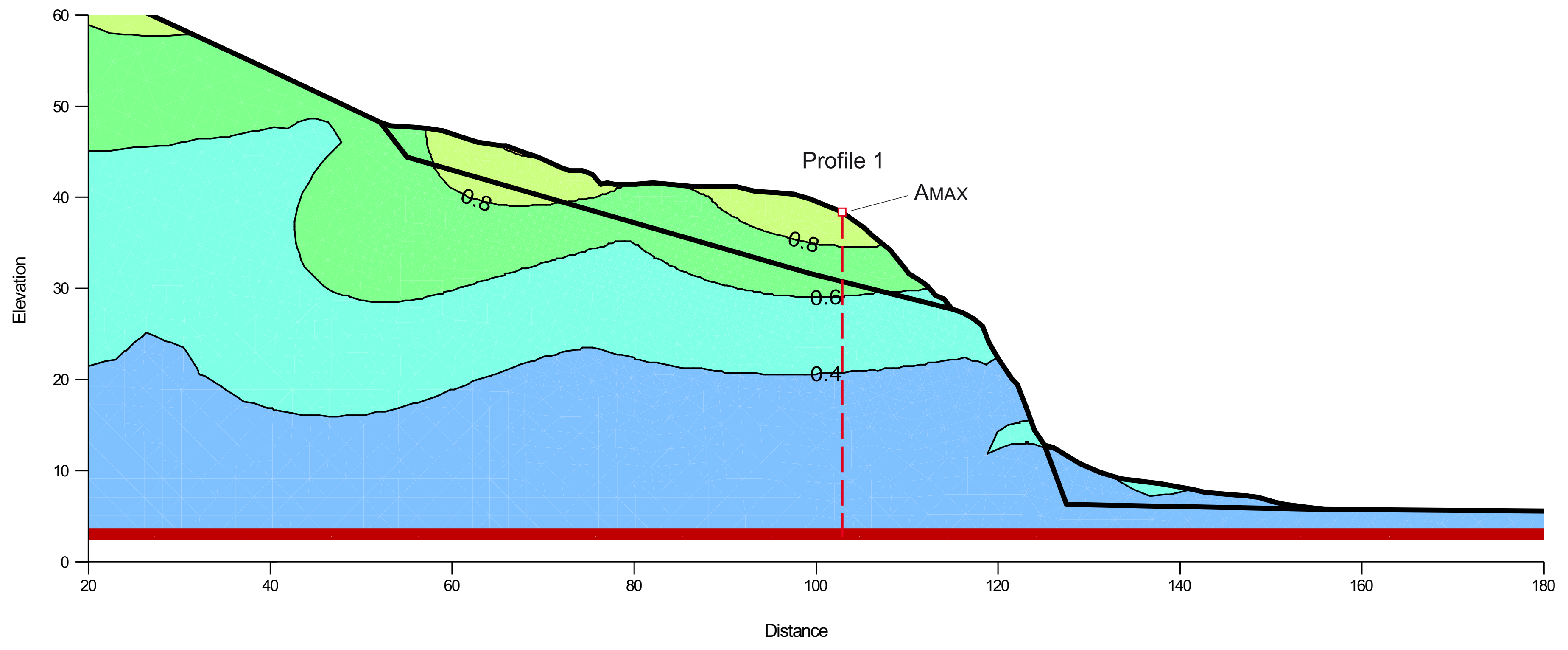
Defender Lane Christchurch

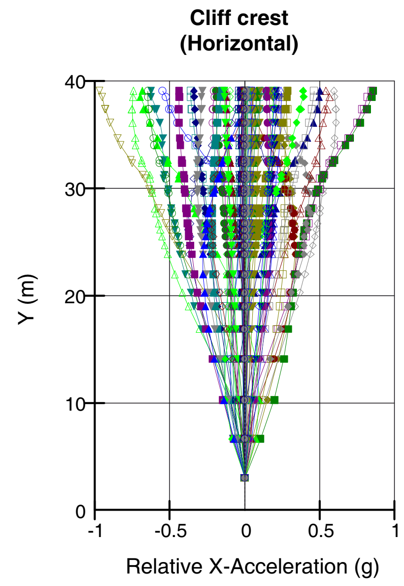
Figure A4.3b

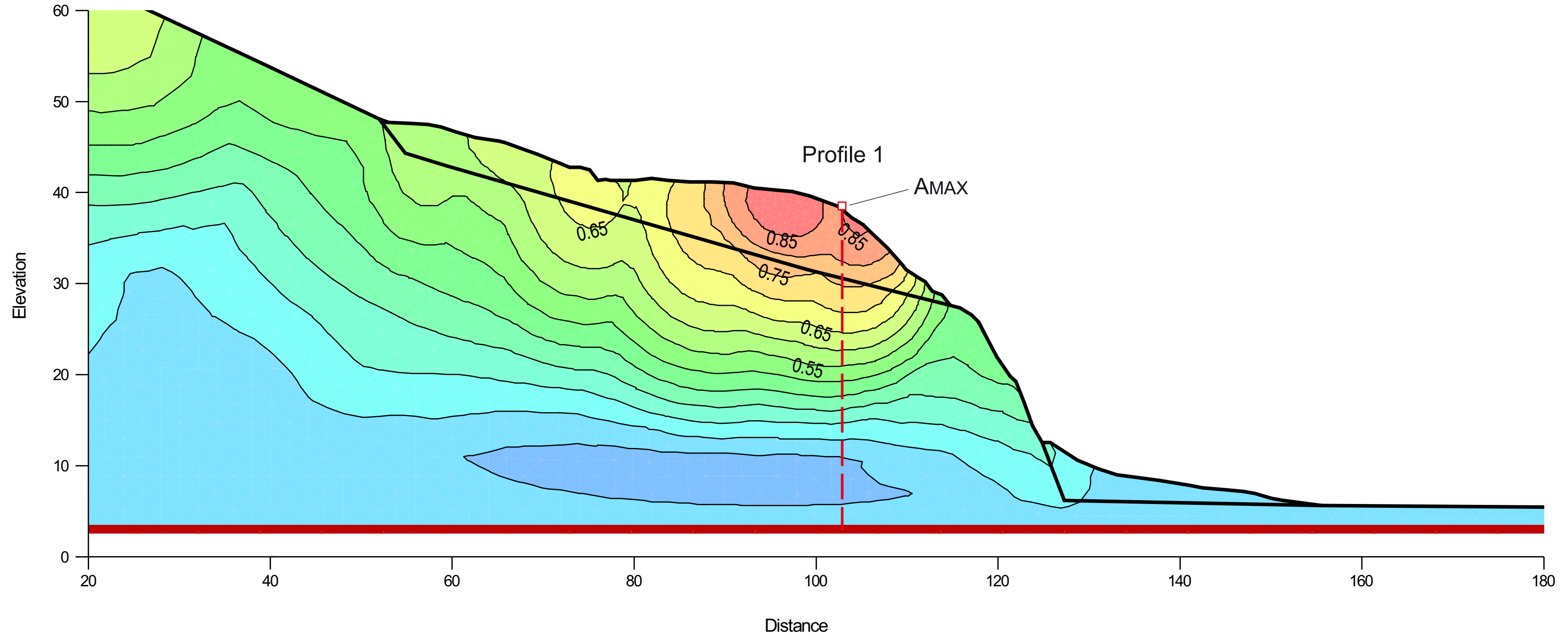
FINAL

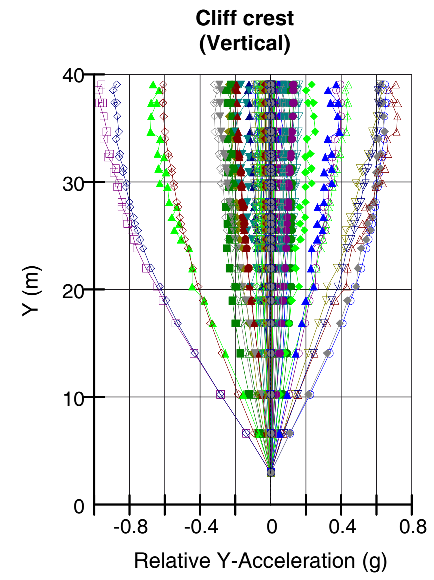
REPORT: CR2014/67

DATE: Jun 2014









DRW:
PC

CHK:
CM

QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 5) to the 13 June 2011, earthquake

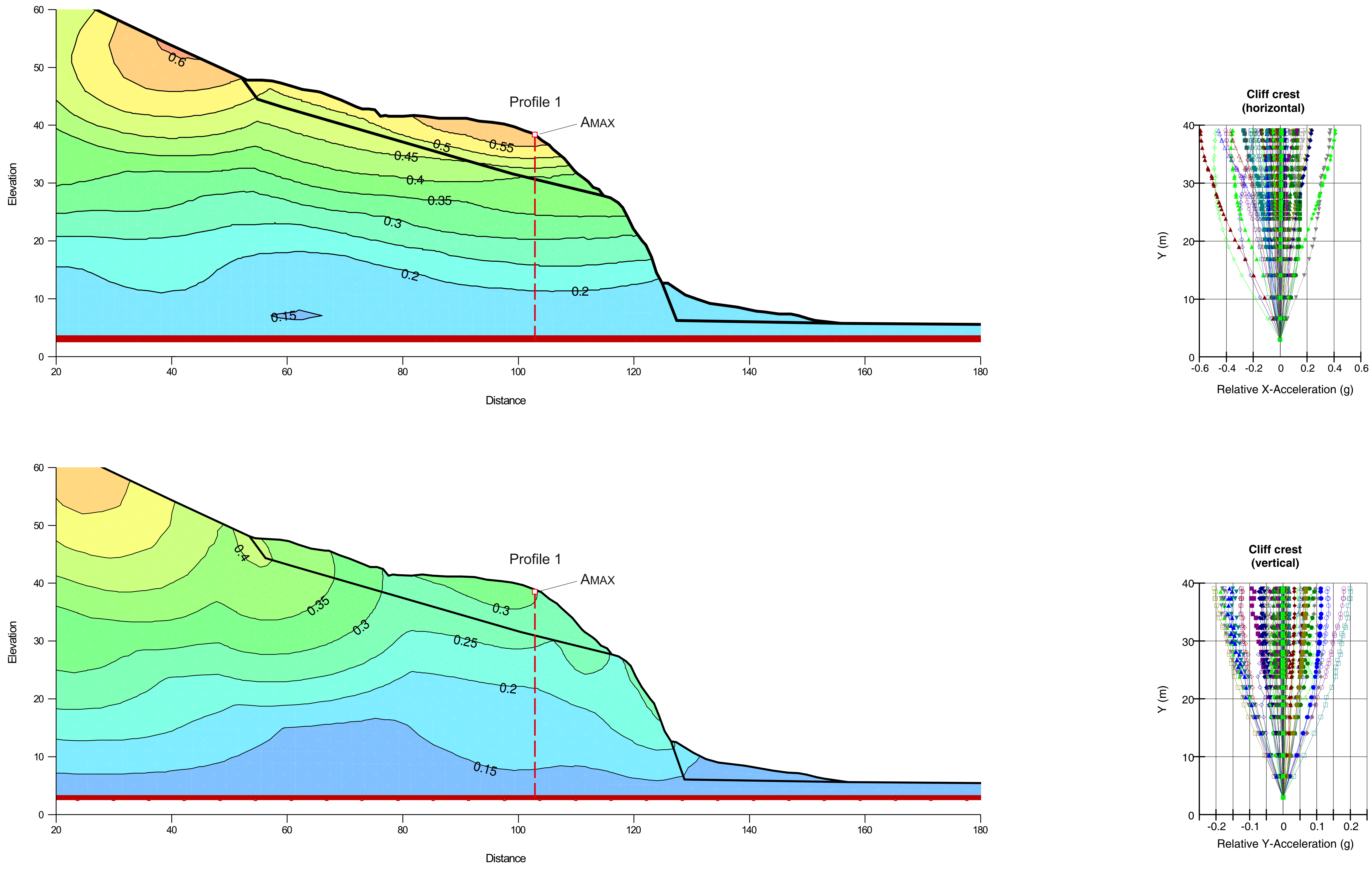
Defender Lane
Christchurch

Figure A4.3c

FINAL

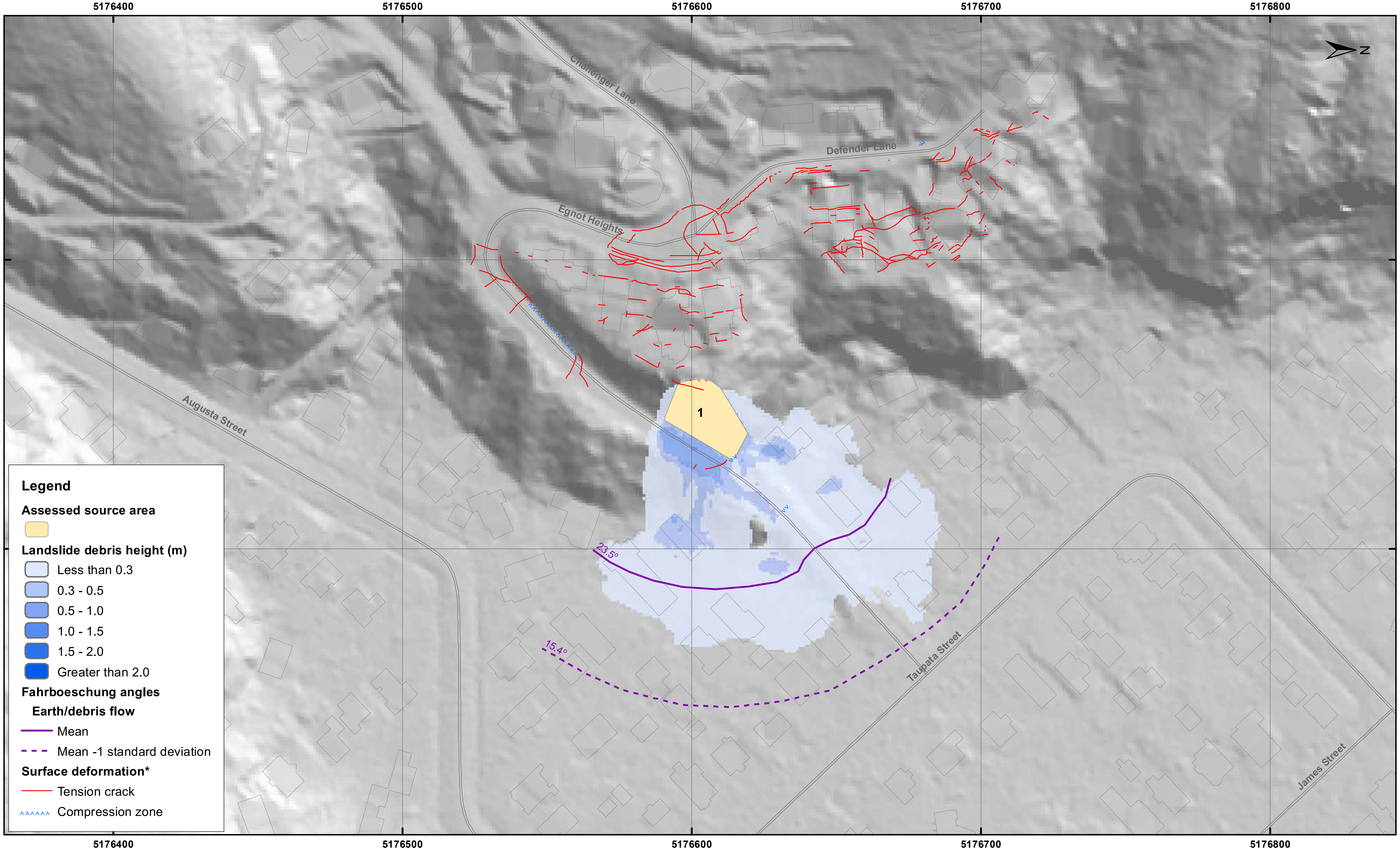
REPORT: CR2014/67

DATE: Jun 2014



DRW: PC	**QUAKE/W DYNAMIC RESPONSE ASSESSMENT (section 5) to the 23 December 2011, earthquake**	Figure A4.3d
CHK: CM	**Defender Lane Christchurch**	FINAL
		REPORT: CR2014/67 — DATE: Jun 2014

A5 APPENDIX 5: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1–10 ADOPTING THE MIDDLE ESTIMATES OF SOURCE VOLUME. ESTIMATED LANDSLIDE RUNOUT HEIGHT



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 1 - Middle Volume (290 m^3)

Defender Lane - Port Hills
Christchurch

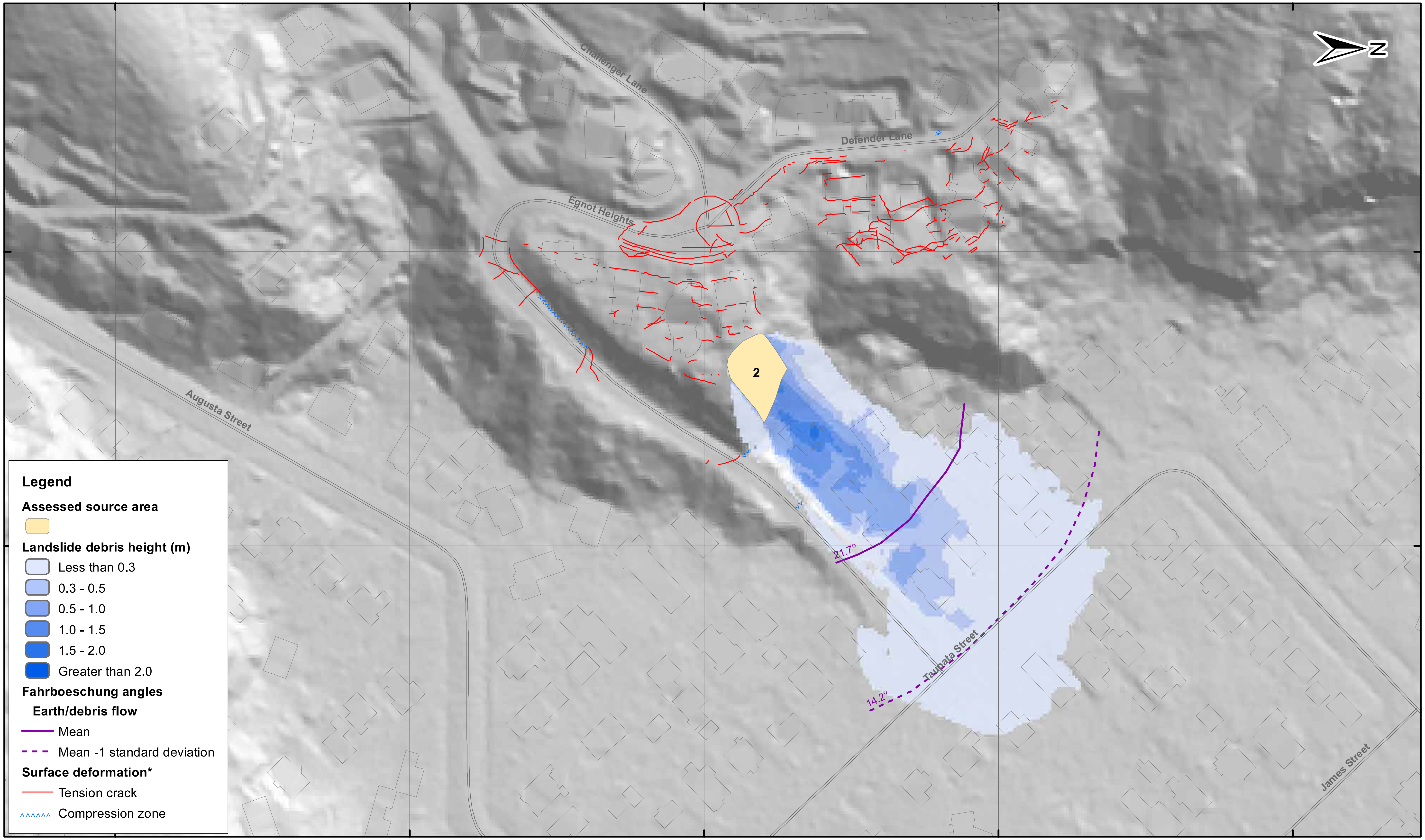
APPENDIX 5

Map 1

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 2 - Middle Volume (660 m^3)

Defender Lane - Port Hills Christchurch

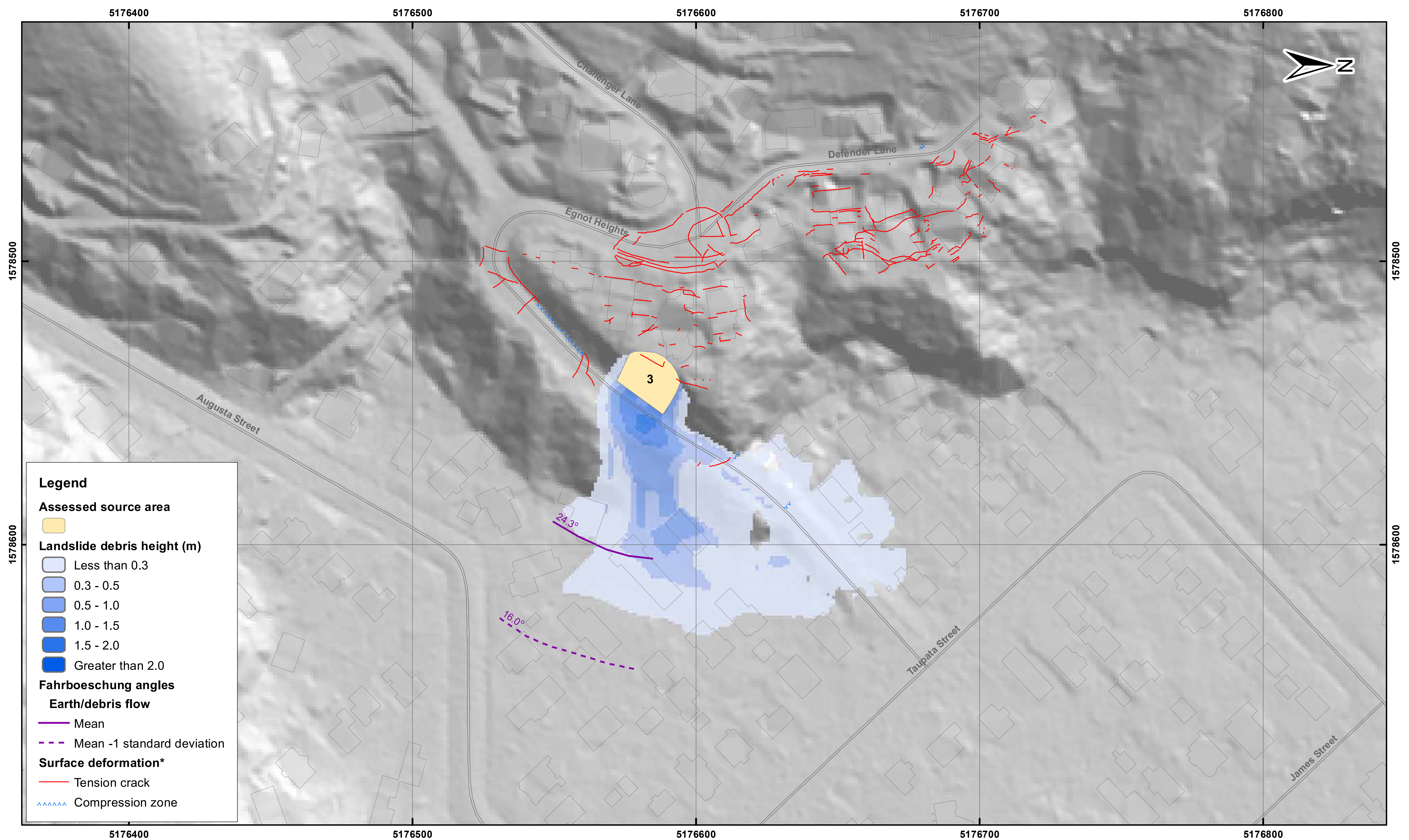
APPENDIX 5

Map 2

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 3 - Middle Volume (200 m^3)

Defender Lane - Port Hills Christchurch

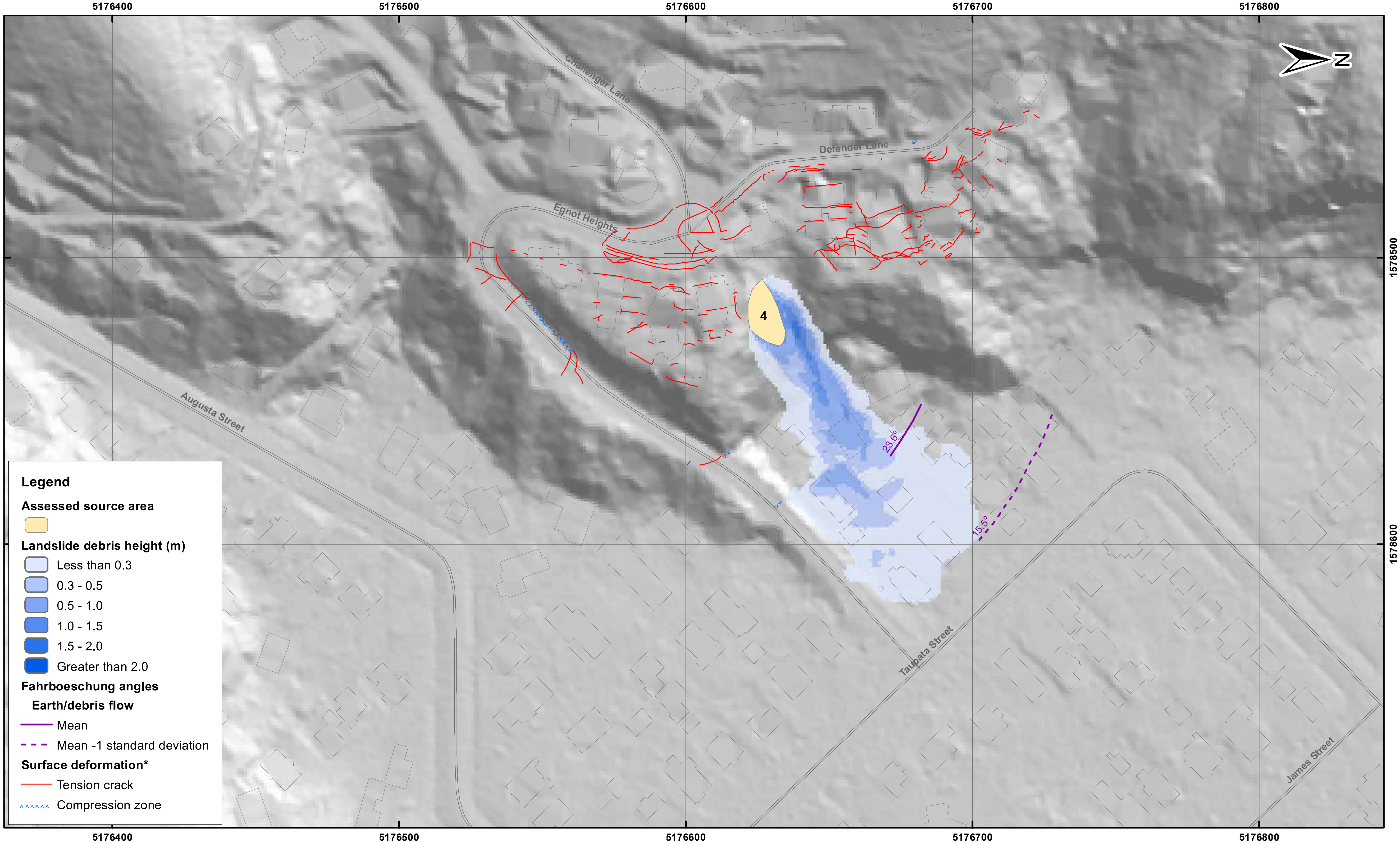
APPENDIX 5

Map 3

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 4 - Middle Volume (280 m^3)

Defender Lane - Port Hills
Christchurch

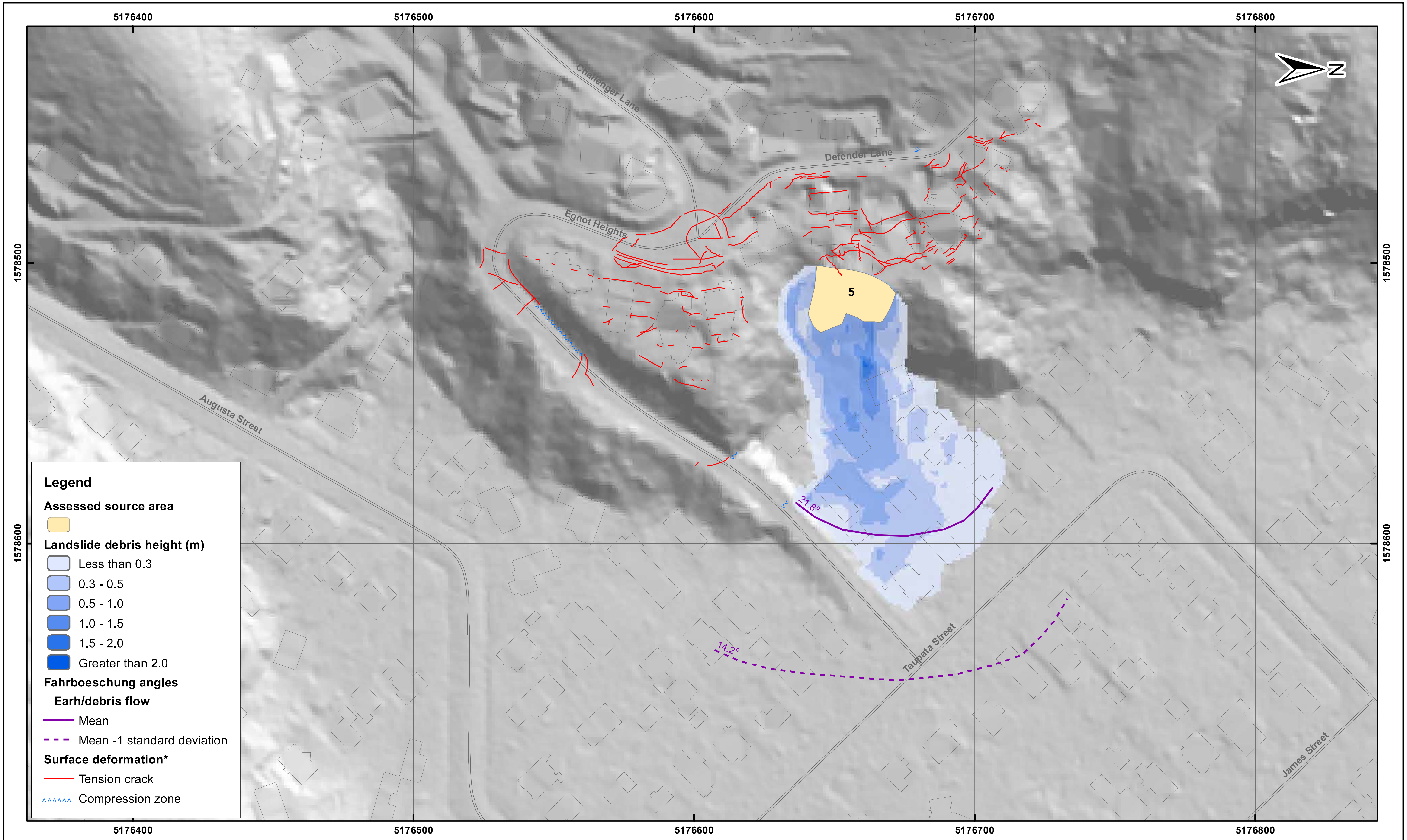
APPENDIX 5

Map 4

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 5 - Middle Volume (620 m^3)

Defender Lane - Port Hills
Christchurch

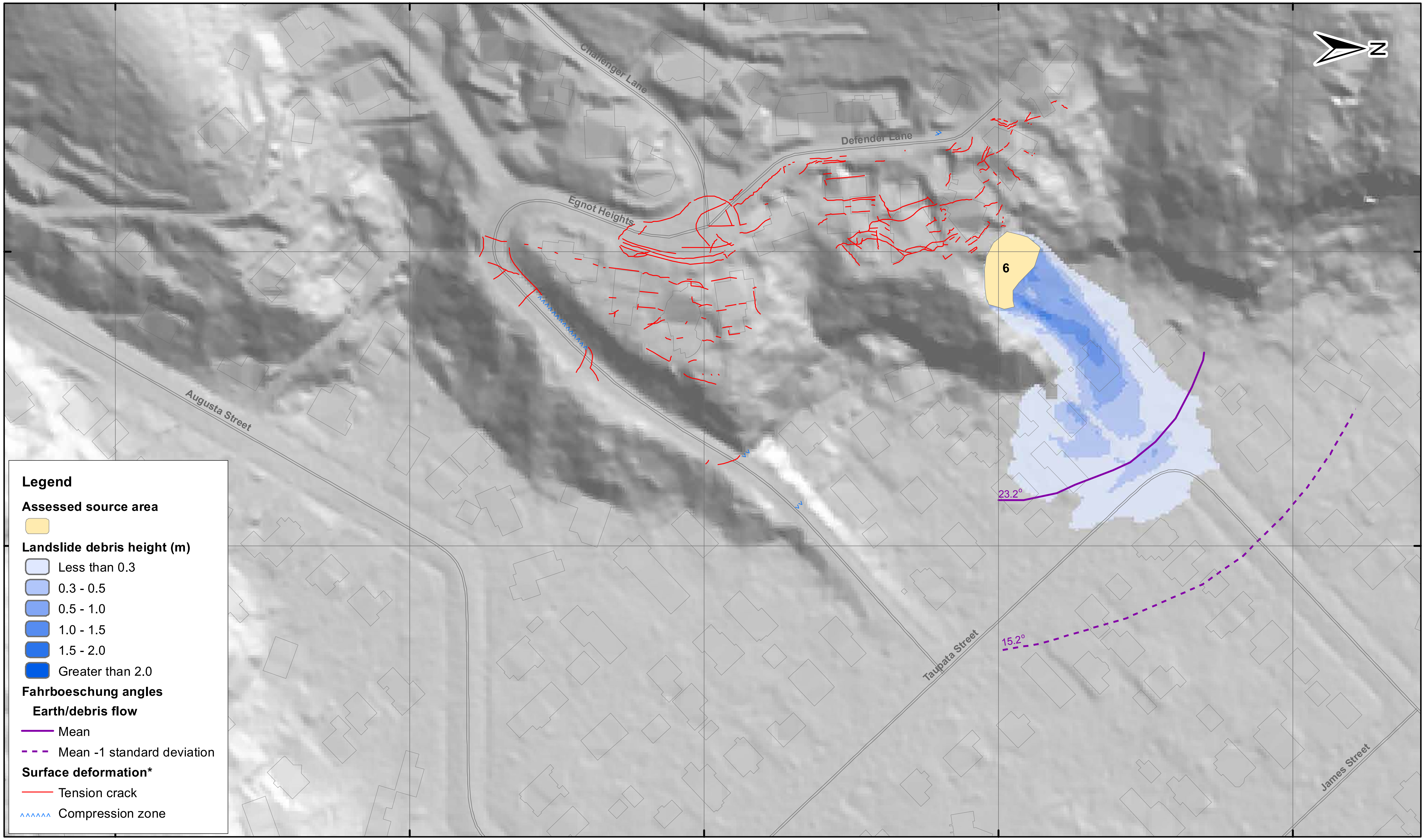
APPENDIX 5

Map 5

FINAL

REPORT: CR2014/67

DATE: June 2014



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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 6 - Middle Volume (330 m^3)

Defender Lane - Port Hills
Christchurch

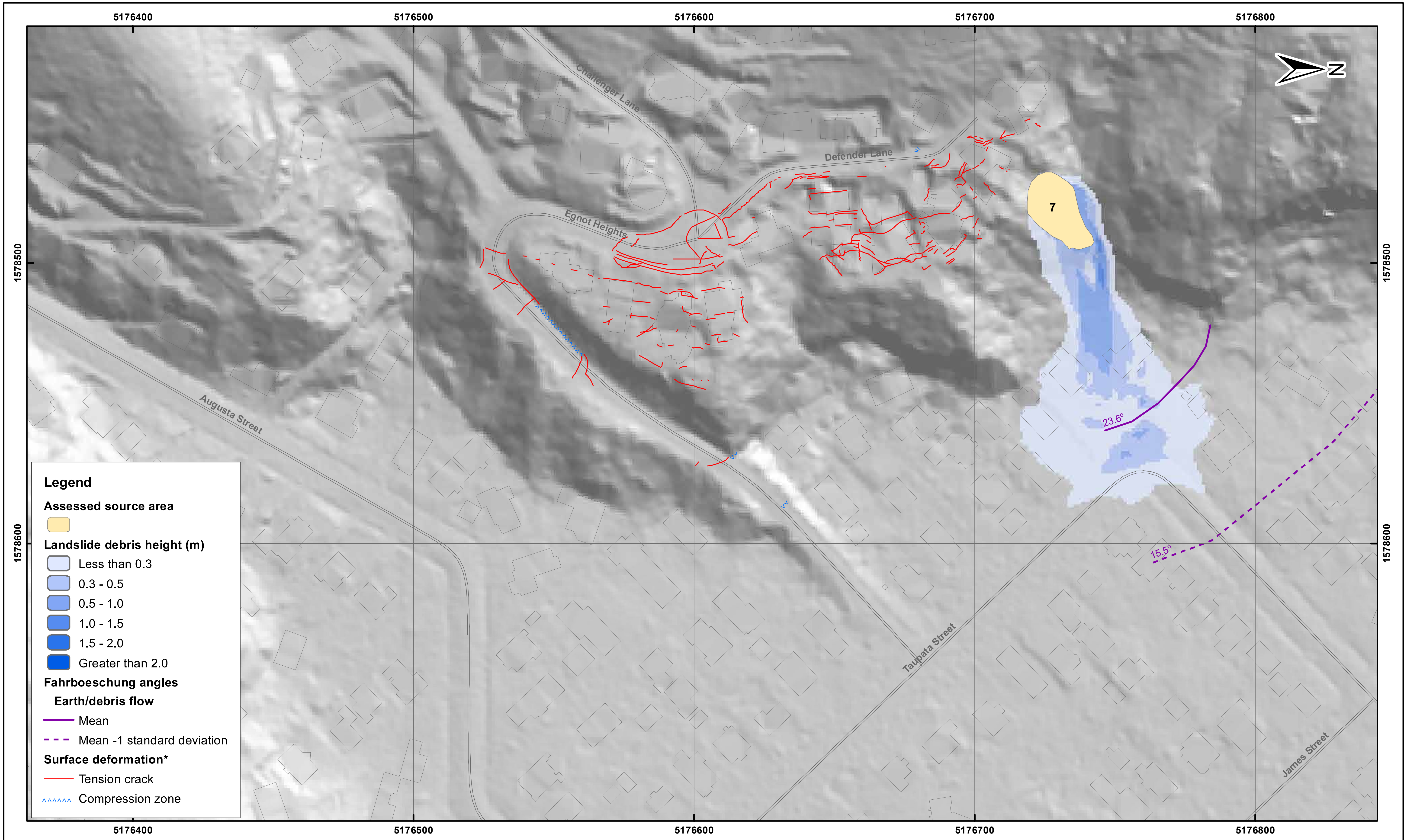
APPENDIX 5

Map 6

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 7 - Middle Volume (270 m^3)

Defender Lane - Port Hills
Christchurch

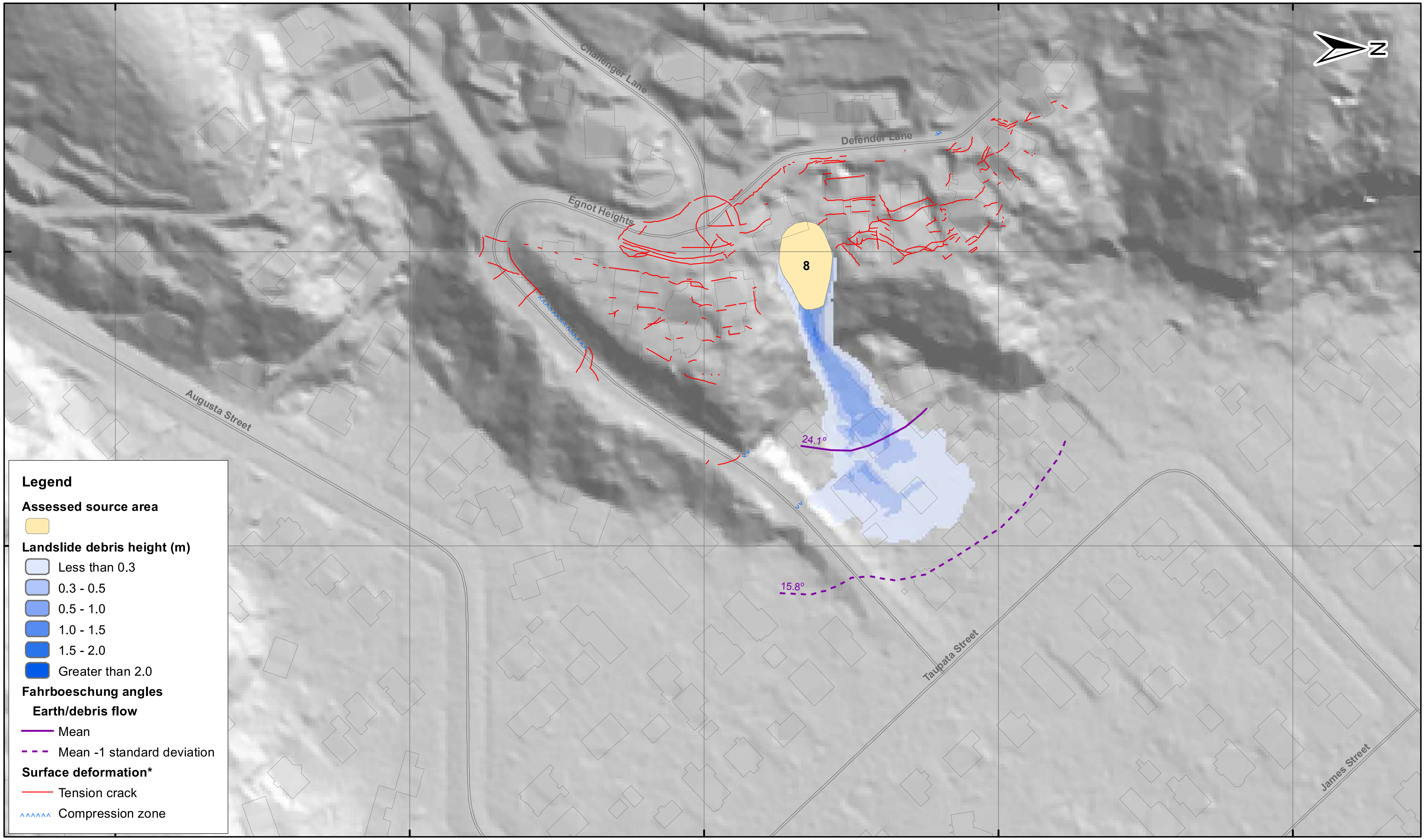
APPENDIX 5

Map 7

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 8 - Middle Volume (230 m^3)

Defender Lane - Port Hills
Christchurch

APPENDIX 5

Map 8

FINAL

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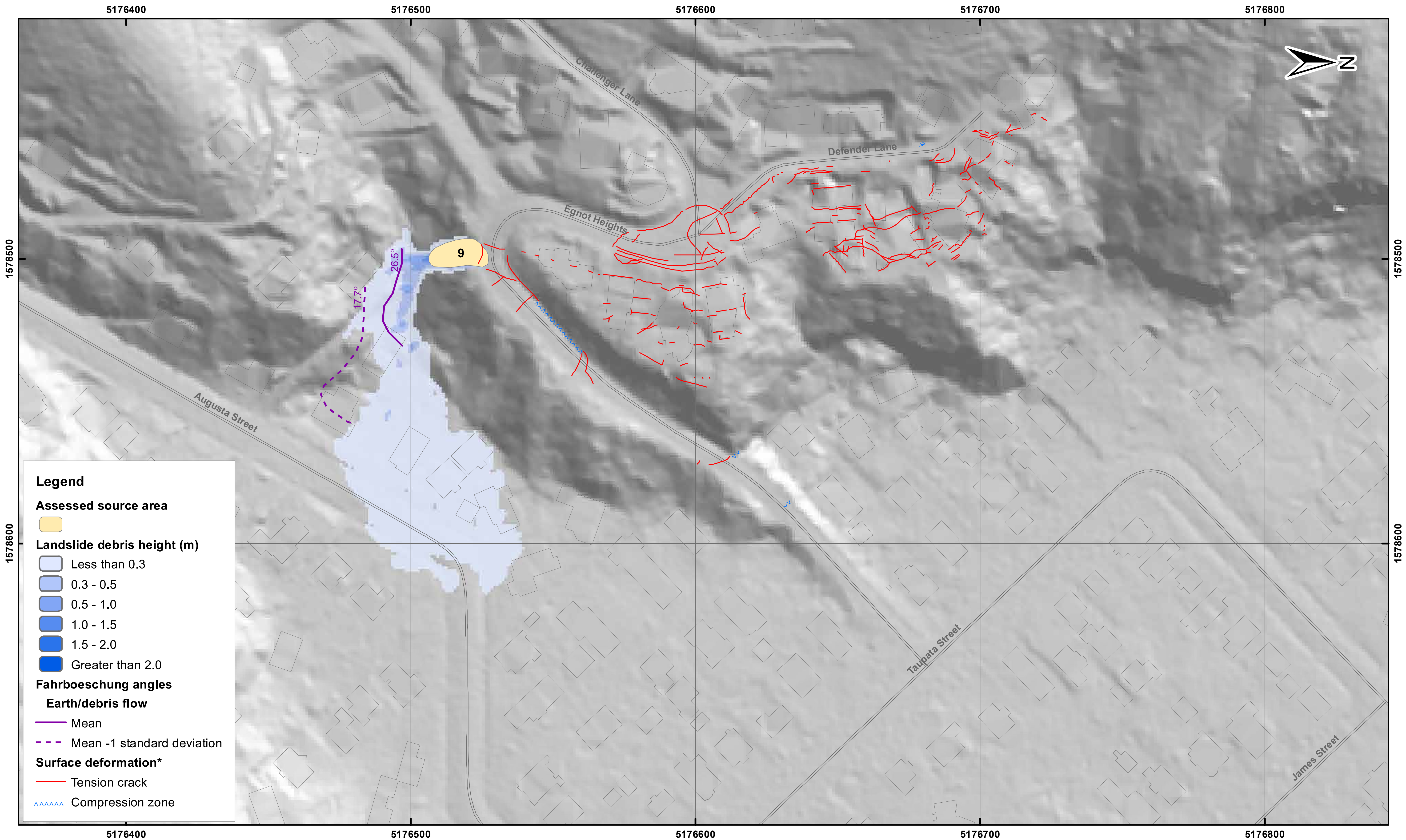
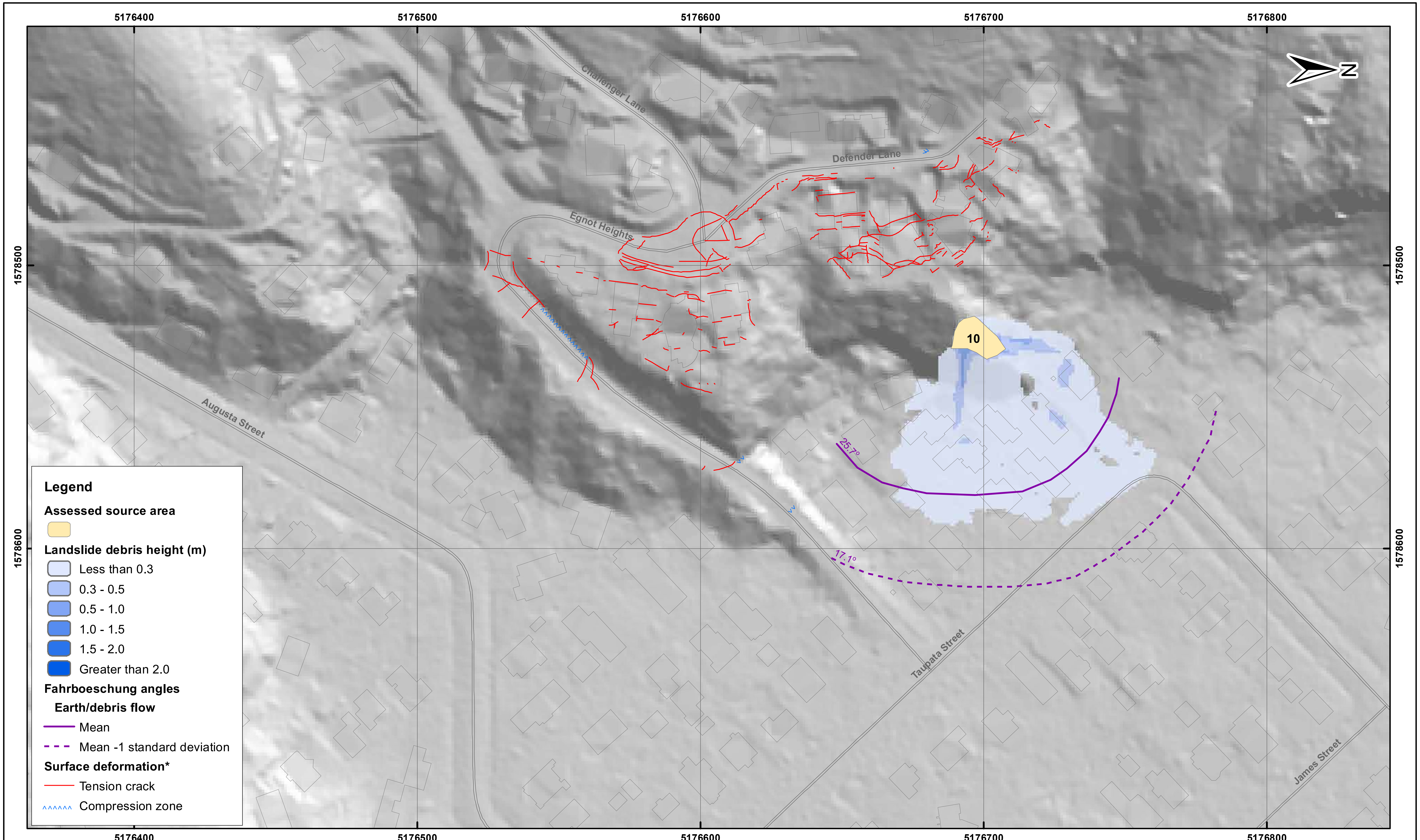


SCALE BAR: 0 50 100 m			ESTIMATED LANDSLIDE RUNOUT HEIGHT Source 9 - Middle Volume (80 m^3)	APPENDIX 5
EXPLANATION: * Taken from report CR2012/317 Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution. Roads and building footprints and types provided by Christchurch City Council (20/02/2012). PROJECTION: New Zealand Transverse Mercator 2000	DRW: BL, WR CHK: CM, FDP	GNS SCIENCE TE PŪ AO	Defender Lane - Port Hills Christchurch	Map 9 FINAL REPORT: CR2014/67 DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 10 - Middle Volume (110 m^3)

Defender Lane - Port Hills
Christchurch

APPENDIX 5

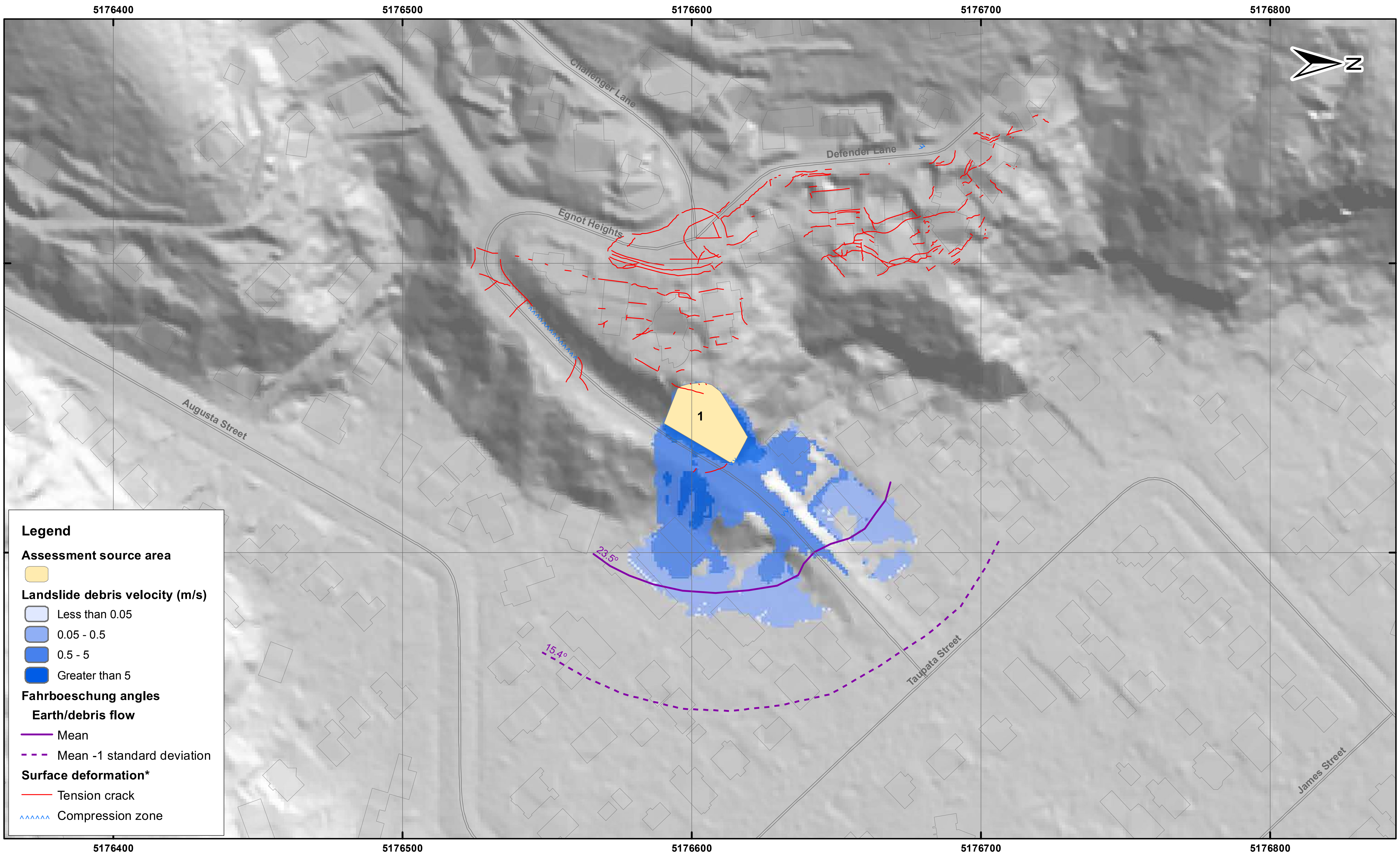
Map 10

FINAL

REPORT: CR2014/67

DATE: June 2014

A6 APPENDIX 6: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1–10 ADOPTING THE MIDDLE ESTIMATES OF SOURCE VOLUME. ESTIMATED LANDSLIDE RUNOUT VELOCITY



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 1 - Middle Volume (290 m^3)

Defender Lane - Port Hills
Christchurch

APPENDIX 6

Map 1

FINAL

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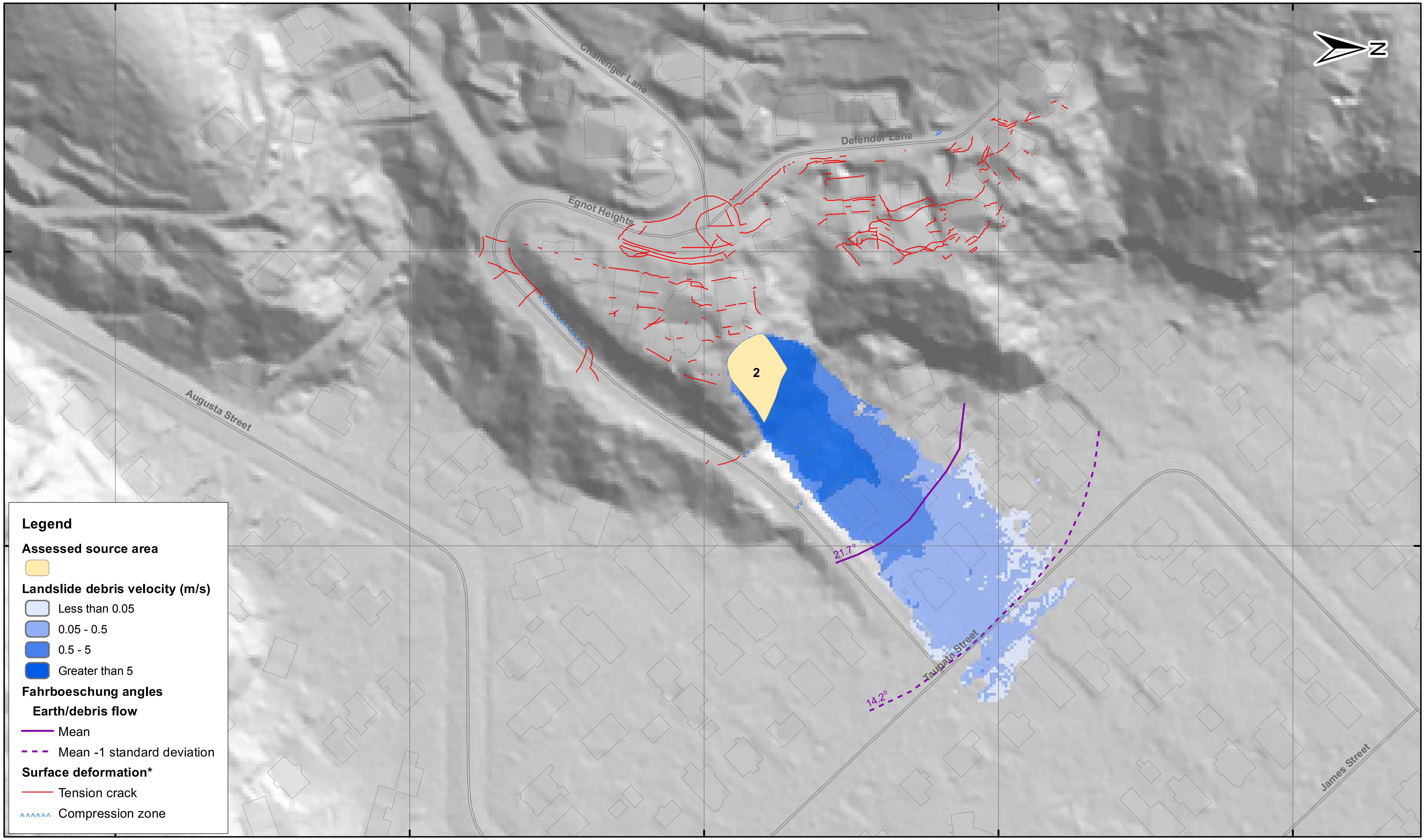

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 2 - Middle Volume (660 m^3)

Defender Lane - Port Hills Christchurch

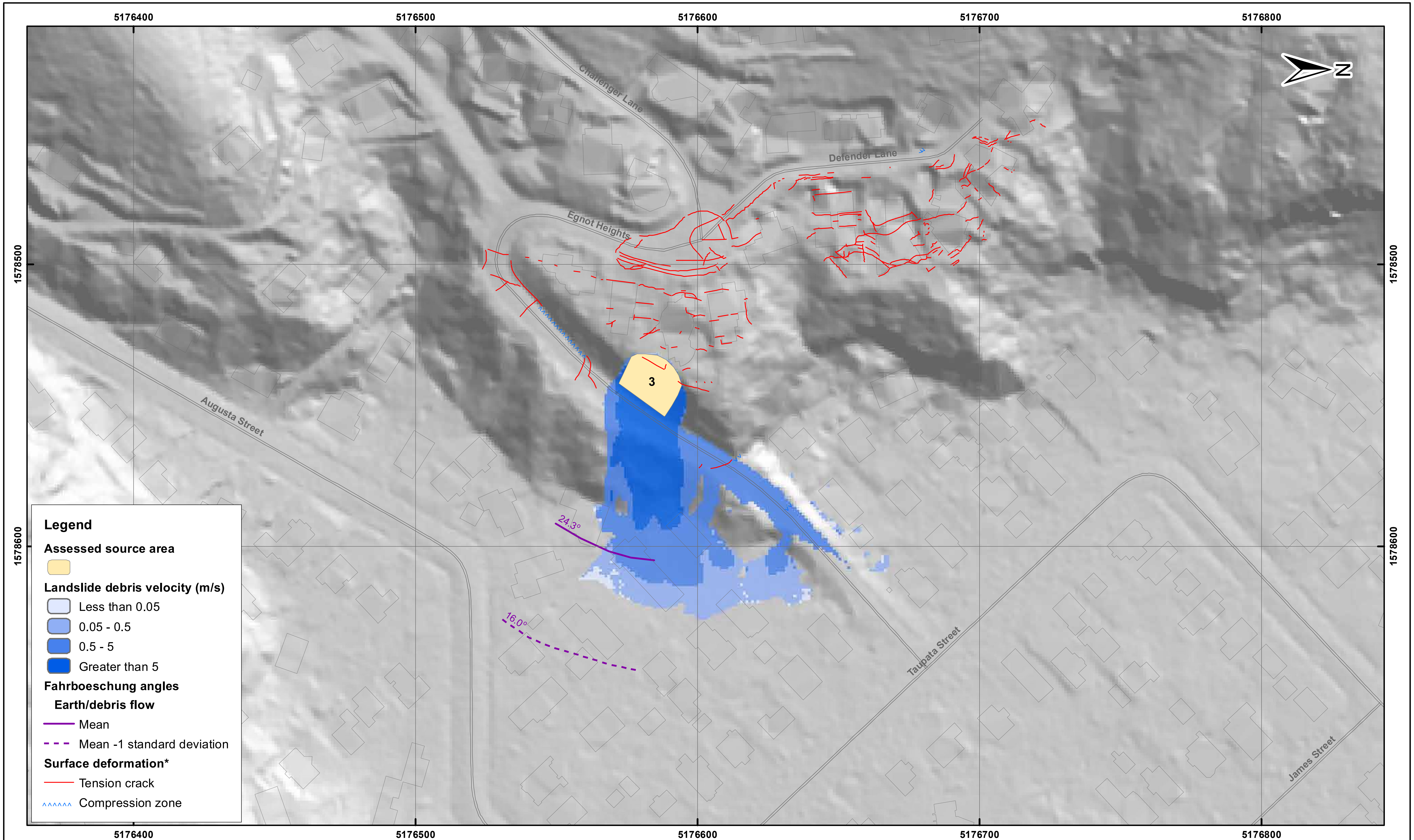
APPENDIX 6

Map 2

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 3 - Middle Volume (200 m^3)

Defender Lane - Port Hills
Christchurch

APPENDIX 6

Map 3

FINAL

REPORT: CR2014/67

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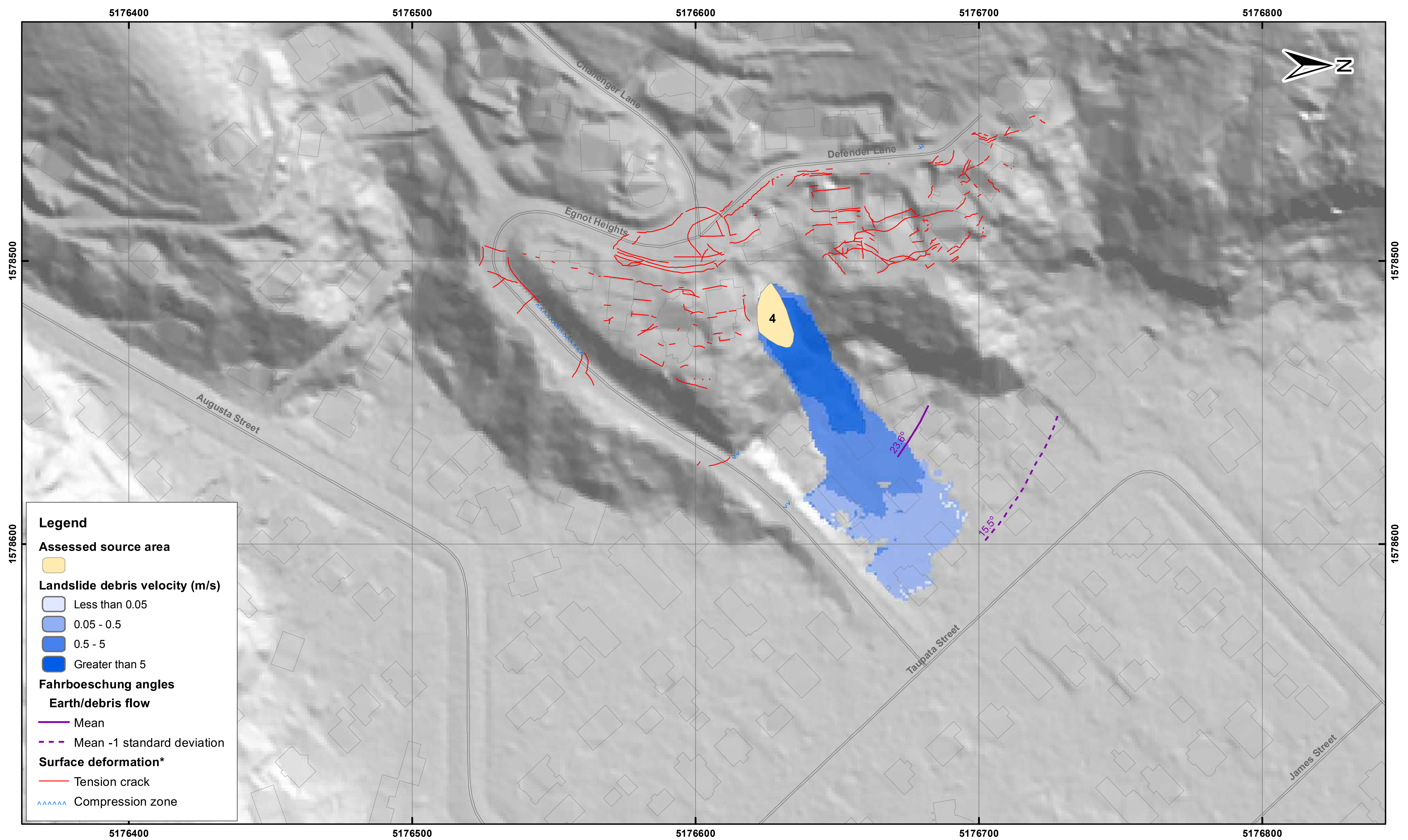

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 4 - Middle Volume (280 m^3)

Defender Lane - Port Hills Christchurch

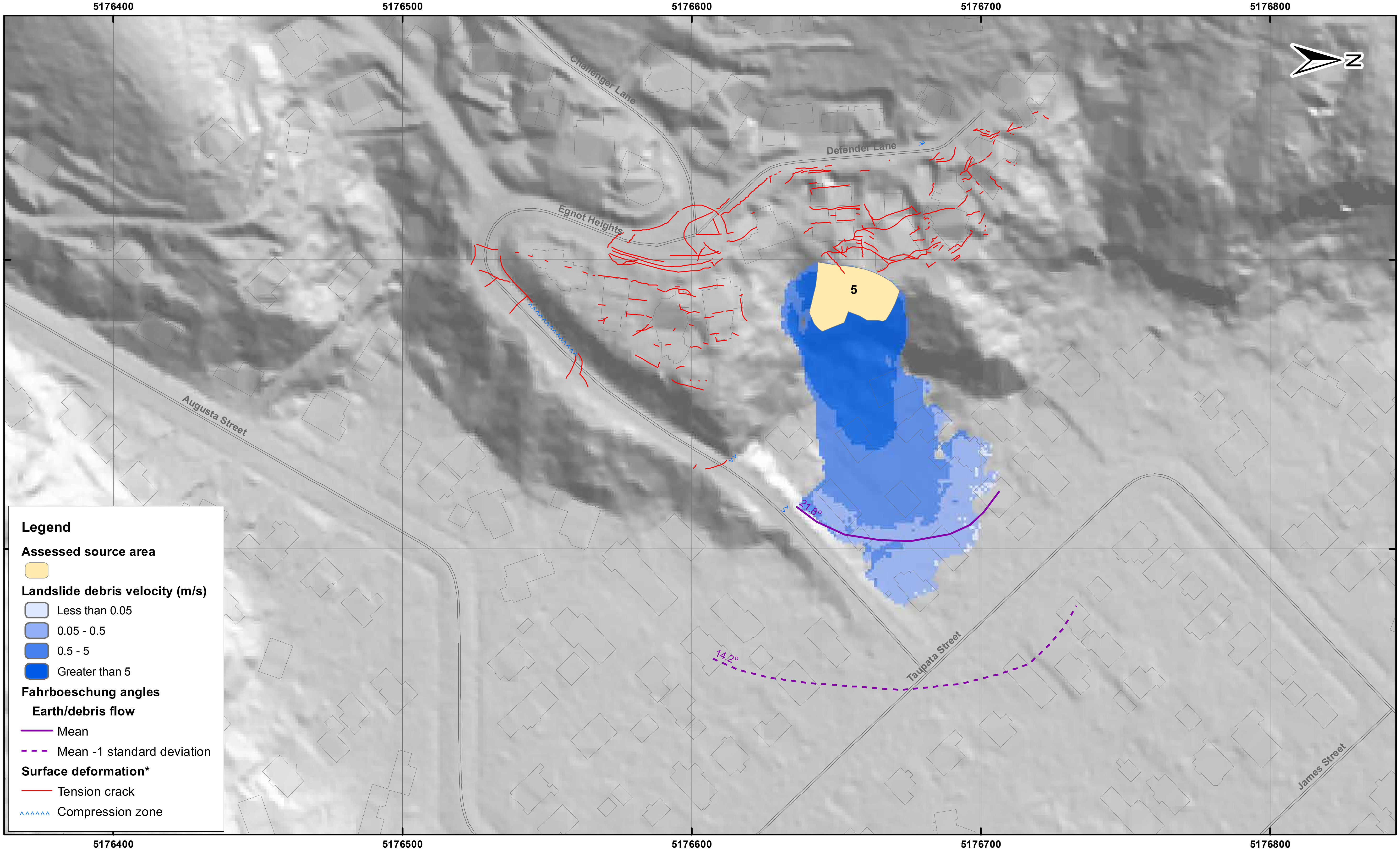
APPENDIX 6

Map 4

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 5 - Middle Volume (620 m^3)

Defender Lane - Port Hills
Christchurch

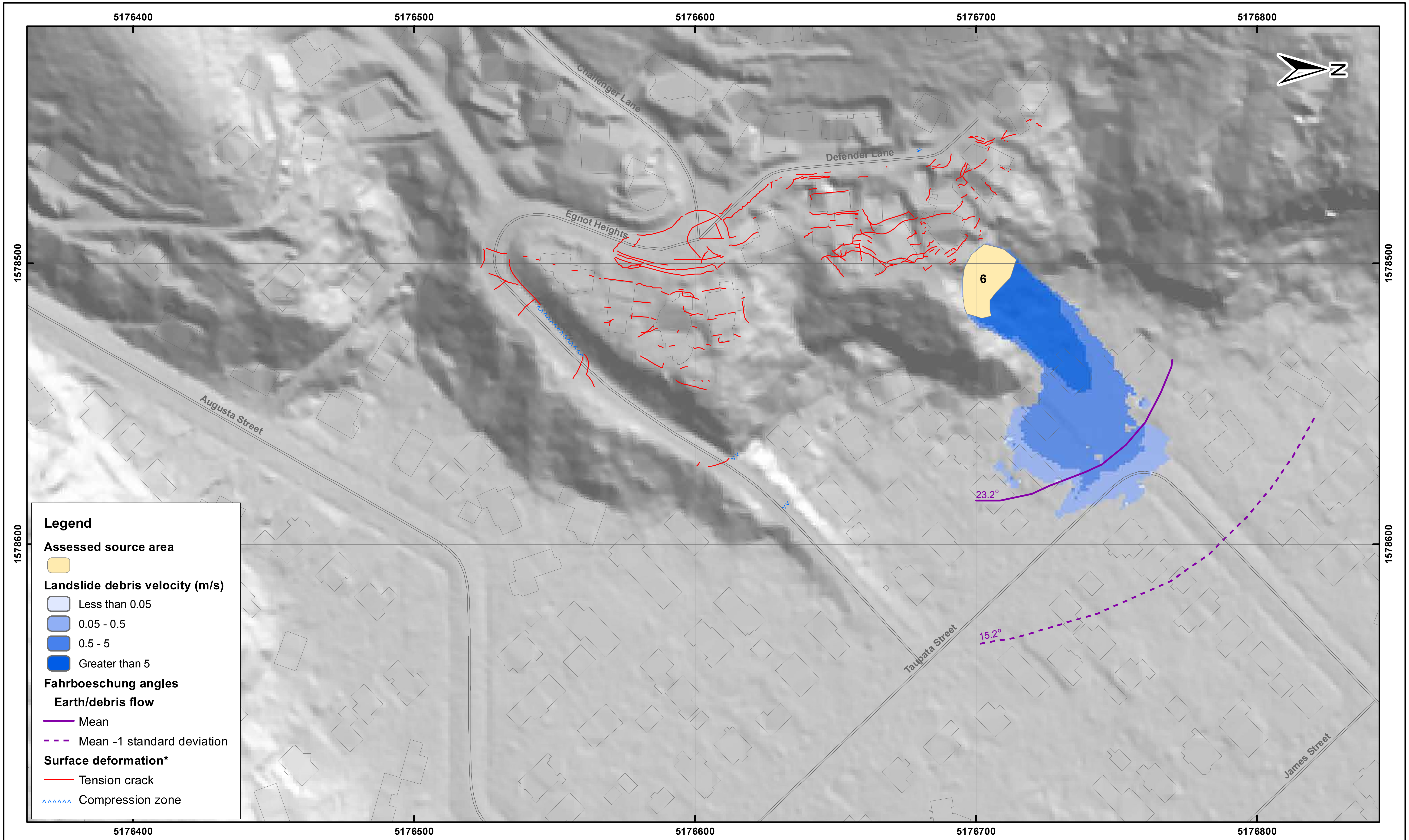
APPENDIX 6

Map 5

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 6 - Middle Volume (330 m^3)

Defender Lane - Port Hills Christchurch

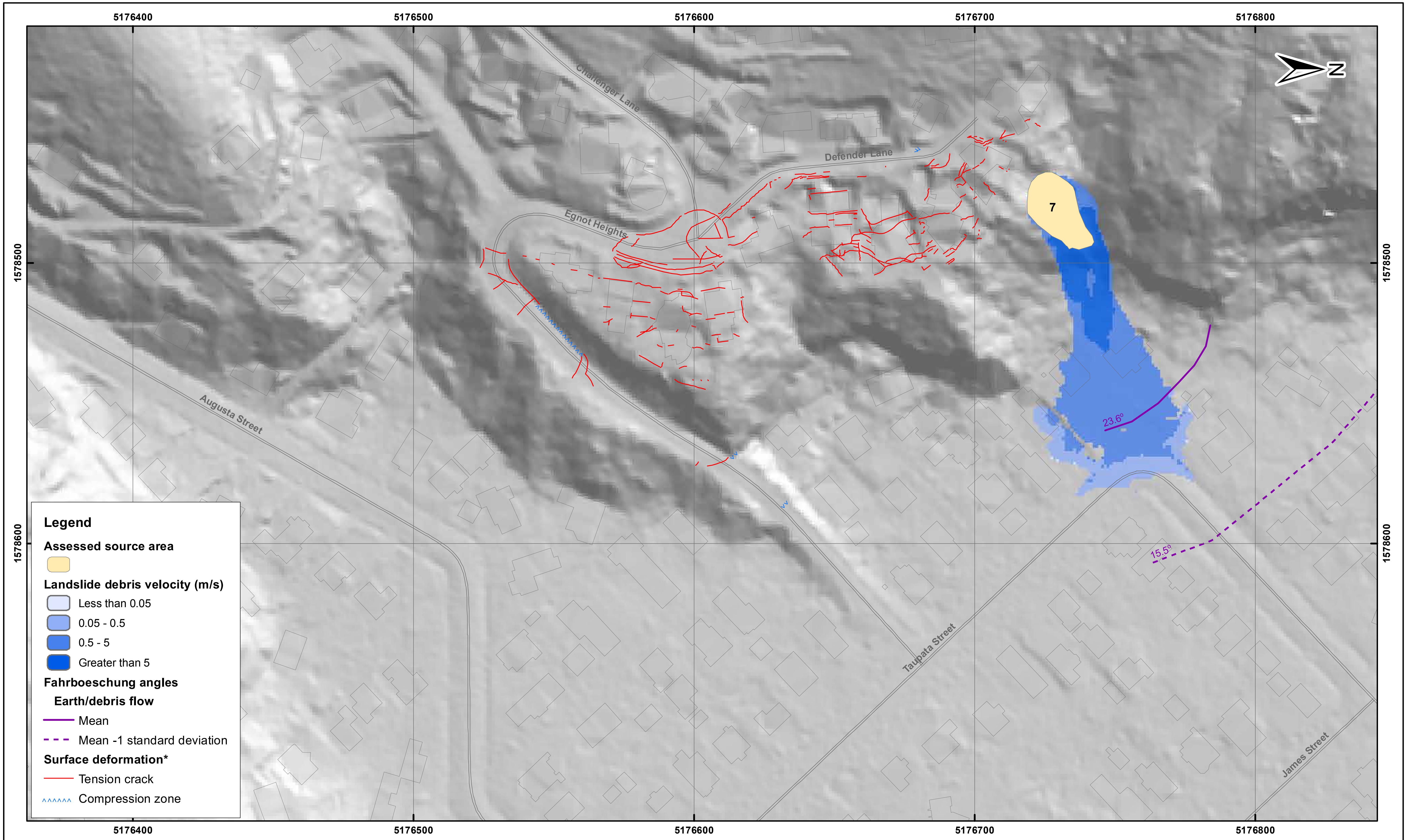
APPENDIX 6

Map 6

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 7 - Middle Volume (270 m^3)

Defender Lane - Port Hills Christchurch

APPENDIX 6

Map 7

FINAL

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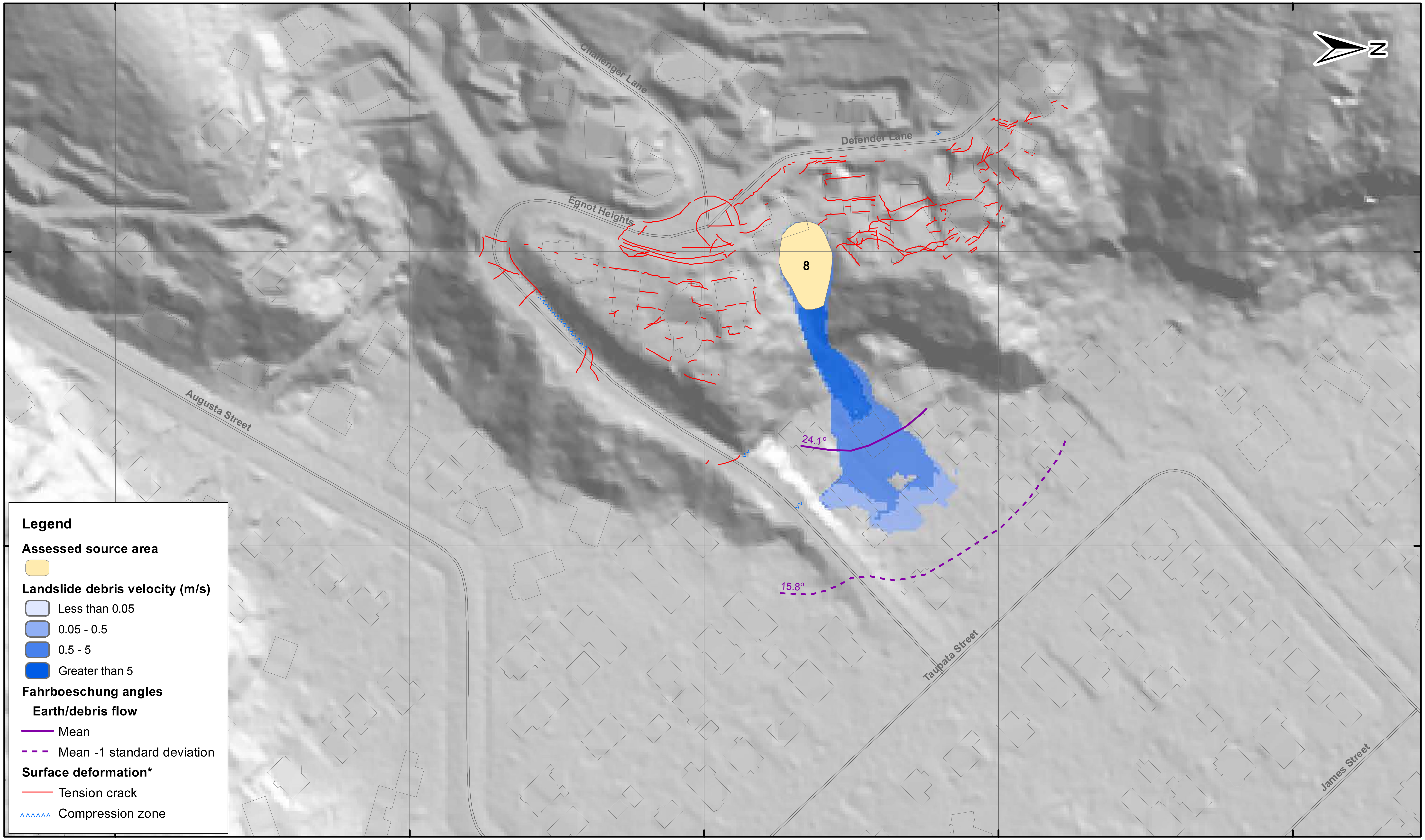
DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 8 - Middle Volume (230 m^3)

Defender Lane - Port Hills Christchurch

APPENDIX 6

Map 8

FINAL

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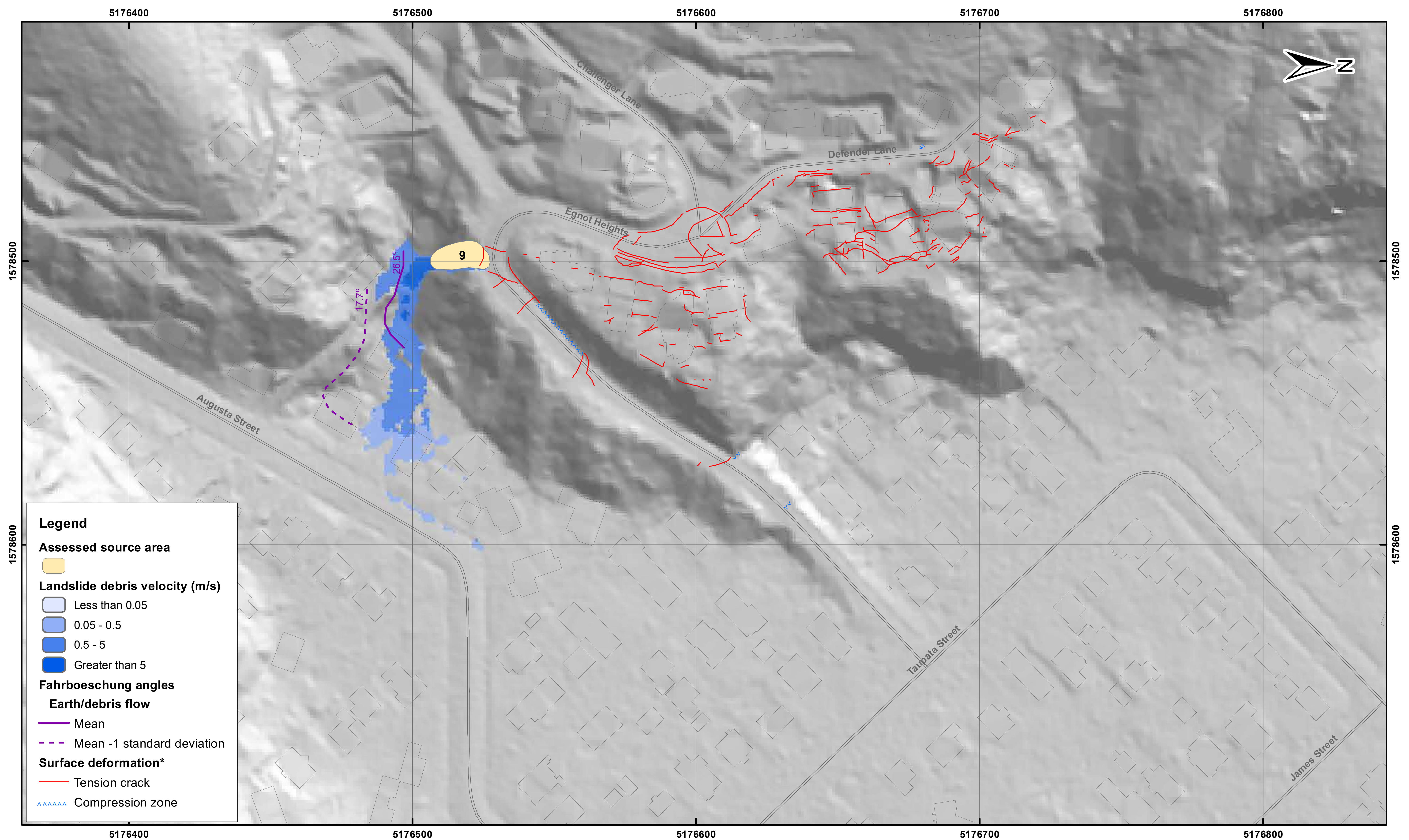
DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 9 - Middle Volume (80 m^3)

Defender Lane - Port Hills Christchurch

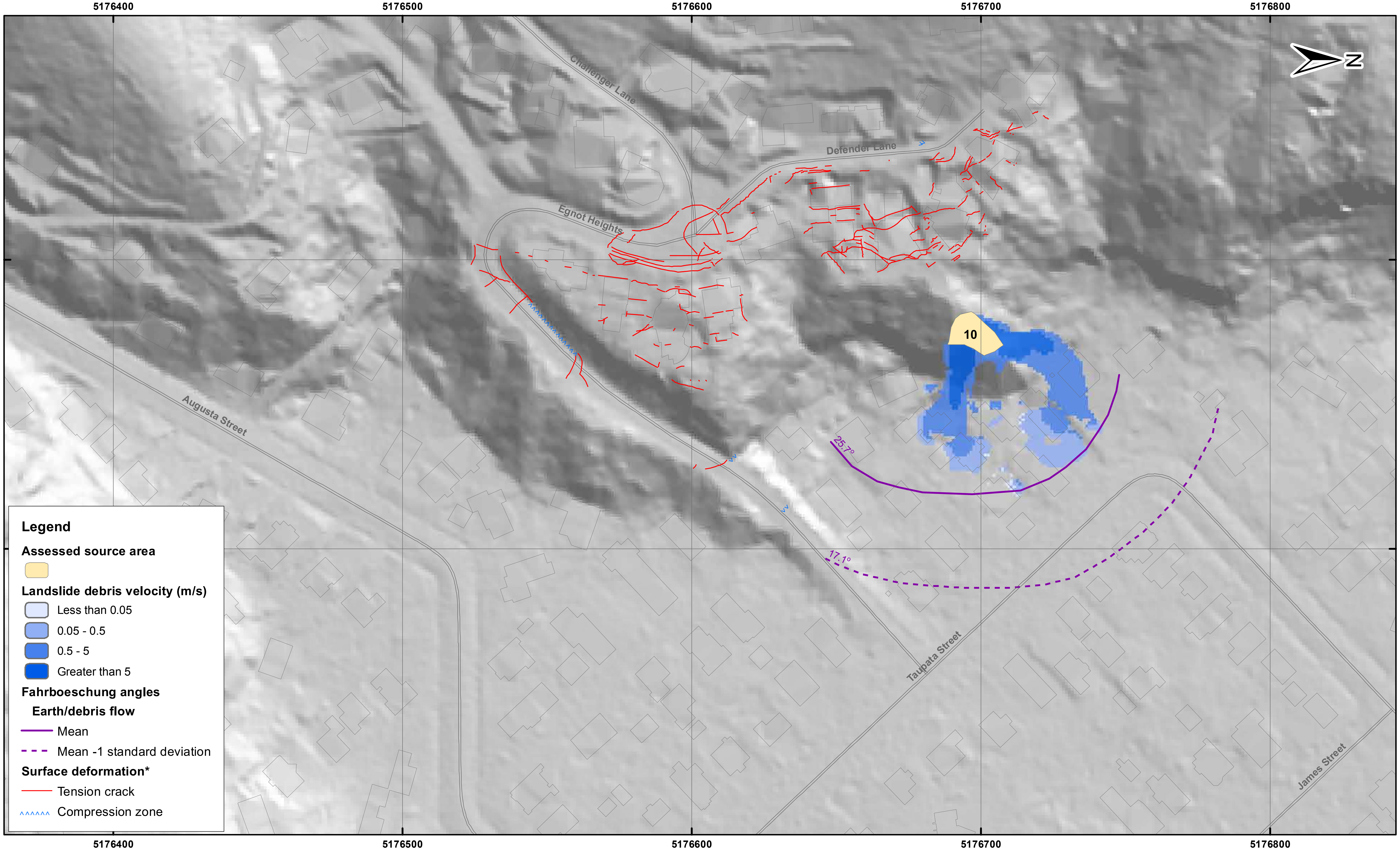
APPENDIX 6

Map 9

FINAL

REPORT: CR2014/67

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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Roads and building footprints and types provided by Christchurch City Council (20/02/2012).

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 10 - Middle Volume (110 m^3)

Defender Lane - Port Hills
Christchurch

APPENDIX 6

Map 10

FINAL

REPORT: CR2014/67

DATE: June 2014

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