Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Risk assessment for Maffeys Road

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DISCLAIMER

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REVIEW DETAILS

This report in draft form was independently reviewed by Dr L. Richards, T. Taig (TTAC Ltd.), and Dr J. Wartman. Internal GNS Science reviews of drafts were provided by N. Litchfield, M. McSaveney and R. Buxton.

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

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EXECUTIVE SUMMARY

ES 1 INTRODUCTION

This report combines recent field information collected from the Maffeys Road 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 Maffeys Road 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 3 investigations, presents the risk assessment results for the Maffeys Road Class I mass movement.

ES 2 INVESTIGATION PROCESS AND FINDINGS

Detailed investigations of the site and its history were carried out by URS Ltd. (Yetton and Engel, 2014). Yetton and Engel (2014) note evidence of historical and pre-historical landslide failure scars at the site.

The slopes were significantly cracked by the 22 February 2011 earthquakes, again by the 13 June 2011 earthquakes, but relatively little movement was observed in the other moderate sized earthquakes (e.g., 23 December 2011).

The absolute ground displacements at this site through the 2010/11 Canterbury earthquakes have been measured from survey markers installed prior to the earthquakes, which have enabled before-and-after measurements to be made. Total recorded permanent displacements are estimated to be slightly less than one metre horizontally.

The bulk strength of the loess slopes was weakened by cracking; and in particular, the presence of open surface cracks have made the slopes more susceptible to the ingress of run-off water, which is expected to weaken them further (possibly critically in a severe weather episode).

The main types of landslide hazard identified at the site are earth/debris flows originating from the loess slopes. By mapping cracks and relating these to the results of stability assessments, it has been possible to identify five potentially significant landslide 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 Maffeys Road assessment area, in particular the five potential landslide sources. Analyses have considered both:

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

ES2.1 Earth/debris flows

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 five 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 to reflect the associated uncertainties, and the overall annual individual fatality risk for nearby residents assessed.

ES 3 CONCLUSIONS

With reference to the assessed source area boundaries (Figures 1 and 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 1–5) of mixed loess and colluvium.
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. There are several dwellings located in the earth/debris flow runout zone, and at the crest of the assessed source areas. The annual individual fatality risk to dwelling occupants in the runout zone are estimated to range from greater than 10^{-4} to less than 10^{-4}, depending on the location of the dwelling with respect the location of the assessed source areas. Occupants of dwellings closest to the assessed source areas expose their occupants to the highest assessed levels of risk.
2. The uncertainties in the key input parameters used in the risk assessment combine to give an order of magnitude uncertainty in either direction. For example, the estimated risk at a dwelling within the estimated annual individual fatality risk of 10^{-4} per year uncertainty zone could feasibly be as low as 10^{-5}, or as high as 10^{-3}.

3. There are several dwellings located in the assessed source areas. It is difficult to assess what the levels of risk to the dwellings in the source areas are, given the uncertainties associated with the triggering event, source volume and area that could be affected. The levels of risk in the assessed source areas are likely to be similar to the highest values of risk estimated in the associated earth/debris flow runout zones.
4. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings from future earthquakes is still relatively high. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.4 m. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the National Seismic Hazard Model.

ES3.2.3 Risk management

1. A risk-management option of monitoring rainfall, soil moisture and pore-pressure in the source areas, may be of some value in providing warning of conditions approaching critical levels, but:
    a. Such early warning could not be assured, as experience in the Port Hills and elsewhere is that water levels in open tension cracks can rise very rapidly to critical values.
    b. There would be little time to evacuate potentially at-risk residents given the rapid nature of the hazard.
    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.

ES 4 RECOMMENDATIONS

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

ES4.1 Policy and planning

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

ES4.2 Short-term actions

ES4.2.1 Hazard monitoring strategy

1. Include the report findings in a slope stability monitoring strategy with clearly stated aims and objectives, and list how these would be achieved, aligning with the procedures described by McSaveney et al. (2014);
2. Ensure that the existing emergency management response plan for the area identifies the dwellings that could be affected by movement and runout, and outlines a process to manage a response.

ES4.2.2 Risk monitoring strategy

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

ES4.2.3 Surface/subsurface water control

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 water main currently runs along Maffeys Road at the crest of the assessed earth/debris flow source areas, and should if possible be either relocated away from this area or be designed to accommodate any future ground movement; 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

1. Assess the cost, technical feasibility and effectiveness of alternative longer term engineering and relocation solutions, for example (but not limited to):
    a. Removal/stabilisation of the slopes in the assessed source areas;
    b. Installation of drainage works;
    c. Relocation of houses to alternative locations within existing property boundaries;
    d. Withdrawal and rezoning of the land for non-residential use.
2. 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 Maffeys Road 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.

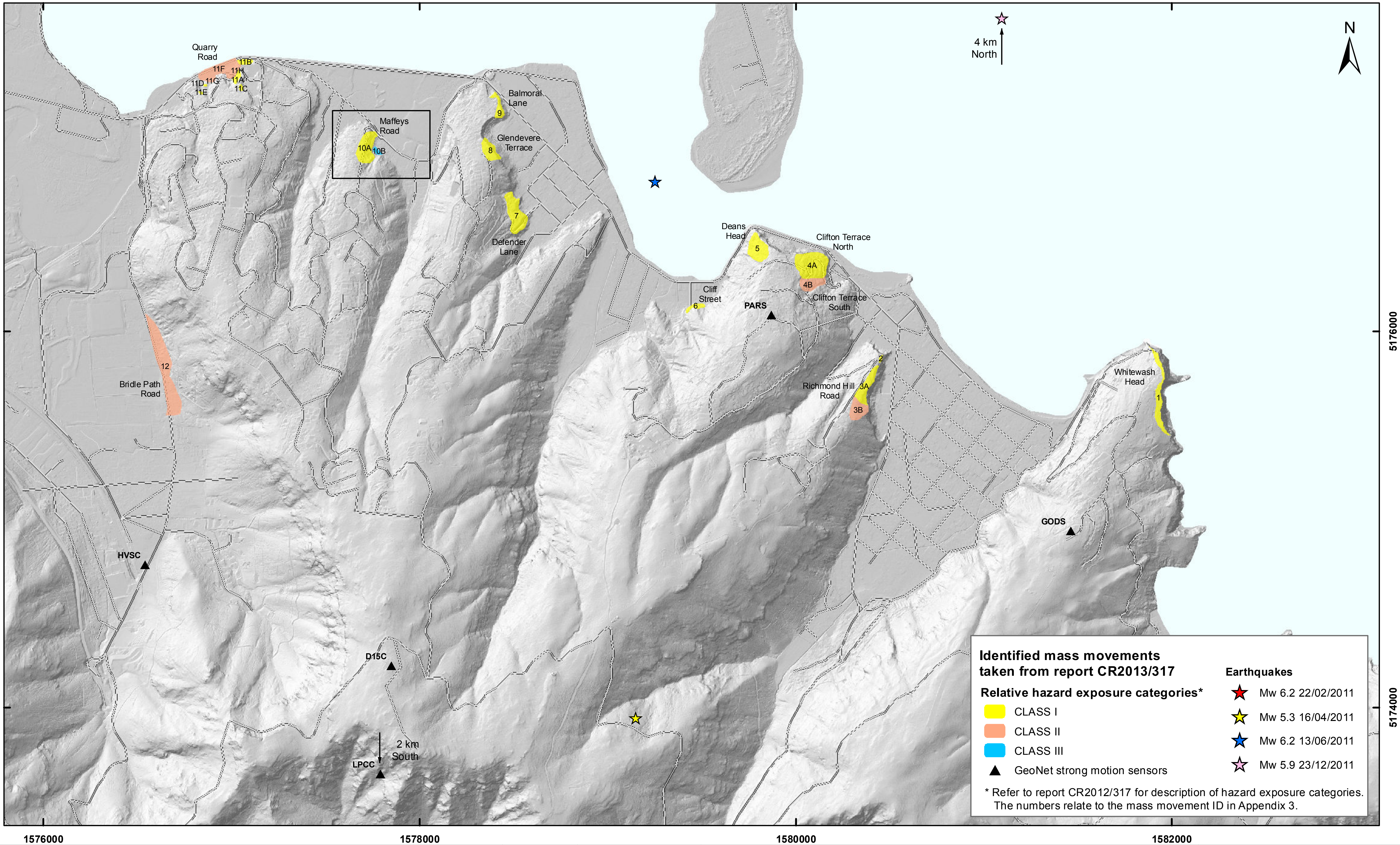
Christchurch City Council contracted GNS Science to carry out detailed investigations of the identified areas of mass movement, in order to assess the nature of the hazard, the frequency of the hazard occurring, and whether the hazard could pose a risk to life, a risk to existing dwellings and/or a risk 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) 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 Stages 2 and 3, which includes this Stage 3 report on the Maffeys Road 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

LOCATION MAP

Port Hills Christchurch

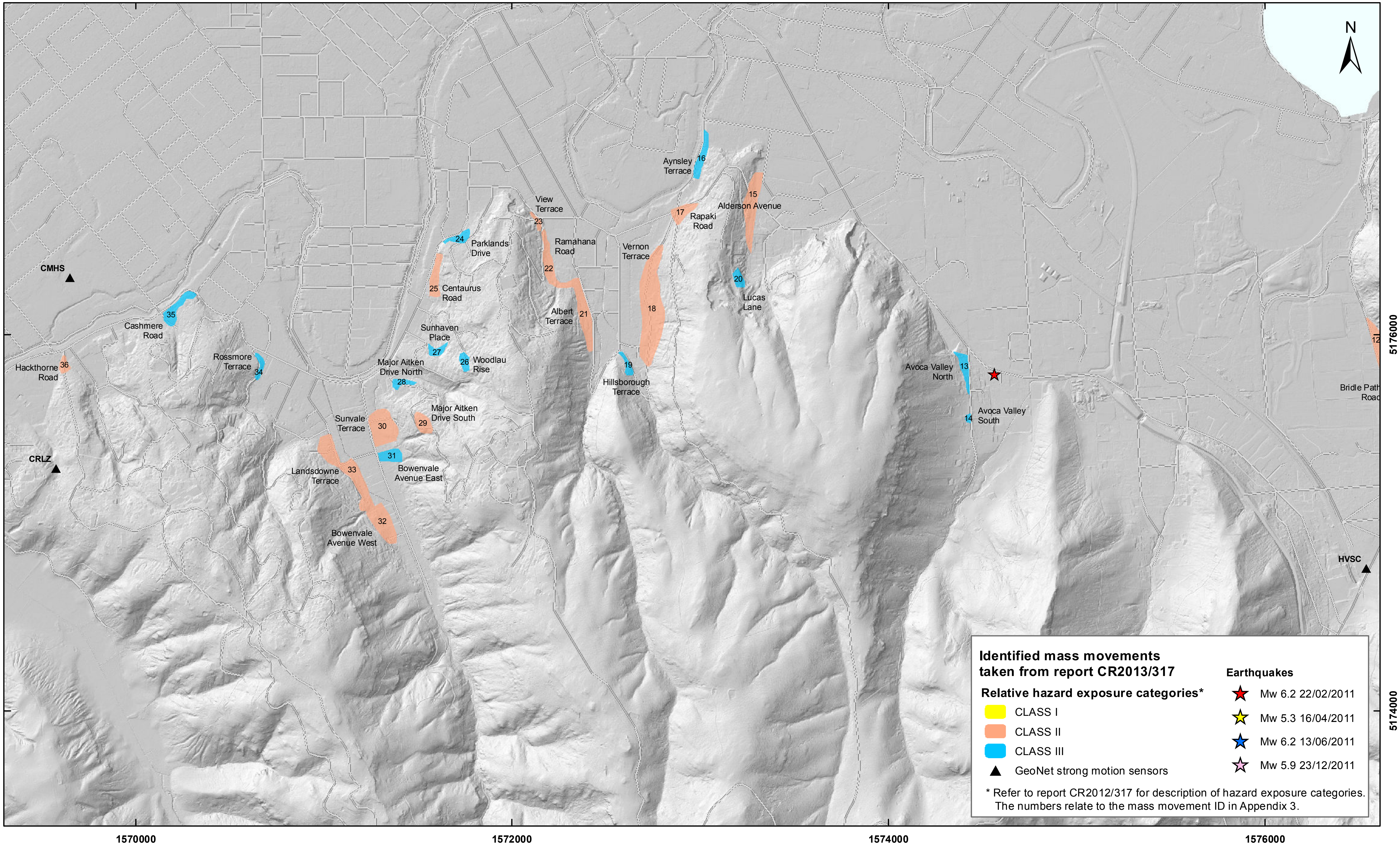
FIGURE 1

Map 1

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 0.5 1 km

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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 2

FINAL

REPORT: CR2014/79

DATE: June 2014

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

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

*Note: Displacements for each 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.

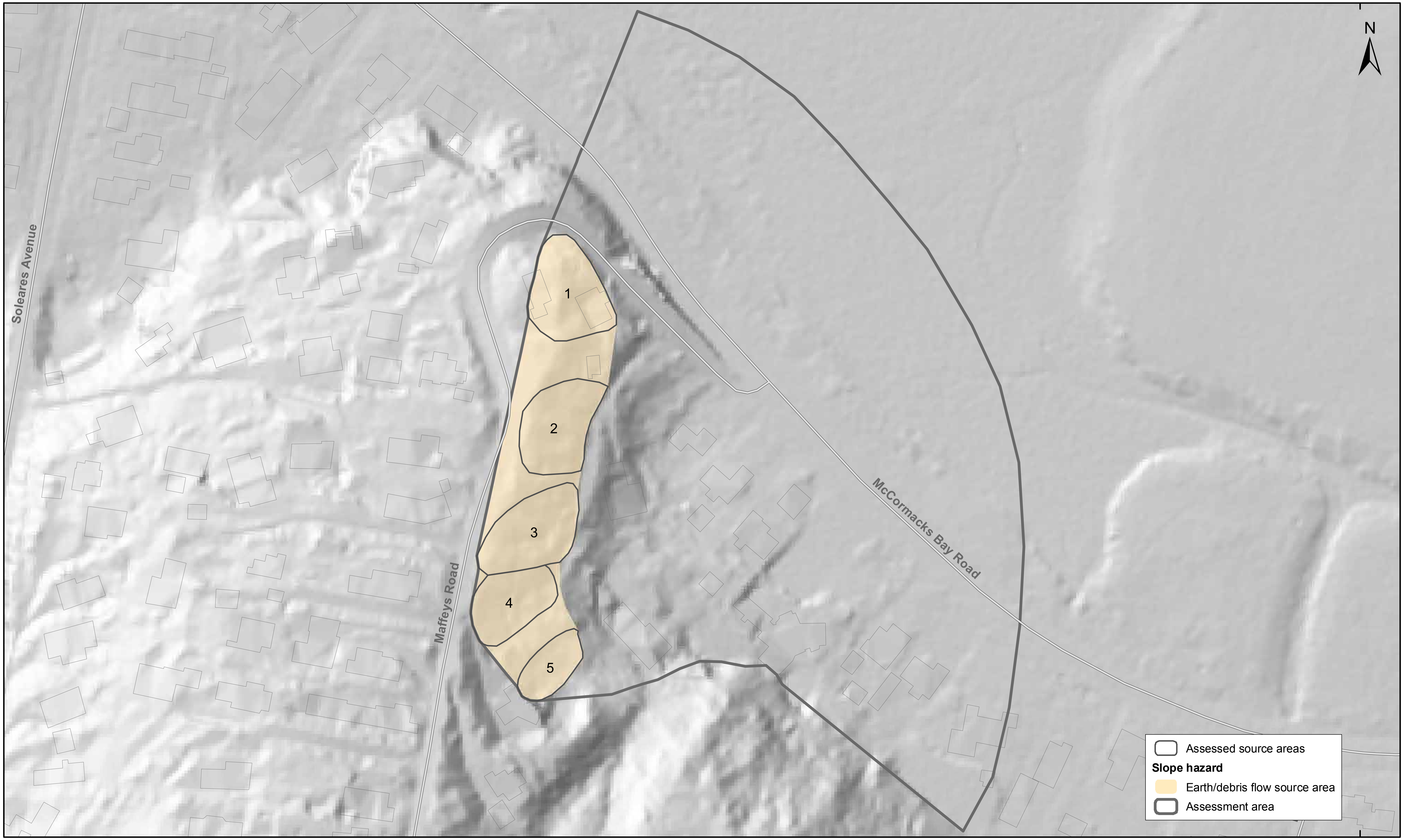
[1] Critical infrastructure is defined, for the purpose of this report, as infrastructure vital to public health and safety. It includes transport routes (where there is only one route to a particular destination), telecommunication networks, all water related mains and power networks (where there is 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 MAFFEYS ROAD MASS MOVEMENT

The Maffeys Road assessment area (mass movement number 8 in the Stage 1 report) is shown in Figures 1, 2 and 3. This mass movement 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 earth/debris flows from the five assessed source areas identified within the Maffeys Road mass movement. The map in Figure 2 outlines the assessed potential landslide source areas within the assessment area, while shows the approximate boundary of the assessment area. Figure 4 shows a view looking along Maffeys Road.



SCALE BAR: 0 50 m

EXPLANATION:

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
Roads 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

Maffeys Road
Christchurch

FIGURE 2

FINAL

REPORT: CR2014/79

DATE: June 2014

Figure 3 Aerial view, looking west, of the Maffeys Road assessment area. The approximate extent of the assessment area is shown by the yellow dashed lines, the actual mapped extent of the assessment area is shown on Figure 2). Photograph taken by M. Yetton (2013).

Figure 4 Aerial view, looking south, of the Maffey Road mass movement. Photograph taken by M. Yetton (2013).

1.3 PREVIOUS WORK AT THE MAFFEYS ROAD SITE

Following the 22 February 2011 earthquakes, significant localised cracking was noted on the slope surface within the Maffeys Road mass movement. Previous investigations of the site comprised:

1. 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).
2. Ground investigation of the site was carried out by Tonkin and Taylor Ltd, under contract to the Earthquake Commission (Tonkin and Taylor, 2012a).
3. Further ground investigation of the site as carried out by URS New Zealand Ltd., for Christchurch City Council. This involved drilling, field mapping and groundwater and drillhole inclinometer monitoring.

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 risk assessment results contained in this report supersede the preliminary risk assessment results contained in the Working Note 2013/07 (Massey and Della Pasqua, 2013).

1.5 REPORT STRUCTURE

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

1.6 METHODS OF ASSESSMENT

The site assessment comprised three stages:

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

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 URS New Zealand Ltd. (Yetton and Engel, 2014), in consultation with GNS Science.

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.

1.6.2.1 Estimation of landslide volumes

The results of the URS Ltd. engineering geological assessments (Yetton and Engel, 2014) and the slope stability modelling carried out by GNS Science have been used to define five potential landslide source areas. 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.

1.6.3 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:

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

where:

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

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

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

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

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

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

1.6.3.1 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 National Seismic Hazard Model of New Zealand (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 1946, 1975, 1975 and 1984 by various agencies and orthorectifed by NZ Aerial Mapping and GNS Science (at variable ground resolutions).	NZ Aerial mapping and GNS Science	1946, 1975, 1975 and 1984	Used to assess the site history before the 2010/11 Canterbury earthquakes.
LiDAR digital elevation model (2011c)	Digital Elevation Model derived from post-13 June 2011 earthquake LiDAR survey; re-sampled to 1 m ground resolution.	NZ Aerial Mapping	18 July–26 August 2011	Used to generate contours and shade models for the maps and cross-sections used in the report.
Christchurch building footprints	Footprints are derived from aerial photographs. The data originate from 2006 but have been updated at the site by CCC using the post-earthquake aerial photos.	Christchurch City Council	Unknown	Used to identify the locations of residential buildings in the site.
GNS Science landslide database	Approximate location, date, and probably trigger of newsworthy landslides	GNS Science	Updated monthly	Used to estimate the likely numbers and volumes of pre-earthquake landslides in the areas of interest.
Earthquake Commission claims database	Location, date and brief cause of claims made in the Port Hills of Christchurch since 1993.	Earthquake Commission	1993–August 2010	Used to estimate the likely numbers and volumes of pre-earthquake landslides in the areas of interest.

Data	Description	Data source	Date	Use in this report
Synthetic earthquake time/ accelerations	Earthquake time acceleration history's for the four main 2011 earthquakes: 22 February, 16 April, 13 June and 23 December.	GNS Science	February 2014	Used as inputs for the seismic site response analysis.
Rainfall records for Christchurch	Rainfall records for Christchurch from various sources, extending back to 1873.	NIWA	1873–present	Used to assess the return periods of past storms triggering landslides of known magnitudes in the Port Hills.
Downhole shear wave surveys	Downhole shear wave velocity surveys carried out in the URS Ltd. drillholes.	Southern Geophysical Ltd. (2013)	February 2014	Used to determine the dynamic properties of the materials in the slope for the seismic site response analysis.
Geotechnical laboratory data	Geotechnical strength parameters for selected soil and rocks in the Port Hills.	GNS Science	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.	URS Ltd, GNS Science and the Port Hills Geotechnical group	22 February 2011–present	Used in generating the engineering geological models of the site. Results from field checks used to update risk maps.
Port Hills Land Damage Studies Maffeys Road Field Investigations	Field mapping, aerial photograph interpretation, interpretation of drillholes and assessment of engineering geological slope hazards present at the site	URS Ltd.	Final report May 2014	Used as the basis for the hazard and risk assessments.

3.0 SITE ASSESSMENT RESULTS

The site assessment and engineering geological conceptual models for the site were developed by URS Ltd. in consultation with GNS Science. The URS Ltd. engineering geological map, cross-sections and conceptual model for the site are presented in Figures 5 and 6. These figures are reproduced from Yetton and Engel (2014), and the main results are summarised below.

3.1 SITE HISTORY

3.1.1 Before the 2010/11 Canterbury earthquakes

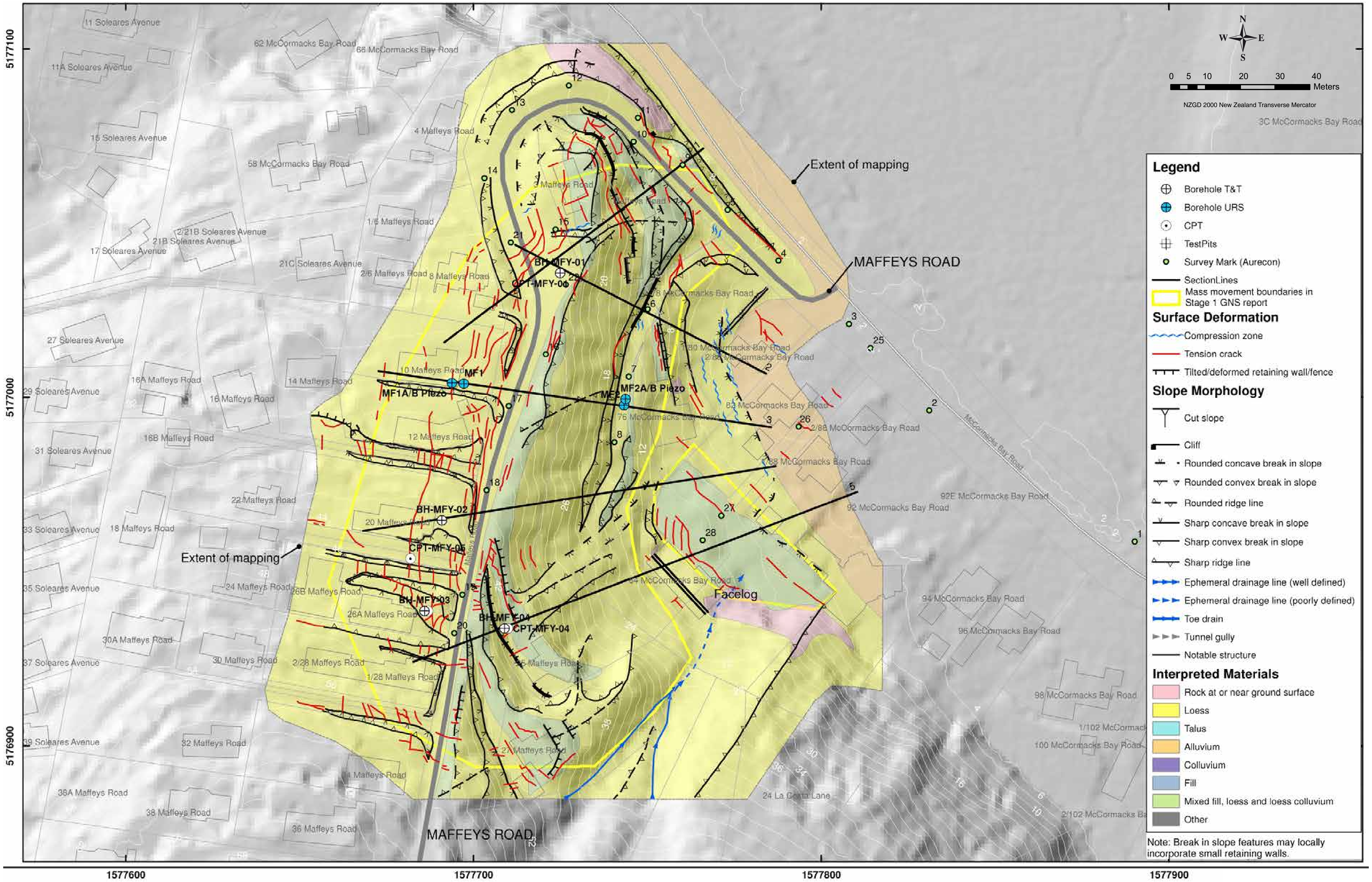
- No large-scale slope deformation had been recorded since European settlement (ca. 1840 AD).
- The morphology of the area comprises an old embayment within the now abandoned coastal cliffs, which might be a relict landslide scar.
- Geomorphic expression of several small relict landslide scars within the larger former embayment along the steep slopes below Maffeys Road, are apparent in the earliest available aerial photographs (1946).
- Presence of a possible debris fan extending from the drainage line above 84 McCormacks Bay Road, towards 82, 1/88 and 2/88 McCormacks Bay Road.
- In 2008, a debris flow from 25 Maffeys Road was triggered by water from a pipe break. This earth/debris flow damaged 86 McCormacks Bay Road (now demolished).
- No cracking reported or observed following 4 September 2010 earthquake.

3.1.2 During the 2010/11 Canterbury earthquakes

- *22 February 2011 earthquakes* – cracks (shown on the map in Figure 5), were mainly generated on 22 February 2011 by one or more earthquakes that occurred on that day. Permanent displacement of the area, inferred from crack apertures and the results of surveying of cadastral survey marks, was in the order of about 0.5 m (Table 3).
- *16 April 2011 earthquake* – No displacements were reported to GNS Science.
- *13 June 2011 earthquakes* – Permanent displacement of the area in response to the earthquakes on 13 June 2011, inferred from the results of surveying of monitoring marks (by Aurecon New Zealand Ltd.), was in the order of about 0.1 m.
- *23 December 2011 earthquake* – Permanent displacement of the area in response to the earthquake on 23 December 2011, inferred from the results of surveying of monitoring marks (by Aurecon New Zealand Ltd.), was in the order of about 0.01 m, and just large enough to be indicative of permanent slope displacement (i.e., just outside survey error).

3.1.3 After the 2010/11 Canterbury earthquakes

- In the northern part of the slope, post-earthquake displacement of the area occurred in response to retaining-wall excavation at the junction of Maffeys Road and McCormacks Bay Road, below 11 Maffeys Road.
- Very slow, post-earthquake creep at the interface between the loess and rock is suggested by survey and inclinometer data (MFY02 only, Figure 5).



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

DRW: PC

CHK: CM

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

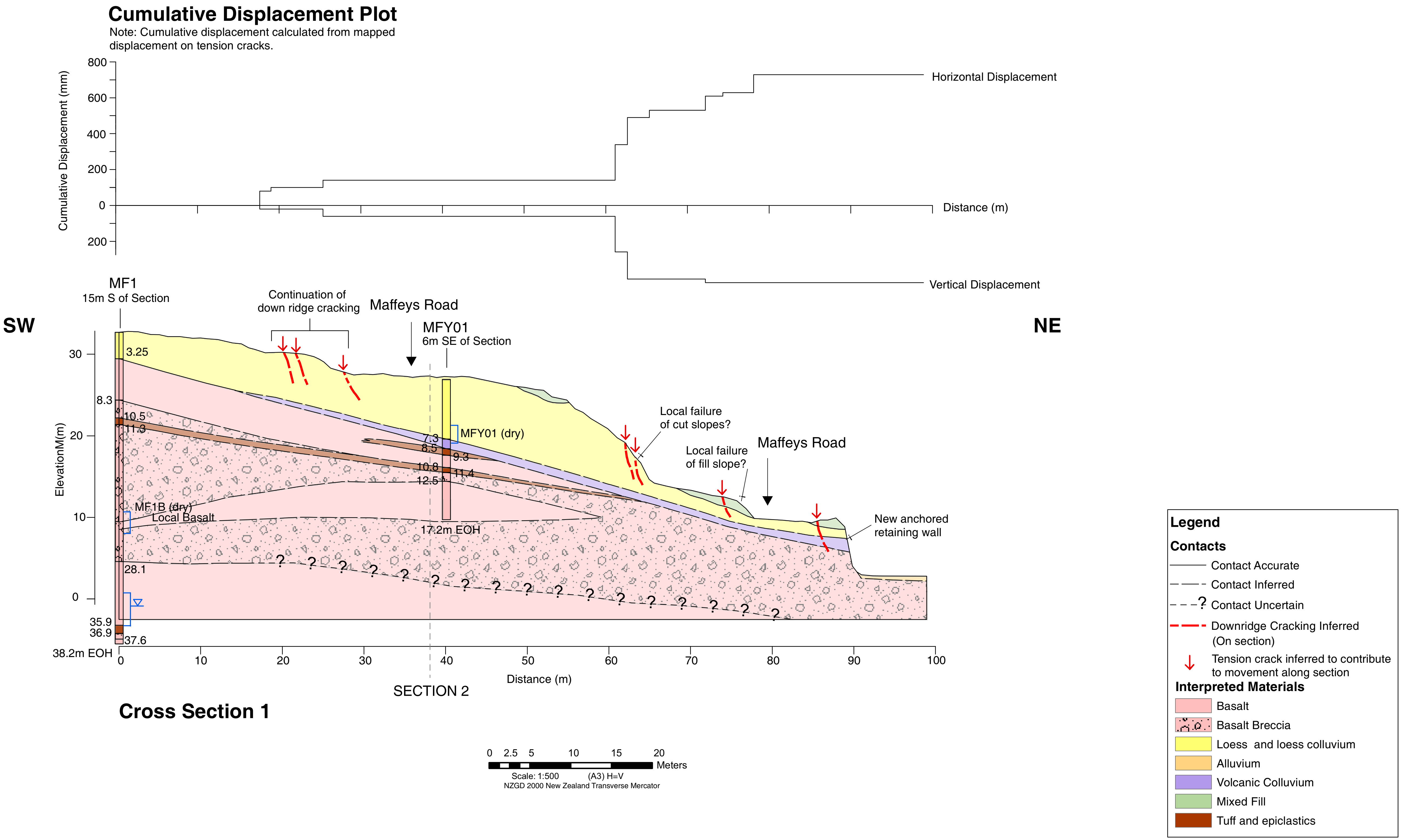
Maffeys Road
Christchurch

FIGURE 5

FINAL

REPORT: CR2014/79

DATE: June 2014



Note: Groundwater levels shown as at 3/10/2013

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

DRW: PC

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

Maffeys Road
Christchurch

FIGURE 6A

FINAL

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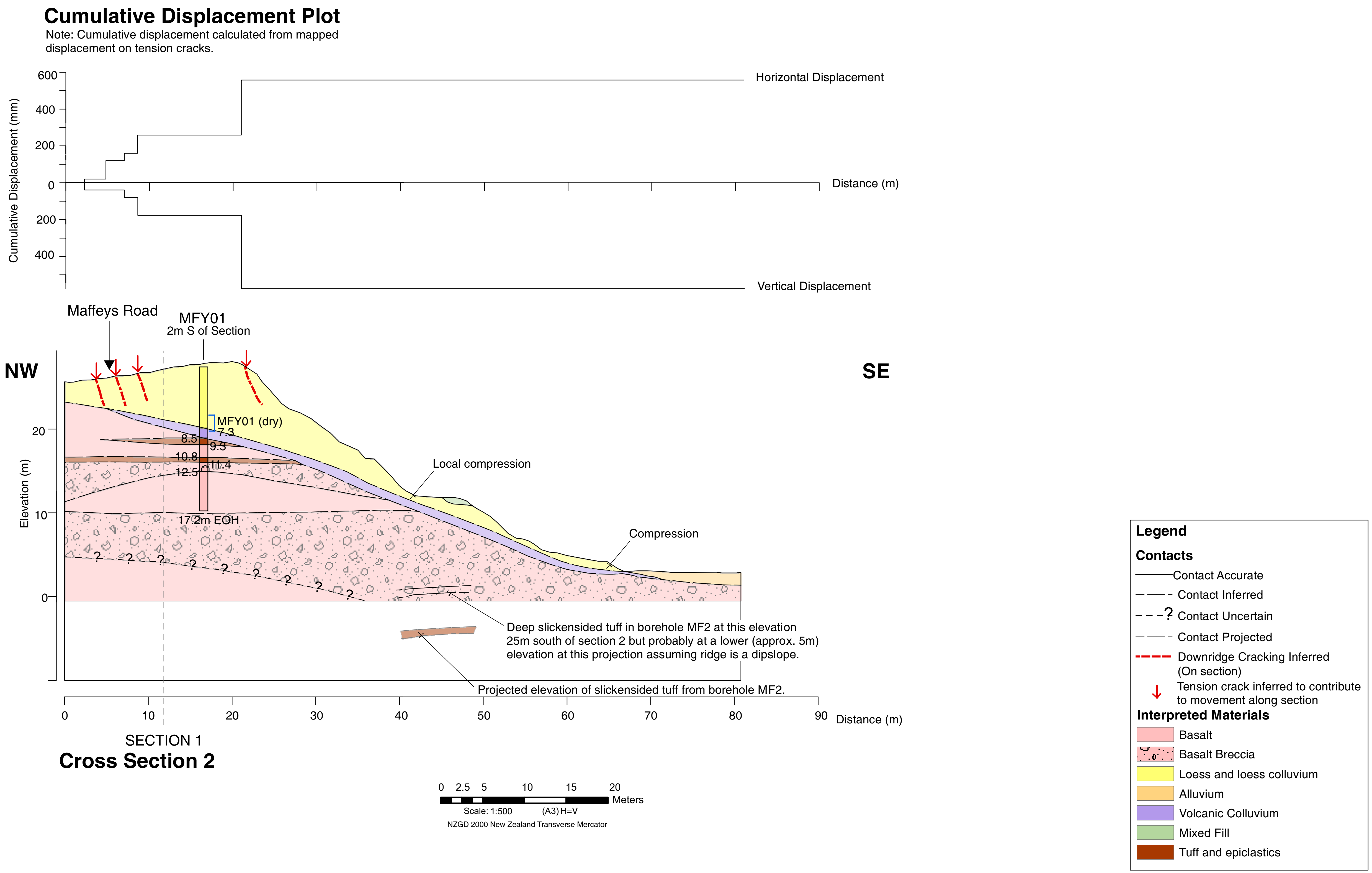

Note: Groundwater levels shown as at 3/10/2013

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

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Maffeys Road
Christchurch

FIGURE 6B

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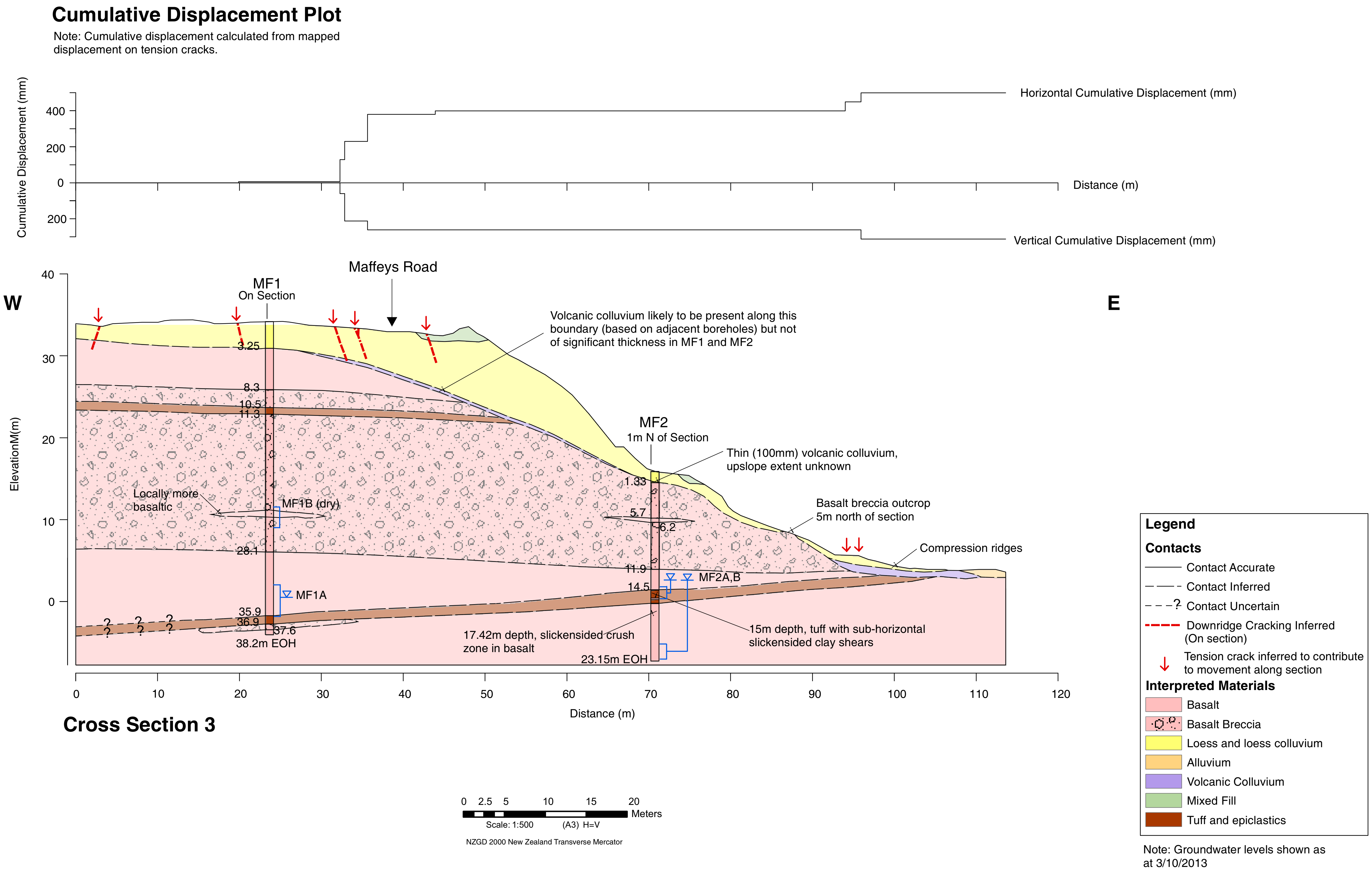


The drawing is reproduced from URS (2014), Port Hills land damage studies Quarry Road field investigations. Report prepared for Christchurch City Council. (Yetton, M. and Engel, M. 2014).

DRW: PC

CHK: CM

URS LTD. ENGINEERING GEOLOGY CROSS SECTION 3
Reproduced from Yetton and Engel (2014)

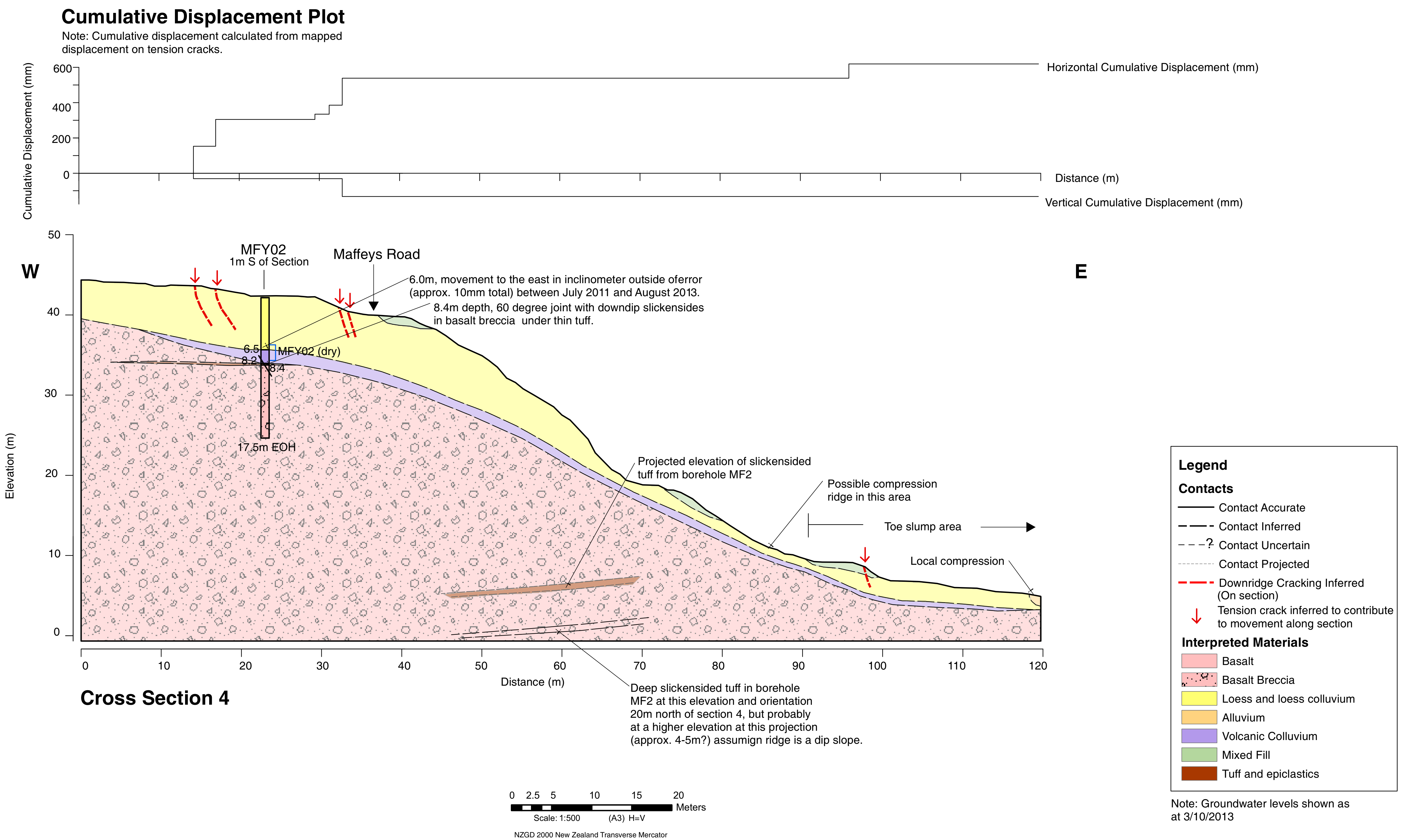
FIGURE 6C

Maffeys Road
Christchurch

FINAL

REPORT: CR2014/79

DATE: June 2014



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

DRW: PC

CHK: CM

URS LTD. ENGINEERING GEOLOGY CROSS SECTION 4
Reproduced from Yetton and Engel (2014)

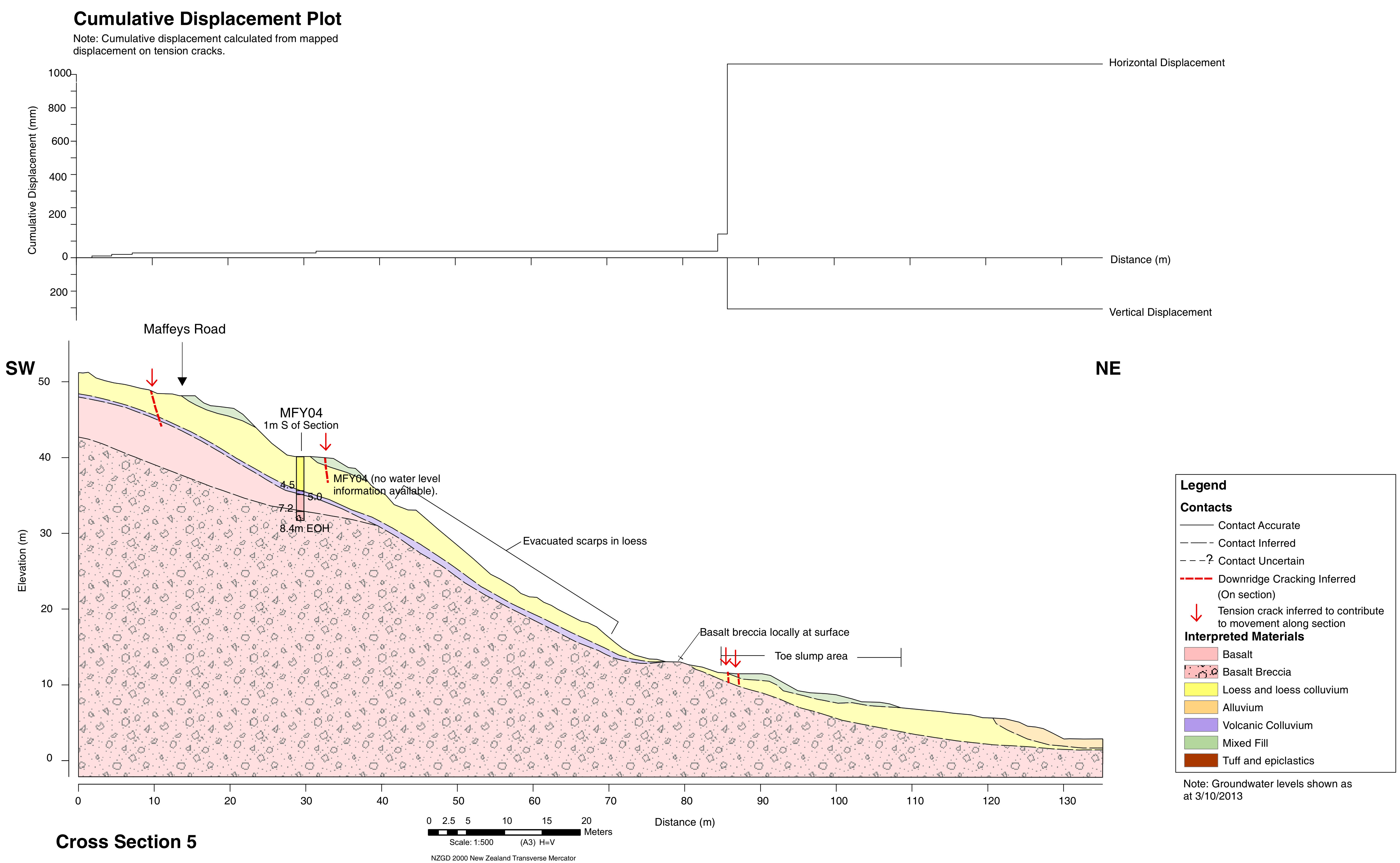
FIGURE 6D

Maffeys Road
Christchurch

FINAL

REPORT: CR2014/79

DATE: June 2014



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

DRW: PC

CHK: CM

URS LTD. ENGINEERING GEOLOGY CROSS SECTION 5
Reproduced from Yetton and Engel (2014)

FIGURE 6E

Maffeys Road
Christchurch

FINAL

REPORT: CR2014/79

DATE: June 2014

3.2 SITE INVESTIGATIONS

3.2.1 Geomorphological mapping

The results from field mapping of slope morphology, interpreted surface materials and their genesis and surface deformation mapping carried out by URS Ltd. (Yetton and Engel, 2014) are shown in Figures 5 and 6.

3.2.2 Subsurface trenching and drilling

The following site investigations have been undertaken by URS Ltd. (Yetton and Engel, 2014) for this report (see Figure 5 for locations):

- Diamond core drilling with core recovery (drillholes MF1 and MF2);
- Non-core drilling (drillholes MF1A/B and MF2A/B);
- Installation of four standpipe piezometers in two drillholes (MF1A/B and MF2A/B);
- Installation of two inclinometer tubes in drillholes (MF1 and MF2).

Previous site investigations were carried out by Tonkin and Taylor Ltd (Tonkin and Taylor, 2012a) for the Earthquake Commission. These comprised:

- Diamond core drilling with core recovery (drillholes BH MFY01-04);
- Cone pentrometer probing (CPT MFY01-05);
- Installation of four standpipe piezometers in two drillholes (BH MFY01-02 and CPT MFY03-04); and
- Installation of two inclinometer tubes in drillholes (CPT MFY01-02).

3.2.3 Surface movement

3.2.3.1 Inferred slope displacement from crack apertures

Total cumulative displacement of the slope inferred from crack apertures along cross-sections 1–5, in response to the 2010/11 Canterbury earthquakes, is in the order of about 0.1–0.8 m (Table 3).

Table 3 Measured cumulative crack apertures, which formed mainly during the 22 February, and less so during the 13 June 2011 earthquakes, measured by the Port Hills Geotechnical Group (Yetton and Engel, 2014). Displacements were measured from field mapping of tension crack apertures along survey lines. Errors are nominally estimated as being ±0.01 m.

URS cross-section	Cumulative vertical component (crest) (mm)	Cumulative horizontal component (crest) (mm)	Resultant vector		Apparent dip along cross-section (°)
			Magnitude (mm)	Dip (°)	
1	430	730	847	30	~15
2	585	565	813	46	~20
3	260	390	469	34	~20
4	130	525	541	14	~20
5	0	50	50	0	~30

3.2.3.2 Surveyed slope displacements

The survey monitoring data is presented in Appendix 2 and is summarised from Yetton and Engel (2014) below. There are two datasets:

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.

Cadastral survey marks (source: LINZ)

Available cadastral survey marks were measured by GNS Science to detect absolute ground movements spanning the earthquake period from before the 22 February 2011 earthquakes to 16 April 2013. The measurements therefore include any displacement of the survey marks in response to the earthquakes within this time period – tectonic displacements were removed from the recorded displacements.

The results of this survey are contained in Appendix 2. Vector displacements indicate permanent ground displacements, within the middle part of the mass movement, that range up to 530 mm towards bearing 065° (east) (e.g., cadastral mark ID 40, Map 2 Appendix 2).

The largest recorded displacements of 694 mm towards bearing 080° (Cadastral mark ID 46, Map 2 Appendix 2) were at the bottom of Maffeys Road, adjacent to the area of the recently constructed retaining wall along McCormacks Bay Road (below Maffeys Road).

Calculated displacements from the cadastral survey marks are summarised in Table 4.

Monitoring marks (source: Aurecon NZ Ltd.)

The displacements calculated using the Aurecon survey data span the time period 26 May 2011–10 April 2013 and there are approximately 30 observations per mark. The dates covered and the numbers of observations vary per survey mark.

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

From the survey time series relating to each mark it has been possible to determine the magnitudes and bearings of any displacement caused by these two earthquakes. The largest total displacements calculated from the survey marks in the middle part of the mass movement (monitoring mark ID 17, Maps 3 and 4, Appendix 2), were about:

- 155 mm towards bearing 080° on 13 June 2011; and
- 31 mm towards bearing 080° on 23 December 2011.

Both magnitudes of displacements are well outside the estimated survey error, and therefore represent permanent displacement of the slope. Survey errors are shown as error ellipses on the maps in Appendix 2. Calculated displacements from the cadastral survey marks are summarised in Table 4.

Estimating the 22 February 2011 displacements

No data are available that directly measures the main movement that occurred on 22 February 2011 and caused virtually all the observed ground damage. However, it is possible to estimate the likely amount of displacement of the main cracked area (outside No. 20 Maffeys Road) during the 22 February 2011 earthquake, by subtracting the combined displacement (13 June and 23 December 2011) estimated from the monitoring marks (surveyed by Aurecon NZ Ltd.), from the total displacement estimated from the cadastral surveys as follows:

Measured displacement of the cadastral survey mark ID 40 (Map 2, Appendix 2) in the middle of the mass movement (for all events) is 530 mm. The sum of measured displacements (of the nearest monitoring mark (ID 18, Maps 3 and 4, Appendix 2) recorded during the 13 June and 23 December 2011 earthquakes is 115 mm. Therefore the estimated 22 February 2011 displacement in this area is approximately 415 mm (530 mm minus 115 mm) (or between 410 and 420 mm, taking into account survey error).

This suggests that the majority of the displacement of the main part of the cracked area was in the 22 February 2011 earthquake(s). This assumes that displacement caused by the 16 April 2011 earthquake or other smaller events in this period was negligible.

Table 4 Summary of slope displacements inferred from crack apertures and the surveying of cadastral and monitoring marks installed on the slope.

Date	Survey type	Slope crest Maffeys Road (southern end)[1]	Slope crest Maffeys Road (middle)	Bottom of Maffeys Road (northern end)
22 February 2011–16 April 2013	Cadastral marks	38 mm* (36) 33 mm* (42) 19 mm* (43)	532 mm (40)	138 mm (44) 164 mm (45) 694 mm (46) 159 mm (47)
22 February 2011	Crack apertures	50 mm (cross-section 5)	470 mm (cross-section 3) 540 mm (cross-section 4)	850 mm (cross-section 1)
22 February 2011	Inferred from survey of cadastral and monitoring marks	0–15 mm	340–480 mm	120–690 mm
13 June 2011	Monitoring marks	26 mm (19) 25 mm (20)	102 mm (18) 155 mm (17) 41 mm (16)	15 mm* (15)
23 December 2011	Monitoring marks	Not surveyed (19) Not surveyed (20)	13 mm* (18) 31 mm (17) 10 mm* (16)	Not outside error

1 The ID of the survey mark is given in brackets; the locations are shown in Appendix 2.

* Indicates measurements outside the 95% confidence error of the survey.

3.2.4 Subsurface movement (inclinometers)

Drillhole inclinometer tubes were used to: 1) monitor displacements at depth; 2) assess whether movement was occurring along single or multiple slide-surfaces; and 3) to independently verify the results of surface monitoring. Monitoring is undertaken manually by commercial contract (Geotechnics Ltd.).

Inclinometer tubes were installed in drillholes MF1 and MF2 (Yetton and Engel, 2014) and CPT MFY01 and MFY02 (Tonkin and Taylor, 2012d). The inclinometer displacements are monitored at 0.5 m intervals and the inclinometer accuracy is quoted as ±6 mm over 25 m of tubing (Slope Indicator, 2005). The measurement details are summarised in Table 5.

The inclinometers installed in drillholes CPT MFY01, MF1 and MF2 show no subsurface displacements of the inclinometer tubes outside survey error, within the period between the initial reading and the most recent survey (Table 5).

The inclinometer in CPT MFY02 has a deflection in the monitoring tube between the 5.75 to 6.75 m (below the collar elevation) interval. This deflection is about 3 mm towards bearing 088° in the tilt change plot, with a cumulative displacement of about 5 mm in the profile change plot. The deflection has been recorded in multiple surveys, and is outside the error of the surveys. It was first identified in the survey on 24 December 2011 (between the 12 October and 24 December 2011 surveys), one day after the 23 December 2011 earthquake.

The depth of movement is coincident with the volcanic colluvium-rock (rockhead) interface, and the direction of movement is down dip on this surface and out of the slope. The magnitude and direction of the inclinometer displacement is consistent with the magnitude and bearing inferred from the surface monitoring marks in response to the 23 December 2011 earthquake (Appendix 2).

Table 5 Summary of drillhole inclinometer surveys.

Drillhole ID Measuring date	**CPT MFY01** (3 Maffeys Road)	**CPT MFY02** (20 Maffeys Road)	**MF1** (10 Maffeys Road)	**MF2** (78 McCormacks Road)
25/08/2011	Base reading	Base reading	Not yet installed	Not yet installed
20/10/2011	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
24/12/2011	No inferred landslide movement[1]	Displacement detected at approx. 6.75 m depth towards bearing 088 (east)[1]. Corresponds to volcanic colluvium layer in the drillhole logs (Yetton and Engel, 2014).	Not yet installed	Not yet installed
30/05/2012	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
13/12/2012	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
12/03/2013	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
18/06/2013	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
9/08/2013	No inferred landslide movement[1]	No inferred landslide movement[1]	Not yet installed	Not yet installed
19/09/2013	Not surveyed	Not surveyed	Base reading[2]	Base reading[2]
12/03/2014	No inferred landslide movement[2]	No inferred landslide movement[2]	Not surveyed	Not surveyed
24/03/2014	Not surveyed	Not surveyed	No inferred landslide movement[2]	No inferred landslide movement[2]

[1] Source: Geotechnics Ltd Report 720273.000/RPT/1, 13 August 2013

[2] Geotechnics Ltd Report 720085.000/RPT, 3 April 2014.

3.2.5 Groundwater

Standpipe (Casagrande) piezometers were installed in drillholes carried out by Tonkin and Taylor and URS Ltd. The Tonkin and Taylor piezometers were shallow and targeted the loess and volcanic colluvium. The URS Ltd. piezometers were deeper and targeted specific horizons within the rock. The installation details are summarised in Table 6.

Table 6 Summary of the piezometer installations at Maffeys Road.

Drillhole ID	Data source	Piezometer response zone (m below drillhole collar)		Material	Piezometric head level (m below drillhole collar)
		From	To		
BH MFY01	Tonkin and Taylor	5.6	7.6	Loess	7.6 (dry)
BH MFY02	Tonkin and Taylor	4.2	6.2	Loess	6.2 (dry)
CPT MFY03	Tonkin and Taylor	2.9	4.9	Loess	5.89 recorded but base of CPT hole recorded at 4.9.
CPT MFY04	Tonkin and Taylor	1.6	2.6	Loess	4.6 recorded but base of CPT at 2.9 (dry)
MF1 A (upper)	URS Ltd.	20	24	Basalt breccia	Dry
MF1 B (lower)	URS Ltd.	30	36	Epiclastics (tuff)	33.5
MF2 A (upper)	URS Ltd.	13.5	17	Epiclastics (tuff)	13.2
MF2 B (lower)	URS Ltd.	21	23.2	Basalt lava	13

3.2.5.1 Shallow piezometers (source: Tonkin and Taylor)

Three of the four shallow piezometers in the loess and volcanic colluvium have been reported by Tonkin and Taylor as being dry, with the exception of CPT MFY03. However, the depth of the piezometer CPT MFY03 shown in the report (Tonkin and Taylor, 2012d) does not match the depth of the water levels reported in the graphs within the same report. Initial readings showed this piezometer to be dry, but subsequent readings showed a piezometric head level of 5.89 below the elevation of the collar, where the base of the piezometer response zone in CPT MFY04 is reported as being at 2.6 m below ground level. It is possible that these piezometric head measurements were made in the adjacent drillhole BH MFY04 (Yetton and Engel, 2014), which would place the recorded piezometric head level, of 5.89 m below the collar elevation, at the same level as the lava and about 1 m below rockhead. In light of the uncertainties associated with these records, the data is not considered fit for purpose.

Monitoring data from these piezometers comprised the manual measurement of water levels in the standpipes. Approximately 12 measurements or less were made over the reporting period 3 August 2011–17 July 2012 (Tonkin and Taylor, 2012d), indicating a poor temporal resolution. No more recent data have been provided to GNS Science. It is possible that ephemeral groundwater is present, but the poor temporal resolution of the monitoring has not captured the response.

3.2.5.2 Deep piezometers (Source: URS Ltd.)

The piezometers in the drillholes MF1 and MF2 were installed to check the bedrock water levels. During drilling, consistent water loss in the volcanic materials was noted by the driller during the drilling of MF1. In MF1 the deeper piezometer currently has an automated recorder (‘diver’) installed that takes hourly readings and shows water level at approximately 33.5 m, which is about 2 m above mean sea level. The water level fluctuates daily by small amounts, which may be a tidal response (Yetton and Engel, 2014).

MF2 has two piezometers installed. The upper piezometer has a response zone corresponding to the tuff layer with clay seams and slickensides at 15–17.5 m depth (below the collar elevation of the hole). The water level, which is monitored with a ‘diver’ (hourly readings), has been dropping slowly from the time of drilling, but now appears to have stabilised at about 13.2 m. The deeper piezometer MF2, with a response zone at the bottom of the hole, is manually monitored, but readings show a water level of approximately 13 m (below the collar elevation of the hole), which is consistent with the levels recorded in the upper piezometer in MF2.

3.3 ENGINEERING GEOLOGICAL MODEL

3.3.1 Slope materials

3.3.1.1 Loess

The loess mantling the slope at Maffeys Road is similar to that found in other areas of the Port Hills. It is a relatively cohesive silt-dominated soil with only minor clay mineral content. Its strength is largely controlled by the soil moisture content and this has been well studied, e.g., Bell et al. (1986), Bell and Trangmar (1987), McDowell (1989), Goldwater, (1990), Yetton (1992) and Carey et al. (2014). The loess is highly hydrophyllic and when exposed to water (rain) it quickly disintegrates into muddy silt.

3.3.1.2 Volcanic colluvium

The volcanic colluvium forms a thin layer of variable thickness underlying the loess across much of the site. It mainly comprises clay and silty gravel, and represents the deposits of debris from past landslides and other erosion processes prior to the main period of loess deposition. The material derives mainly from weathered volcanic breccia and lava and less so from the loess, although remobilised loess is present in varying proportions (Yetton and Engel, 2014).

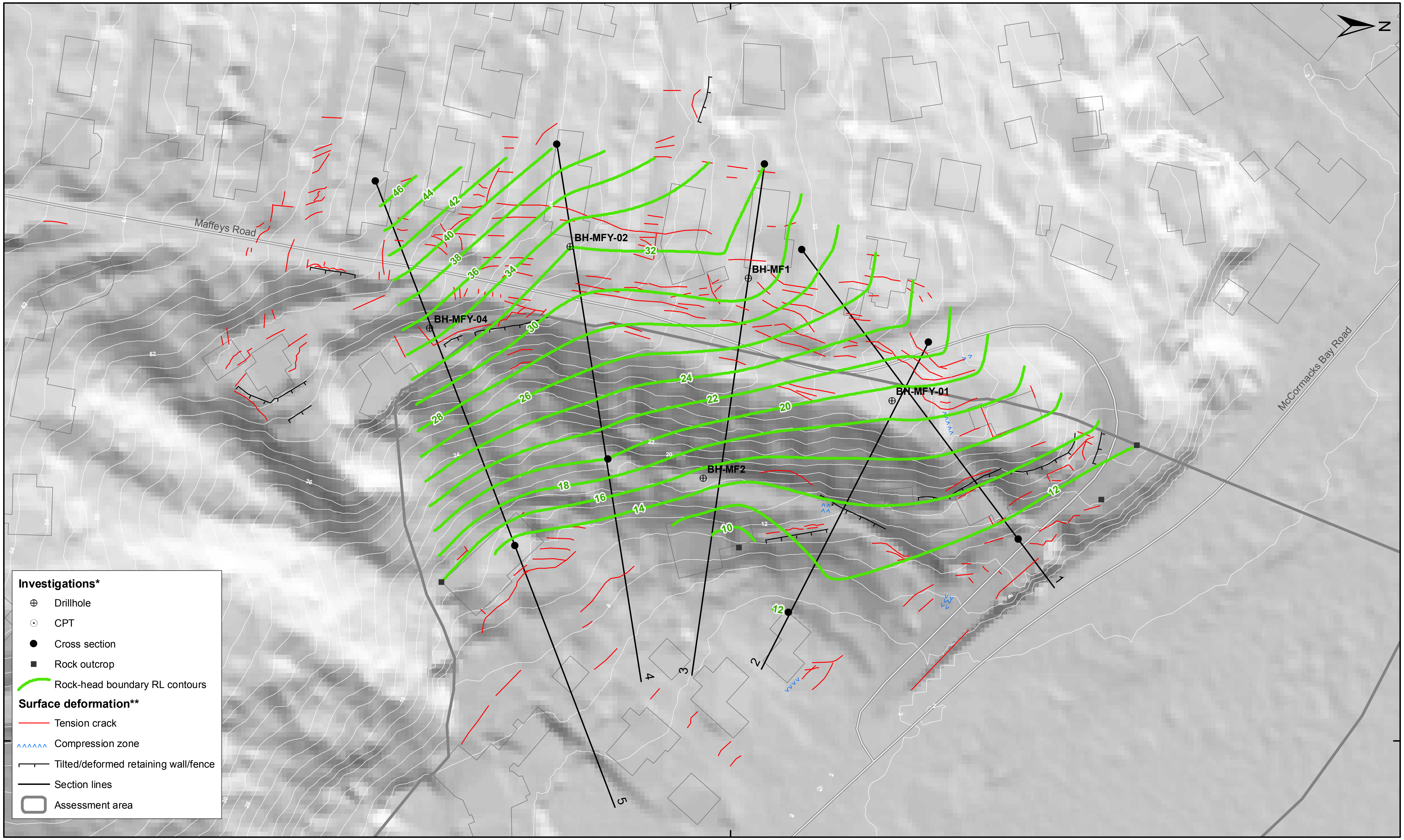
3.3.1.3 Volcanic breccia (bedrock)

The main bedrock unit underlying much of the loess and volcanic colluvium is volcanic basalt breccia, which contains occasional flows of basalt lava, which are discontinuous and vary both laterally and vertically. The basalt lava breccia is described (Yetton and Engel, 2014) as being weak to very weak with a variable rock quality designation as low as 15%. Slickensides (indicating deformation) were identified in drillhole BH MFY02 (Tonkin and Taylor, 2012a) in the volcanic breccia near the base of colluvium.

Slickensides were also noted in a thin layer of clay, within a thicker layer of epiclastic volcanic derived material (possible tuff and reworked volcanic breccia and lava), in drillhole MF2 (Yetton and Engel, 2014) (Figure 6).

3.3.1.4 Volcanic colluvium/loess and rock boundary

The boundary between the base of the volcanic colluvium and loess and the underlying volcanic rock (rockhead) was interpolated by GNS Science using: drillhole intersections, cone penetrometer test depths of refusal and field mapped rock outcrop exposures, as control points. The resulting rockhead surface (Figure 7) shows an overall dip towards the east-southeast of about 16°, which is approximately coincident with the surface slope aspect. It should be noted that the trend of cracks is roughly parallel to the strike of rockhead.



5177000

SCALE BAR: 0 50 m

EXPLANATION:

* Taken from report by Yetton and Engel (2014)
** 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

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INTERPOLATED ROCK- HEAD BOUNDARY

Maffeys Road
Christchurch

FIGURE 7

FINAL

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DATE: June 2014

3.3.2 Geotechnical properties

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 material in the assessment area are shown in Table 13. 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 8). 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 drillcore samples were all substantially higher than those for the block samples. The difference may reflect the sampling method, where drilling includes using water as a flush, and block sampling does not, or that slope faces in loess tend to be free-draining and relatively dry (Table 7).

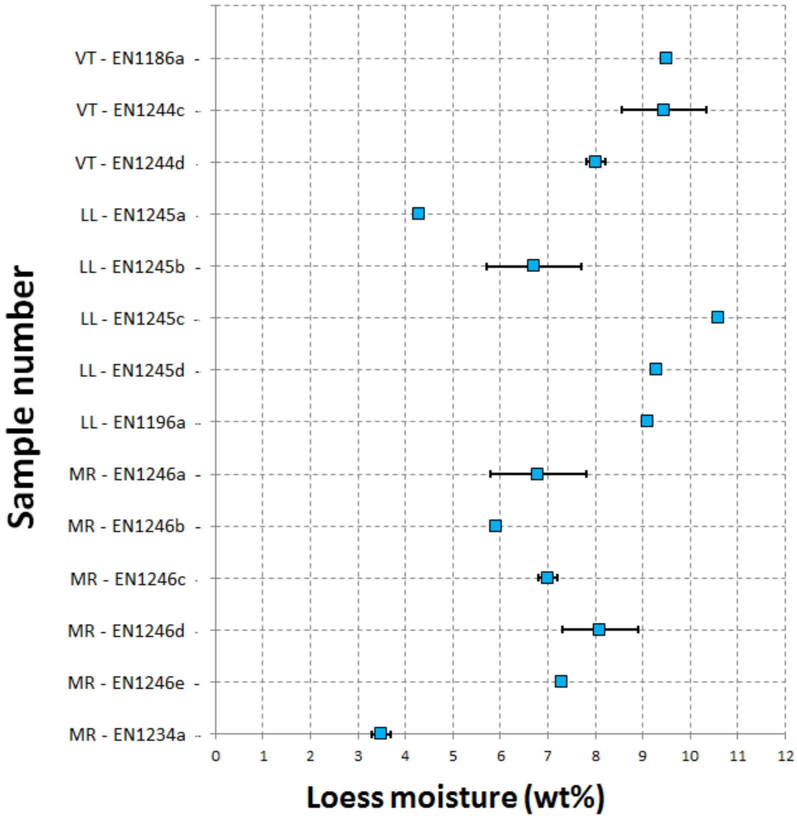


Figure 8 In-ground moisture (water, wt%) content of collected loess samples.

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 7 and plotted in Figure 9, together with results from other studies. 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 datapoints 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 7 are considered to be more representative of the bulk residual strength parameters for the loess slope rather than peak strength parameters.

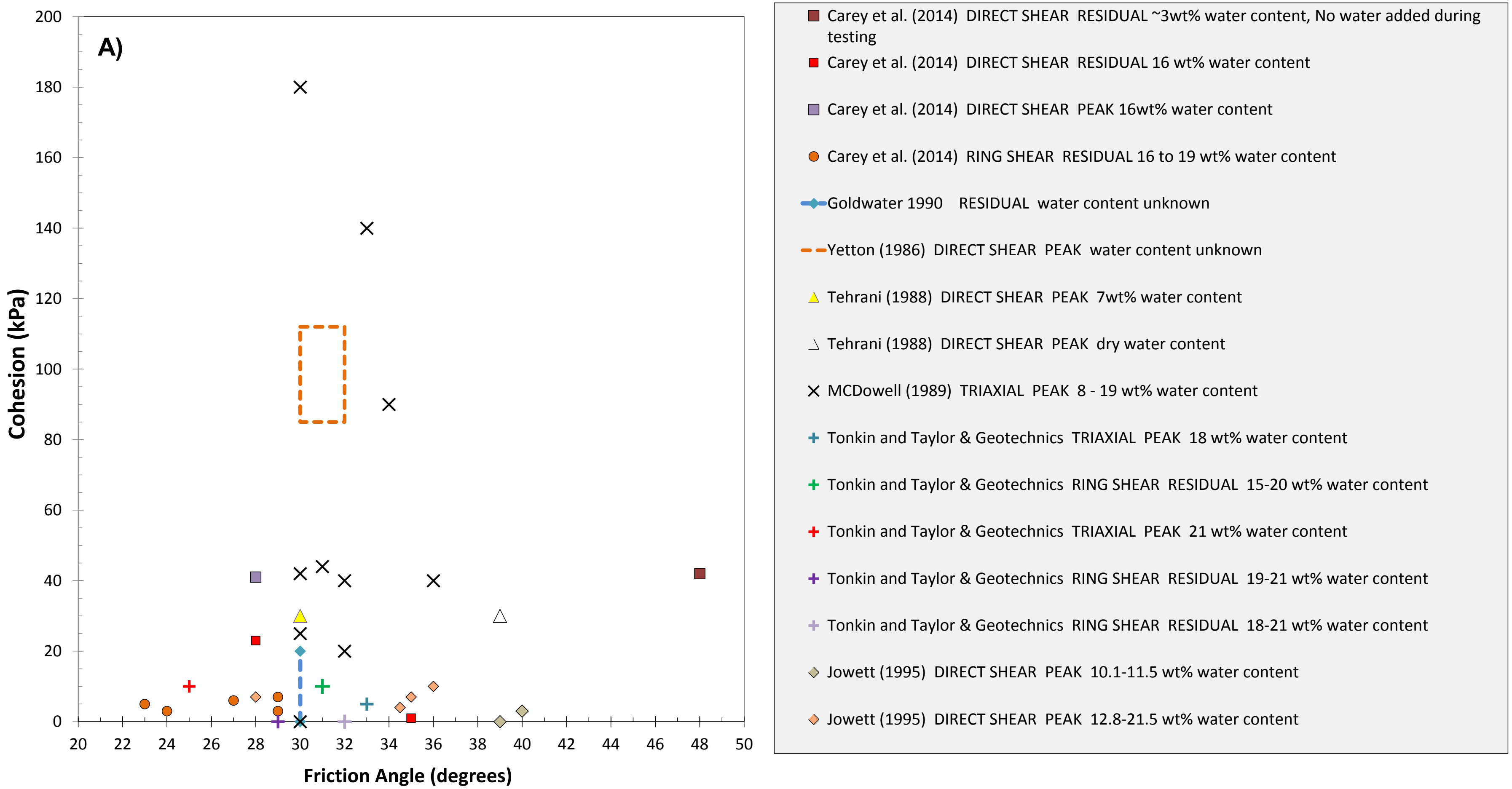
Effect of moisture content on loess shear strength

Comparison can be made with shear strength results from other published Port Hills investigations (Table 8) by plotting them alongside the results of the GNS Science testing (Figure 9).

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 9 probably represent the range of strength parameters at the likely range of moisture contents that could be anticipated in the Port Hills loess.


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

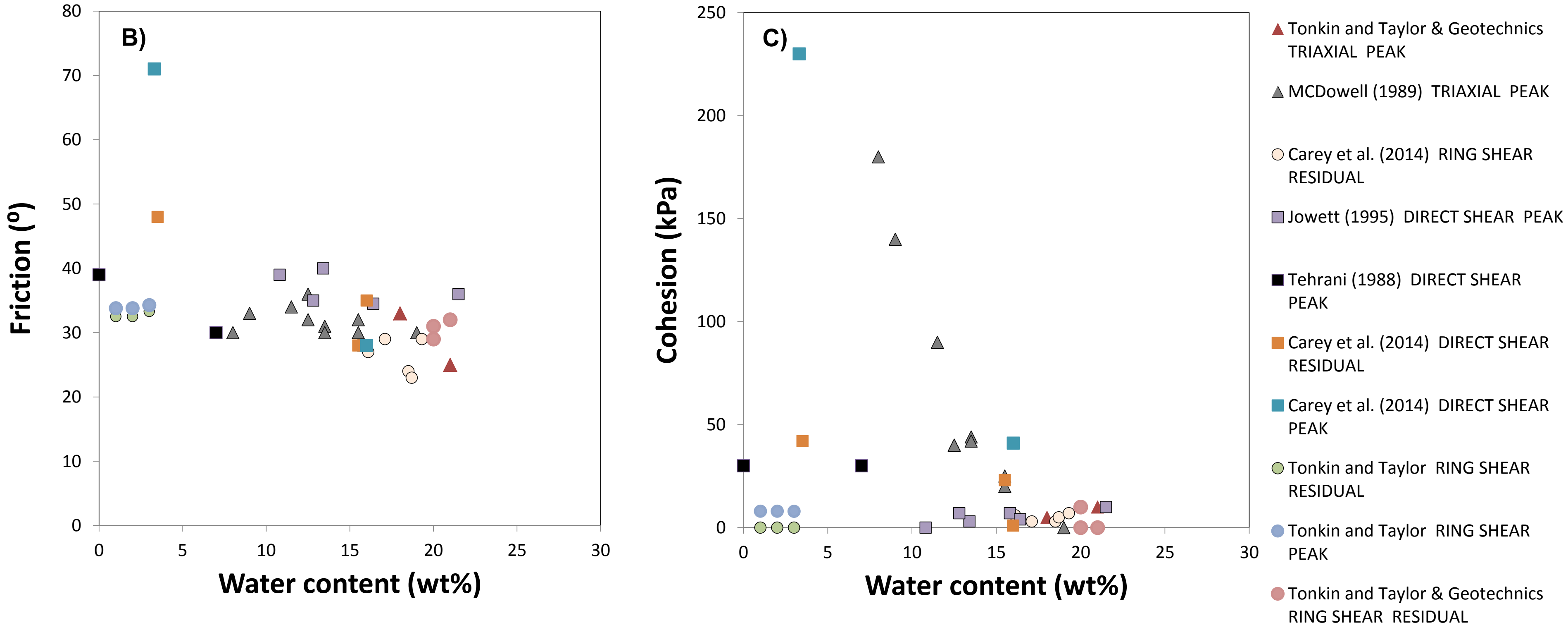


Figure 9 Loess residual shear strength results (from Table 7 and Table 8). A) Cohesion and friction laboratory results plotted for loess. B) Loess residual cohesion plotted against water content. C) Loess residual friction plotted against water content.

Table 7 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 8 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 9. Unconfined compressive strength tests were carried out to constrain the range in loess Young's modulus value for finite element numerical models.

Figure 10 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 10–Figure 12, 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 9 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 10 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)

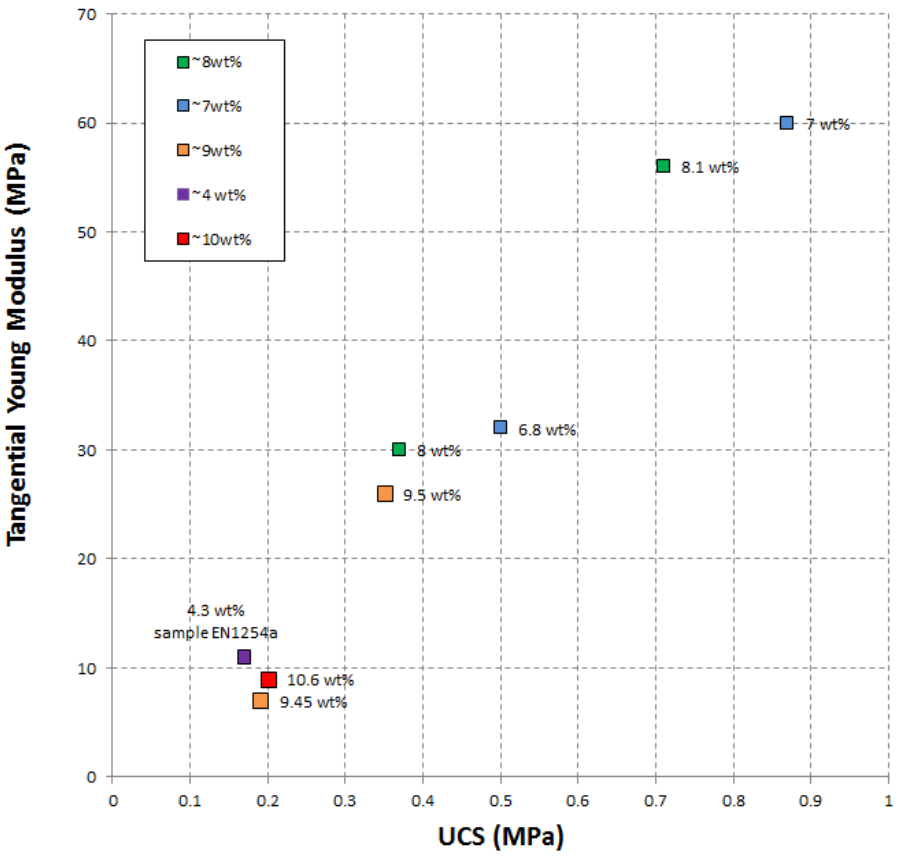


Figure 10 Loess Young's modulus versus the unconfined compressive strength and moisture content (wt%).

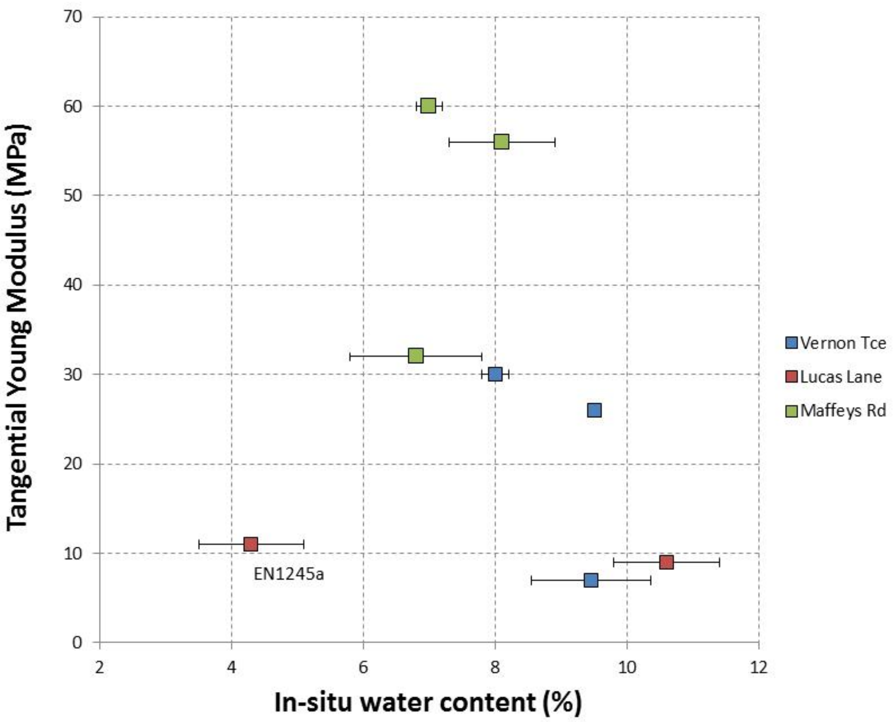


Figure 11 Loess Young's modulus versus water content (moisture content).

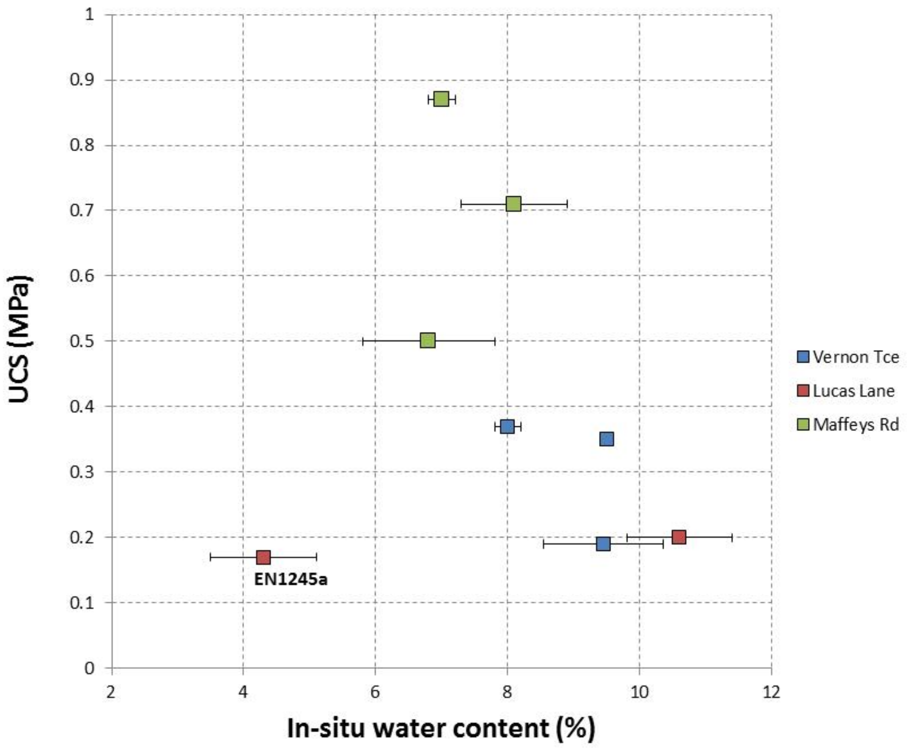


Figure 12 Loess compressive strength versus water content.

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 13 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 one are shown as diamonds and factors of safety less than one are shown as crosses. If a slope has a static factor of safety of one, 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:

- *In situ* measurements of loess samples from Maffeys Road range from 6 to 9 wt%, which are below the range moisture content of ring shear tests. The sensitivity of the loess slope to changing water contents was therefore addressed in this study by adopting a range of strength parameters for the stability assessments (of friction angle (ϕ) of 25–35° and cohesion (c) of 5–30 kPa) to reflect the range in moisture content.

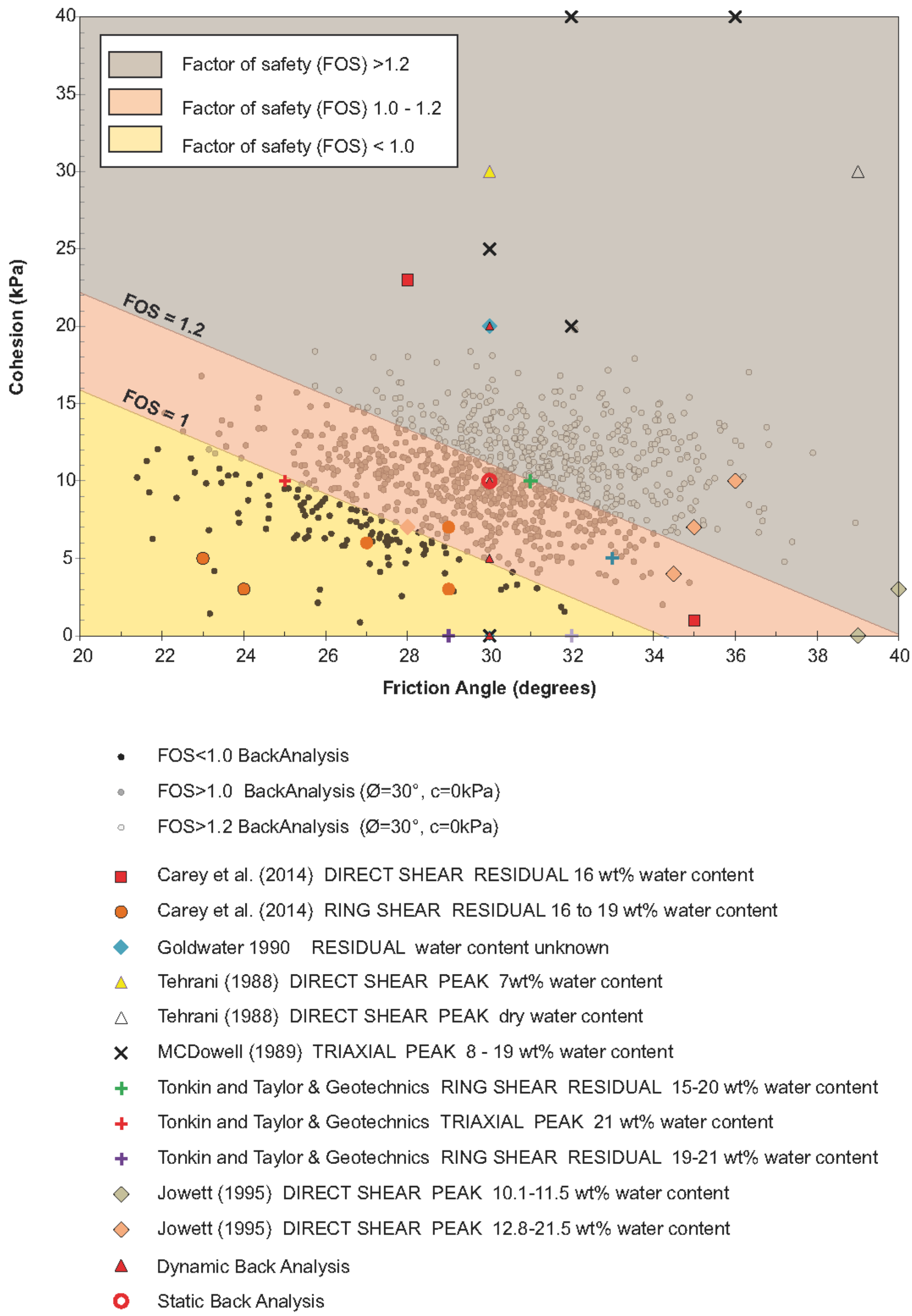


Figure 13 Back-analysis of the loess material based on cross-section 4. Note: each data point represents a modelled slide surface at a given combination of cohesion and friction adopted for loess. Those slide surfaces shown as crosses, represent those combinations of cohesion and friction that would yield a static factor of safety of less than 1.

Other loess parameters

A Young's modulus of 30 MPa was adopted based on laboratory testing results from the loess (Table 9 and Table 10). 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 11.

Table 11 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)	

Loess Bulk Shear Modulus

The *in situ* shear modulus of the loess and volcanic colluvium were derived from the downhole shear-wave velocity surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) based on the survey results from drillholes MF1 and MF2, and results from the dynamic probing carried out by Tonkin and Taylor for the Earthquake Commission at Clifton Terrace (Tonkin and Taylor, 2012b). The results from the dynamic probing are summarised in Figure 13. The mean shear wave velocity is 306 m/s (±93 m/s at one standard deviation) and the mode is 222 m/s.

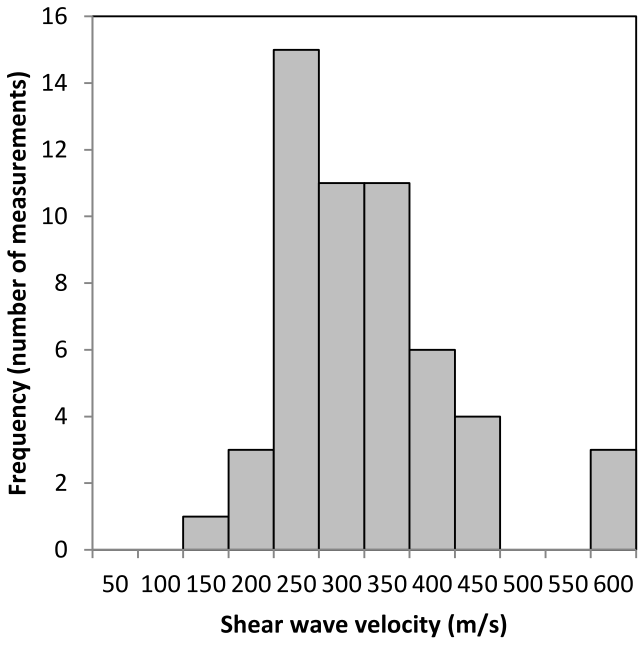


Figure 14 Loess shear wave velocity results from dynamic probing reported by Tonkin and Taylor (2012b) for the loess at Clifton Terrace.

The corresponding shear wave velocity measured in the drillhole MF1 was 252 m/s (for the loess intersection 2.0–4.0 m), which is consistent with the results from the dynamic probing, and 656 m/s (for the loess intersection (4.0–8.0 m), which is higher than the results from the dynamic probing.

The lower values are consistent with shear wave velocity trends defined by Rinaldi et al. (2001) for other loess as a function of normal stress and moisture content (Table 12) where in the 2–14 m depth range (corresponding to 30–240 kPa range of overburden pressure) the range of loess shear wave velocity was 280–300 m/s, at a water contents of ~16%, and 300–320 m/s for water contents of 6.4%.

Applying the relationship for shear wave velocity:

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

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

Material	Shear wave velocity V_S (m/s)	Source
Port Hill loess from Redcliff borehole MF1 Inferred moisture content 6–10 wt%	288	Southern Geophysical Ltd. (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)

3.3.2.2 Volcanic colluvium

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

It mainly comprises clay and silty gravel and derives mainly from weathered volcanic breccia and lava and less so from the loess, although remobilised loess is present in varying proportions.

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 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.

3.3.2.3 Volcanic bedrock

In the absence of strength tests of Maffeys Road rock samples, the results from testing of similar materials carried out on rock samples from Redcliffs (Carey et al., 2014), were used as a basis for the numerical models.

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

Given the anisotropic nature of the materials, especially with regards to the lateral and vertical extent of the lava sequences within the predominant volcanic breccia, and the generally low shear wave velocities derived from the down-hole surveys, it was decided to treat the volcanic materials as one unit, comprised mainly of breccia. The geological strength index values adopted for the breccia are shown in Figure 15.

Mohr-Coulomb parameters (cohesion and friction) were derived from RocLab by line fitting over the appropriate stress range of the slope, using intact laboratory test results from volcanic breccia material from Redcliffs.

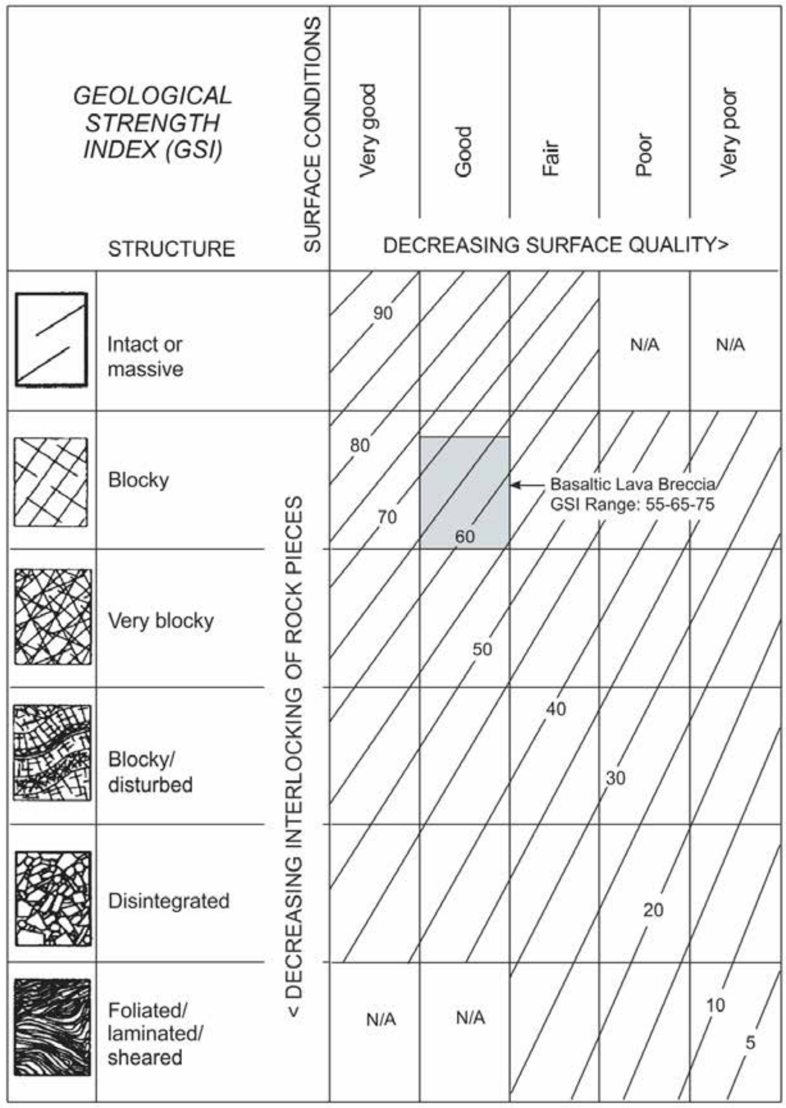


Figure 15 Geological strength index plot for volcanic breccia at Maffeys Road (modified after Hoek, 1999).

The shear strength of the slickensided thin layer of clay, within the thicker layer of epiclastic volcanic derived material (possible tuff and reworked volcanic breccia and lava), in drillhole MF2 (Yetton and Engel, 2014), has been derived by adopting the results from the laboratory testing of the clay exposed in the cut-slope parking area for 12 Cliff Street (Massey et al., 2014). The clay was tested in-house using ring shear equipment. Test results gave residual shear strength values of friction (ϕ) of 12° (±2°) and cohesion (c) of 3 kPa (±3), errors represent one standard deviation.

Shear Modulus

The shear moduli for mixed volcanic breccia and lava were derived from the downhole geophysical surveys carried out by Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) in drillholes MF1 and MF2 (Figure 16).

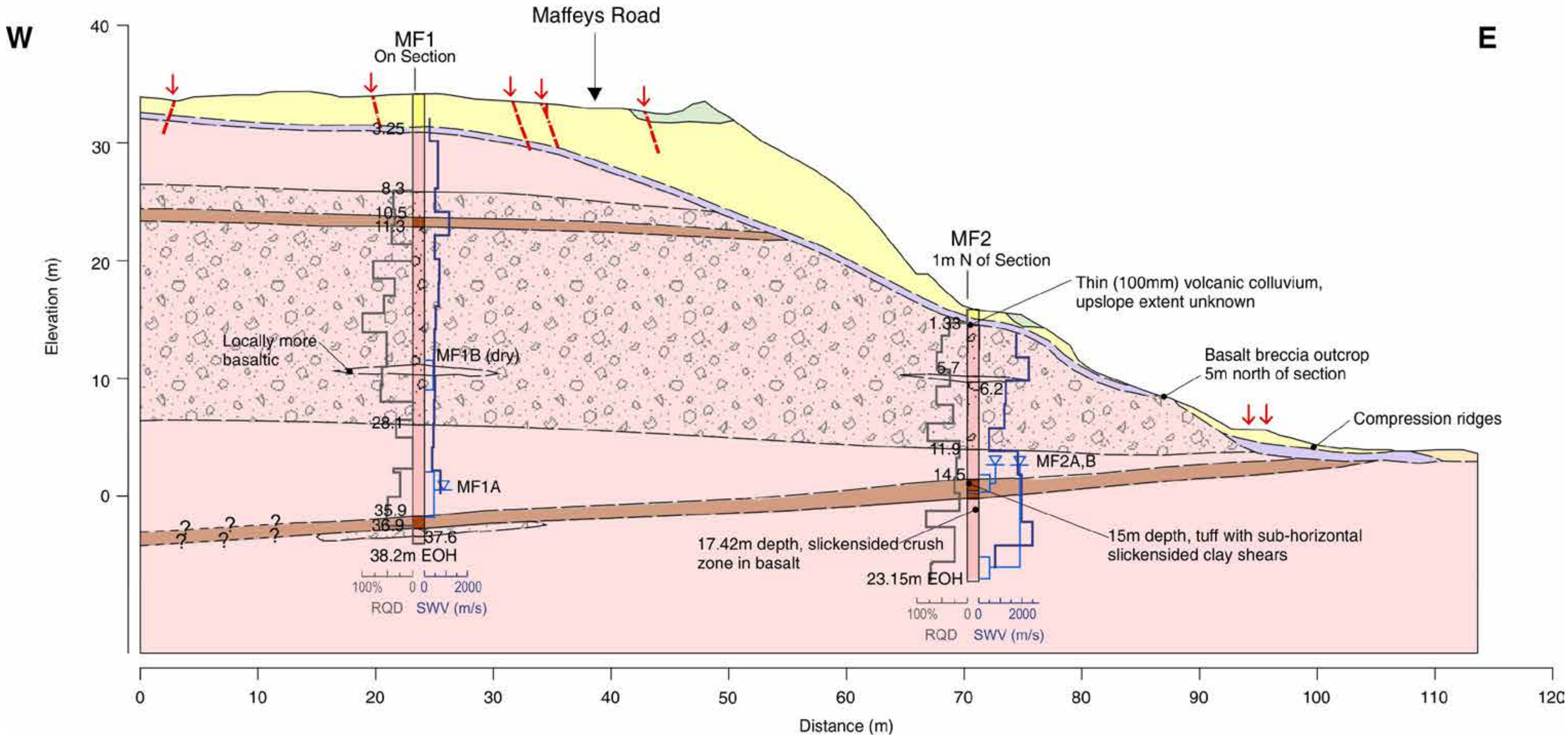


Figure 16 Cross-section 3 (Yetton and Engel, 2014) modified by including the shear wave velocity profile after Southern Geophysical Ltd. (Southern Geophysical Ltd., 2013) and rock quality designation values for drillholes MY1 and MY2.

3.3.3 Adopted parameters for numerical models

For the purpose of stability assessment, material strength parameters were selected as shown in Tables 13 and 14.

Table 13 Range of bulk geotechnical material parameters adopted for Maffeys Road soil materials.

Material	Unit weight (kN/m^3)	Intact modulus E_i (MPa)	Poisson's ratio	Cohesion c (KPa)	Friction ϕ (°)	Tensile strength (KPa)	Shear wave velocity (m/sec)	Shear modulus G_S (MPa)
Loess	17	30	0.3	0–30	30	10	200–400	70–270
Colluvium	17	30	0.3	0–15	21–28	0	200–400	70–270
Weak layer in rock	20	10	0.4	0–6	10–14	0	200–400	70–270

Table 14 Range of adopted rockmass strength parameters for Maffeys Road volcanic breccia.

Unit		Lab UCS (MPa)	Unit weight (kN/m^3)	Tensile (MPa)	Intact modulus E_i (MPa)	Poisson's ratio	Slope height (m)	GSI	m_i[2]	Cohesion[3] c (KPa)	Friction[3] ϕ (°)	Tensile strength (KPa)	Rock mass modulus E_M (MPa)	Shear Wave Velocity (m/sec)	Shear modulus G_S (MPa)[4]
Basalt lava breccia	MIN[1]	1.3	18	0.2	310	0.1	20–40	55	3	44	27	15	127	400	288
	AV[1]	1.9	18	0.3	824	0.2		65	6	78	38	23	520	550	545
	MAX[1]	2.9	19	0.5	1,500	0.3		75	10	140	49	44	1225	700	931

1 MIN, AV and MAX represent the range of test results and field measurements.

2 The m_i values shown, represent the range in the ratio of unconfined compressive strength (UCS) to tensile strength, derived from tested samples of basalt lava breccias (Carey et al., 2014).

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

4 Shear Modulus (G_s) is derived from down-hole shear wave velocity $G_s=\rho*V_s^2$ survey of drillhole MF1 and MF2, where $G_s=\rho*V_s^2$ and ρ=density (Kg/m^3) and V_s = shear wave velocity (m/s).

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 Peninsula 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 17).

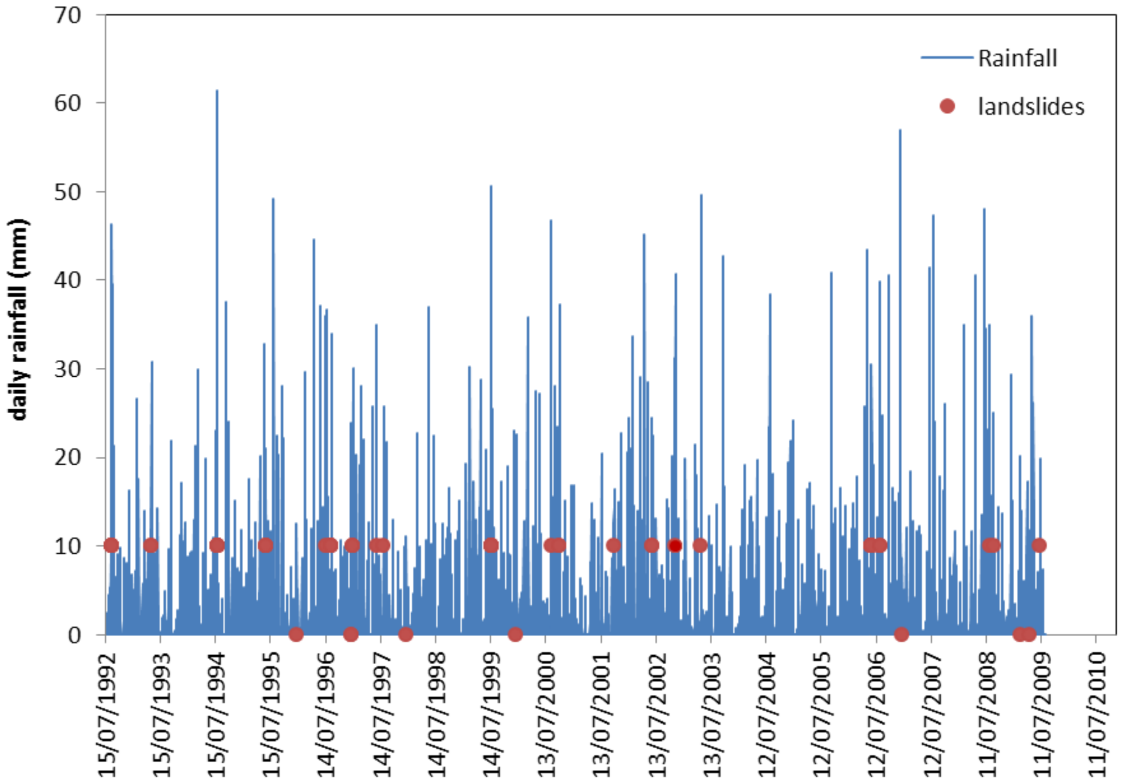


Figure 17 Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills. Daily rainfalls at Christchurch Botanic Gardens and landslides in the Port Hills investigated by Geotechnical Consulting Ltd, or listed by the Earthquake Commission as causing damage to homes. Landslides without 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 15):

- 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 18; 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 15. 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 15 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.

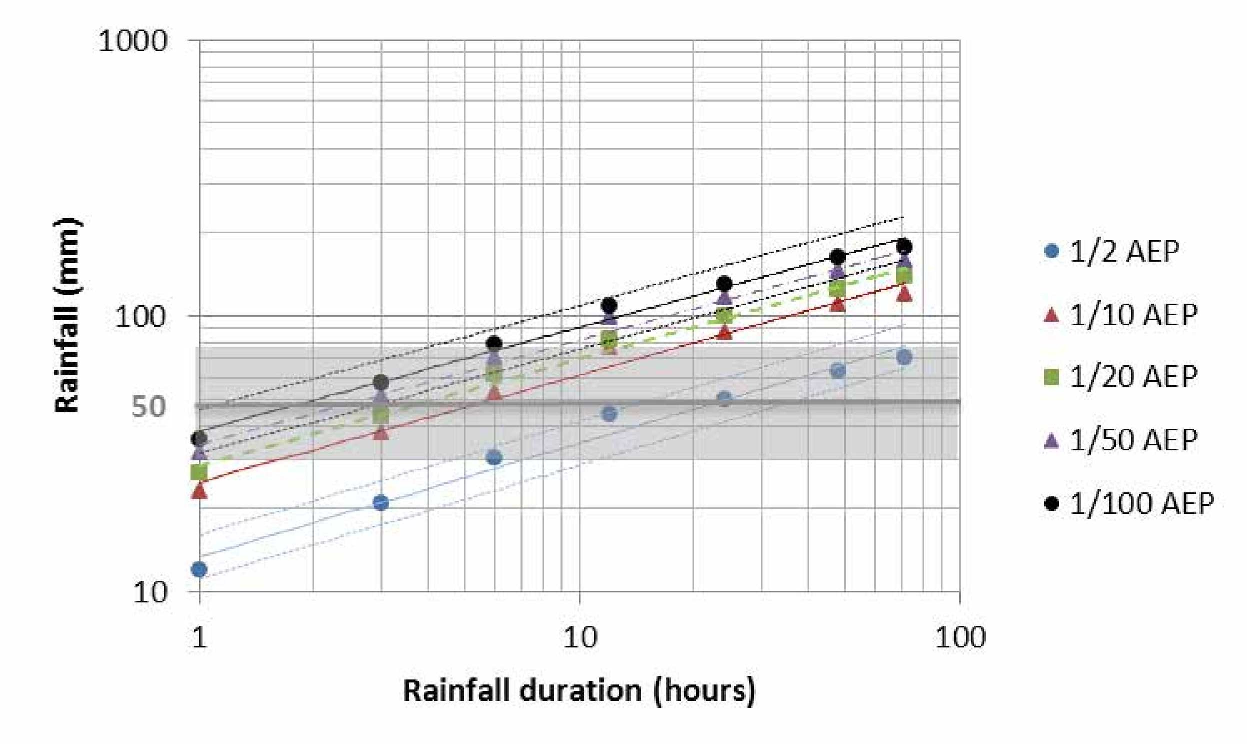


Figure 18 Rainfall depth-duration-return period relations estimated for Christchurch Gardens by Griffiths et al. (2009) using recorded rainfall data. Error limits of 20% are shown by dotted lines for the 1/2 and 1/100 AEP curves. Shaded area covers the range of 30–75 mm of rainfall over which the expected 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.

Yetton and Engel (2014) and Tonkin and Taylor (2012a) found no permanent piezometric head levels within the loess or volcanic colluvium. Tonkin and Taylor Ltd. do note groundwater at the base of piezometer CPT MFY03. However, the groundwater monitoring data are unusable, as it is not known which piezometer the data are from given the depth of the piezometer and CPT are shallower than the listed piezometric head level. Piezometric heads are present in the deeper piezometers within the rock, but they are deep within the slope, and unlikely to affect shallower failures in the loess and volcanic colluvium.

At present data of loess water content and ground-water levels are not being collected and so cannot be correlated to climatic conditions, i.e., rainfall, either short duration high intensity, or longer duration low intensity.

3.4 SLOPE FAILURE MECHANISMS

3.4.1 Landslide types affecting the site

Yetton and Engel (2014) have identified the following main landslide types and failure mechanisms within the assessment area:

- Earth/debris flows:
    a. Shallow failure of loess, forming several discrete landslide source areas, which could form earth/debris flows.
    b. Deeper failure of the loess through the underlying volcanic colluvium layer, forming several discrete landslide source areas. The presence of a loess colluvium layer (Yetton and Engel, 2014) was intercepted in all drillholes. 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.
- Deep-seated translational slides: Deep seated failure of the entire cracked area, through weak clay layers within the underlying volcanic breccia.
- Toe slumps: Failure of the loess and alluvium at the toe of the site. This failure mechanism has not been assessed in this report.

Based on the results in Yetton and Engel (2014), 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:

- The slope is currently cracked allowing surface water to infiltrate the slope more readily.
- 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.
- 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 fan originates from a few infrequent but large landslides, or from the accumulation of debris from many smaller landslides.

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 (1–5);
- 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; and.
- Mechanism 3: Deep seated failure of the entire cracked area, through weak clay layers within the underlying volcanic breccia.

4.0 HAZARD ASSESSMENT RESULTS

4.1 SLOPE STABILITY – STATIC CONDITIONS

The engineering geological cross-sections from Yetton and Engel (2014) (Figures 5 and 6) were used as the basis of the static slope stability modelling. Geotechnical material strength parameters used in the modelling are from Tables 13 and 14, and models using variable shear strength parameters for the key materials were run to assess the sensitivity of the slope.

Graphic examples of stability assessment outputs are shown in Figures 19–24 for cross-section 4. Cross-section 4 was chosen as it best represented the site conditions where the angle of rockhead was steepest, and it is the cross-section closest to where the permanent slope displacements were measured from survey marks.

Table 16 shows the results from the assessment of the three failure mechanisms:

- *Mechanism 1 – Failure through the loess:* The results from the assessment, adopting loess shear strength parameters of friction angle (ϕ) of 30° and cohesion (c) of 10 kPa suggest that the slope factor of safety is about 1.2. The factors of safety of the trial slide surfaces are very sensitive to the adopted cohesion value.
- *Mechanism 2 – failure through the volcanic colluvium layer:* As for mechanism 1, the presence of a weak layer results in a low factor of safety. When a weak volcanic colluvium layer is taken into account, the factor of safety is about 1.1. This result suggests that the slope is marginally stable under static conditions if a weak layer of volcanic colluvium, above rockhead, is included in the assessment. The sensitivity of the factor of safety to changes in friction angle of the colluvium is shown in Figure 25. The range of friction-angle values in Figure 25 are within the range of residual strength values obtained from ring- and direct-shear tests, and they are likely to represent a lower bound range of the bulk strength conditions of the loess and volcanic colluvium. Nonetheless, the results highlight the sensitivity of the slope to the presence of a weak layer.
- *Mechanism 3 – Failure through a weak clay layer within the rock:* The factor of safety for the modelled conditions ranges from 1.2 to greater than 1.7. This variability is due to: 1) the different assessed failure path method, e.g., circular versus block-search methods; and 2) the bulk strength adopted for the volcanic rock mass.

Table 16 Example results from static slope stability assessment of cross-section 4.

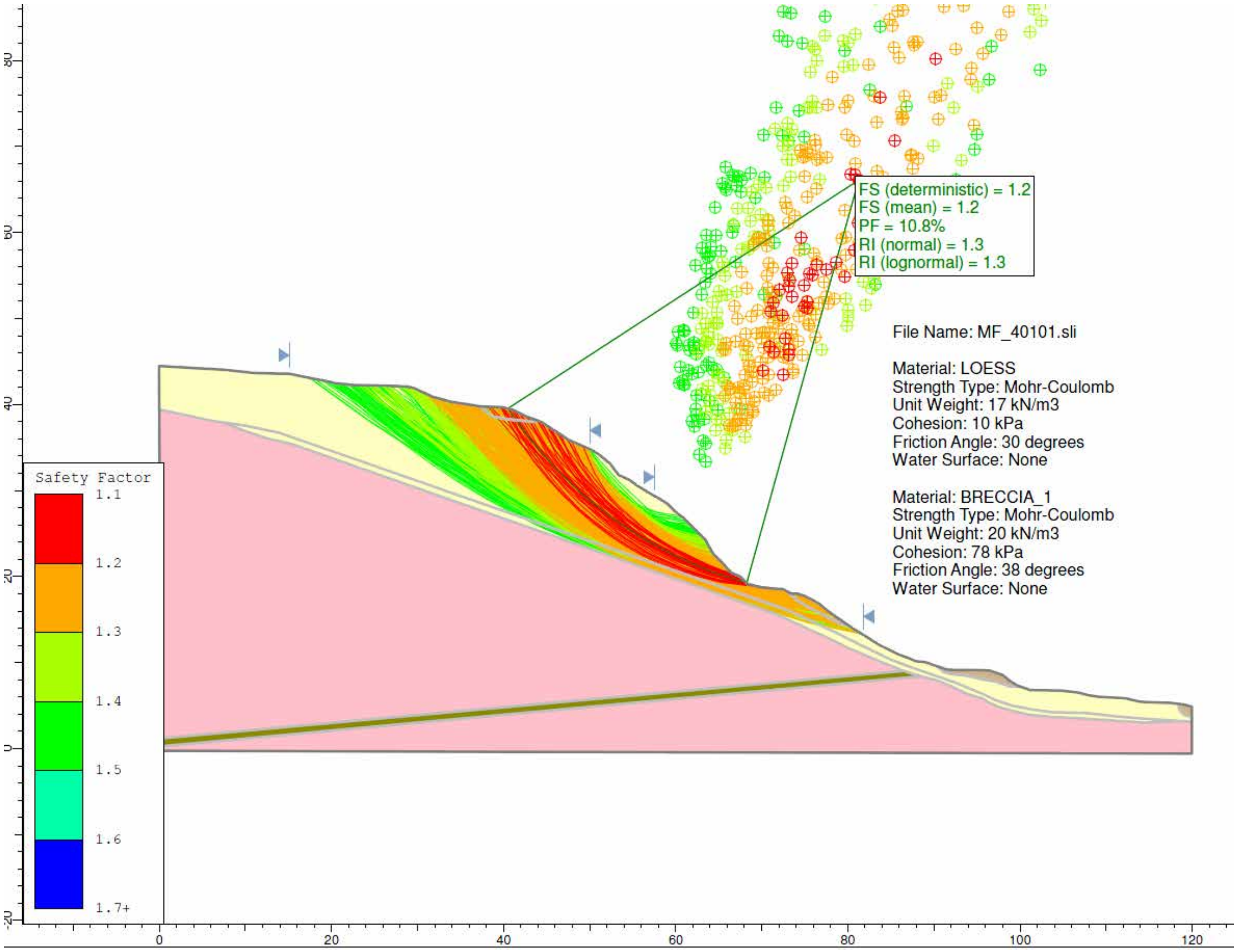
Failure mechanism	Failure method[1]	Loess Cohesion (kPa) / Friction (°)	Colluvium Cohesion (kPa) / Friction (°)	Volcanic breccia Cohesion (kPa) / Friction (°)	Weak layer Cohesion (kPa) / Friction (°)	FOS[2] SLIDE	Piezometric head level[3] (m)	SRF[4] PHASE
1) Shallow failure of Loess slope	Circular	10 / 30	Not simulated	78 / 38	n/a	1.2	Drained	1.2
						1	3	
2) Failure through volcanic colluvium	Circular	10 / 30	0 / 28	78 / 38	n/a	1.1	Drained	1.1
						1	1	
	Block	10 / 30	0 / 28	78 / 38	n/a	1.1	Drained	
						1	1	
3) Rock slope with weak clay layer	Circular	10 / 30	0 / 28	78 / 38	0 / 12	2.2	Drained	>5
	Circular	10 / 30	0 / 28	44 / 27	0 / 12	1.4	Drained	
	Block	10 / 30	0 / 28	78 / 38	0 / 12	1.7	Drained	
						1	20	
	Block	10 / 30	0 / 28	44 / 27	0 / 12	1.2	Drained	
						1	13	

[1] Block refers to the block slide surface method and circular refers to the circular slide surface method.

[2] FOS is the factor of safety derived using the General Limit Equilibrium method of Morgenstern and Price (1965).

[3] For mechanisms 1 and 2 the piezometric head levels are the levels above rockhead. For mechanism 3, they represent the level above the weak clay layer.

[4] The finite element model was also used for comparison. Where the slope has been assessed using the finite element model, the stability of the slope is assessed in terms of the stress reduction factor Note the shear strength reduction method is used to determine the stress reduction factor (SRF) or factor of safety value that brings a slope to the verge of failure (Dawson et al., 1999).



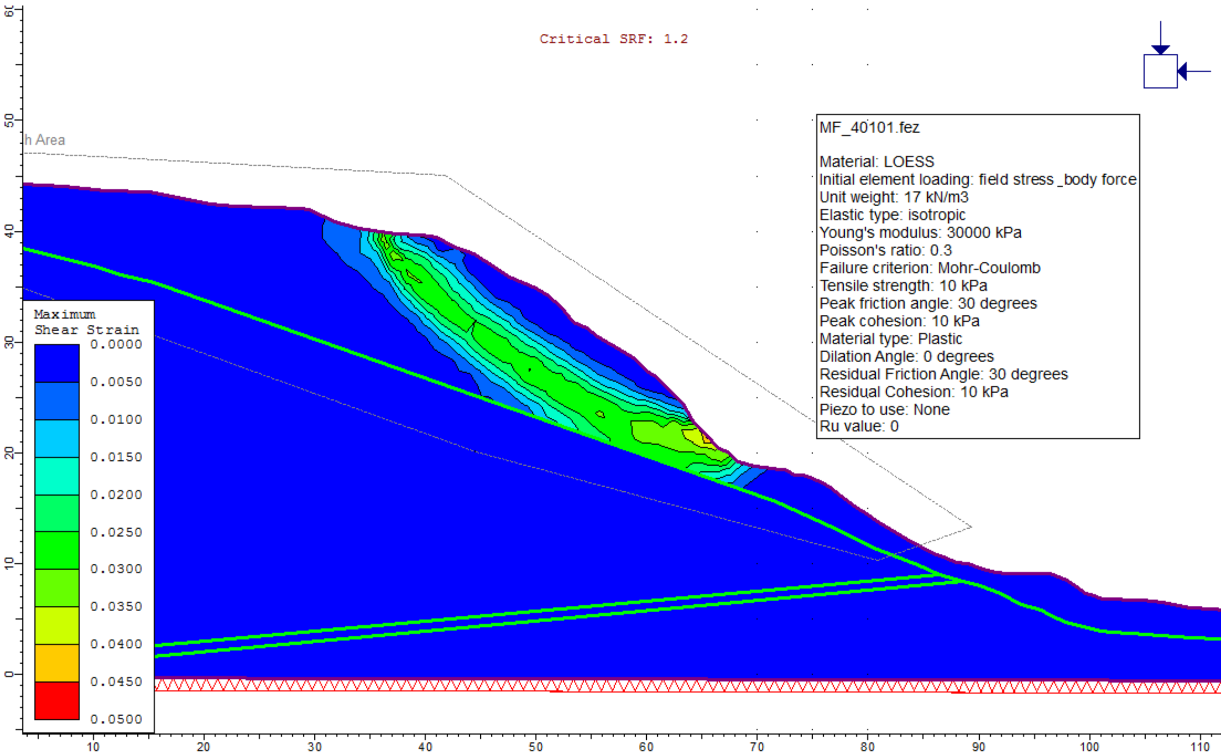


Figure 19 Example of limit equilibrium and finite element modelling results for cross-section 4, mechanism 1 assessment through loess (circular failure model).

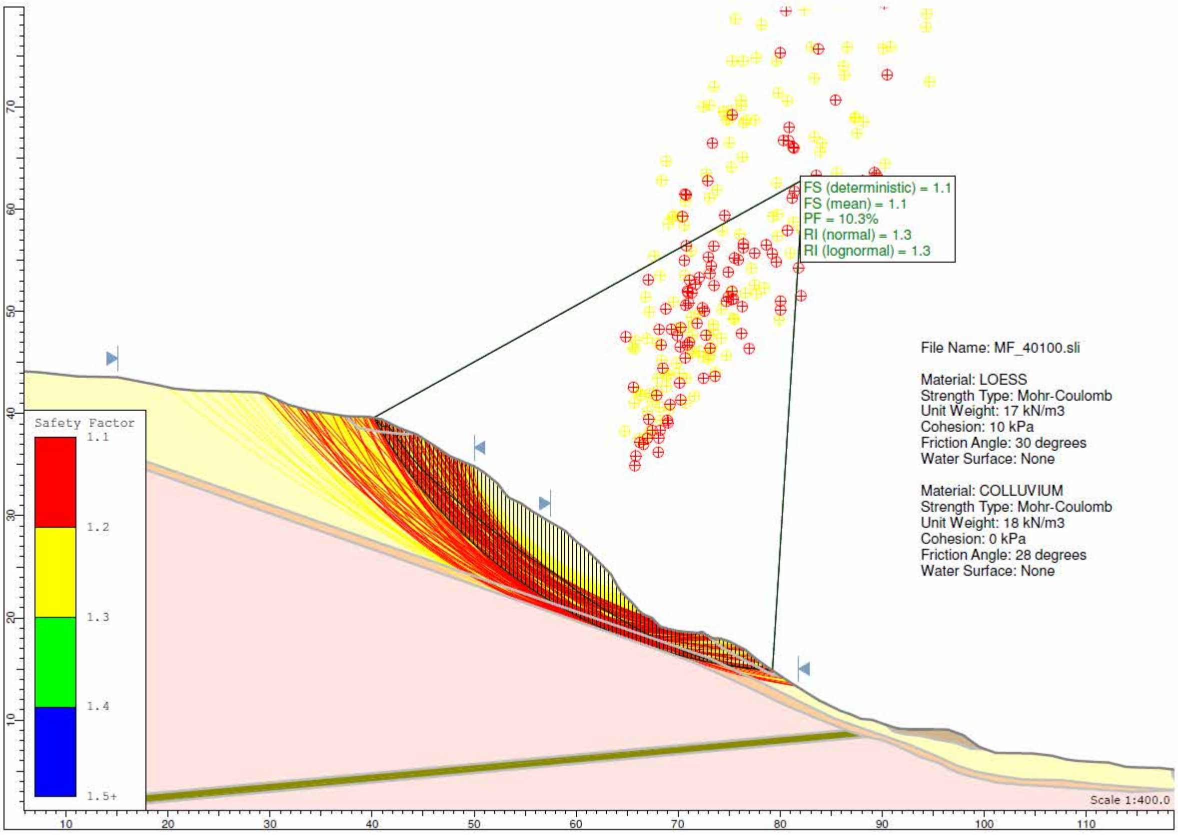


Figure 20 Example of limit equilibrium modelling results for cross-section 4, mechanism 2, with weak colluvium layer (circular failure model).

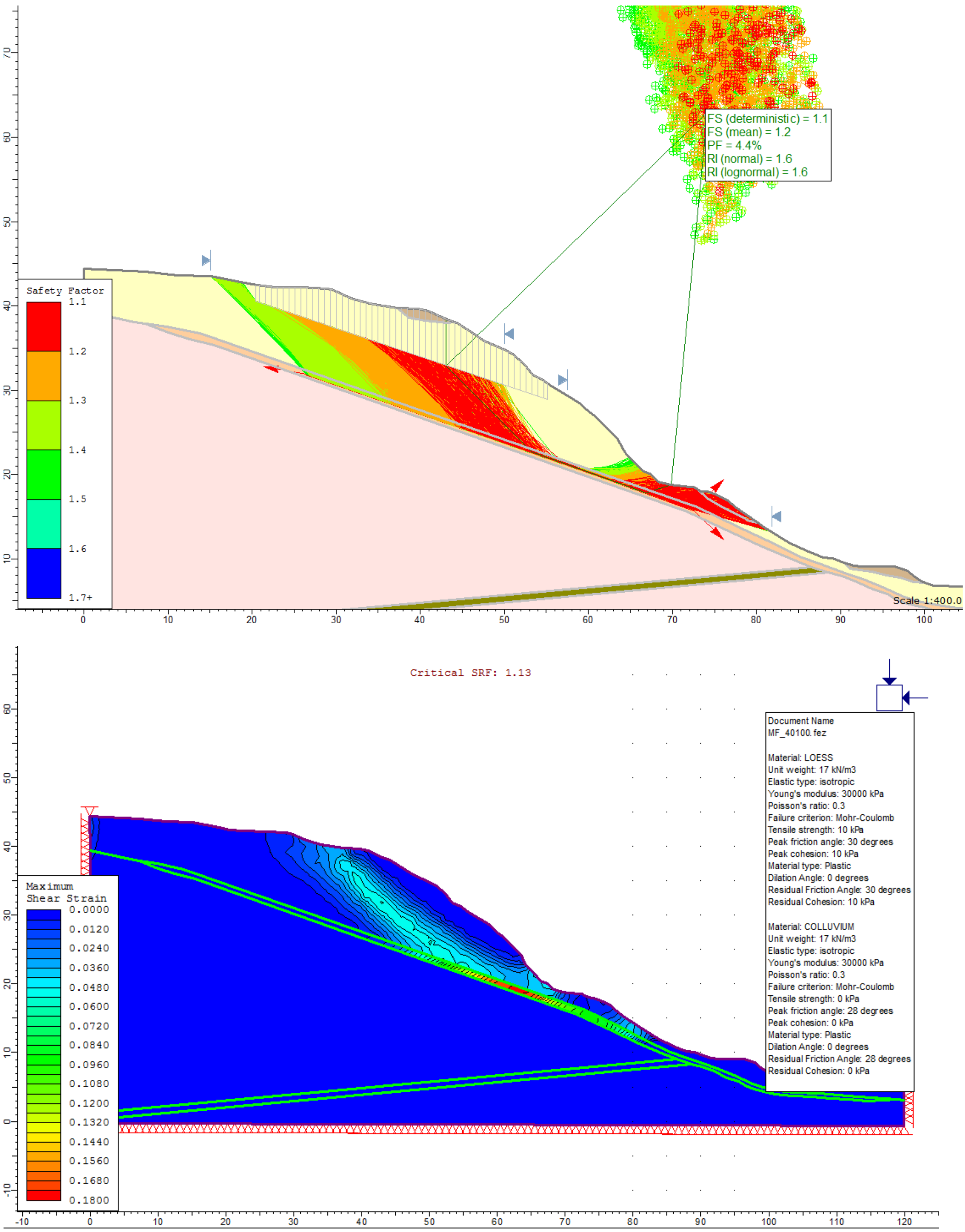


Figure 21 Example of limit equilibrium and finite element modelling results for cross-section 4, mechanism 2, with weak colluvium layer, assuming a block slide failure model for the limit equilibrium model.

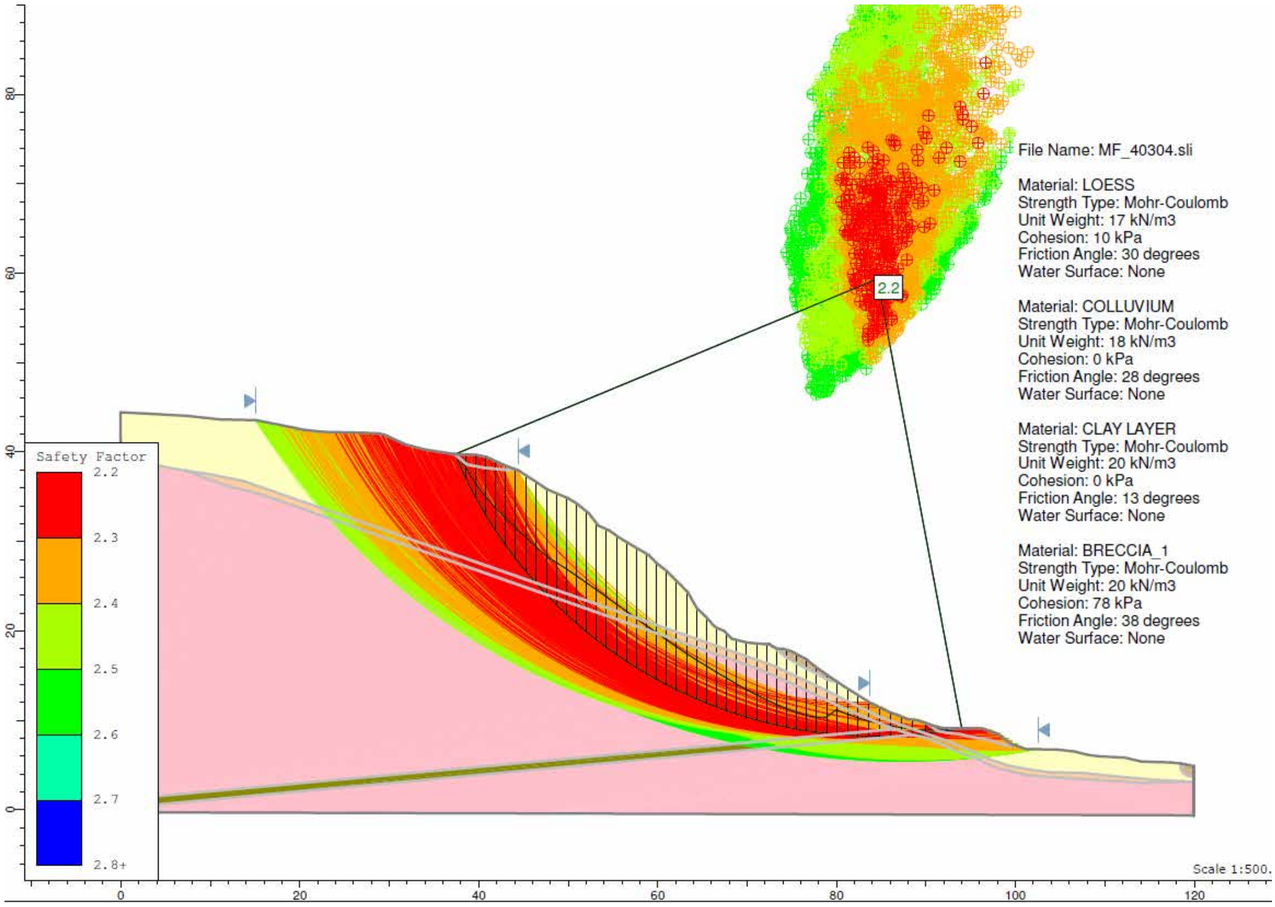


Figure 22 Example of limit equilibrium modelling results for cross-section 4, mechanism 3 (circular failure model).

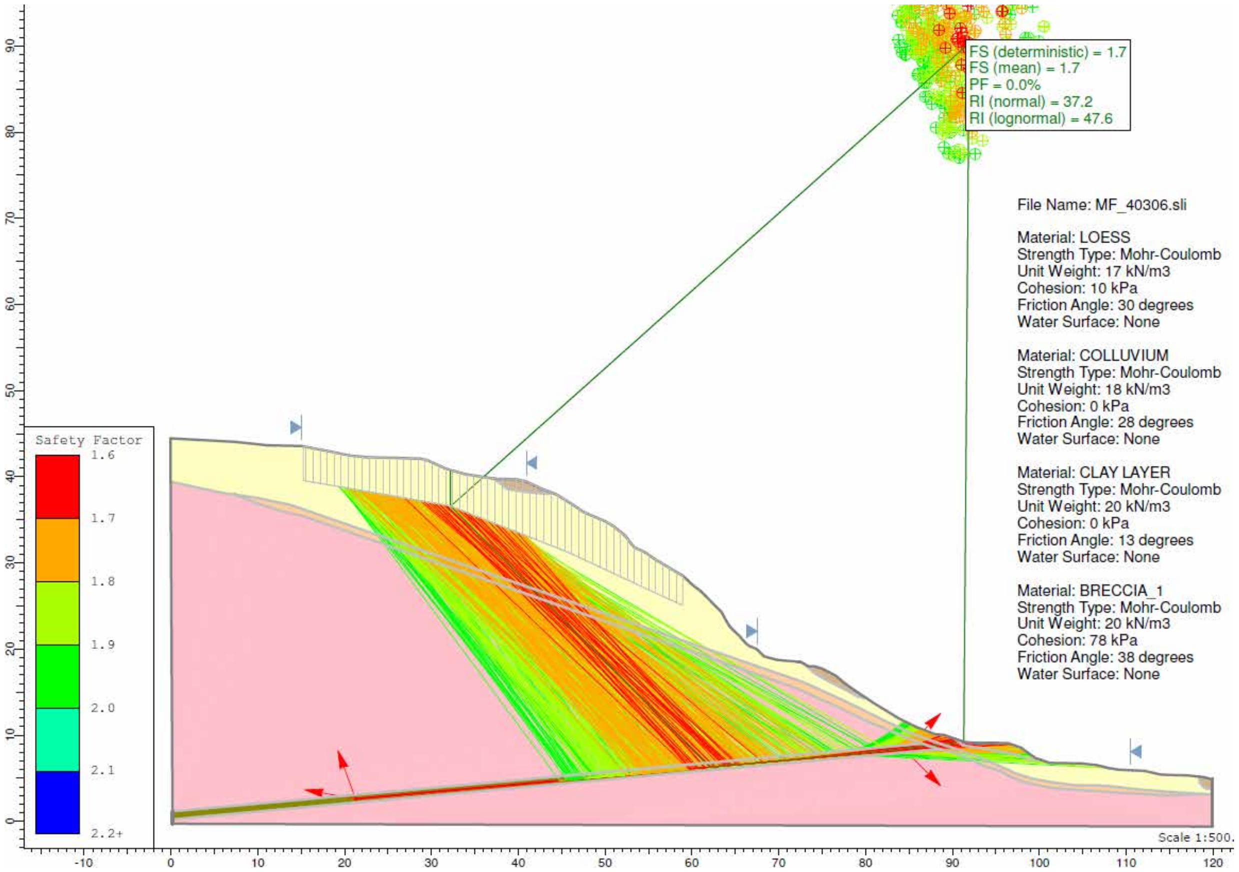


Figure 23 Example of limit equilibrium modelling results for cross-section 4, mechanism 3 (block slide).

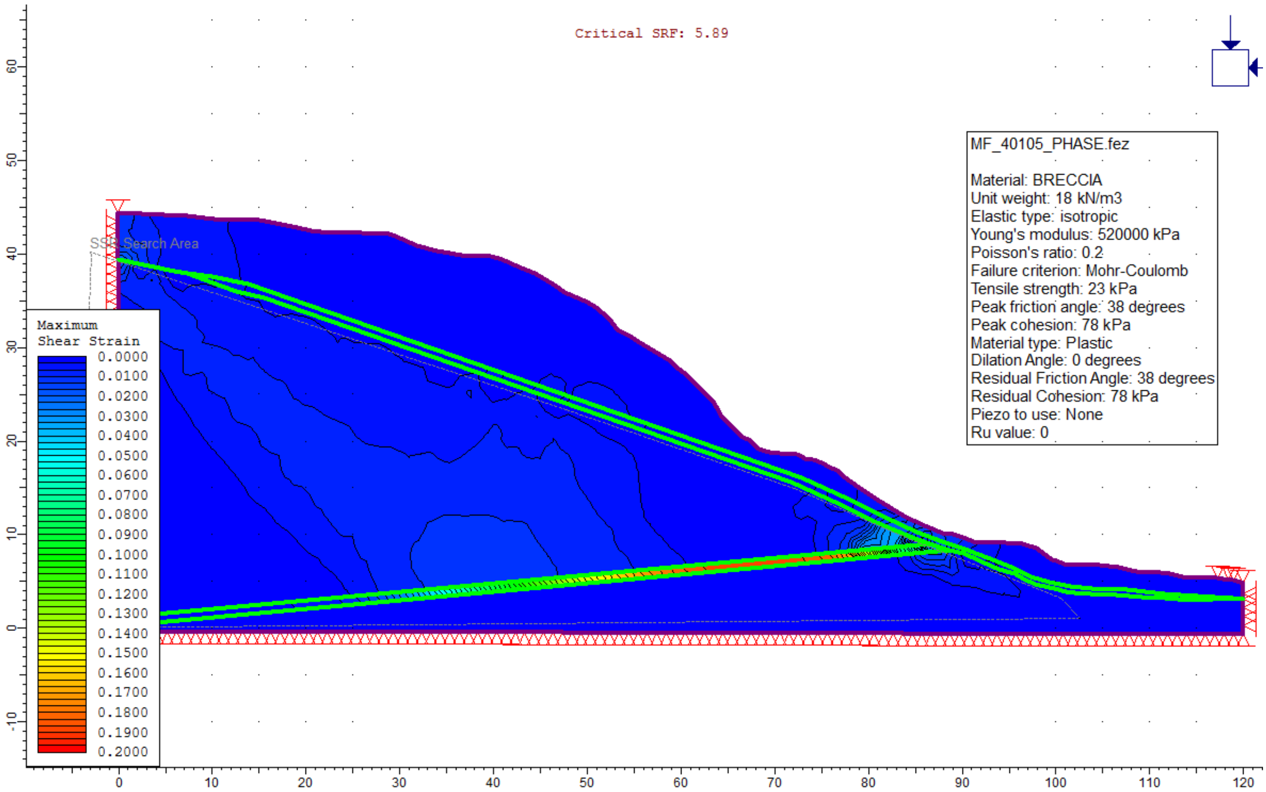


Figure 24 Example of finite element modelling results for cross-section 4, mechanism 3 (block slide).

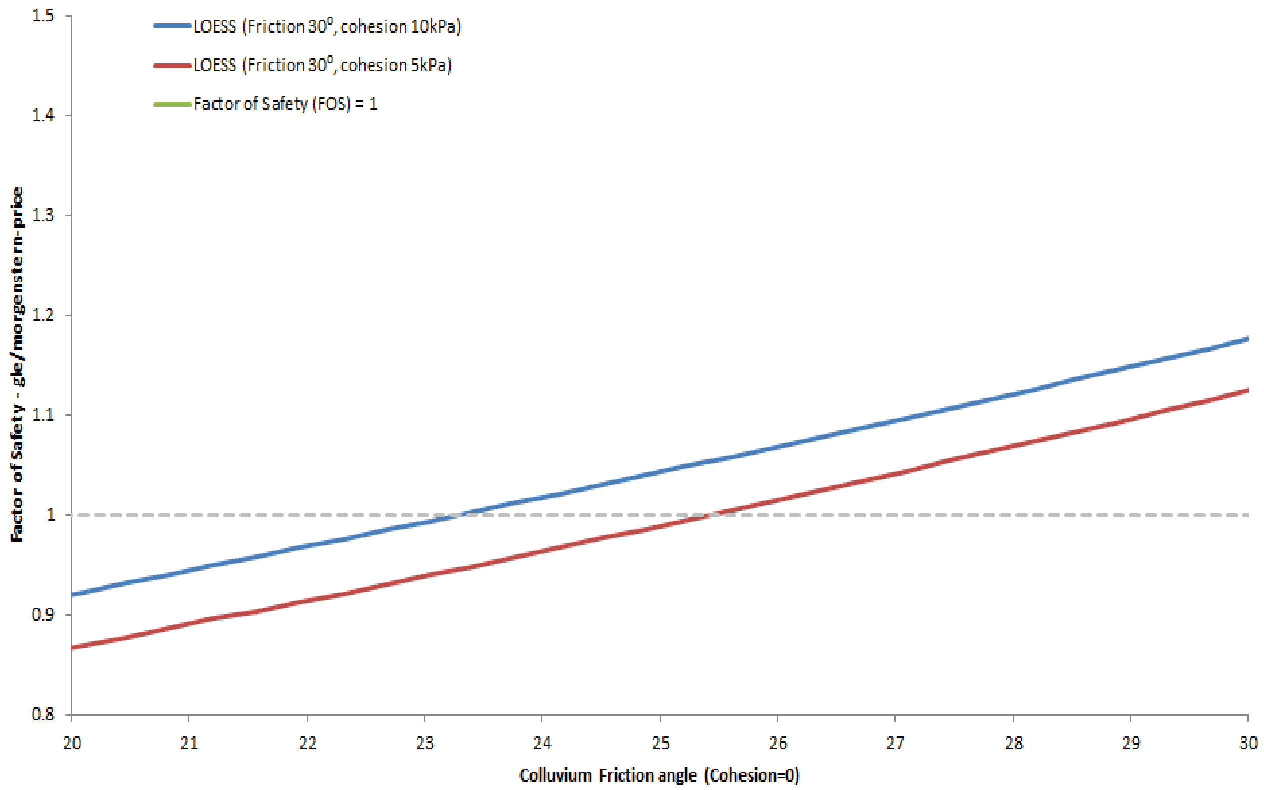


Figure 25 Mechanism 2, sensitivity of the slope factor of safety to changes in the friction angle of the volcanic colluvium, for cross-section 4. The cohesion (c) of the volcanic colluvium is fixed at 0 kPa and a variable friction angle (ϕ) was adopted. The assessmrnt adopted two different shear strength conditions for the overlying loess. The blue line represents loess shear strength of friction (ϕ) 30° and cohesion (c) of 10 kPa. The red line represents loess shear strength of friction (ϕ) 30° and cohesion (c) of 5 kPa.

4.1.1 Model sensitivity to groundwater

The effects of changing water content of the loess on the slope factor of safety were modelled using variable cohesion values. The results from this sensitivity assessment are shown in Figure 26.

The sensitivity of the slope factor of safety to changes in transient ground water (pore pressure) has been simulated by modelling an initial piezometric line at: 1) rockhead for mechanisms 1 and 2; and 2) at the weak clay layer for mechanism 3. Piezometric head levels were increased, from the initial starting level, at given increments. Results are shown in Table 16 and Figure 26.

Mechanism 1:

- The results show that an increase in piezometric head levels of 3 m above rockhead reduces the factor of safety to approximately one, indicating that failure mechanism 1 is relatively sensitive to increases in piezometric head levels.
- When the weak volcanic colluvium layer is included in the model, a 1 m rise in the piezometric head level is needed to reduce the factor of safety to below one.

Mechanism 2:

- A small increase in piezometric head levels of approximately one metre above rockhead is needed to reduce the factor of safety to approximately one. This suggests that failure mechanism 2 is relatively sensitive to increases in piezometric head levels.

Mechanism 3:

- Adopting the low bulk strength parameters for the mixed volcanic breccia and lava (of friction (ϕ) of 27° and cohesion (c) of 44 kPa), a piezometric head level of approximately 13 m above the weak clay layer is needed to reduce the factor of safety to less than one (adopting parameters of friction (ϕ) of 12° and cohesion of 0 kPa for the clay).
- Using the mid rock bulk strength parameters (of friction (ϕ) of 38° and cohesion (c) of 78 kPa), a piezometric head level of approximately 20 m above the weak clay layer is needed to reduce the factor of safety to less than one.

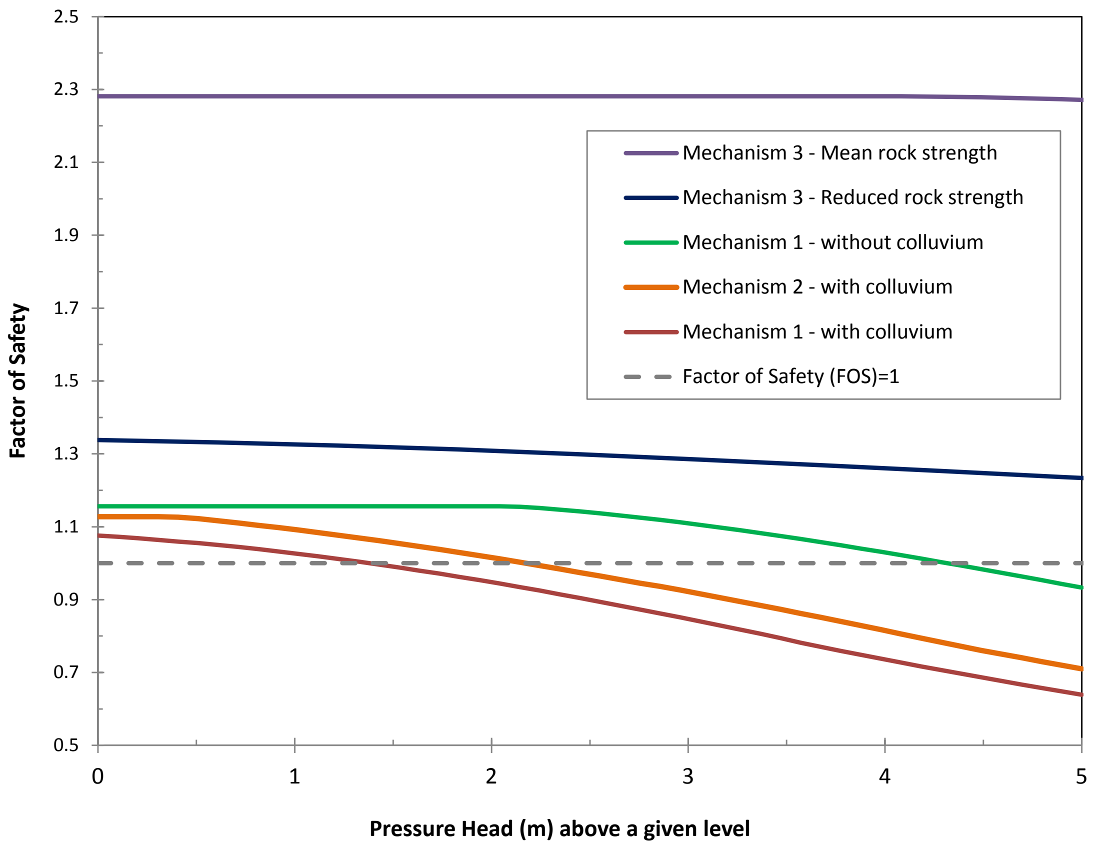


Figure 26 Sensitivity of the slope factor of safety (cross-section 4) in response to changing piezometric head levels, and the effects on the assessed failure mechanisms 1–3.

4.2 Slope Stability – Dynamic Conditions

Dynamic stability assessment comprised: 1) back-analysing the performance of the slope during the 2010/11 Canterbury earthquakes to calibrate the models and check that the calculated 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. One cross-section has been assessed (cross-section 4) under dynamic conditions, assuming a drained slope.

4.2.1 Amplification of ground shaking

The first stage of the assessment was to calculate the maximum acceleration at the cliff crest (A_{MAX}) to quantify any amplification effects caused by topography and or contrasting materials. Results from the dynamic site response assessment are contained in Appendix 4.

Results from this assessment suggest that modelled peak acceleration at the cliff crest (A_{MAX}) varies approximately linearly with the peak ground acceleration of the free-field input motion. Over the range of modelled peak horizontal accelerations, the peak ground acceleration amplification factor (S_T) for cross-section 4, is about 3.1 (±0.1) for horizontal motions and 3.2 (±0.1) for vertical motions, times the input free-field peak accelerations. The input peak accelerations are those derived from the synthetic free-field rock outcrop earthquake time acceleration histories described by Holden et al. (2014).

The results from cross-section 4, 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 using the decoupled method (Makdisi and Seed, 1978) and the Slope/W software. The three failure mechanisms assessed were: 1) shallow failure in loess; 2) failure through the volcanic colluvium underlying the loess; and 3) failure of the rock mass through the slickensided clay. For each failure mechanism, a range of slide surfaces were assessed. Permanent displacements were estimated along the slide surface, where the displacing mass was treated as rigid-plastic body and no internal plastic deformation of the mass was accounted for, and the mass accrued no displacement at accelerations below the yield acceleration.

The synthetic rock outcrop earthquake time acceleration histories from the 22 February, 13 June and 23 December 2011 earthquakes were used as inputs for the modelling, as permanent coseismic displacement of the Maffeys Road slopes were recorded or inferred (from site observations) during these events. The synthetic rock outcrop earthquake time acceleration histories from the 16 April 2011 earthquake were also modelled to ensure that either no modelled movement or very minor (undetectable) movement of the slopes occurred. Variable material strength parameters were used for the critical materials present, and the different parameters used in the modelling are listed in Table 17.

For these assessments, the displacements inferred from the cadastral and monitoring surveys are assumed to represent the coseismic permanent displacement of the slopes, along cross-section 4, during the 22 February, 13 June and 23 December 2011 earthquakes. The results from each modelled scenario were then compared to the recorded coseismic permanent slope displacements for each earthquake.

Table 17 Material strength parameters used for modelling permanent coseismic displacements for cross-section 4. Coseismic displacements are inferred from survey records and field mapping of cracks.

Mechanism	Earthquake	Material	Cohesion c (kPa)	Friction ϕ (º)	Total recorded coseismic displacement (m)
1	22 Feb, 13 Jun, 16 Apr, and 23 Dec 2011	Loess Loess Loess Mixed basalt lava and	10 20 30 38	30 30 30 78	0.4–0.5 (22 Feb) 0.1–0.15 (13 Jun) 0 (16 Apr) <0.05 (23 Dec)
2	22 Feb, 13 Jun, 16 Apr, and 23 Dec 2011	Loess Volcanic colluvium Volcanic colluvium Volcanic colluvium Volcanic colluvium Mixed basalt lava and breccia	20 0 0 5 10 38	30 28 30 30 30 78	0.4–0.5 (22 Feb) 0.1–0.15 (13 Jun) 0 (16 Apr) <0.05 (23 Dec)
3	22 Feb, 13 Jun, 16 Apr, and 23 Dec 2011	Loess Volcanic colluvium Mixed basalt lava and breccia Slickensided tuff (clay)	20 0 78 0	30 30 38 12	0.4–0.5 (22 Feb) 0.1–0.15 (13 Jun) 0 (16 Apr) <0.05 (23 Dec)

The results from the modelling of the 22 February and 13 June 2011 earthquakes, adopting the parameters listed in Table 17, are summarised in Table 18, and Figures 27–34 show the results for the different failure mechanisms.

Table 18 Results from the dynamic modelling of cross-section 4. Total inferred coseismic displacements are from measurements of survey marks. Yield accelerations and permanent displacements are calculated from the decoupled assessment and represent the modelled slide surface with the lowest yield acceleration for the given material parameters and failure mechanism. Those rows highlighted in grey represent the material parameters that give the best correlation between the modelled and recorded permanent displacements, for a given earthquake and failure mechanism.

Failure mechanism	Earthquake	Failure method and critical material	Friction ϕ (º)	Cohesion c (kPa)	Lowest yield acceleration (g)	Modelled coseismic displacement (m)	Total inferred coseismic displacement (m)
1	22 February 2011	Circular, loess	30	10	0.18	1.8	0.4–0.5
1	22 February 2011	Circular, loess	30	20	0.36	0.3	0.4–0.5
1	22 February 2011	Circular, loess	30	30	0.5	0.2	0.4–0.5
1	13 June 2011	Circular, loess	30	10	0.18	0.1	0.1–0.15
1	13 June 2011	Circular, loess	30	20	0.37	0.02	0.1–0.15
1	13 June 2011	Circular, loess	30	30	0.50	0	0.1–0.15
2	22 February 2011	Block, colluvium	28	0	0.08	1.8	0.4–0.5
2	22 February 2011	Block, colluvium	30	0	0.11	1.2	0.4–0.5
2	22 February 2011	Block, colluvium	30	5	0.18	0.6	0.4–0.5
2	22 February 2011	Block, colluvium	30	10	0.24	0.4	0.4–0.5
2	13 June 2011	Block, colluvium	28	0	0.11	0.2	0.1–0.15
2	13 June 2011	Block, colluvium	30	0	0.11	0.2	0.1–0.15
2	13 June 2011	Block, colluvium	30	5	0.18	0.1	0.1–0.15
3	22 February 2011	Block, clay layer	12	0	0.30	0.1	0.4–0.5
3	13 June 2011	Block, clay layer	12	0	0.30	0	0.1–0.15

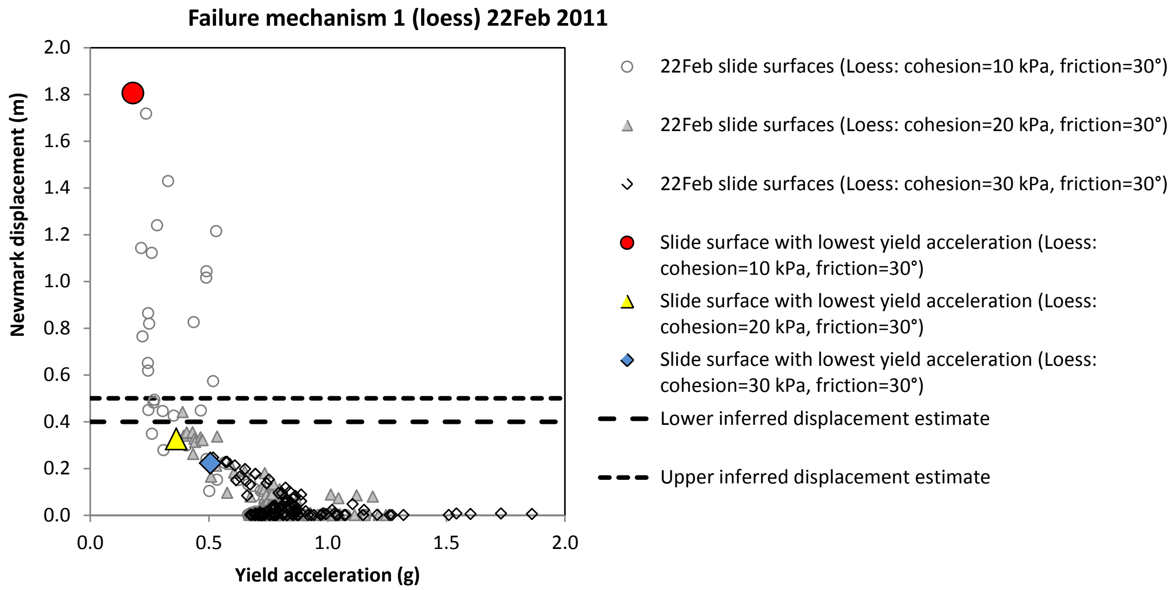


Figure 27 Modelled Slope/W decoupled displacements of cross-section 4 for the 22 February 2011 earthquake and adopting variable estimates of the material strength of the loess. Each datapoint represents a modelled circular slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic permanent displacement for the cross-section during the given earthquake.

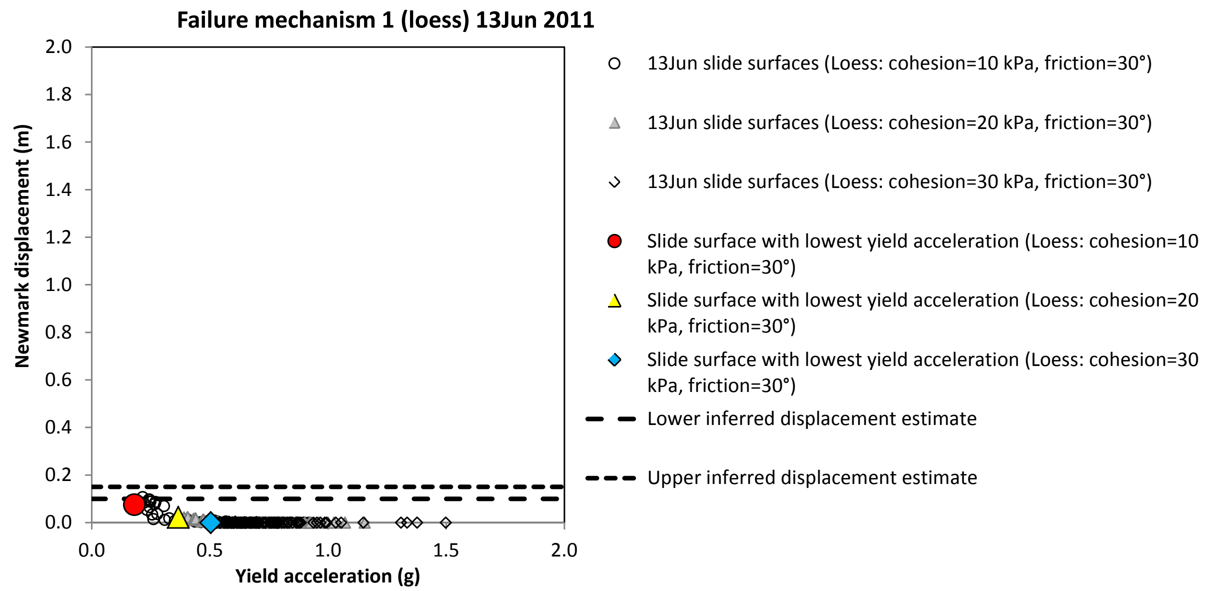


Figure 28 13 June 2011 earthquake, modelled Slope/W decoupled displacements for cross-section 4, and adopting variable estimates of the material strength of the loess. Each datapoint represents a modelled slide surface and the corresponding estimate of its displacement as a result of the 13 June 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the inferred coseismic permanent displacement for the cross-section during the given earthquake.

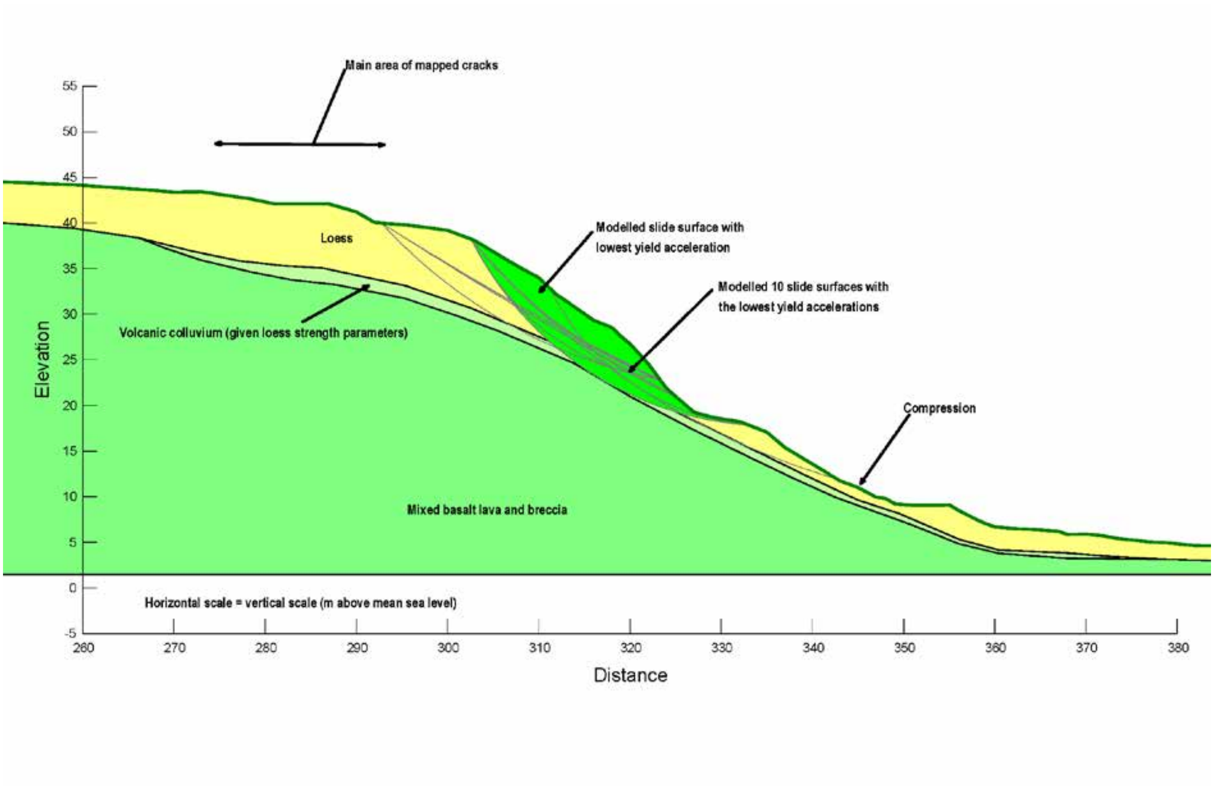


Figure 29 Results from the seismic slope stability assessment for failure mechanism 1, cross-section 4, for the 22 February 2011 earthquake.

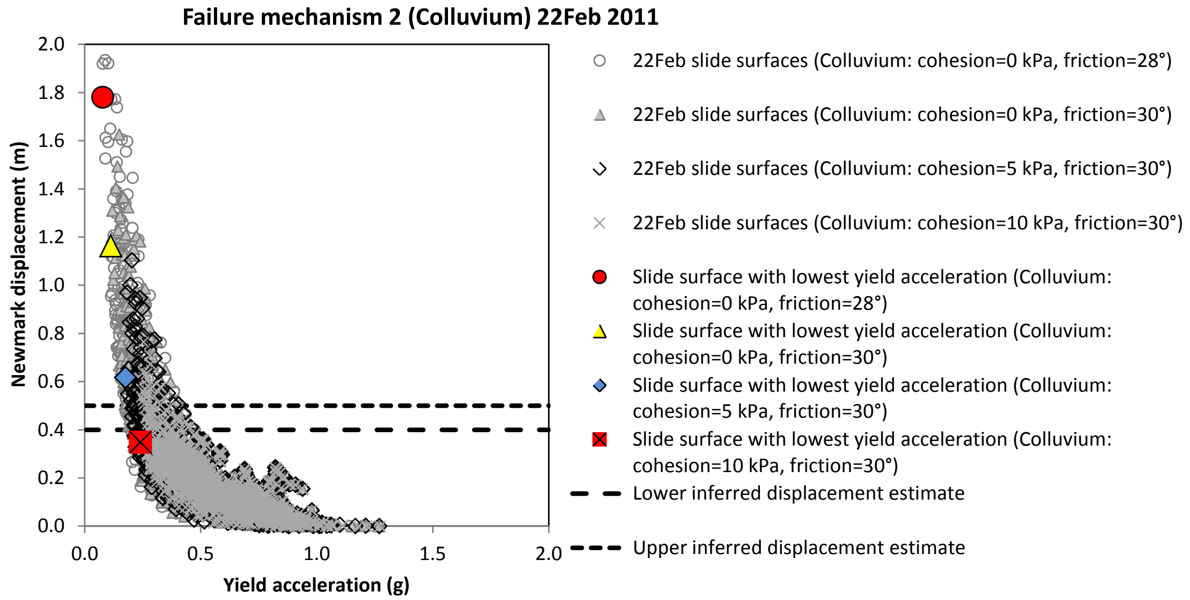


Figure 30 22 February 2011 earthquake, modelled Slope/W decoupled displacements for cross-section 4, and adopting variable estimates of the material strength of the volcanic colluvium. Each datapoint represents a modelled block slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic permanent displacement for the cross-section during the given earthquake.

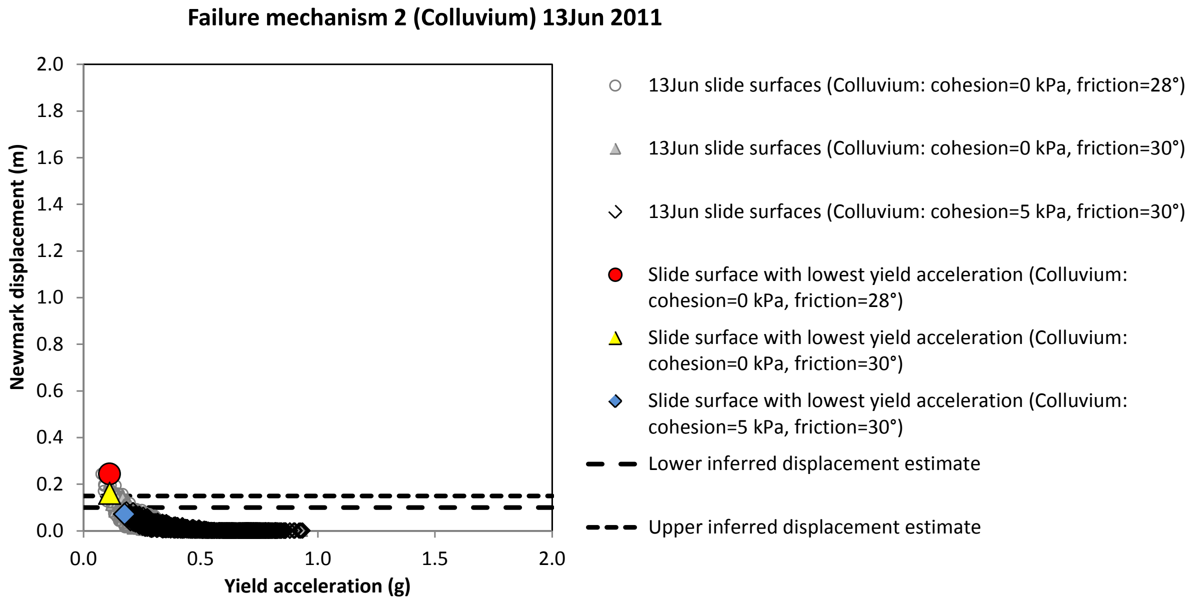


Figure 31 13 June 2011 earthquake, modelled Slope/W decoupled displacements for cross-section 4, and adopting variable estimates of the material strength of the volcanic colluvium. Each datapoint represents a modelled block slide surface and the corresponding estimate of its displacement as a result of the 13 June 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic permanent displacement for the cross-section during the given earthquake.

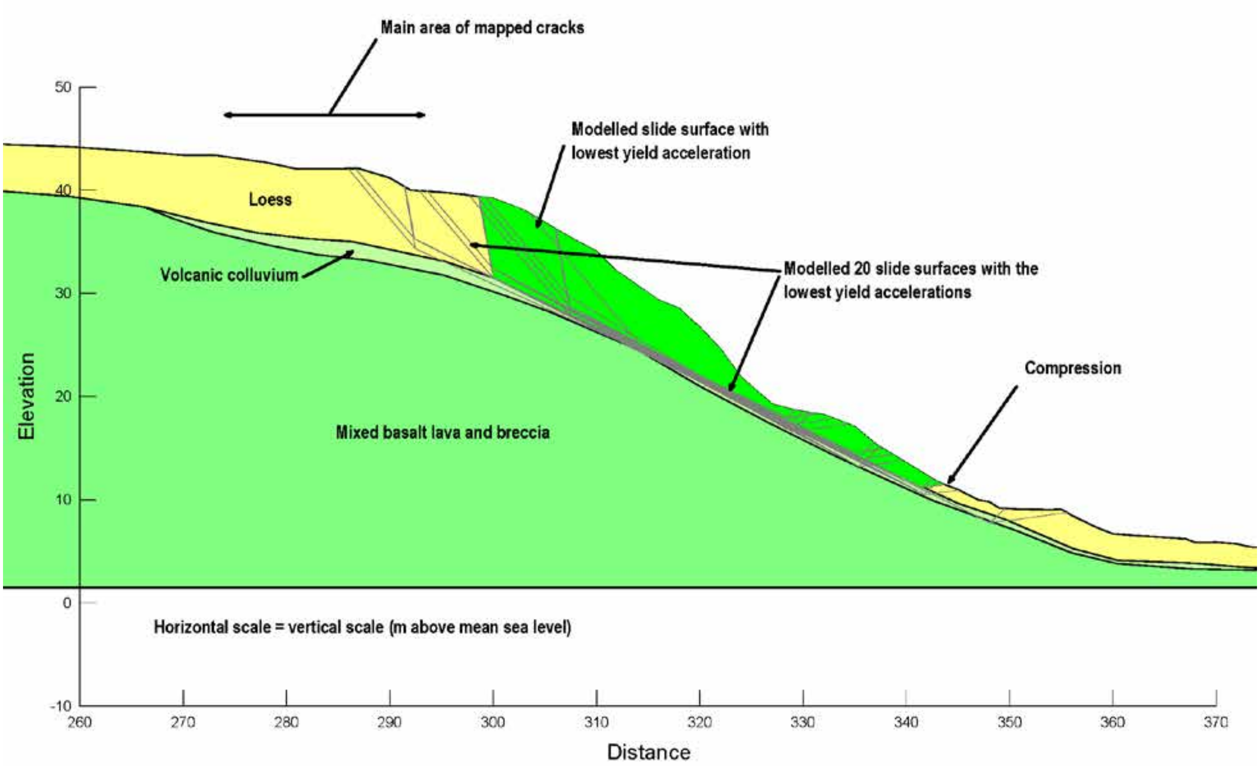


Figure 32 Results from the seismic slope stability assessment for failure mechanism 2, cross-section 4, for the 22 February 2011 earthquake.

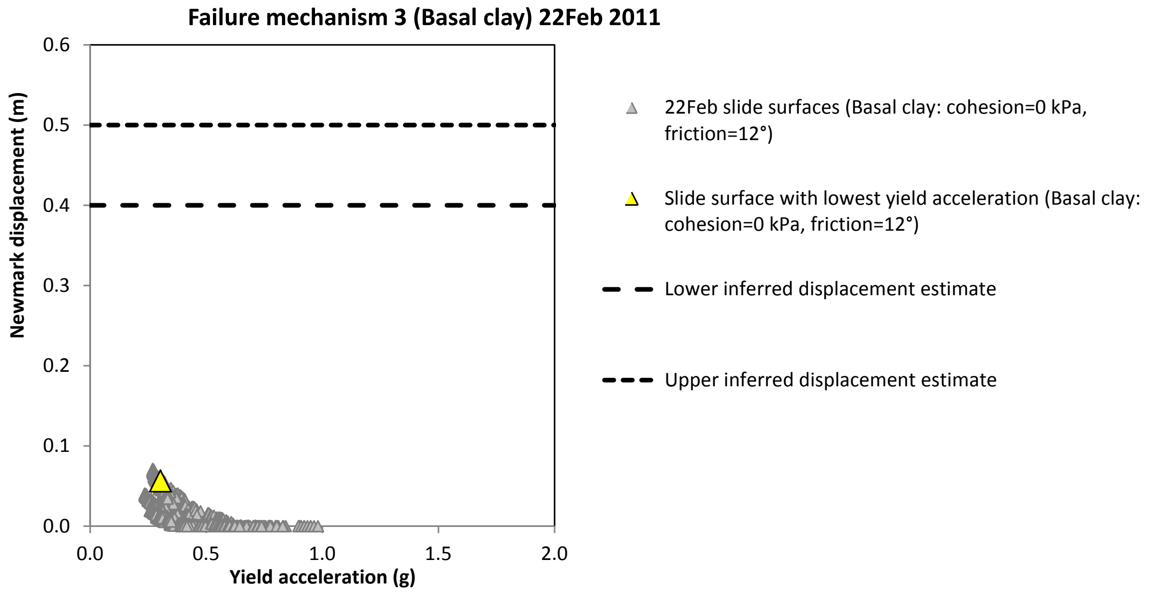


Figure 33 22 February 2011 earthquake, modelled Slope/W decoupled displacements for cross-section 4, and adopting variable estimates of the material strength of the volcanic colluvium. Each datapoint represents a modelled block slide surface and the corresponding estimate of its displacement as a result of the 22 February 2011 earthquake – adopting the synthetic free-field rock outcrop earthquake acceleration time histories. The dashed lines represent the total inferred coseismic permanent displacement for the cross-section during the given earthquake.

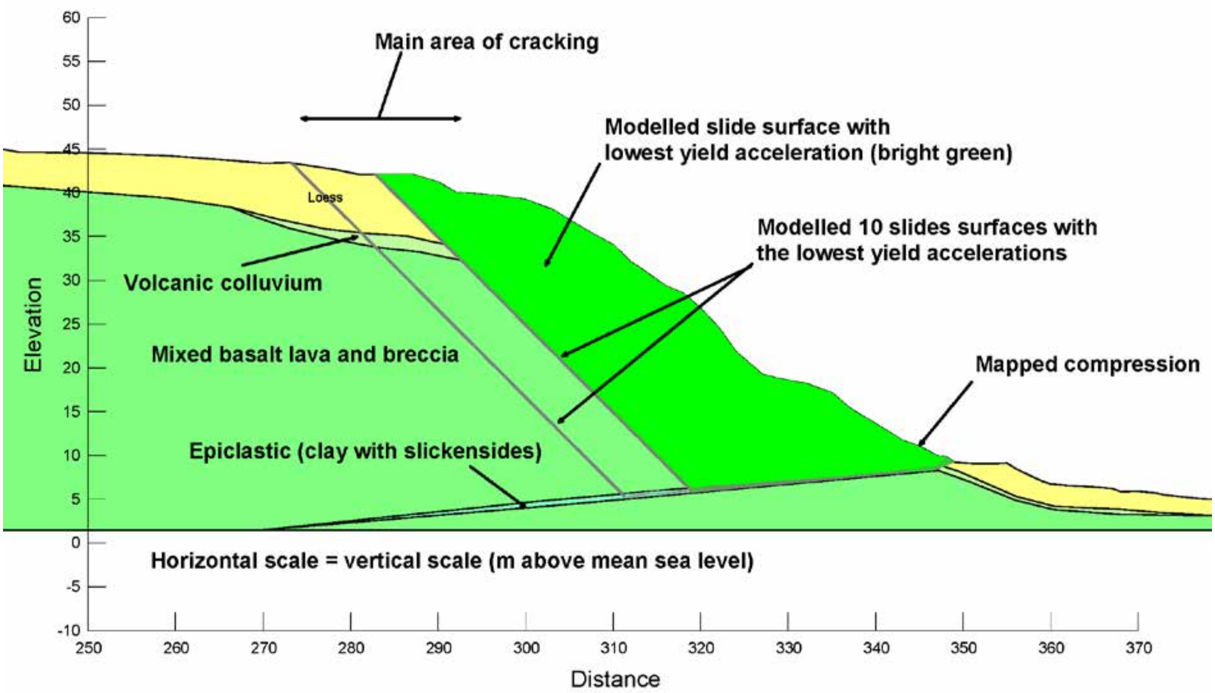


Figure 34 Results from the seismic slope stability assessment for failure mechanism 3, cross-section 4, for the 22 February 2011 earthquake.

The results show that:

Mechanism 1:

- A good correlation between the recorded permanent coseismic displacements and modelled displacements for the 22 February and 13 June 2011 earthquakes was obtained for modelled slide surfaces adopting shear strength parameters for the loess of cohesion (c) of 20 kPa and friction (ϕ) of 30°, for the 22 February 2011 earthquake, and cohesion (c) of 10 kPa and friction (ϕ) of 30° for the 13 June 2011 earthquake.
- The lowest yield accelerations for the modelled slide-surface geometries is 0.2–0.4 g (adopting strength parameters for the loess of cohesion 10–20 kPa and friction (ϕ) of 30°).
- Modelled permanent displacements for the 23 December 2011 earthquake was less than 0.01 m, adopting shear strength parameters for the loess of cohesion (c) 10–20 kPa and friction (ϕ) of 30°. For the 16 April 2011 earthquake, the estimated displacements were zero, adopting the same material parameters.
- There is a good correlation between the locations and shape of the slide surfaces derived from the limit equilibrium and finite element static stability modelling, and those from the dynamic modelling. However, the locations of the modelled slide surfaces with the lowest yield accelerations do not correspond particularly well with the locations of the mapped cracks and slope toe deformation.

Mechanism 2:

- A good correlation between the recorded permanent coseismic displacements and modelled displacements for the 22 February and 13 June 2011 earthquakes was obtained for modelled slide surfaces adopting shear strength parameters for the volcanic colluvium of cohesion (c) of 5–10 kPa and friction (ϕ) of 30°, for the 22 February 2011 earthquake, and cohesion (c) of 0–5 kPa and friction (ϕ) of 30° for the 13 June 2011 earthquake.
- The lowest yield accelerations for the modelled slide-surface geometries is 0.1–0.2 g (adopting strength parameters for the colluvium of cohesion 0–10 kPa and friction (ϕ) of 30°).
- Modelled permanent displacements for the 23 December 2011 earthquake were less than 0.01 m, adopting shear strength parameters for the colluvium of cohesion (c) 0–10 kPa and friction (ϕ) of 30°. For the 16 April 2011 earthquake, the estimated displacements were zero, adopting the same material parameters.
- There is a good correlation between the locations and shape of the slide surfaces derived from the limit equilibrium and finite element static stability modelling, and those from the dynamic modelling. There is also a good correlation between the locations of the modelled slide surfaces with the lowest yield accelerations and the locations of the mapped cracks and slope toe deformation.

Mechanism 3:

- There is a good correlation between the mapped cracks at the surface and the modelled slide surface with the lowest yield accelerations.

- The estimated displacements for mechanism 3, on their own, could not account for the total inferred displacement of the slope, even by adopting the lower rock shear strength parameters (cohesion (c) of 44 kPa and friction (ϕ) of 27°, which are at the lower end of the range thought to be credible), and the lowest credible shear strength of the clay layer (cohesion (c) of 0 kPa and friction (ϕ) of 12°).
- However, it is still possible that movement through the weak clay layer within the rock did occur, but that any such displacements would be minor in comparison with those estimated for mechanisms 1 and 2.
- Modelled permanent displacements in response to the modelled 16 April, 13 June and 23 December 2011 earthquakes were zero.
- The lateral persistence and strength of such a weak layer is uncertain and cannot be constrained by the current field mapping and drillholes alone.
- The lowest yield acceleration for the modelled slide-surface geometries is about 0.3 g, which is at the upper end of the calculated yield accelerations for mechanisms 1 and 2.

4.2.3 Forecast modelling of permanent slope deformation

Permanent displacements, from the decoupled assessment results from the 22 February and 13 June 2011 modelled earthquakes, were calculated for a range of slide-surface geometries with different ratios of yield acceleration (Ky) to the maximum average acceleration of the failure mass (K_{MAX}). The maximum average acceleration (K_{MAX}) was calculated for each selected slide surface by taking the maximum value of the average acceleration time history from the response to the synthetic earthquake. About 10 slide surfaces were chosen to represent the results from each earthquake input motion, adopting different estimates of the shear strength of the key materials (listed in Table 17).

The results from the assessment, adopting failure mechanisms 1 and 2, are shown in Figure 35 for those slide surface shown in Figures 27–34. The results show that between Ky/K_{MAX} values of 0.1 and 0.5, and Ky/A_{FF} values of 0.1 and 0.8, the data are well fitted to a straight line (exponential trend line) in semi-log space. The coefficient of determination (R^2) is 0.78 for Ky/K_{MAX} and 0.62 for Ky/A_{FF}, and includes all of the plotted data (N = 30). The gradients of the fitted lines are different, with the Ky/K_{MAX} line having a slightly steeper gradient, indicating, as expected, that for the same magnitude of displacement the ratio of Ky/K_{MAX} is lower than the corresponding Ky/A_{FF} ratio. The poorer coefficient of determination for ratios of Ky/A_{FF} is not unusual as Newmark (1965) displacements are highly sensitive to the high frequency components of the input motions, which can vary from event to event. By comparison, K_{MAX} "filters" the higher frequency components, and thus is less sensitive to the input motion characteristics.

The peak ground acceleration of the input motion (A_{FF}) does not take into account amplification effects caused by the slope geometry, and at this site, the material contrasts within the slope, between the fill and underlying rock (Appendix 4). From the data in Figure 35, the mean ratio of K_{MAX} to A_{FF} for cross-section 4 is 1.9 (±0.4 at one standard deviation), meaning that K_{MAX} is on average 1.9 times greater than the peak horizontal ground acceleration of the input motion.

For ratios of Ky/A_{FF} in Figure 35, the estimated magnitudes of displacement are consistent with those reported by Jibson (2007), where these data plot between the ranges for earthquakes of M6.5–7.5 as reported by Makdisi and Seed (1978) and plotted by Jibson (2007).

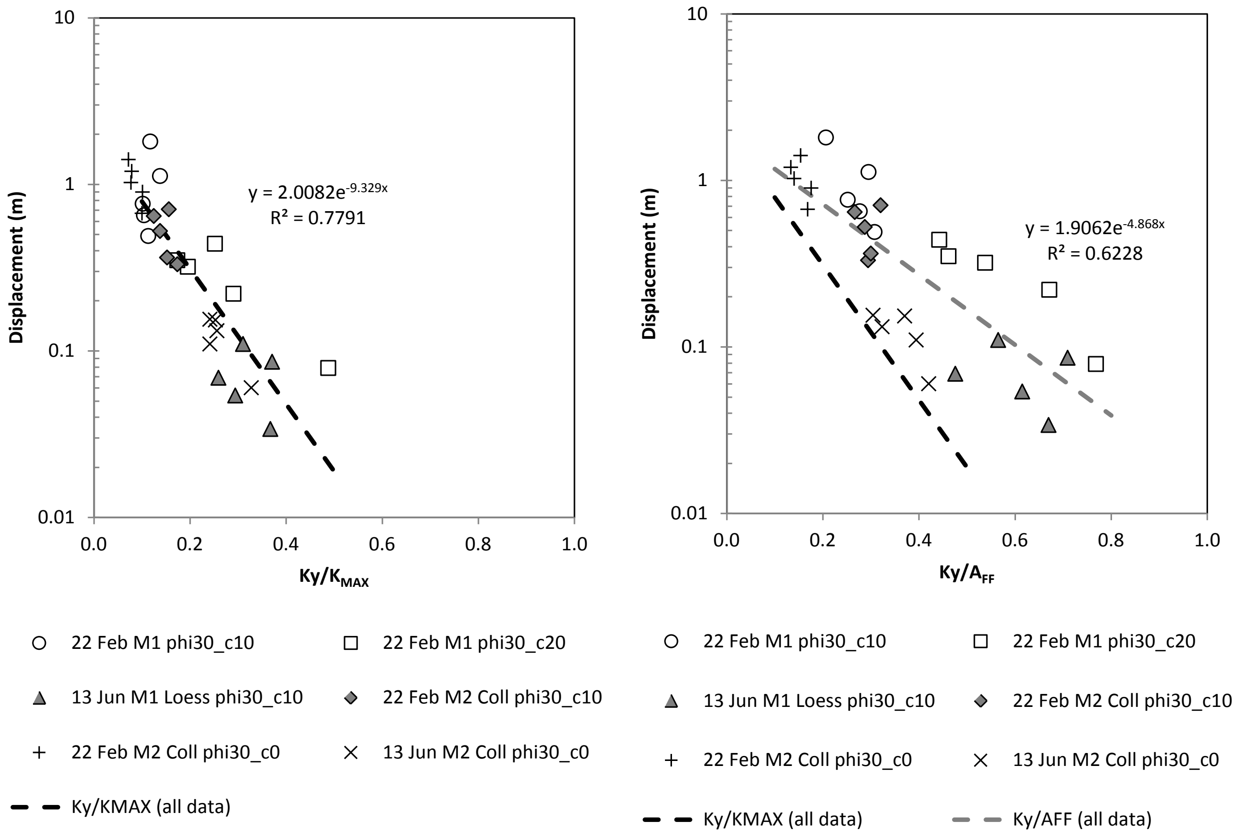


Figure 35 Cross-section 4, failure mechanisms 1 and 2 (M1 and M2). Decoupled Slope/W displacements calculated for different ratios of yield acceleration to maximum average acceleration of the mass (Ky/K_{MAX}), and maximum acceleration of the mass (Ky/A_{FF}), for selected slide-surface geometries, and given material shear strength parameters. A_{FF} is the peak acceleration of the input earthquake time acceleration history. Synthetic rock outcrop time acceleration histories for the 22 February and 13 June 2011 earthquakes were used as inputs for the assessment. N = 30. The dashed lines are exponential trend lines fitted to the semi–log data. The formula and the coefficient of determination (R^2) for the trend lines are shown.

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 35) 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 19. Conservative yield accelerations have been adopted to take into account the possibility that the current shear strength of the materials is now degraded as a result of the past movement.

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 19 Forecast modelling results from the dynamic slope stability assessment for cross-section 4. 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}[2] to A_{FF} ratio (mean + 1 standard deviation)			2.3			
Equivalent K_{MAX}			0.5	1.2	1.7	2.4
Cross-section	**Failure mechanism**	**Yield acceleration[3] (g)**	**Estimated displacement (m)**			
4	1 and 2	0.1	0.3	0.9	1.1	1.4
4	1 and 2	0.2	0.0	0.4	0.6	0.9
4	1 and 2	0.3	0.0	0.2	0.4	0.6

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

2 K_{MAX} represents the maximum average acceleration of a given slide mass

4.3 SUMMARY OF RESULTS FROM THE STATIC AND DYNAMIC STABILITY ASSESSMENTS

1. In their current state, it is possible for failure of the trial slide surfaces to occur under either static or dynamic conditions. However, material strengths – and therefore the slope factors of safety – may reduce with time (weathering), water content, and further movement of the slope under either static or dynamic conditions. The key results are summarised below.

2. The modelled slide surfaces through the volcanic colluvium (mechanism 2) underlying the loess and the weak clay layer within the rock (mechanism 3), simulated best the locations of cracks and compression features mapped in the field.

    a. Based on the dynamic back-analysis of slope stability, values of cohesion (c) and friction (ϕ) of the volcanic colluvium of about 0–10 kPa and 30°, would achieve the recorded displacements of the slope (during the 2010/11 Canterbury earthquakes) inferred from crack apertures. It should be noted that:

        i. The results show that for the 22 February 2011 earthquake, upper estimates of the strength parameters for the volcanic colluvium are needed in order to simulate displacements matching the recorded permanent displacements.

        ii. For the 13 June 2011 earthquake, lower estimates of the strength parameters for the volcanic colluvium are needed to simulate the recorded displacements.

        iii. These results suggest that the strength parameters of the volcanic colluvium (and loess) may have been lower at the time of the 13 June 2011 earthquakes than at the time of the 22 February 2011 earthquakes.

        iv. Two realistic alternative scenarios could account for the lowered strength: *in situ* moisture content in the colluvium may have been higher in June than in February; or the strength of the volcanic colluvium may have degraded with movement during the 22 February 2011 earthquakes (and been further degraded with movement in June 2011).

3. The static factor of safety of the assessed slope ranges between 1 and 1.3 for failure mechanisms 1 and 2, and for the range of material strength parameters assessed (and verified from back-analysis of stability).
4. The modelled slide surfaces with the lowest factors of safety under static conditions are located close to the steeper part of the slope along the down-slope side of Maffeys Road, while the modelled slide surfaces with the lowest yield accelerations, under dynamic conditions, extend further back up slope and correspond to the main area of cracking.
5. The relatively low static factors of safety suggest that a small increase in pore water pressure (piezometric head levels of 1–3 m above rockhead) within the loess and volcanic colluvium, and/or within open tension cracks, could make the slope unstable under static conditions such as during short duration, high intensity rain, and after long periods of wet weather. Increase in the water content of the materials could reduce the cohesion, thereby reducing the static factor of safety.
6. The relatively low estimated yield acceleration of the slope (0.1–0.4 g), suggests that future earthquakes could easily reactivate the slope, leading to permanent displacements that could be large. The amount of coseismic permanent displacement in an earthquake will depend upon:
    a. The shear strength of the materials at the time;
    b. The pore pressure/water content conditions within the slope; and
    c. The duration and amplitude of the earthquake shaking at the site.
7. However, rainfall-induced failures are likely to be more mobile, and triggering rainfalls occur frequently than triggering earthquakes, and therefore contribute most of the risk.
8. The area has already undergone more than 0.5 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 vulnerable 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).
9. The slopes are also cracked and water can more readily infiltrate the slope. This is particularly important for loess, as its strength is largely controlled by its moisture content (McDowell, 1989; Goldwater, 1990). Larger failures from the slopes are more likely in future large earthquakes and after heavy rain.

4.4 POTENTIAL DEBRIS SOURCE VOLUME ESTIMATION

The likely locations and volumes of potential sources 1–5 have been estimated after consideration of:

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

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

Volumes were calculated by estimating the shape of any future failures as quarter-ellipsoids (half-spoon shaped) (following the method of Cruden and Varnes, 1996) (Figure 36). Estimated volumes are shown in Table 20. Source areas 1–5 represent the range (volume and locations) of potential earth/debris flows that could occur at the site.

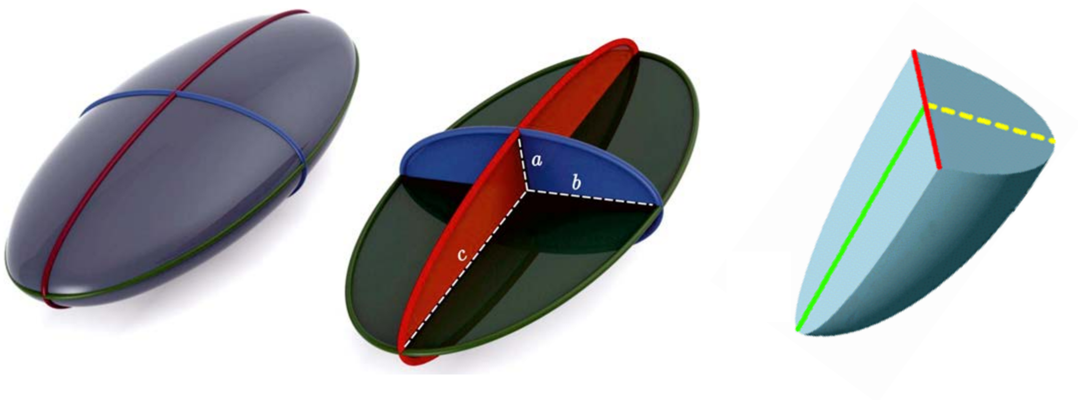


Figure 36 Estimation of landslide volume assuming a quarter-ellipsoid shape.

Table 20 Example of estimated debris flow source volumes (the first digit in the number is significant) and corresponding fahrboeschung angles for Maffeys Road assessed source areas 1–5.

Assessed source area	Slide surface	Source depth (m)	Source length (m)	Source width (m)	Volume estimate (m^3)	Fahrboeschung angle Mean (º)	Fahrboeschung angle Mean - 1STD (º)
	MIN	1.9	13	26	340	23.1	15.2
1	**MID**	**2.9**	**17**	**30**	**770**	**21.4**	**13.9**
	MAX	3.9	21	34	1460	20.1	13.0
	MIN	2.9	23	26	910	21.1	13.7
2	**MID**	**3.9**	**27**	**30**	**1650**	**19.9**	**12.8**
	MAX	4.9	31	34	2700	19.0	12.2
	MIN	3.9	25	20	1020	20.8	13.5
3	**MID**	**4.9**	**29**	**24**	**1780**	**19.7**	**12.7**
	MAX	5.9	33	28	2850	18.9	12.1
	MIN	4.2	30	14	920	21.0	13.7
4	**MID**	**5.2**	**34**	**18**	**1670**	**19.9**	**12.8**
	MAX	6.2	38	22	2710	19.0	12.2
	MIN	2.9	26	8	320	23.3	15.3
5	**MID**	**3.9**	**30**	**12**	**740**	**21.5**	**14.0**
	MAX	4.9	34	16	1390	20.2	13.1

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 37 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 20 (about 320–2,900 m^3) are well within the range of other loess landslides in the Port Hills mapped by Townsend and Rosser (2012).

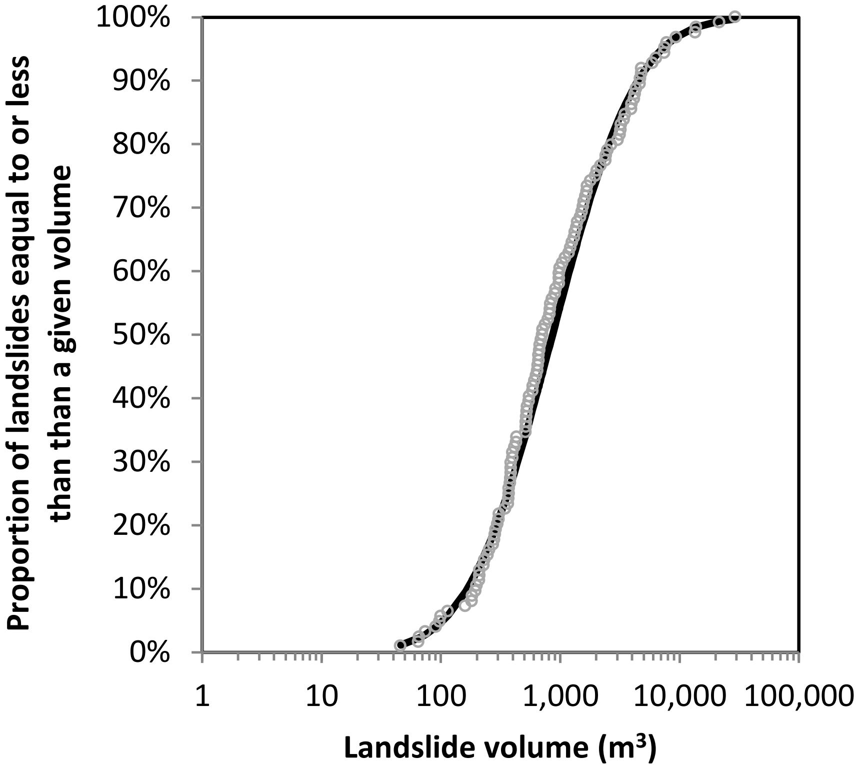


Figure 37 Estimation of landslide volumes in the Port Hills loess from Townsend and Rosser (2012), adopting the area depth relationships of Larsen et al. (2010).

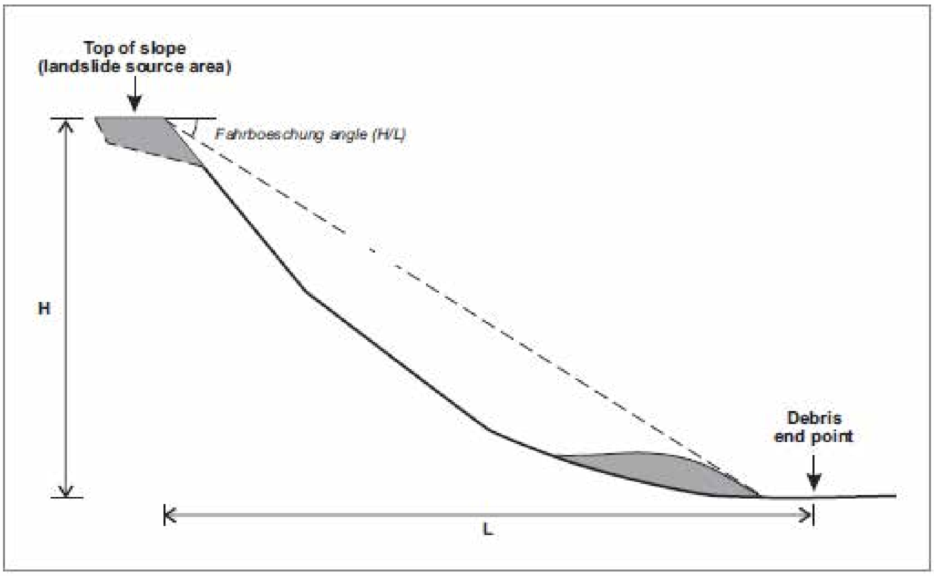
4.5 POTENTIAL DEBRIS RUNOUT

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

4.5.1 Empirical data

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

Figure 38 shows the estimated mean and mean minus one standard deviation debris flow fahrboeschung angles derived for assessed source areas 1–5. For each source a range in fahrboeschung angles is estimated based on the range in volumes (min, mid, max) as shown in Table 20.



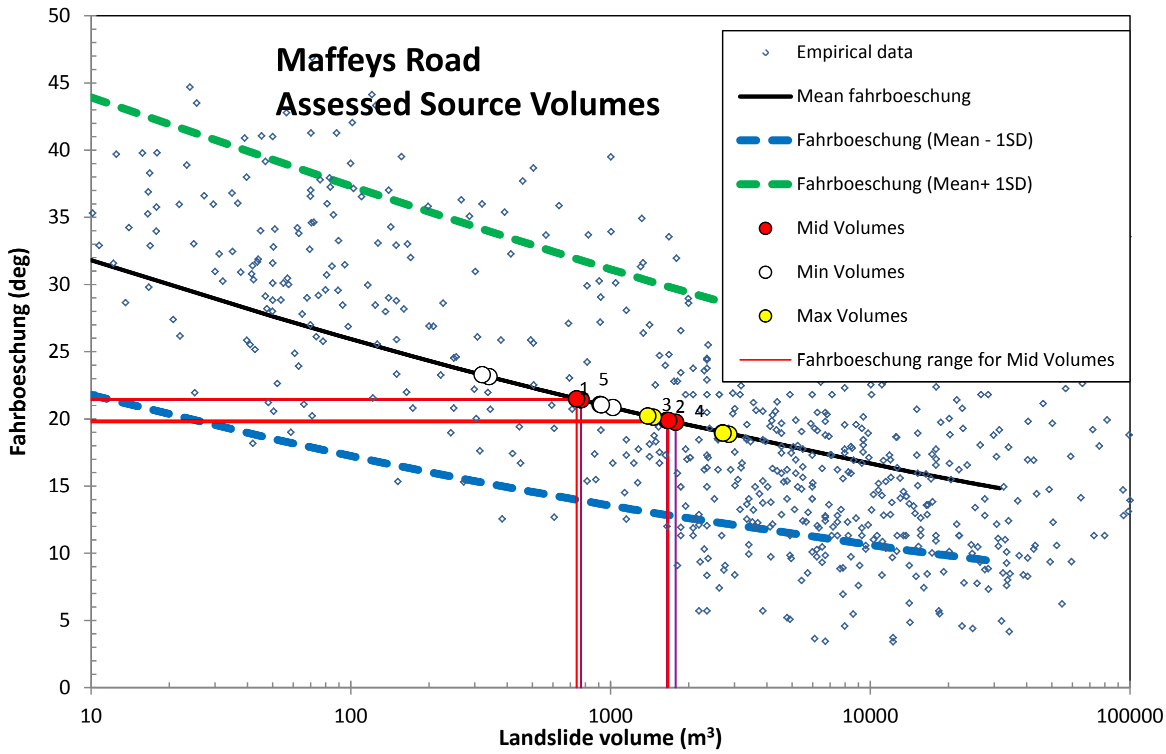


Figure 38 Estimation of debris flow fahrboeschung angles based on empirical runout data presented by Massey and Carey (2012).

4.5.2 Numerical modelling

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. RAMMS software takes into account the slope geometry of the site when modelling debris runout.

The RAMMS model parameters were calculated from the back-analysis of four Port Hills debris flows. The modelled parameters μ and *xi* were optimised to obtain a good correlation between the modelled versus actual runout and deposited debris heights. The model, with calibrated input parameters (μ = 0.06 (7°) and x_i = 200 m/s^2), were used to estimate the likely velocity and depth of the debris at given locations down the slope for the given failure volumes. The μ and x_i values are comparable to results from other assessments compiled by Andres (2010) for debris flows (Figure 39).

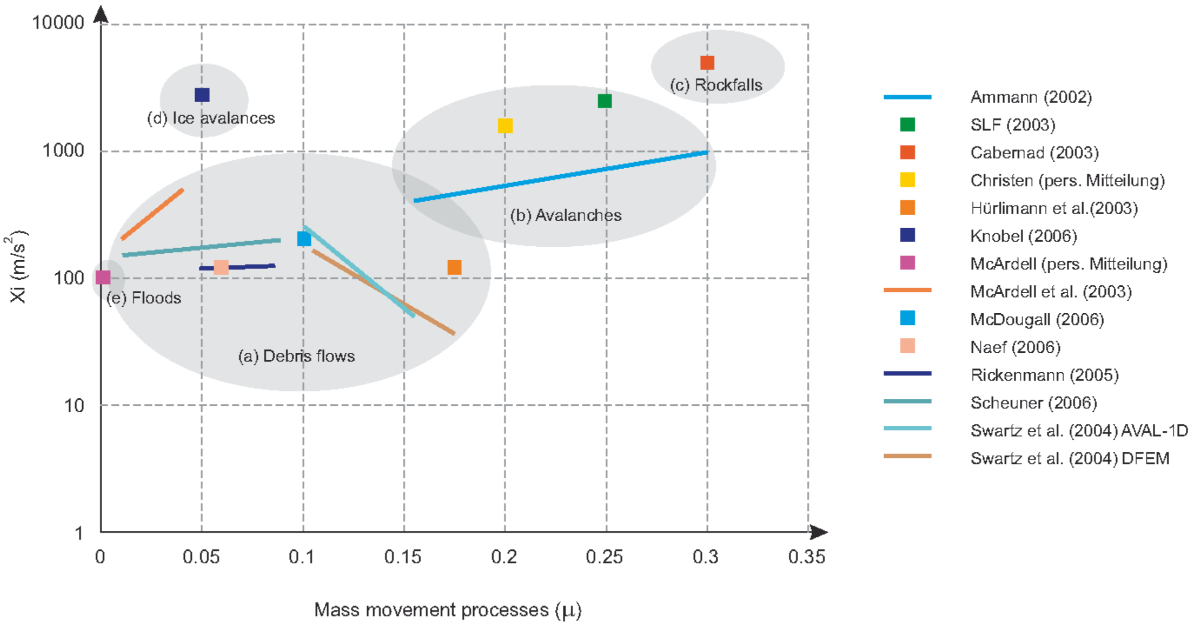
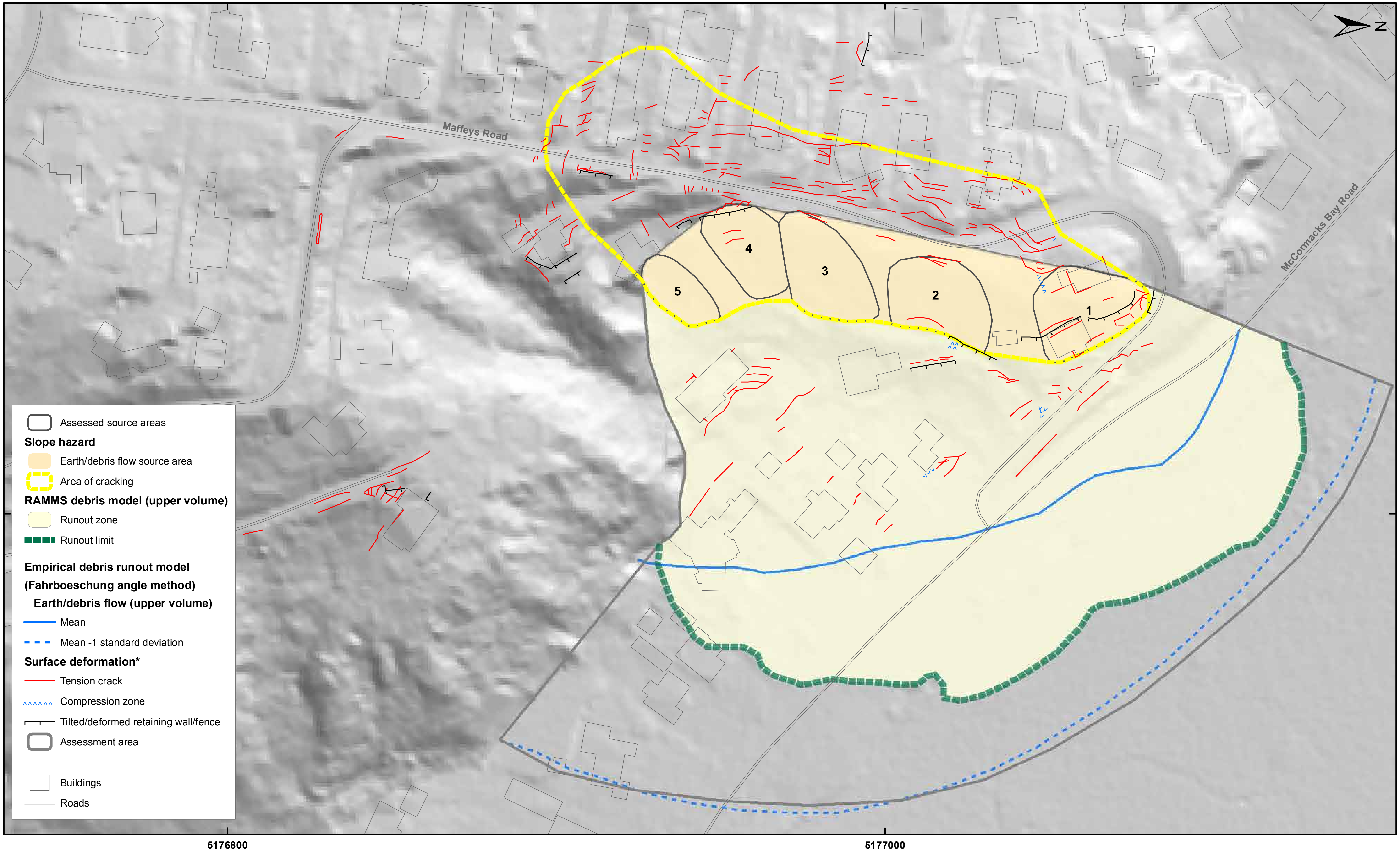


Figure 39 Range of parameters for different assessed source areas processes: **a**) debris flows, **b**) snow avalanches, **c**) snow avalanches, **d**) ice avalanches, **e**) debris floods. Modified from Andres (2010).

4.5.3 Forecast runout modelling

A hazard map (Figure 40) presents the empirical and numerical runout limits from all of the modelled source areas (1–5) for the upper volume source estimates.

The mean and mean minus one standard deviation fahrboeschung angles for each potential source area (1–5) are shown in Figure 40. 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 1–5, along with the corresponding mean and mean minus one standard deviation fahrboeschung angles.



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

Maffeys Road
Christchurch

FIGURE 40

FINAL

REPORT: CR2014/79

DATE: June 2014

5.0 RISK ASSESSMENT RESULTS

5.1 TRIGGERING EVENT FREQUENCIES

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

5.1.1 Frequency of earthquake triggers

For earthquake triggers, the frequency of a given free-field peak ground acceleration occurring is obtained from the National Seismic Hazard Model of New Zealand (Table 21) (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.

Total permanent slope displacements during the 2010/11 Canterbury earthquakes estimated from crack apertures were about 0.1–0.7 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 area of cracking still suffered significant damage.

The estimated magnitude of permanent slope displacement, in the assessed source areas and area of cracking (Figure 40), in a future earthquake, was based on the decoupled assessment results. 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 35). 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 0.5 g, assuming a ratio of 2.4.

Table 21 The annual frequency of a given peak ground acceleration occurring on rock (Site Class B) for different years adopting the 2012 National Seismic Hazard Model 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	2.3 (mean + 1 STD)			
Equivalent K_{MAX} for the given A_{FF}	0.5	1.2	1.7	2.4
Estimated displacements (m) for *Ky* of 0.1	**0.3**	**0.9**	**1.1**	**1.4**
Estimated displacements (m) for *Ky* of 0.2	**0.0**	**0.4**	**0.6**	**0.9**
Estimated displacements (m) for *Ky* of 0.3	**0.0**	**0.2**	**0.4**	**0.6**

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 35 is based on past performance of the slope; however, the slope moved about 0.7 m during the 2010/11 Canterbury earthquakes. It is now possible that catastrophic failure of the slope could occur at further permanent slope displacements of less than 0.7 m. At free-field peak ground accelerations of greater than 1.0 g the amount of permanent displacement could lead to catastrophic failure and runout of debris, as performance of the slope at these levels of ground acceleration is unknown.

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 35 represent displacements in response to these earthquakes (adopting failure mechanisms 1 and 2). Earthquakes of longer duration may affect the site in different ways. For example, the response of the loess and volcanic colluvium (at higher water contents representative of winter conditions) may be non-linear, and could lead to larger permanent displacements. Conversely, the peak amplitudes relating to longer duration earthquakes from more distant sources are likely to be lower and may not trigger displacement of the slope.

It should also be noted that parts of the slope have already moved more than 0.7 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 probabilistic seismic hazard model

The seismic performance of the slope in future earthquakes was inferred from assessing its performance in past earthquakes, mainly the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, using the relationship established between peak ground acceleration and the amount of permanent slope displacement. These earthquakes varied in magnitude between M5.2 and M6.3, and were “near-field” i.e., their epicentres were very close, within 10 km, to the Maffeys Road site.

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

Buxton and McVerry (personal communications 2014) suggest that it is magnitude M5.3–6.3 earthquakes on unknown active faults, within 20 km of the site that contribute most to the 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 historic and prehistoric earth/debris flows at the site.
- The 5 March 2014 rainstorm in Lyttelton (130 mm), triggered several large earth/debris flows. The return period of the rainfall at Lyttelton was about 100 years, but the lower amount of rainfall at the site (89 mm) had a return period of only about 10–20 years.
- It is likely that the slope could fail if the water content of the loess increases, but the likelihood of this happening is not known.
- Even though the slope survived a 10–20 year return period event, the loess water content of the slope 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 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 (1–5) 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 41 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.

5.2.1 Earth/debris flow risk

The risk from earth/debris flows from source areas 1–5, adopting the estimated lower, middle and upper source volumes and a return period of 50 years for the triggering event, are shown in Figure 41.

Based on the results from the static and dynamic stability assessments, the potential earth/debris flow source areas are located along the crest of the steeper slope (Figure 41). 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 41.

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

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

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

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

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 41 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.

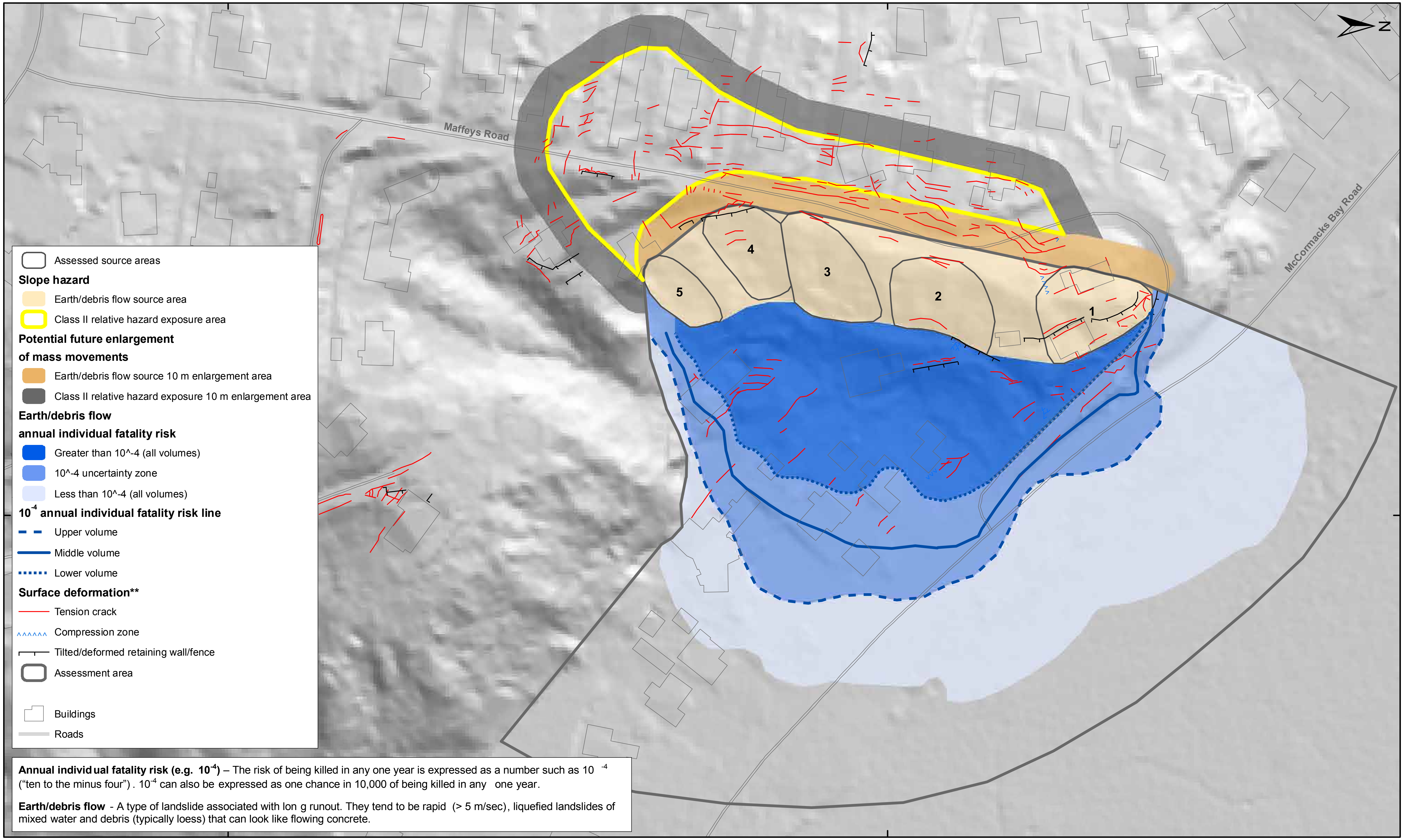
5.3 SLUMPING AND CRACKING

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 zone has been included around the 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-section 4 are shown for various values of peak free field ground accelerations (A_{FF}) in Table 21, for the area behind the slope crest as shown in Figure 39.



5176800 5177000

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 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 ANNUAL INDIVIDUAL FATALITY RISK

Maffeys Road Christchurch

FIGURE 41

FINAL

REPORT: CR2014/79

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 42. The risks adopting the 200 year triggering event frequency are all less than 10^{-4} and cannot therefore be plotted in Figure 42.

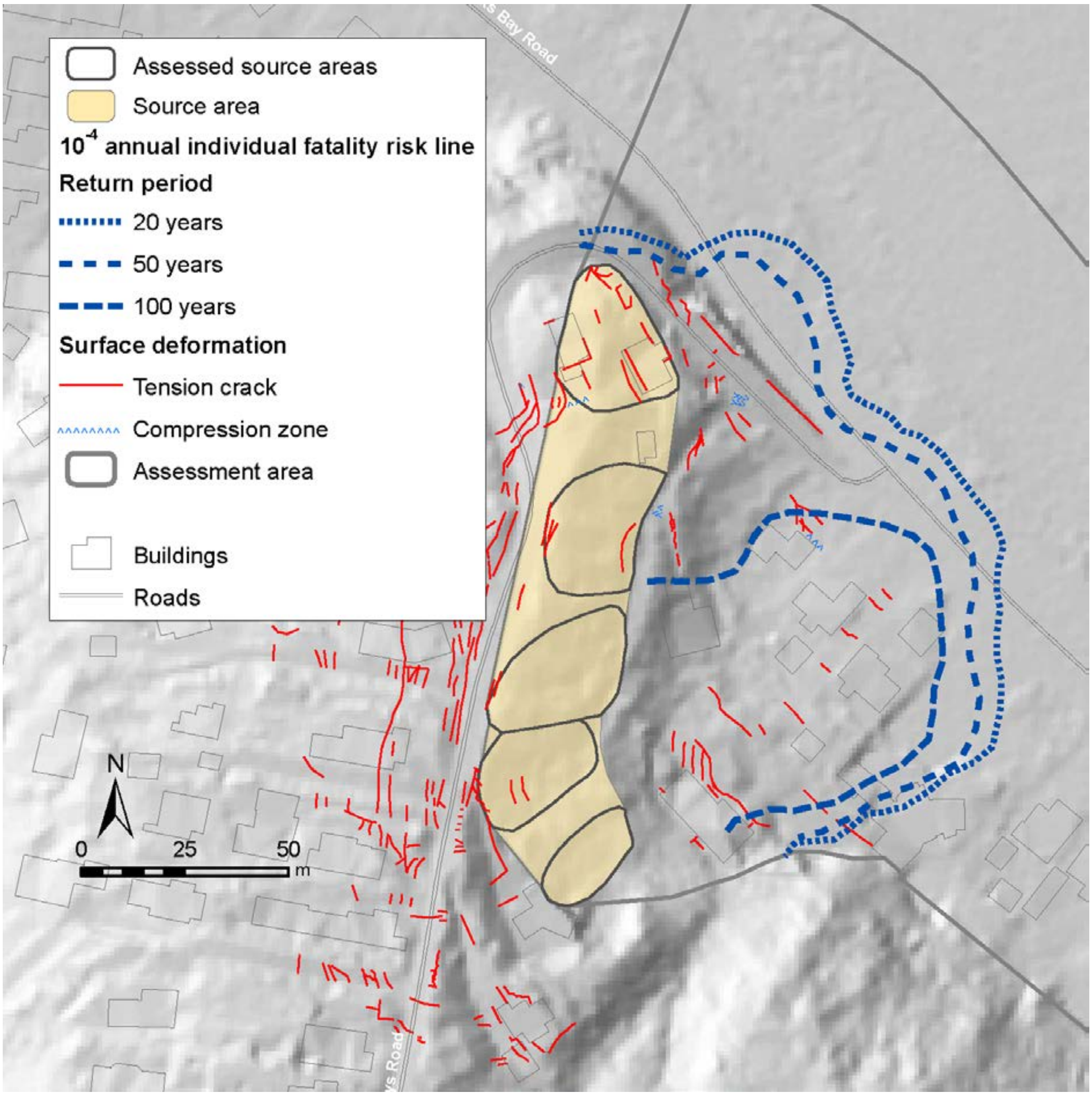

Figure 42 Sensitivity of the risk estimates, upper volume estimates, for triggering event return periods of 20, 50, 100 and 200 years. Note: No 200 year 10^{-4} annual individual fatality risk line is shown, as the risk is less than 10^{-4} for this return period.

The results show that area within the 10^{-4} risk contour reduces between the 20 year 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 41 is considered reasonable.

6.0 DISCUSSION

6.1 DWELLING OCCUPANT RISK

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

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 several dwellings located within the "greater than 10^{-4}" annual individual fatality risk zone;
3. There are a few dwellings located within the assessed source areas where the annual individual fatality risk is inferred to be greater than 10^{-4}; and
4. There are several dwellings located in the Class II relative hazard exposure area and the associated 10 m enlargement area, behind the "earth/debris flow source 10 m enlargement area".

6.2 ANNUAL FREQUENCY OF THE EVENT

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

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

No permeability or loess 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 underlying the slope 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 and colluvium 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, 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.
- 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.

6.3 RISK ASSESSMENT SENSITIVITIES AND UNCERTAINTIES

In this section the sensitivity of the risk model to key uncertainties such as source volumes, annual frequency and vulnerability are discussed, and the reliability of the assessments is identified.

6.3.1 Source volumes

The effect of changing the volumes of earth/debris flows that could be triggered in future events. This was done by comparing the three volume ranges (lower, middle and upper) which account for variation in the likely source volumes. The three volume ranges also took into account variability in the debris runout velocities and inundation heights, as larger volumes of debris tend to travel further down slope at higher velocities.

There are quite large differences between the positions of the 10^{-4} annual individual fatality risk contours between the modelled lower and upper source volumes. In Figure 41, the numbers of dwellings inside the 10^{-4} risk contours would increase by three, between the upper and lower source volume estimates (Figure 41).

6.3.2 Annual frequency of the triggering event

The effect on the model to changes in the annual frequency of the event that could trigger failure of source areas was assessed by adopting event annual frequencies of 0.05, 0.02, 0.01 and 0.005, corresponding to return periods of 20, 50, 100 and 200 years respectively (Figure 42). Results from the model show:

- That there is little change between the risk results adopting the 20-year and 100-year return periods. The risk at the dwellings in the greater than 10^{-4} annual individual fatality risk zone decreases by a factor of about 2 (for a return period from 50 to 100 years)
- For return periods greater than 100 years the risk is less than 10^{-4} for all assessed source volumes.
- For return periods of less than 50 years the greater than 10^{-4} annual individual fatality risk zone marginally increases, and the risk at the dwellings increases by a factor of 2–3 (for a return period from 50 to 20 years).

6.3.3 Vulnerability

The probability of a person being killed if present and inundated (buried) by debris is a function of debris height and velocity. The risk assessment does not 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 New Zealand-specific data available for use in the risk assessment. It is possible that the risk could reduce by an order of magnitude or more if the dwelling could withstand inundation by debris without collapsing. This could have a large effect on the risk especially in the distal run out zones where the debris is travelling at lower velocity.

6.3.4 How reliable are the results?

The results (Figure 41) 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.

For source areas 1–5 the uncertainties combine to give 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}. Those dwellings in the greater than 10^{-4} risk zone (all source volume estimates), could still be at relatively high levels of risk (10^{-4} or greater) even if some major uncertainties were resolved so as to reduce the risk.

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

Table 22 Uncertainties and their implications for risk.

Issue	Direction and scale of uncertainty	Implications for risk
a. Choice of whether to use different event annual frequencies other than 0.02 (50-year return period)	Moderate uncertainty between the use of the 50-year and 100-year, return periods. But larger uncertainty between the 50 year and 15 year plus return periods., and 50-year and 200-year, return periods	Longer term risk is potentially 2–4 times lower, but shorter term risk could be 2–3 times higher.
b. Volume of debris produced by a source area	Largest uncertainty between upper volume and the lower volume, and then the lower volume and middle volume.	About a factor of 3 between the upper and lower volume estimates. But a factor of 1.7 between the lower and middle estimates, and a factor of 1.5 between the middle and upper estimates.
c. Changing the assumed debris height where probability of inundation = 0 from 0.30.5 m and 0.1 m	Small uncertainty in either direction	Would change modestly by a factor of about 1.2 in either direction.
d. Occupancy (proportion of time people are at home)	Assumption of 100% occupancy instead of 67% would modestly increase the estimated risk.	Would increase slightly by a factor of about 2.
e. Vulnerability (probability of being killed if inundated by debris)	Variable vulnerabilities have been adopted linked to debris velocity. However, the vulnerability of a person in a dwelling is related to the nature of the structure, for which there is no data available for use in the risk assessment. Potentially large uncertainty in either direction but very difficult to quantify.	Could be relatively large depending on the nature of the dwelling construction and age/ability of the person to get out of the way of the debris. Possibly over an order of magnitude uncertainty in either direction

7.0 CONCLUSIONS

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

7.1 HAZARD

1. There is potential for volumes ranging from many tens to several hundreds of cubic metres of earth/debris flows (source areas 1–5) of mixed loess and colluvium.
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. There are several dwellings located in the earth/debris flow runout zone, and at the crest of the assessed source areas. The annual individual fatality risk to dwelling occupants in the runout zone are estimated to range from greater than 10^{-4} to less than 10^{-4}, depending on the location of the dwelling with respect the location of the assessed source areas. Occupants of dwellings closest to the assessed source areas expose their occupants to the highest assessed levels of risk.
2. The uncertainties in the key input parameters used in the risk assessment combine to give an order of magnitude uncertainty in either direction. For example, the estimated risk at a dwelling within the estimated annual individual fatality risk of 10^{-4} per year uncertainty zone could feasibly be as low as 10^{-5}, or as high as 10^{-3}.
3. There are several dwellings located in the assessed source areas. It is difficult to assess what the levels of risk to the dwellings in the source areas are, given the uncertainties associated with the triggering event, source volume and area that could be affected. The levels of risk in the assessed source areas are likely to be similar to the highest values of risk estimated in the associated earth/debris flow runout zones.
4. Even if failure of these sources does not occur under static conditions (rain), the risk of damage to dwellings from future earthquakes is still relatively high. For example, the estimated amount of permanent slope displacement when subjected to 0.5 g peak ground acceleration is in the order of about 0.4 m. A peak ground acceleration of 0.5 g has a 50-year average annual frequency of occurring of about 1 in every 140 years, adopting the results from the National Seismic Hazard Model.

7.2.1 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 water main currently runs along Maffeys Road at the crest of the assessed earth/debris flow source areas, and should if possible be either relocated away from this area or be designed to accommodate any future ground movement ; and
    b. Control surface water seepage by filling the accessible cracks on the slope and providing an impermeable surface cover to minimise water ingress. However, it is not thought that such works alone are sufficient to reduce the risk.

8.3 LONG-TERM ACTIONS

8.3.1 Engineering measures

1. Assess the cost, technical feasibility and effectiveness of alternative longer term engineering and relocation solutions, for example (but not limited to):
   a. Removal/stabilisation of the slopes in the assessed source areas;
   b. Installation of drainage works;
   c. Relocation of houses to alternative locations within existing property boundaries;
   d. Withdrawal and rezoning of the land for non-residential use.
2. 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.

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 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. As the depth of tension crack is unknown, the simulated tension cracks depth was defined based on the relationship of Craig (1997), in order to satisfy the thrust line verification method in the numerical model. In reality the depth of the tension cracks may be much deeper, but their depths are unknown.

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

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

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 Maffeys Road 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 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. This is based on ground investigations by Yetton and Engel (2014) reporting the lack of evidence for significant groundwater mounding under the spurs. 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 of fill – dynamic testing is currently being carried out by GNS Science as part of a research project. Therefore for loess shear modulus and damping ratio functions

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

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

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.

Forecasting permanent slope displacements

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

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

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

A1.1.2 Estimation of slope failure volumes

The 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 Maffeys Road assessed source area slope and other similar slopes, during the Canterbury 2010/11 earthquakes.

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

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

A1.1.3 Debris runout modelling

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

1. Empirical fahrboeschung method:

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

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

2. Numerical methods:

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

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

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

   d. The modelling results give likely debris runout, area affected, volume, velocity and the maximum and final height of debris in a given location at any moment in the runout – refer to Section 4.3 for details of the calibration.

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

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

A1.2 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.

Fatality risk for dwelling occupants

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

1. Divide the entire study area into a series of 1 m by 1 m grid cells.
2. Consider the possible range of triggering events from non-earthquake triggers (mainly rain). The annual frequency of the event (rainfall) that could trigger failure of any of the identified source areas (source area 2) is difficult to estimate given the lack of precedence in the Port Hills. The variation of risk across the slope has, therefore been assessed using a range of event frequencies and earth/debris flow source volumes:
3. 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.
4. 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.
5. 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.
6. Three scenarios were considered based on: 1) lower; 2) middle; and 3) upper estimates of the source volume.
7. Each source volume scenario was assessed as having an equal probability of failure in a given event, of a given annual frequency.
8. 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. This is discussed in a later section.

9. 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.

10. 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).

11. 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 across their home, GNS Science has assumed that Port Hills homes have a footprint area (assuming a single story dwelling) of A_F = 100 m^2. The probability that a person will be occupying a given area within their home at any one time can be expressed as:

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

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

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.

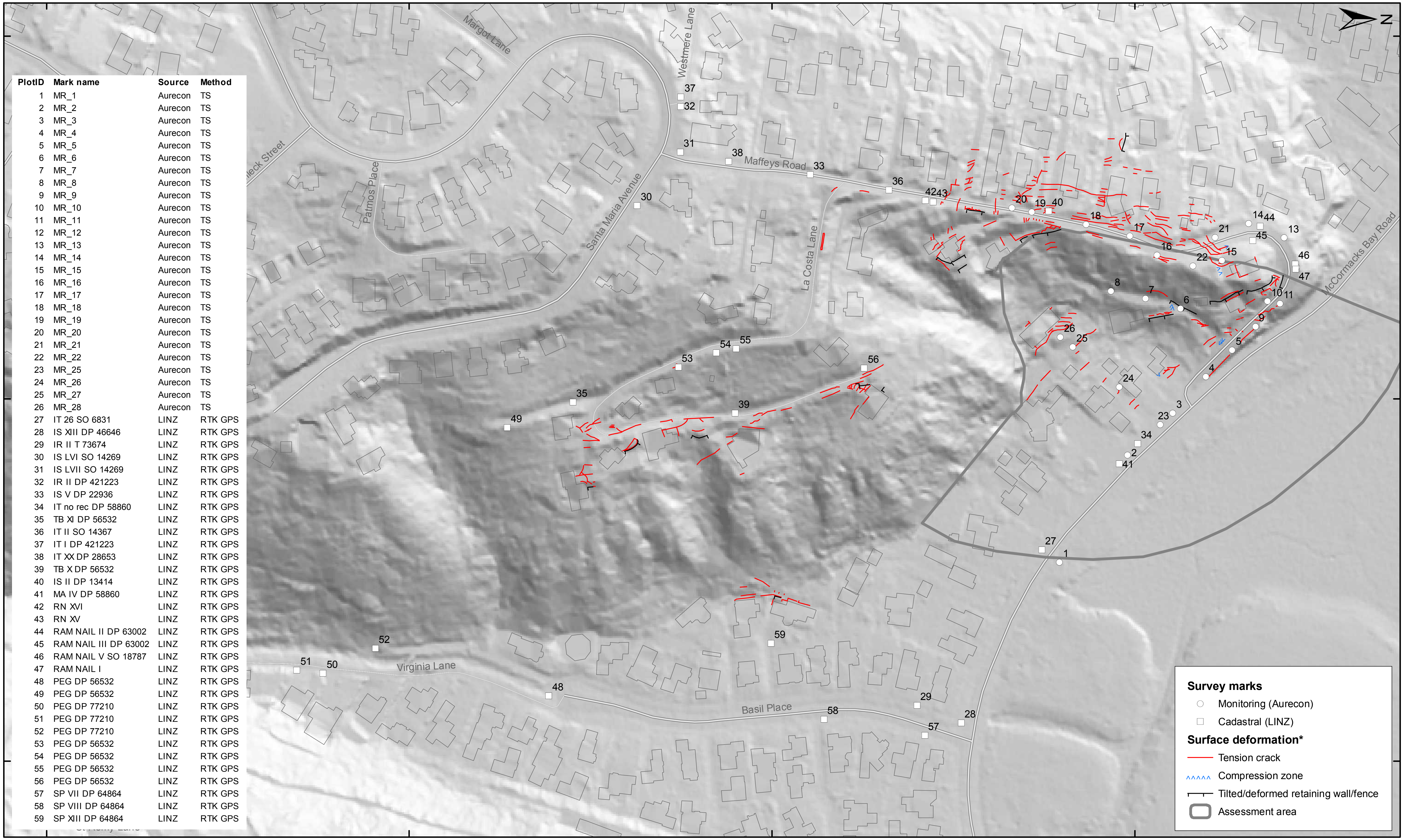
For loess earth/debris flows where the debris tends to be very fluid, it is likely that homes (even wooden ones) would provide some protection from the debris. In this risk assessment the probability of being inundated has been calculated separately as $P_{(S:H)}$. Therefore it would be appropriate to 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 3). The RAMMS model outputs were used to calculate debris velocity at different locations along the earth/debris flow trail, using the ranges given in Table 3.

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

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

A2 APPENDIX 2: RESULTS FROM THE SURVEYING OF CADASTRAL AND MONITORING MARKS INSTALLED IN THE AREA



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24	MR_26	Aurecon	TS
25	MR_27	Aurecon	TS
26	MR_28	Aurecon	TS
27	IT 26 SO 6831	LINZ	RTK GPS
28	IS XIII DP 46646	LINZ	RTK GPS
29	IR II T 73674	LINZ	RTK GPS
30	IS LVI SO 14269	LINZ	RTK GPS
31	IS LVII SO 14269	LINZ	RTK GPS
32	IR II DP 421223	LINZ	RTK GPS
33	IS V DP 22936	LINZ	RTK GPS
34	IT no rec DP 58860	LINZ	RTK GPS
35	TB XI DP 56532	LINZ	RTK GPS
36	IT II SO 14367	LINZ	RTK GPS
37	IT I DP 421223	LINZ	RTK GPS
38	IT XX DP 28653	LINZ	RTK GPS
39	TB X DP 56532	LINZ	RTK GPS
40	IS II DP 13414	LINZ	RTK GPS
41	MA IV DP 58860	LINZ	RTK GPS
42	RN XVI	LINZ	RTK GPS
43	RN XV	LINZ	RTK GPS
44	RAM NAIL II DP 63002	LINZ	RTK GPS
45	RAM NAIL III DP 63002	LINZ	RTK GPS
46	RAM NAIL V SO 18787	LINZ	RTK GPS
47	RAM NAIL I	LINZ	RTK GPS
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51	PEG DP 77210	LINZ	RTK GPS
52	PEG DP 77210	LINZ	RTK GPS
53	PEG DP 56532	LINZ	RTK GPS
54	PEG DP 56532	LINZ	RTK GPS
55	PEG DP 56532	LINZ	RTK GPS
56	PEG DP 56532	LINZ	RTK GPS
57	SP VII DP 64864	LINZ	RTK GPS
58	SP VIII DP 64864	LINZ	RTK GPS
59	SP XIII DP 64864	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

SURVEY MARKS
Index Map

Maffeys Road
Christchurch

APPENDIX 2

Map 1

FINAL

REPORT: CR2014/79

DATE: June 2014

PlotID	Mark name	Offset (mm)
27	IT 26 SO 6831	378
28	IS XIII DP 46646	145
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30	IS LVI SO 14269	20
31	IS LVII SO 14269	147
32	IR II DP 421223	170
33	IS V DP 22936	72
34	IT no rec DP 58860	577
35	TB XI DP 56532	52
36	IT II SO 14367	38
37	IT I DP 421223	170
38	IT XX DP 28653	232
39	TB X DP 56532	68
40	IS II DP 13414	532
41	MA IV DP 58860	528
42	RN XVI	33
43	RN XV	19
44	RAM NAIL II DP 63002	138
45	RAM NAIL III DP 63002	164
46	RAM NAIL V SO 18787	694
47	RAM NAIL I	159
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49	PEG DP 56532	46
50	PEG DP 77210	53
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53	PEG DP 56532	60
54	PEG DP 56532	117
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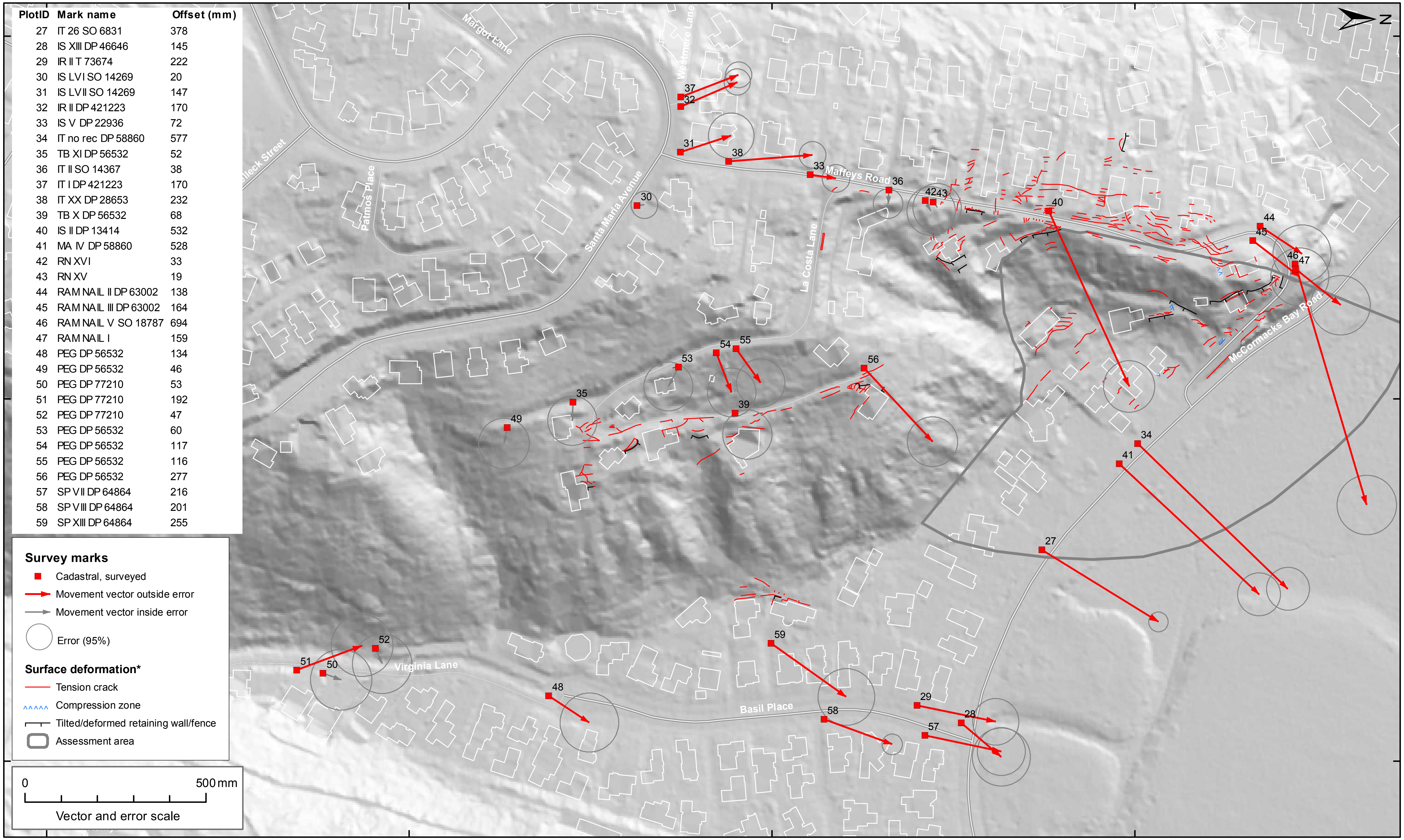


Survey marks
- Cadastral, surveyed
- Movement vector outside error
- Movement vector inside error
- Error (95%)

Surface deformation*
- Tension crack
- Compression zone
- Tilted/deformed retaining wall/fence
- Assessment area

0 500 mm
Vector and error scale

SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from the report CR2012/317

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

DRW: BL

CHK: GA, CM, FDP

GNS SCIENCE TE PŪ AO

MOVEMENT VECTORS
Cadastral Marks (Source: LINZ)
Total Movement - Pre 22-02-2011 to 16-04-2013

Maffeys Road
Christchurch

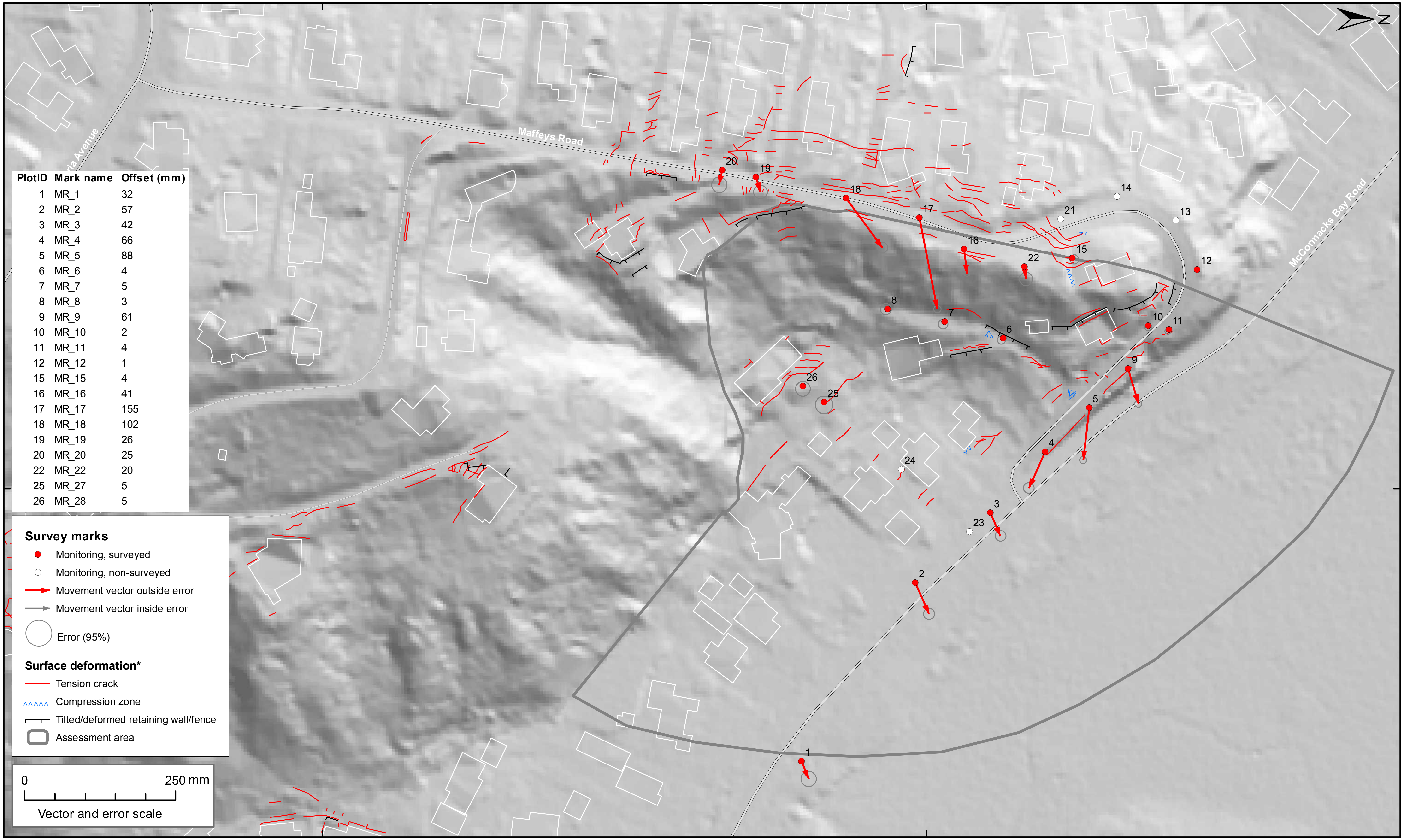
APPENDIX 2

Map 2

FINAL

REPORT: CR2014/79

DATE: June 2014



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9	MR_9	61
10	MR_10	2
11	MR_11	4
12	MR_12	1
15	MR_15	4
16	MR_16	41
17	MR_17	155
18	MR_18	102
19	MR_19	26
20	MR_20	25
22	MR_22	20
25	MR_27	5
26	MR_28	5

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 - 13 Jun 2011 (M 6.0)

Maffeys Road
Christchurch

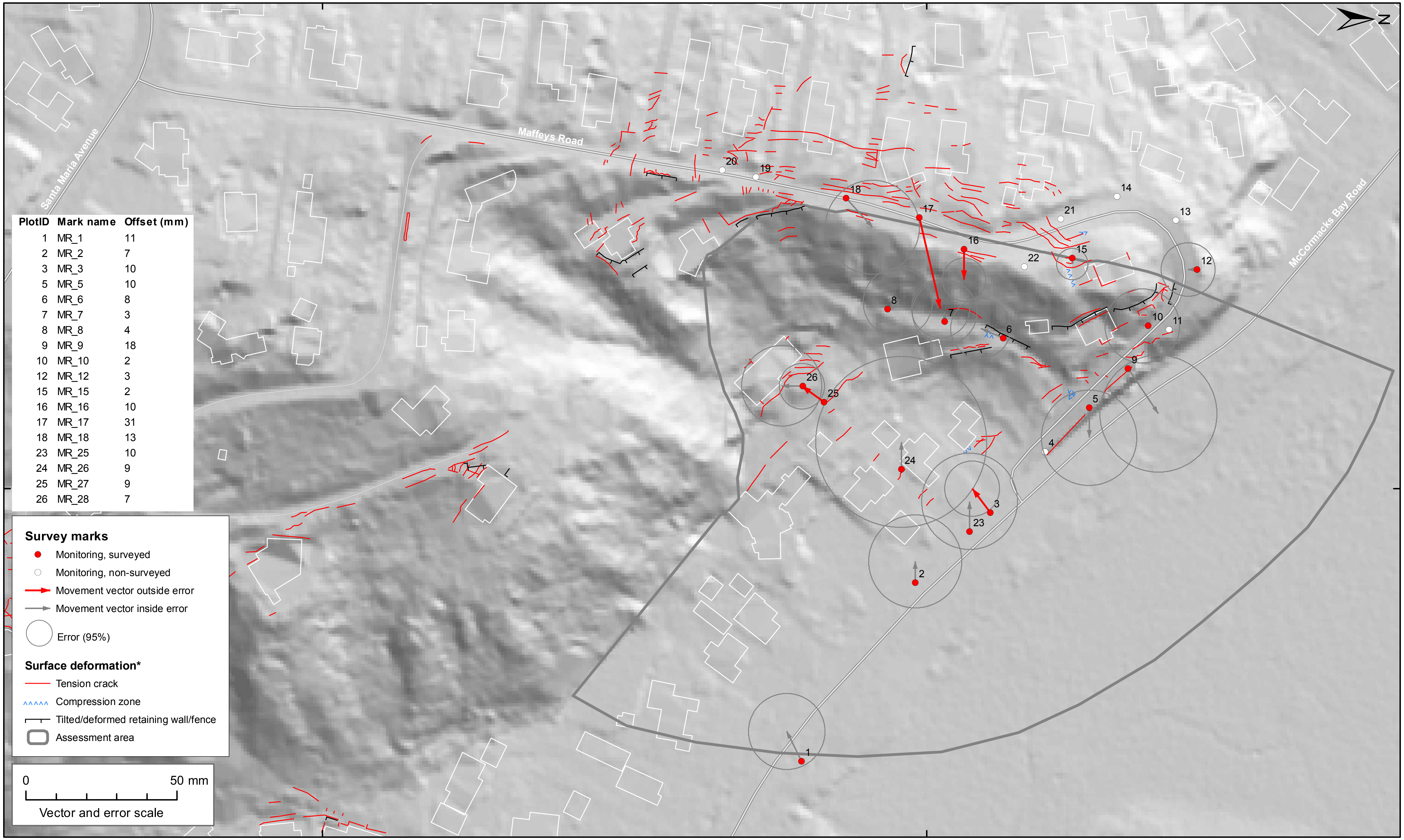
APPENDIX 2

Map 3

FINAL

REPORT: CR2014/79

DATE: June 2014



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6	MR_6	8
7	MR_7	3
8	MR_8	4
9	MR_9	18
10	MR_10	2
12	MR_12	3
15	MR_15	2
16	MR_16	10
17	MR_17	31
18	MR_18	13
23	MR_25	10
24	MR_26	9
25	MR_27	9
26	MR_28	7

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)

Maffeys Road
Christchurch

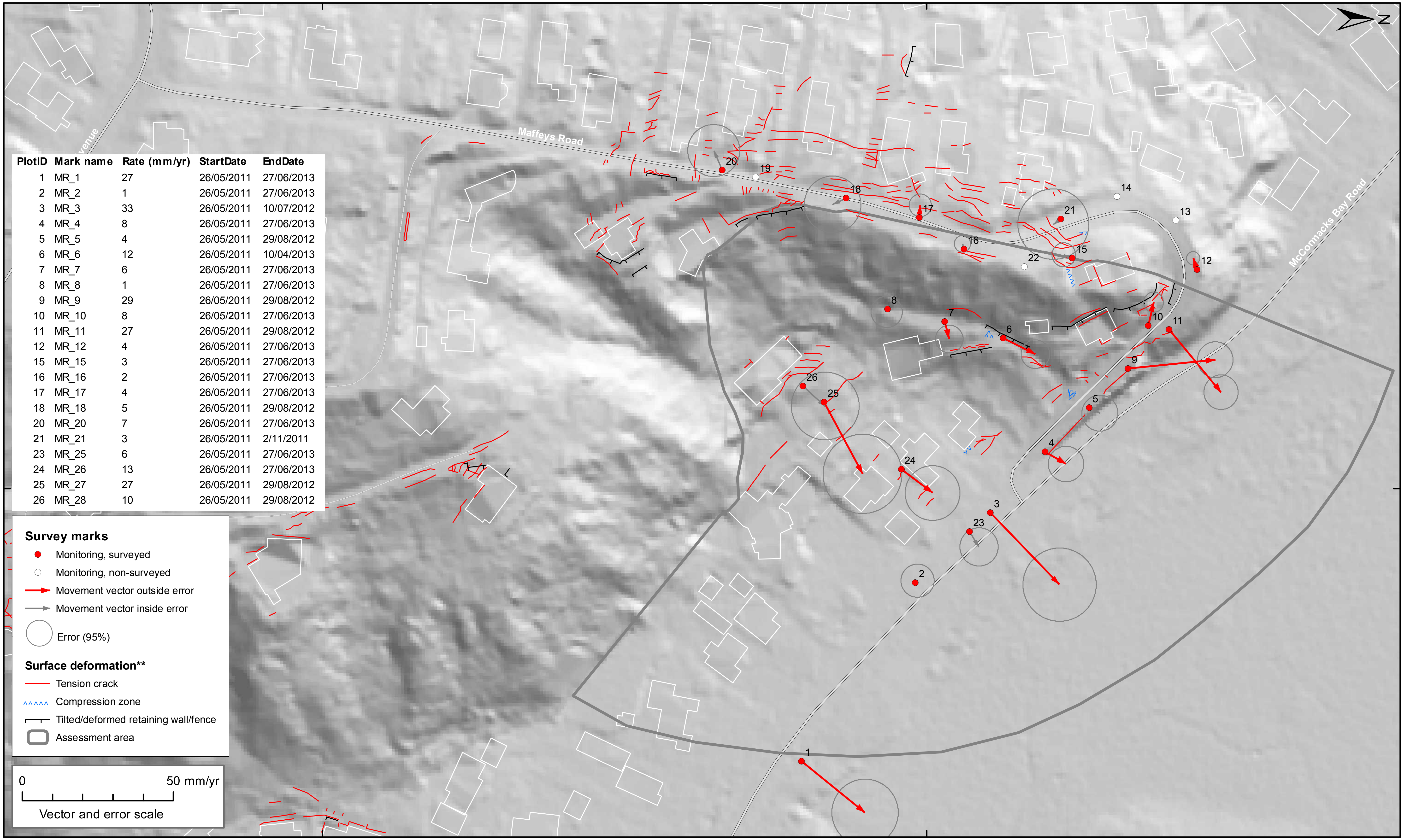
APPENDIX 2

Map 4

FINAL

REPORT: CR2014/79

DATE: June 2014



PlotID	Mark name	Rate (mm/yr)	StartDate	EndDate
1	MR_1	27	26/05/2011	27/06/2013
2	MR_2	1	26/05/2011	27/06/2013
3	MR_3	33	26/05/2011	10/07/2012
4	MR_4	8	26/05/2011	27/06/2013
5	MR_5	4	26/05/2011	29/08/2012
6	MR_6	12	26/05/2011	10/04/2013
7	MR_7	6	26/05/2011	27/06/2013
8	MR_8	1	26/05/2011	27/06/2013
9	MR_9	29	26/05/2011	29/08/2012
10	MR_10	8	26/05/2011	27/06/2013
11	MR_11	27	26/05/2011	29/08/2012
12	MR_12	4	26/05/2011	27/06/2013
15	MR_15	3	26/05/2011	27/06/2013
16	MR_16	2	26/05/2011	27/06/2013
17	MR_17	4	26/05/2011	27/06/2013
18	MR_18	5	26/05/2011	29/08/2012
20	MR_20	7	26/05/2011	27/06/2013
21	MR_21	3	26/05/2011	2/11/2011
23	MR_25	6	26/05/2011	27/06/2013
24	MR_26	13	26/05/2011	27/06/2013
25	MR_27	27	26/05/2011	29/08/2012
26	MR_28	10	26/05/2011	29/08/2012

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*

Maffeys Road
Christchurch

APPENDIX 2

Map 5

FINAL

REPORT: CR2014/79

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.

****

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

THE WEATHER.

Gales in the South.

Landslip Fatality.

(Per United Press Association.)

CHRISTCHURCH, September 22.

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

****

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

BURIED UNDER LANDSLIP.

ONE KILLED TWO INJURED.

AN EXTENSIVE SLIDE.

(By Telegraph—Press Association.)

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

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

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

CANTERBURY LANDSLIDE.

HOWARDS BODY FOUND. MAN WASHED INTO LAKE.

(By Telegraph - Own Correspondent)

CHRISTCHURCH, Saturday.

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

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

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

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

HEAVY FLOODS REPORTED

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

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

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

OTHER LANDSLIDES

Cave near Sumner? – July 1875

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

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

****

The Christchurch Star, Sunday Sept 3 1870

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

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

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

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

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

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

Our Temuka correspondent reports as follows: "This morning the inquiry was, Did you feel the earthquake?" and there was no mistake hut it was felt, and that pretty severely last night. About half-past six p.m. a tremendous rumbling noise was heard, and in a very few seconds the houses and buildings began to shake about in a manner that was certainly anything but pleasant. The motion lasted some seconds, giving unmistakeable evidence as to what it was, and causing the occupiers of houses to vacate the same with all possible speed. The first observation I heard on reaching the road was evidently from a son of the Sister Isle who observed "Faith, this is the first earthquake I ever saw, and I never saw such a big one in my life." But joking apart, the shock was pretty severe, and caused considerable alarm. Most of the brick buildings have sustained damage, and the new store erected by Mr Mendleson has been cracked in many places, rendering it necessary to secure the same by bracing it with iron; and Mr Collins shop felt the effects of the shock. A picture in Dr. Rayner's house was shaken from the wall and the glass broken to pieces, but I do not hear of any real serious injury being the result. A variety of Opinions are expressed as to the direction from which the earthquake proceeded, but I should imagine it was from the north-west and proceeded south east.

Our Waihi Crossing correspondent says: At about a quarter to seven p.m. a severe shock was felt in the neighbourhood of the Waihi Crossing, causing great alarm to the inhabitants, and a sickening sensation was felt by them after the shock, as was plainly visible on their countenances as they flocked together to relate the circumstances. At the Clarendon Hotel the bottles and glasses rattled together on the shelves. It was preceded with a loud rumbling noise, and appeared to move from north-west to southeast. From Oamaru we learn that two very perceptible shocks were felt at about half-past six. Several substantial buildings the Bank of New Zealand among others were visibly shaken, but we have not heard of any actual damage. From Dunedin we learn that there was a smart shock at twenty minutes past six. It lasted for several seconds. The direction was from north to south. No damage done only rang bells and jingled glasses.

Landslide dam failure at Tikau Bay, Akaroa – June 1881

Otago Witness, Issue 1546, 25 June 1881, Page 9

A rather distressing occurrence in connection with the late storm (says the Christchurch Press) took place on the property of Mr A. C. Knight, Tikau Bay, Akaroa. An employe of Mr Knight was living with his wife in a small house near the creek, which it seems had been blocked up with a landslip, thereby causing a stoppage and allowing a large pool of water to get together. The heavy rain of Friday night swelled the creek into a raging torrent, and, the dam giving way, carried the house down the gully, breaking it to pieces with all its contents, the occupants barely escaping with their lives. The poor man not only lost all his clothes and furniture, but £18 in money, which was in his purse. While searching amongst the debris for his money, he discovered his watch, which he had left on a nail in the house, hanging on the branch of a tree, And, strange to say, the watch was going.

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Landslides – Lowry Bay – January 1884

Evening Post, Volume XXVII, Issue 19, 23 January 1884, Page 2

[Wellington]
A very heavy landslip is reported on the upper road to Lowry Bay, entirely blocking it up, and compelling all traffic to deviate to the lower or tidal road. Our informant estimates that the work of clearing a passage must occupy several days even if a strong staff of men should be employed.

Two Italian fishermen had a very narrow escape from sudden death yesterday. They live in a small hut erected at the mouth of a deep gully about three quarters of a mile from Lowry Bay. Owing to the excessive rain of Monday, a heavy landslip occurred during the night in the gully just above this hut. The men were awakened by the rush of the earthy and rocky avalanche that was descending and absolutely brushing past their hut, but, strange to say, without injuring it, although had it been struck fair by any one of the massive boulders, several feet in diameter, which came down in regular volleys, it is morally certain that the building and its inmates would have been crushed to jelly. The two men listened in the utmost terror to the appalling sounds, which they supposed to indicate a tremendous earthquake, and momentarily expected to be dashed into atoms, but the landslip left them unhurt. In the morning they found the face of the immediately adjacent country extraordinarily changed, and were devoutly thankful for their hairbreadth escape.

Star, Issue 5619, 15 May 1886, Page 3

Sumner.

TRAFFIC STOPPED BY A SLIP. [Special to the Star."]
SUMNER, May 15.
A slight slip has taken place on the Sumner road, which has stopped traffic for a time. It is still raining here (12.30 p.m.) Some parts of the township are completely flooded.

****

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-SECTION 4

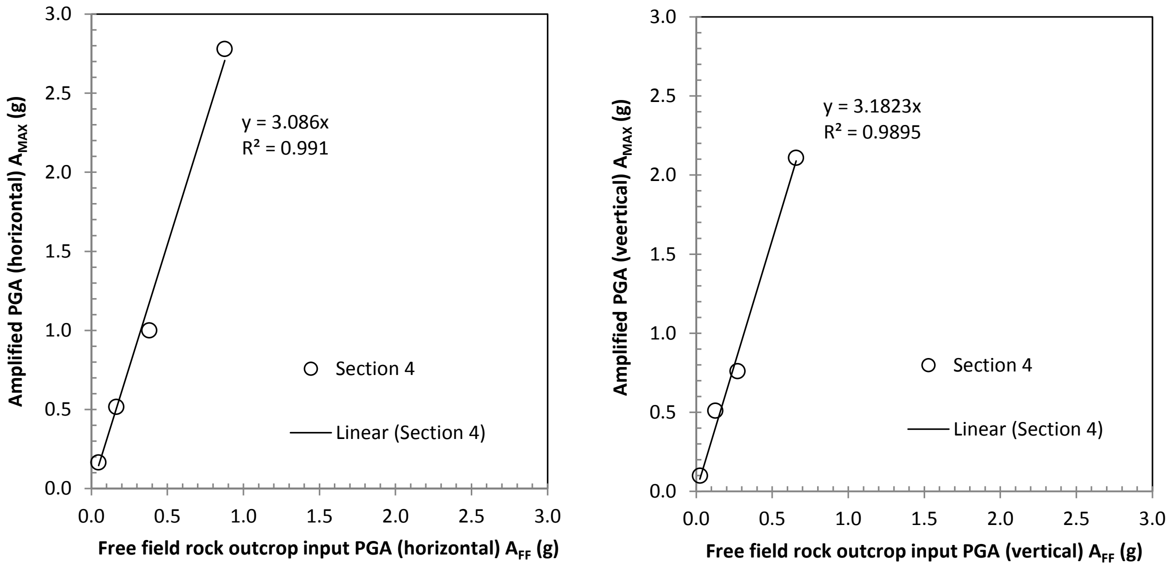
The results from the two-dimensional site response modelling are shown for cross-section 4. The maximum acceleration (A_{MAX}) at the cliff crest derived from the modelling of each synthetic earthquake time history has been plotted in Figure A4.1. Each point on the graph represents the response of the slope crest to a given synthetic free field rock outcrop earthquake input motion (Table A4.1).

The fundamental frequency of the slope varies from 3.8 to 10 Hz based on the equation in Bray and Travasarou (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 400–600 m/s. The dominant frequency of the input motions is between 3.6 Hz and 5.7 Hz. The "tuning ratio" defined as the ratio between the dominant frequency of the input motion and the fundamental frequency of the slope (Wartman et al., 2013), is about 1.1–1.7 for a shear wave velocity of 400 m/s, and 0.4–0.6 for a shear wave velocity of 600 m/s.

Results from the seismic response assessment suggest that the peak ground acceleration amplification factors (S_T) for cross-section 4 is bout 3.1 (±0.1) 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-section 4, using the synthetic free-field rock outcrop motions for the Maffeys Road site by Holden et al. (2014) as inputs to the assessment. PGA is peak ground acceleration. A_{FF} is the maximum acceleration of the input motion and A_{MAX} is the maximum acceleration of the ground response to the input motion measured at the slope crest.

Earthquake (2011)	Free-field input PGA (horizontal) – A_{FF} (g)	Free-field input PGA (vertical) – A_{FF} (g)	Maximum PGA (horizontal) at slope crest – A_{MAX} (g)	Maximum PGA (vertical) at slope crest – A_{MAX} (g)
22 February	0.88	0.66	2.83	2.18
16 April	0.05	0.02	0.17	0.10
13 June	0.38	0.27	1.00	0.76
23 December	0.16	0.13	0.52	0.51



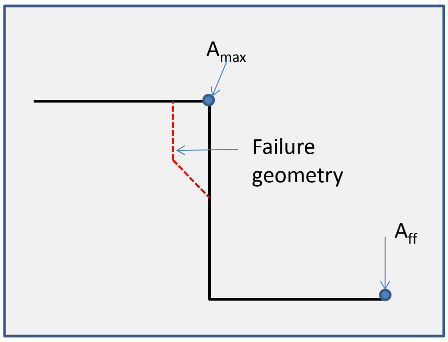


Figure A4.1 Amplification relationship between the synthetic free-field rock outcrop input motions (A_{FF}) and the modelled cliff crest maximum accelerations (A_{MAX}) for cross-section 4. A schematic diagram showing the locations of the various recorded accelerations is shown.

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 approximately linear. In the range of modelled peak horizontal accelerations, the horizontal amplification factor (S_T) is typically in the order of about 3.1 times the input free-field peak horizontal acceleration.

Cross-section 4 comprises about 9 m of loess and colluvium overlying mixed basalt lava and breccia, where the mean shear wave velocities of the materials change from 400–700 m/s in the basalt breccia and lava, to 200–400 m/s in the loess (Figure A4.2). The results suggest that the impedance contrasts between the materials contribute most to the amplification of shaking, but that the peak horizontal accelerations (for all modelled earthquakes) concentrate around the convex break in slope, corresponding to where the largest permanent slope displacements were recorded from cadastral survey marks and inferred from crack apertures. 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, which could explain the significant shaking damage to those dwellings at the slope crest (4, 6 and 8 Maffeys Road) within and above the area of mapped cracks.

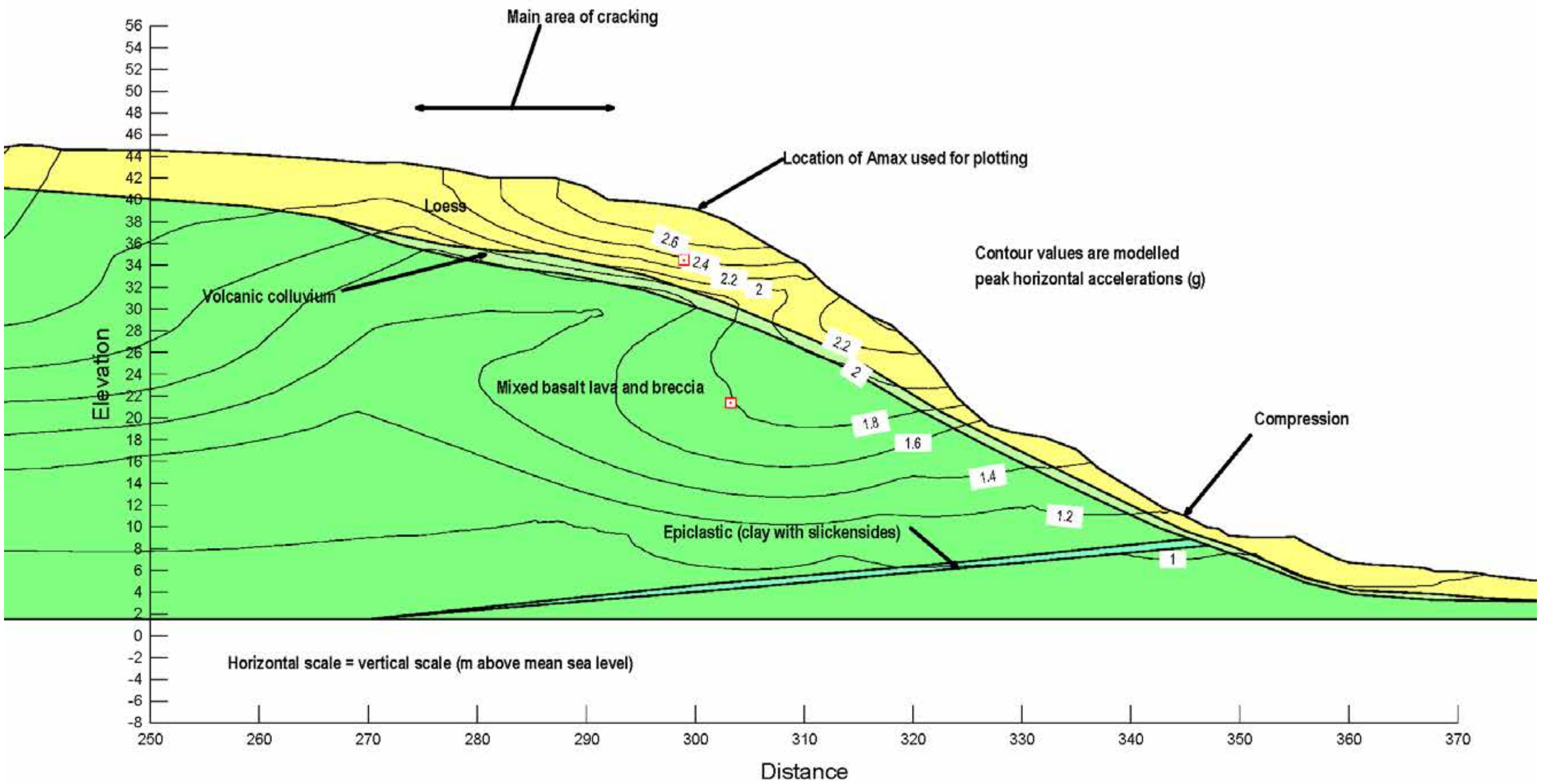


Figure A4.2 Modelled peak horizontal ground acceleration contours for the 22 February 2011 earthquake at Maffeys Road, cross-section 4.

In experimental data, as the slope displaces during an earthquake, the slide surface can "base isolate" the mass above, resulting in lower levels of shaking and displacement. Therefore, the reported amplification factors are near the upper bound of published topographic amplification factors. However, given the impedance contrasts between the loess/fill and rock are so high, this contrast could lead to the trapping of seismic waves within the loess and colluvium. Assessment of this is outside the scope of this report.

The relationship between the modelled vertical and horizontal peak ground accelerations recorded at the slope crest (A_{MAX}) is shown in Figure A4.3. The gradient of the linear fit is 0.77 (±0.03) – errors at one standard deviation. The relationship between horizontal and vertical peak ground accelerations appears linear.

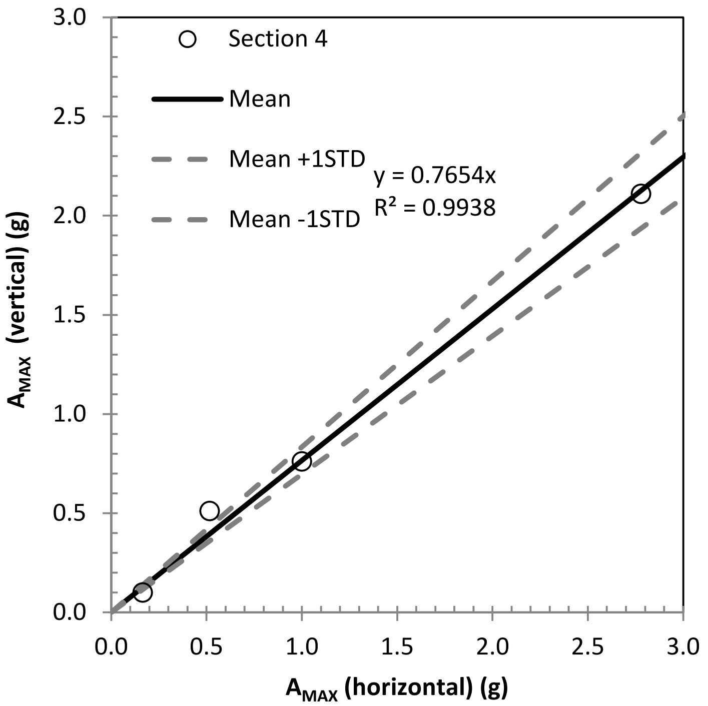


Figure A4.3 Relationship between the modelled horizontal and vertical maximum accelerations modelled at the slope crest (A_{MAX}) for cross-section 4, using the synthetic free-field rock outcrop motions for the Maffeys Road site by Holden et al. (2014) as inputs to the assessment. The mean and standard deviation trend lines are fitted for A_{MAX} all data. Errors are shown as the mean ± one standard deviation (1 STD).

Eurocode 8, Part 5, Annex A, gives some simplified amplification factors for the seismic action used in the verification of the stability of slopes. Such factors, denoted S_T, are to a first approximation considered independent of the fundamental period of vibration and, hence, multiply as a constant scaling factor.

Eurocode 8, Part 5, Annex A recommends:

1. Isolated cliffs and slopes. A value $S_T \geq 1.2$ should be used for sites near the top edge;
2. Ridges with crest width significantly less than the base width. A value $S_T \geq 1.4$ should be used near the top of the slopes for average slope angles greater than 30° and a value S_T >1.2 should be used for smaller slope angles;
3. Presence of a loose surface layer. In the presence of a loose surface layer, the smallest S_T value given in a) and b) should be increased by at least 20%;

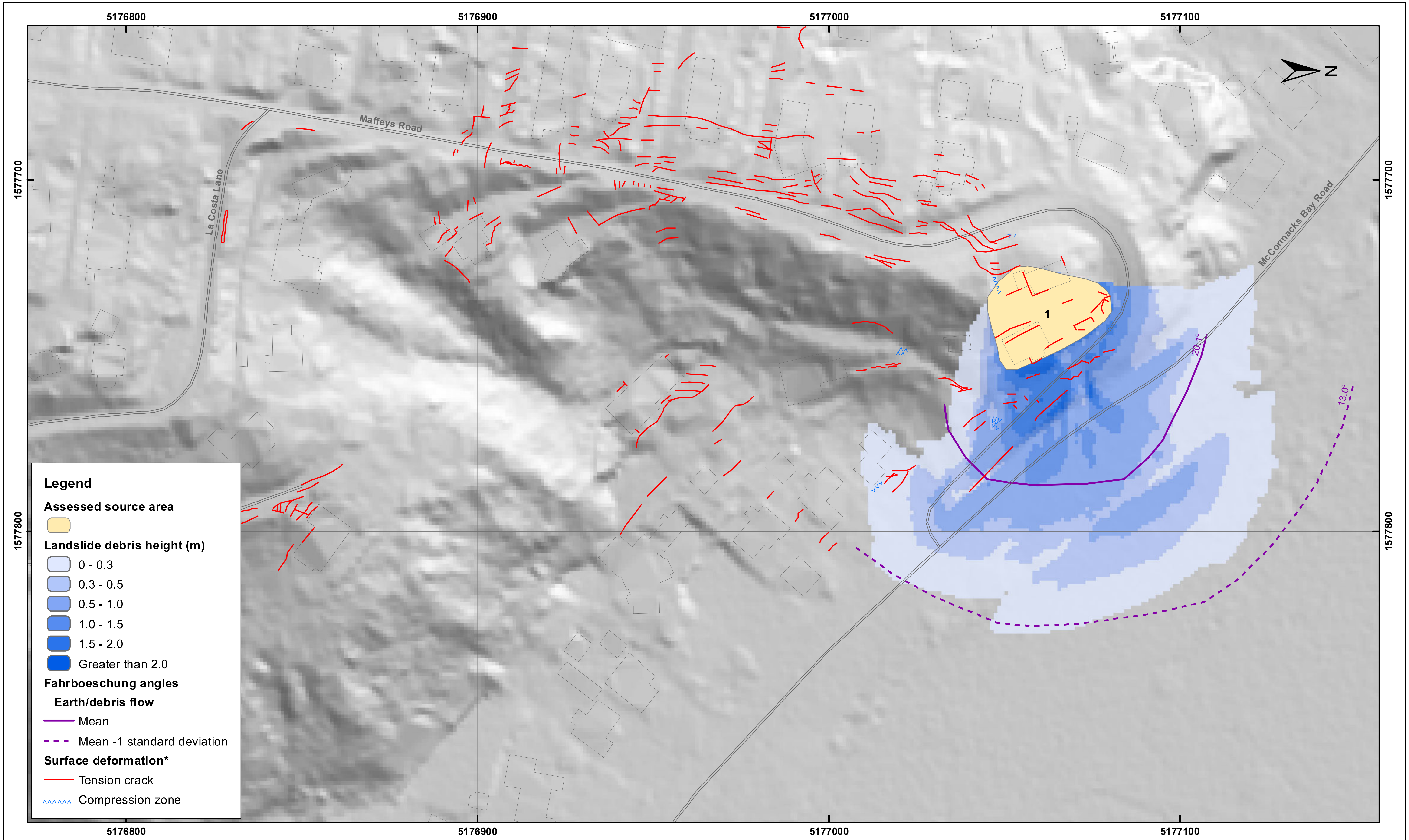
4. Spatial variation of amplification factor. The value of S_T may be assumed to decrease as a linear function of the height above the base of the cliff or ridge, and to be unity at the base; and

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

Ashford and Sitar (2002) recommend an S_T of 1.5 be applied to the maximum free-field acceleration behind the crest based on their assessment of slopes 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 Maffeys Road are about 3 (cross-section 4) 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 Maffeys Road, the site is within 5 km of the epicentres of the 22 February, 16 April, 13 June and 23 December 2011 earthquakes, making them all “near-field” earthquakes.

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



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 1 - Upper Volume (1,500 m^3)

Maffeys Road - Port Hills
Christchurch

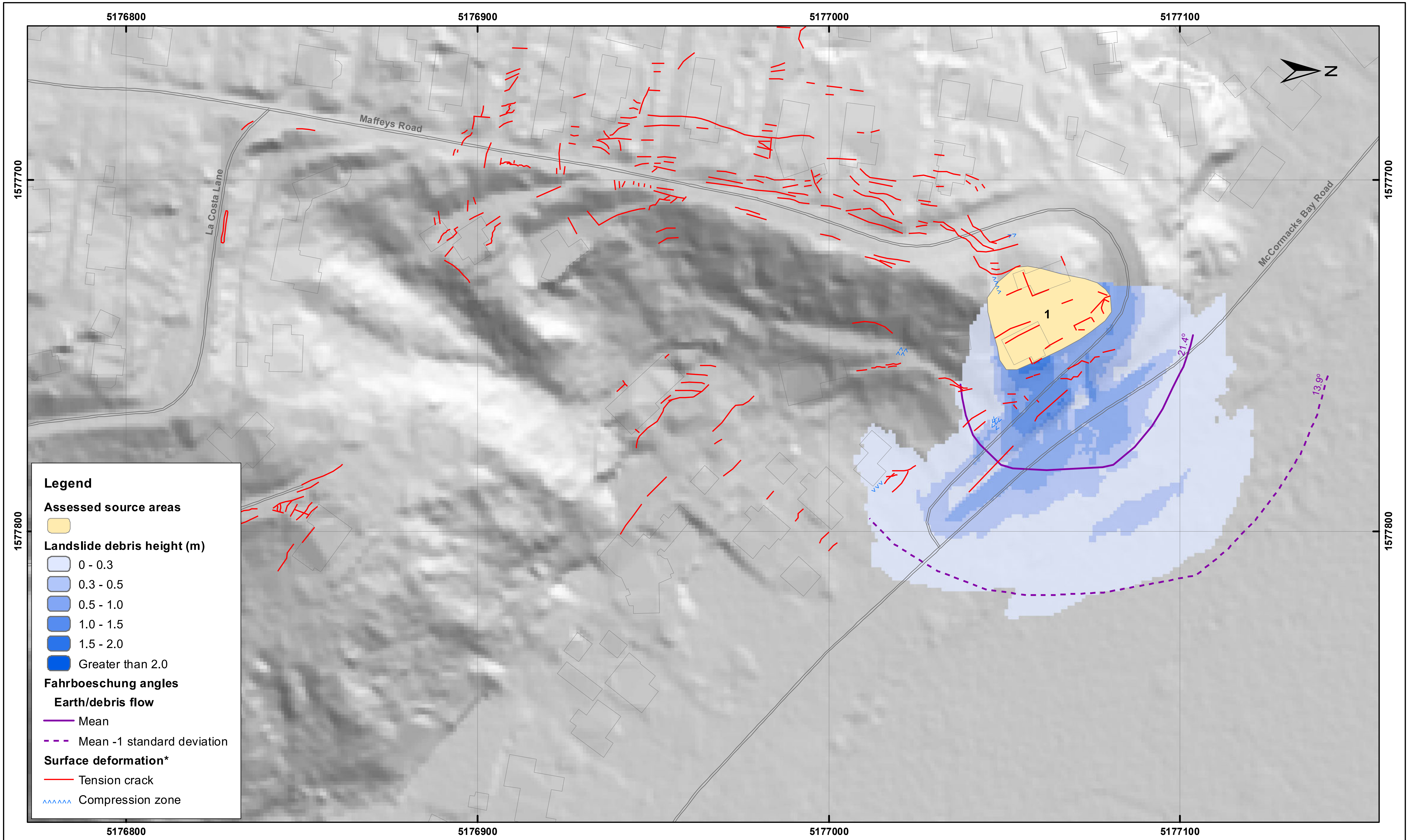
APPENDIX 5

Figure 1

FINAL

REPORT: CR2014/79

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 1 - Middle Volume (770 m^3)

Maffeys Road - Port Hills Christchurch

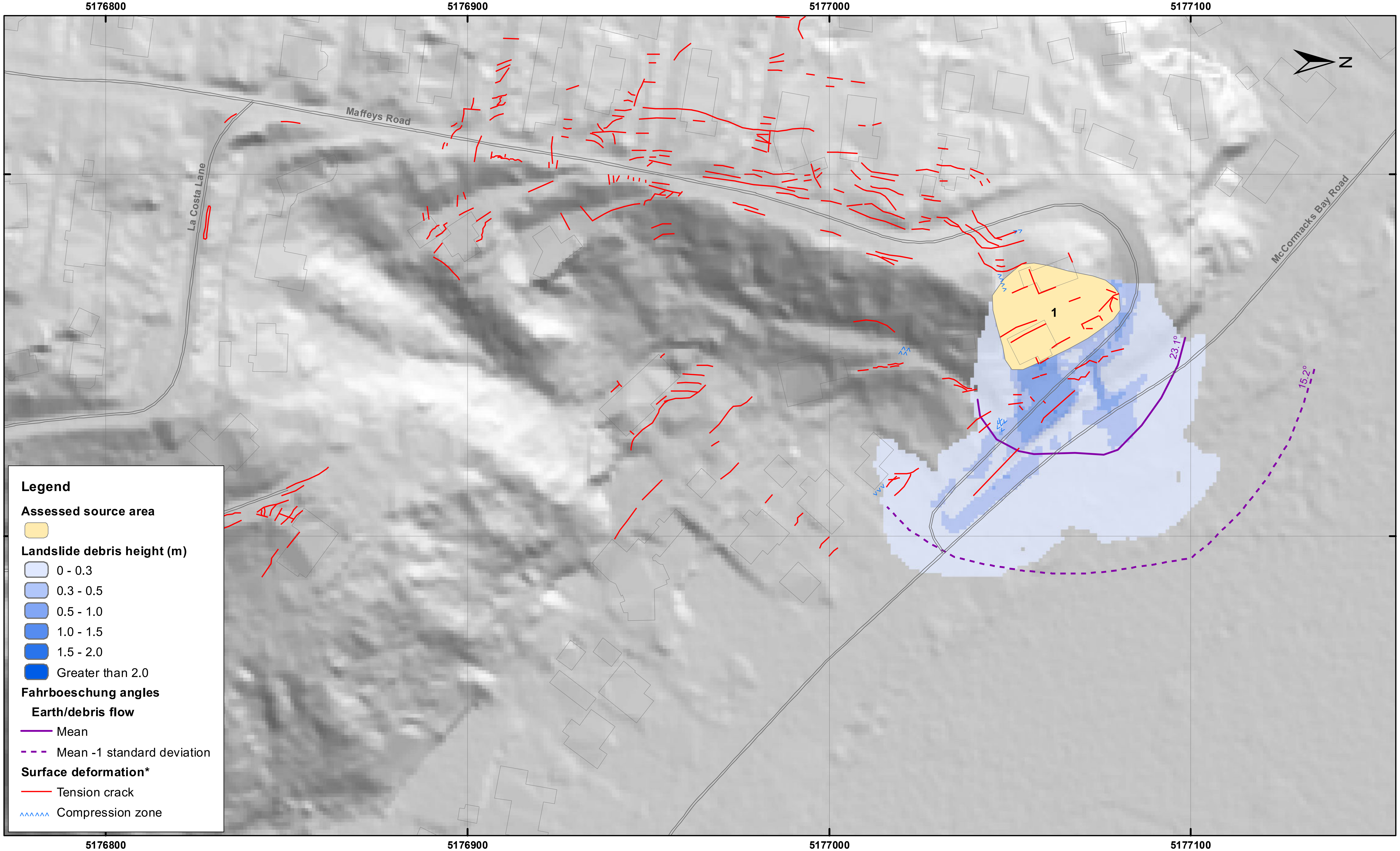
APPENDIX 5

Figure 2

FINAL

REPORT: CR2014/79

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 1 - Lower Volume (340 m^3)

Maffeys Road - Port Hills
Christchurch

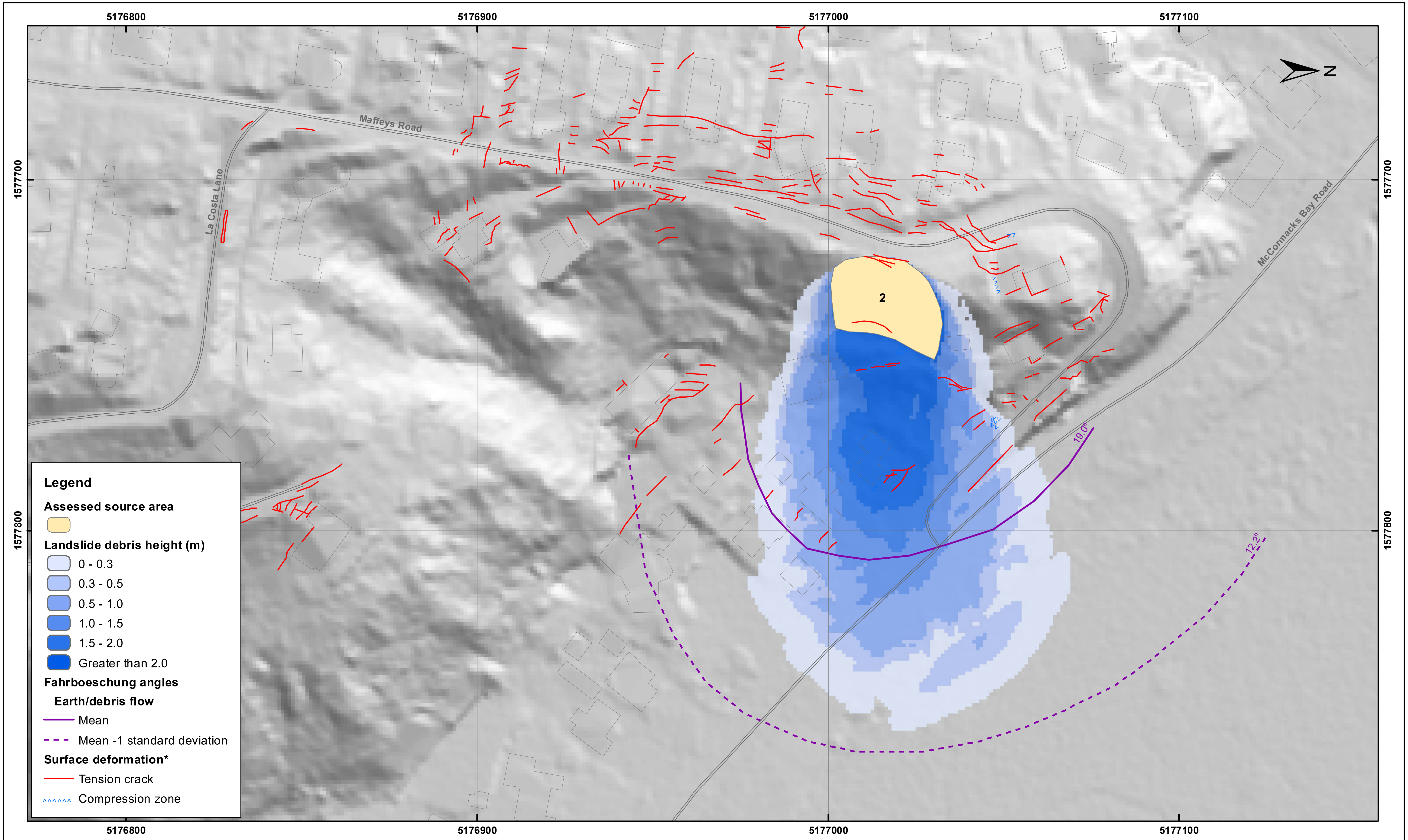
APPENDIX 5

Figure 3

FINAL

REPORT: CR2014/79

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 2 - Upper Volume (2,700 m^3)

Maffeys Road - Port Hills Christchurch

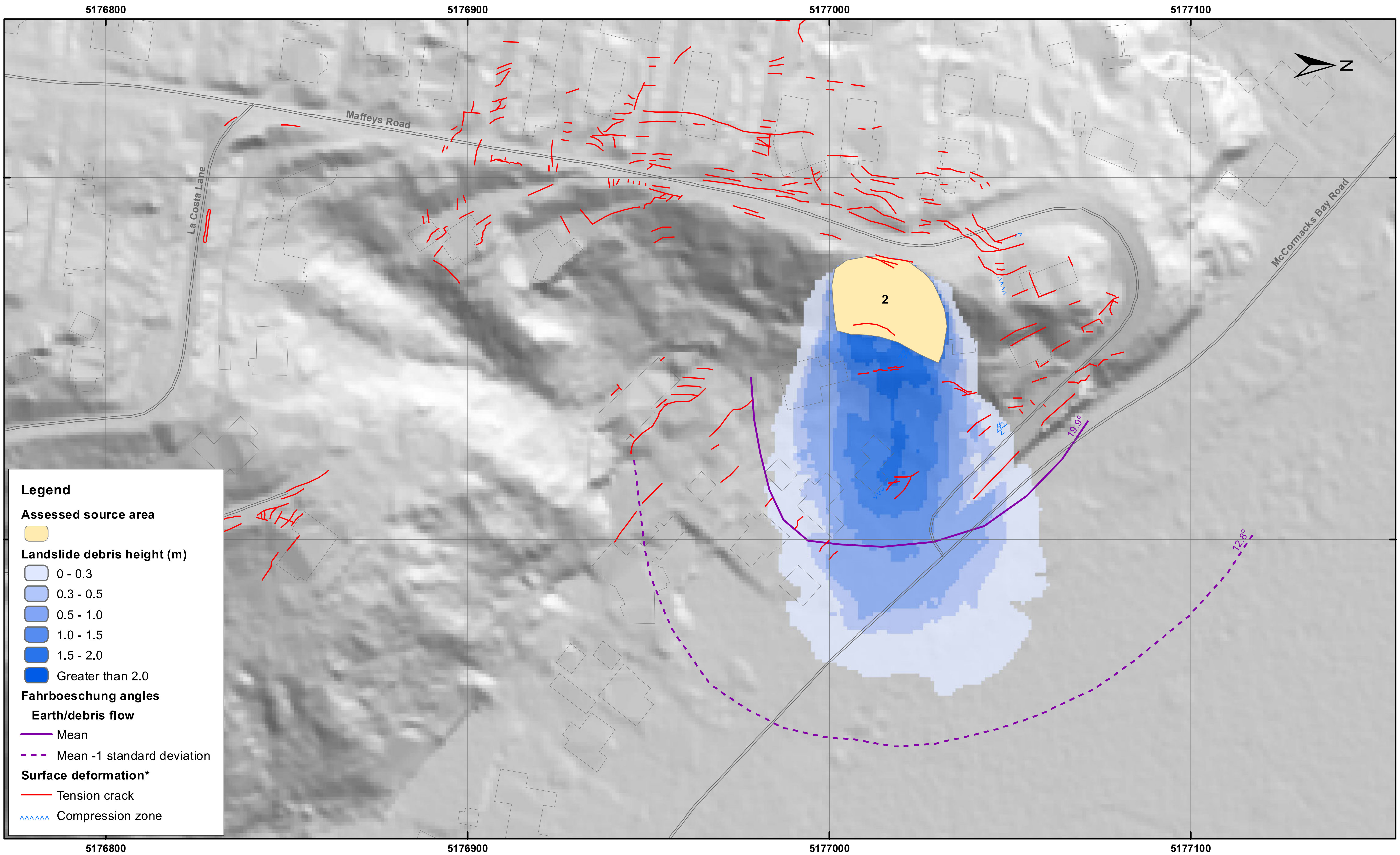
APPENDIX 5

Figure 4

FINAL

REPORT: CR2014/79

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 2 - Middle Volume (1,600 m^3)

Maffeys Road - Port Hills Christchurch

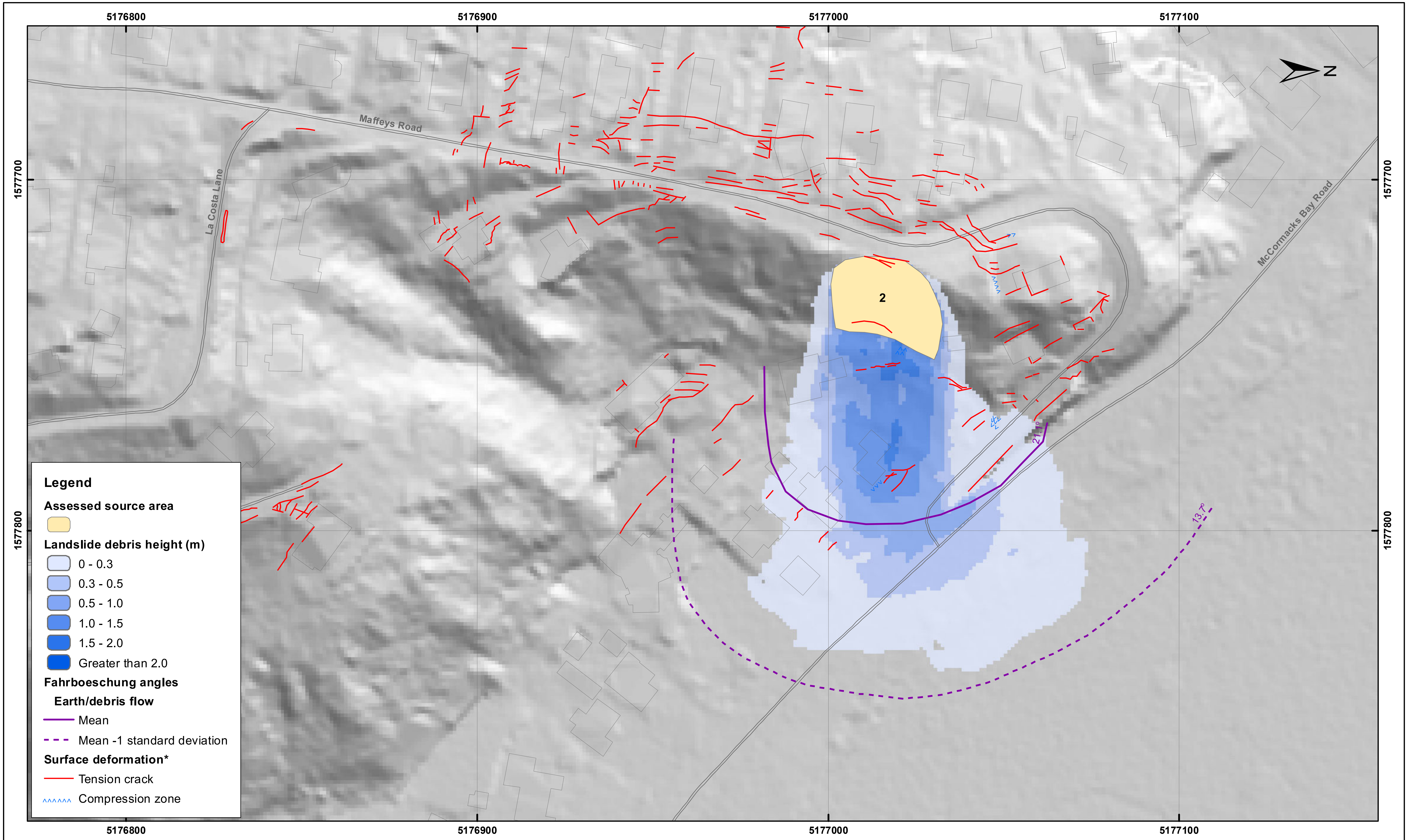
APPENDIX 5

Figure 5

FINAL

REPORT: CR2014/79

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 2 - Lower Volume (910 m^3)

Maffeys Road - Port Hills
Christchurch

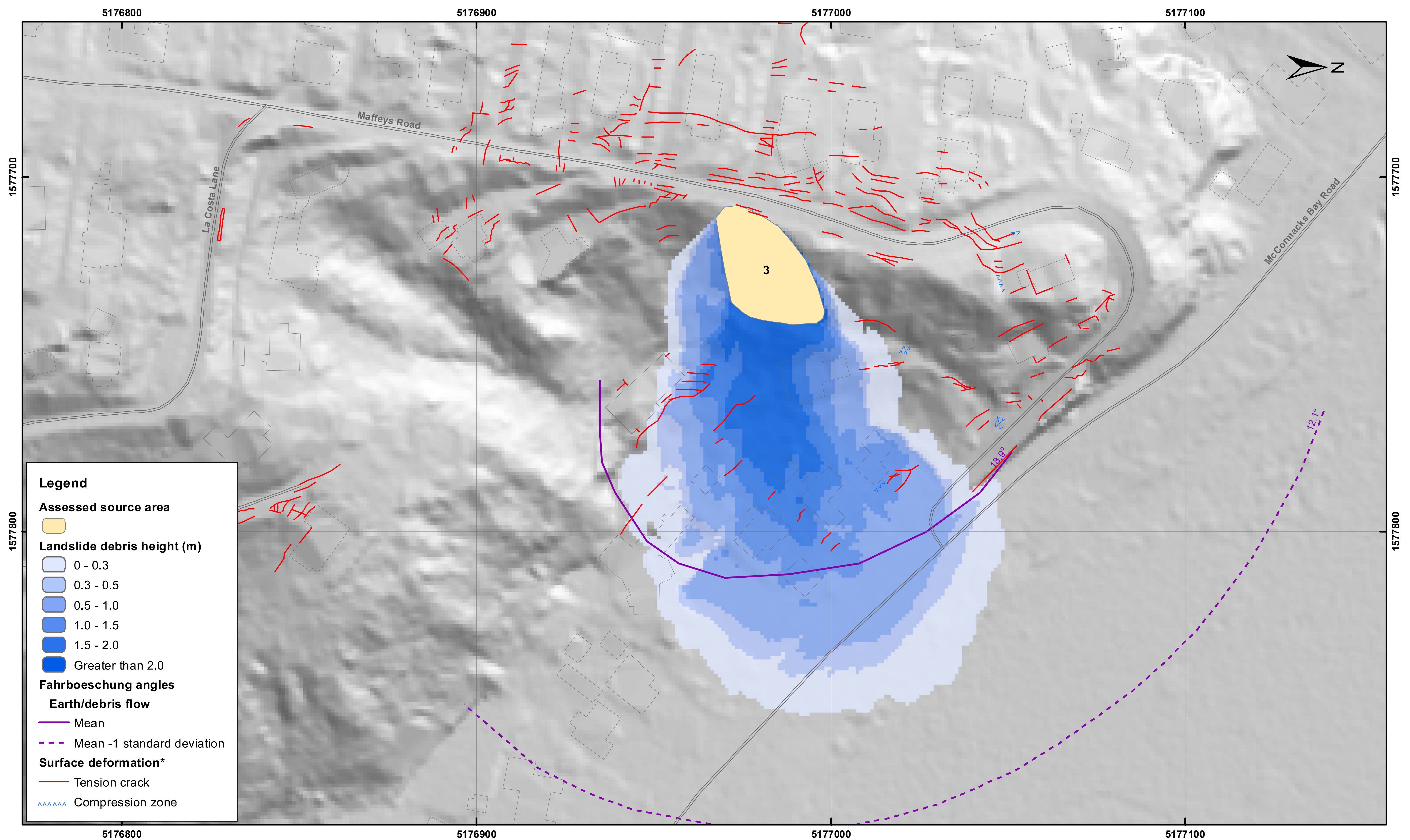
APPENDIX 5

Figure 6

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
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PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 3 - Upper Volume (2,800 m^3)

Maffeys Road - Port Hills
Christchurch

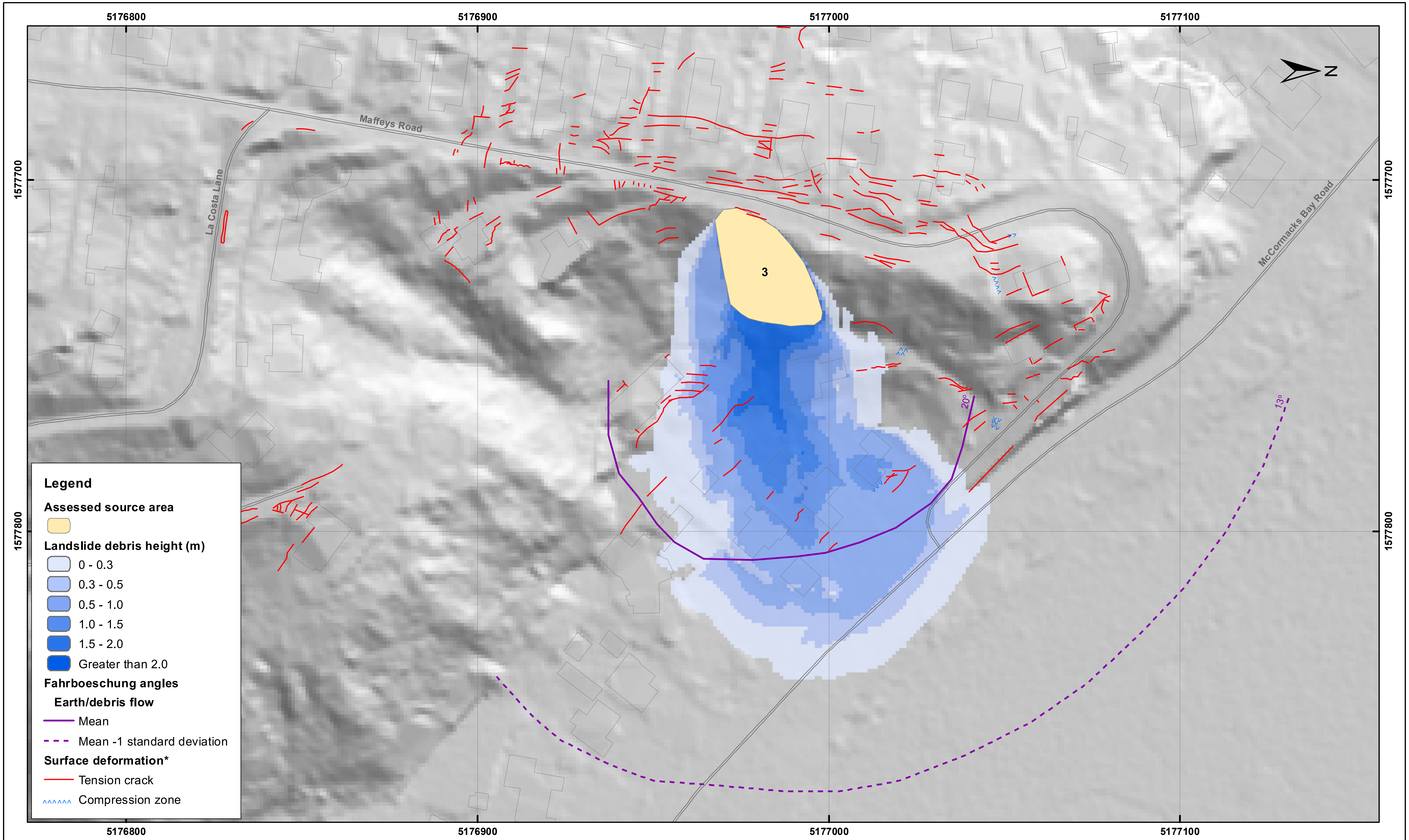
APPENDIX 5

Figure 7

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

Background shade model derived from NZAM post earthquake 2011c (July 2011) LiDAR survey resampled to a 1 m ground resolution.
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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 3 - Middle Volume (1,800 m^3)

Maffeys Road - Port Hills
Christchurch

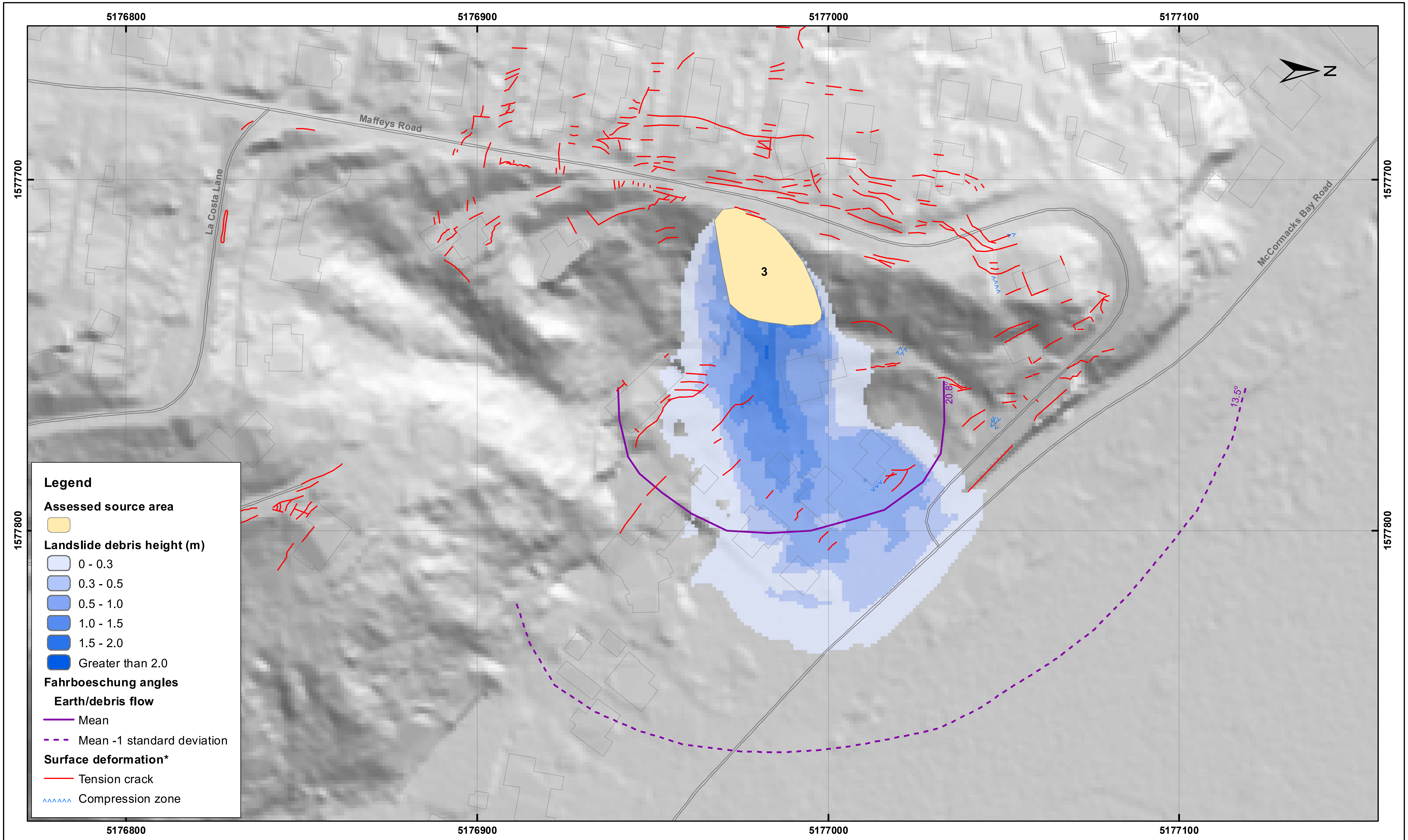
APPENDIX 5

Figure 8

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

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PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 3 - Lower Volume (1,000 m^3)

Maffeys Road - Port Hills
Christchurch

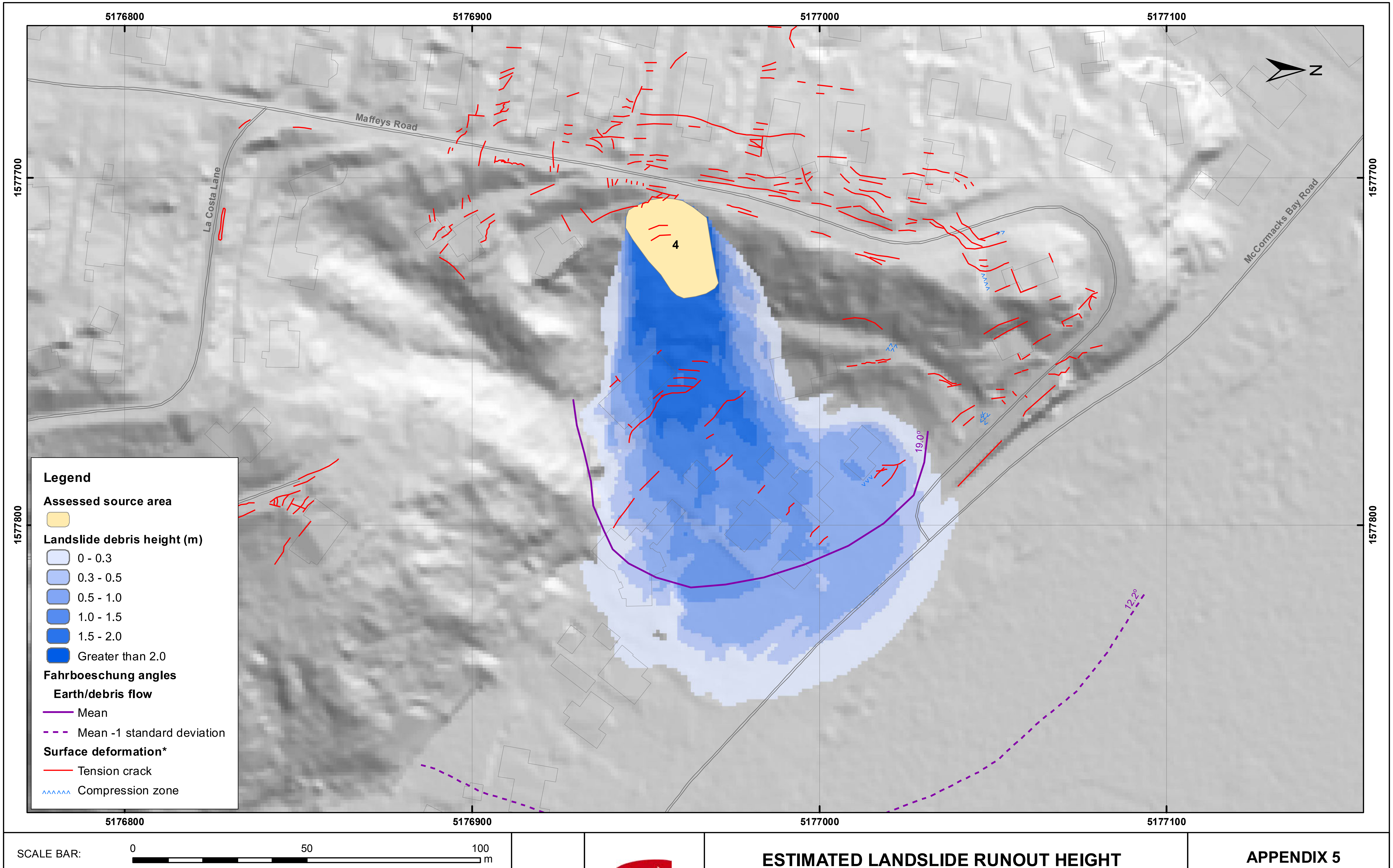
APPENDIX 5

Figure 9

FINAL

REPORT: CR2014/79

DATE: June 2014



EXPLANATION:

* Taken from report CR2012/317

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

PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 4 - Upper Volume (2,700 m^3)

Maffeys Road - Port Hills
Christchurch

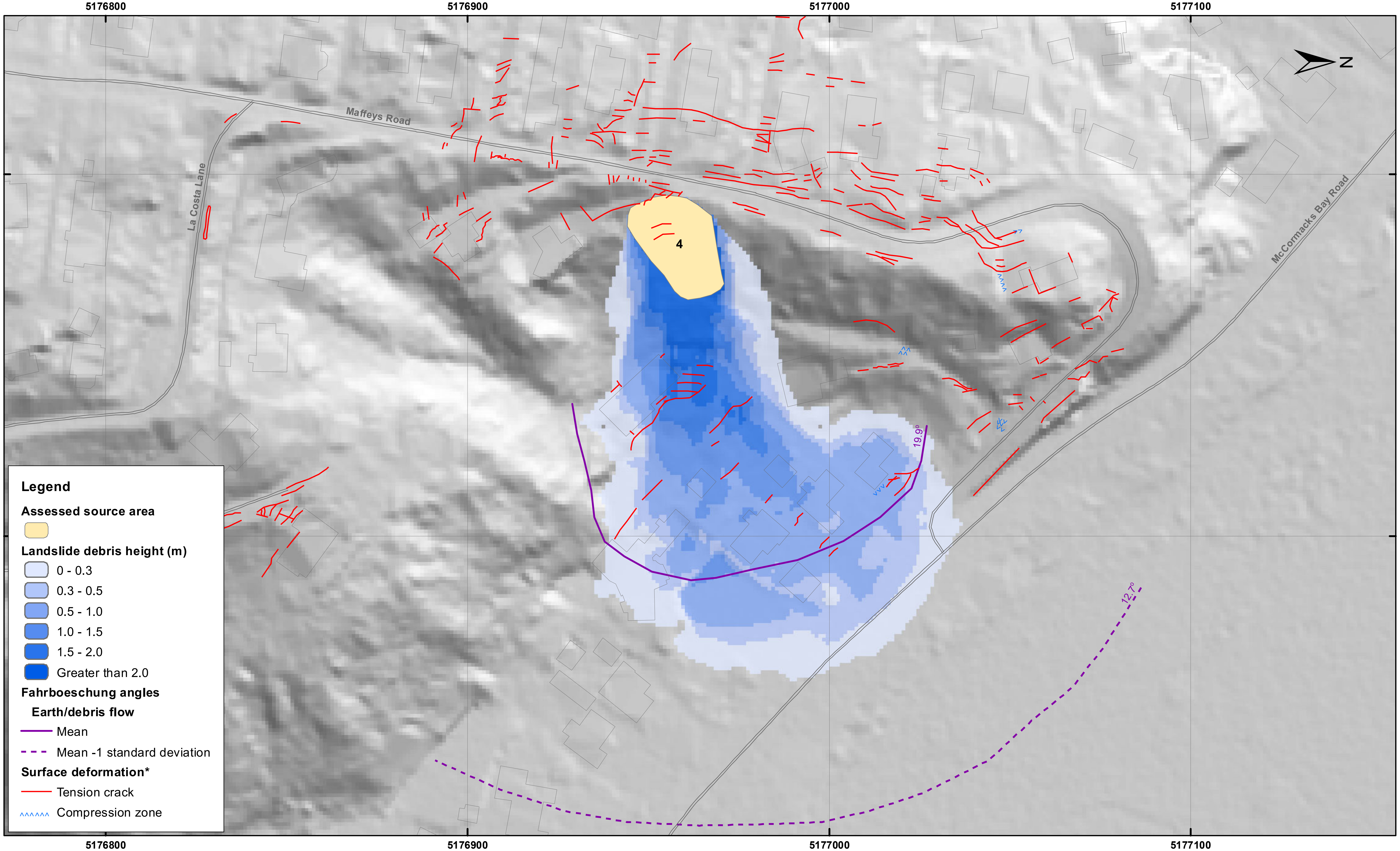
APPENDIX 5

Figure 10

FINAL

REPORT: CR2014/79

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

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 4 - Middle Volume (1,700 m^3)

Maffeys Road - Port Hills Christchurch

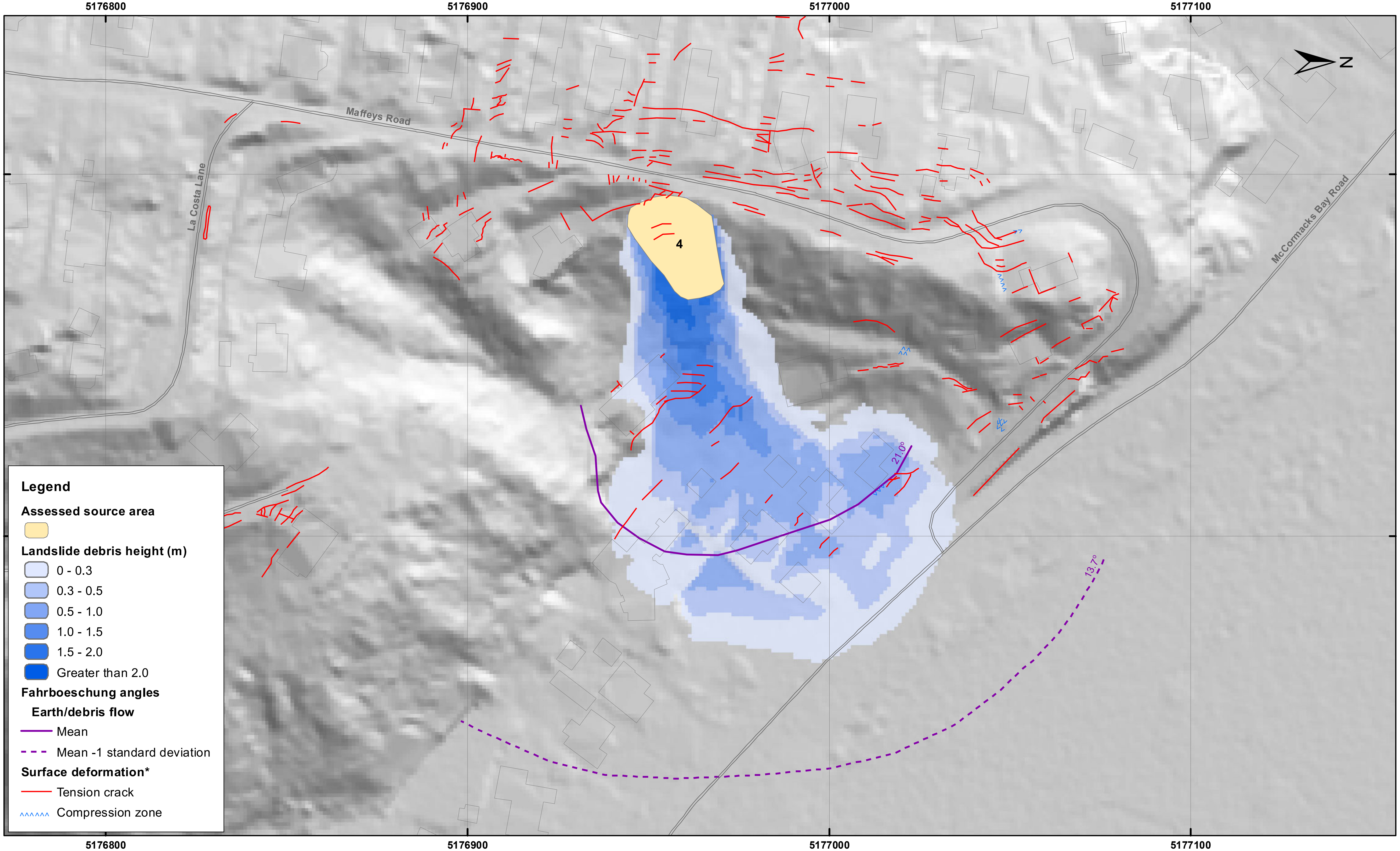
APPENDIX 5

Figure 11

FINAL

REPORT: CR2014/79

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 4 - Lower Volume (920 m^3)

Maffeys Road - Port Hills
Christchurch

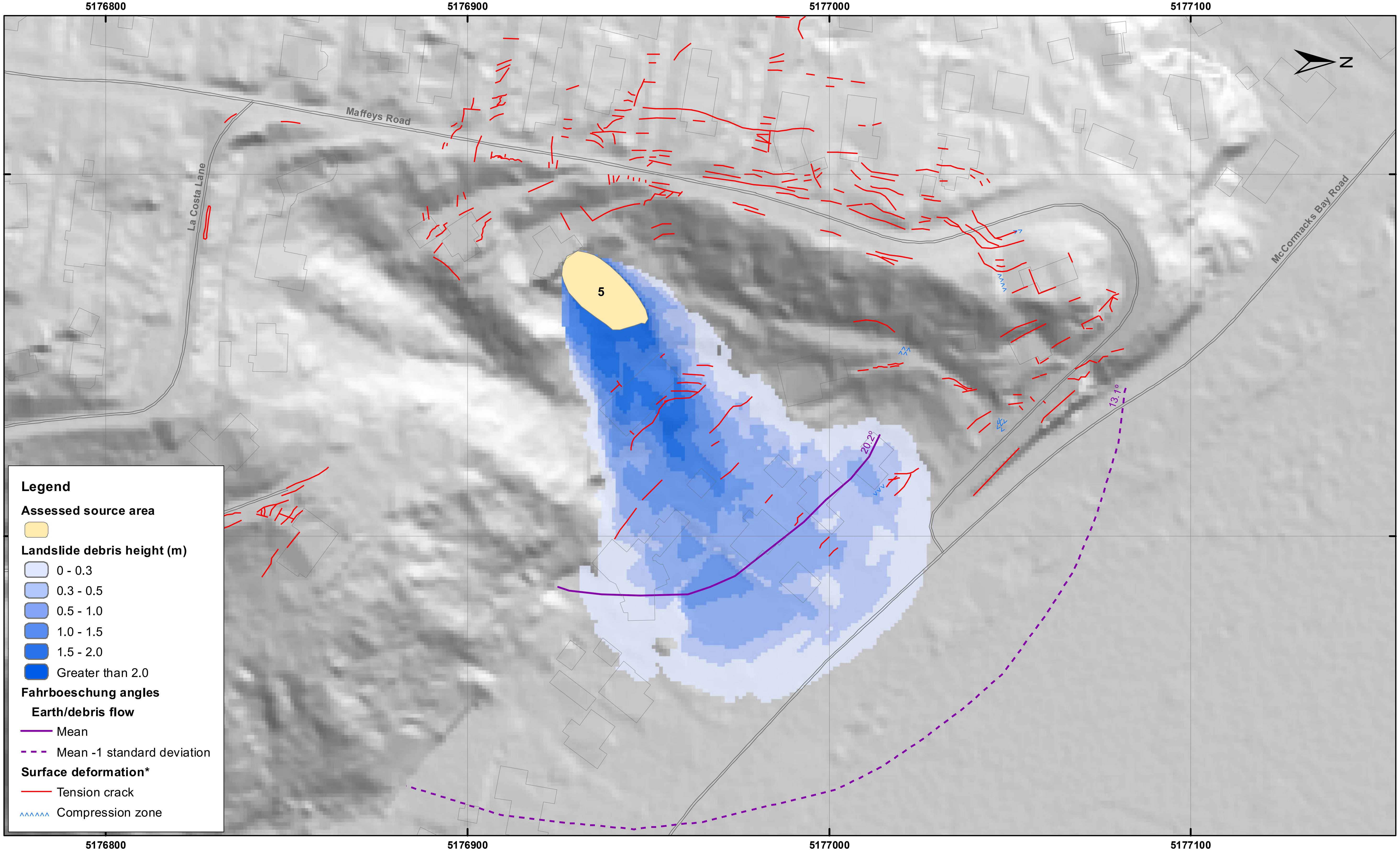
APPENDIX 5

Figure 12

FINAL

REPORT: CR2014/79

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

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 5 - Upper Volume (1,400 m^3)

Maffeys Road - Port Hills
Christchurch

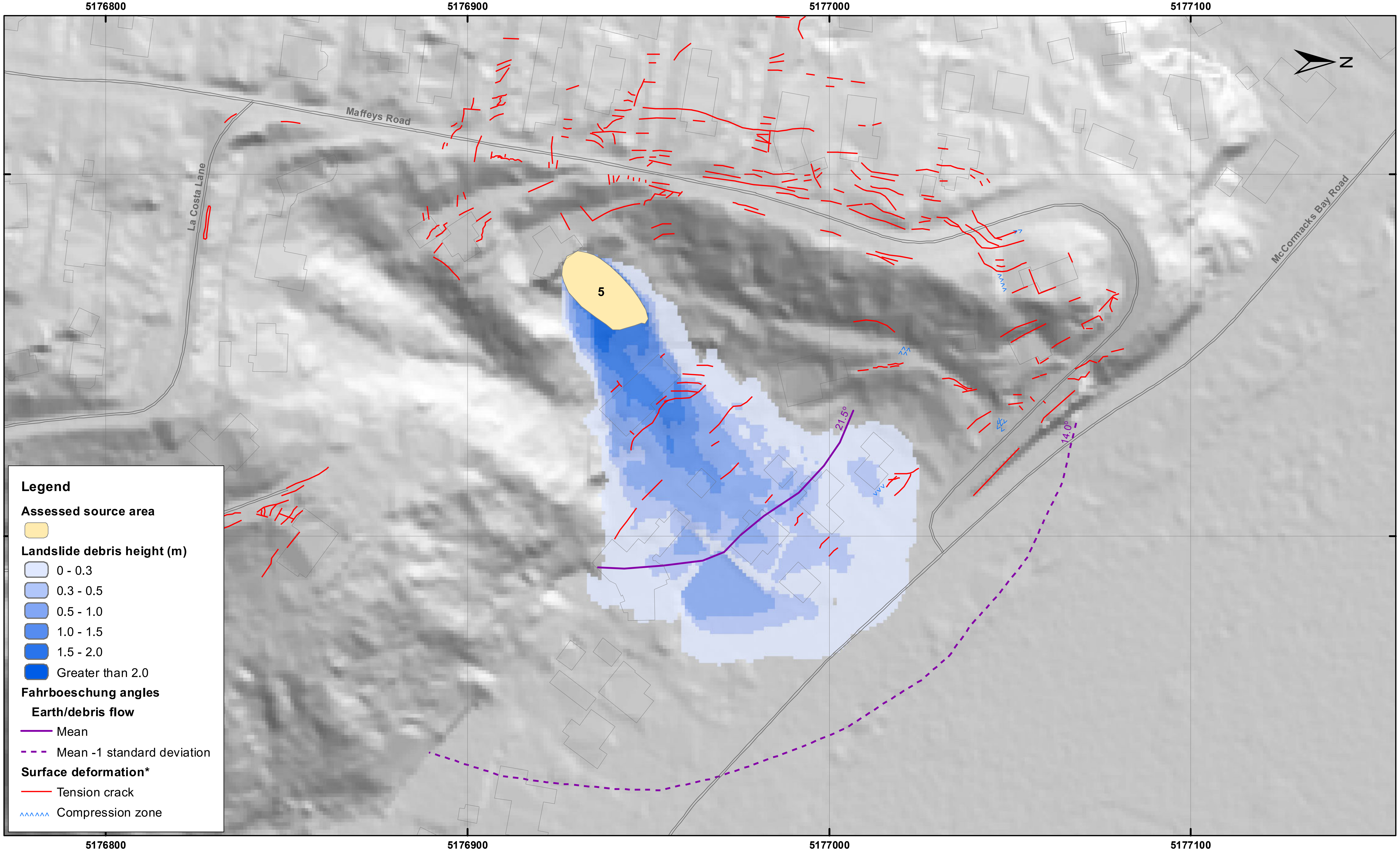
APPENDIX 5

Figure 13

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

* Taken from report CR2012/317

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CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT HEIGHT
Source 5 - Middle Volume (740 m^3)

Maffeys Road - Port Hills
Christchurch

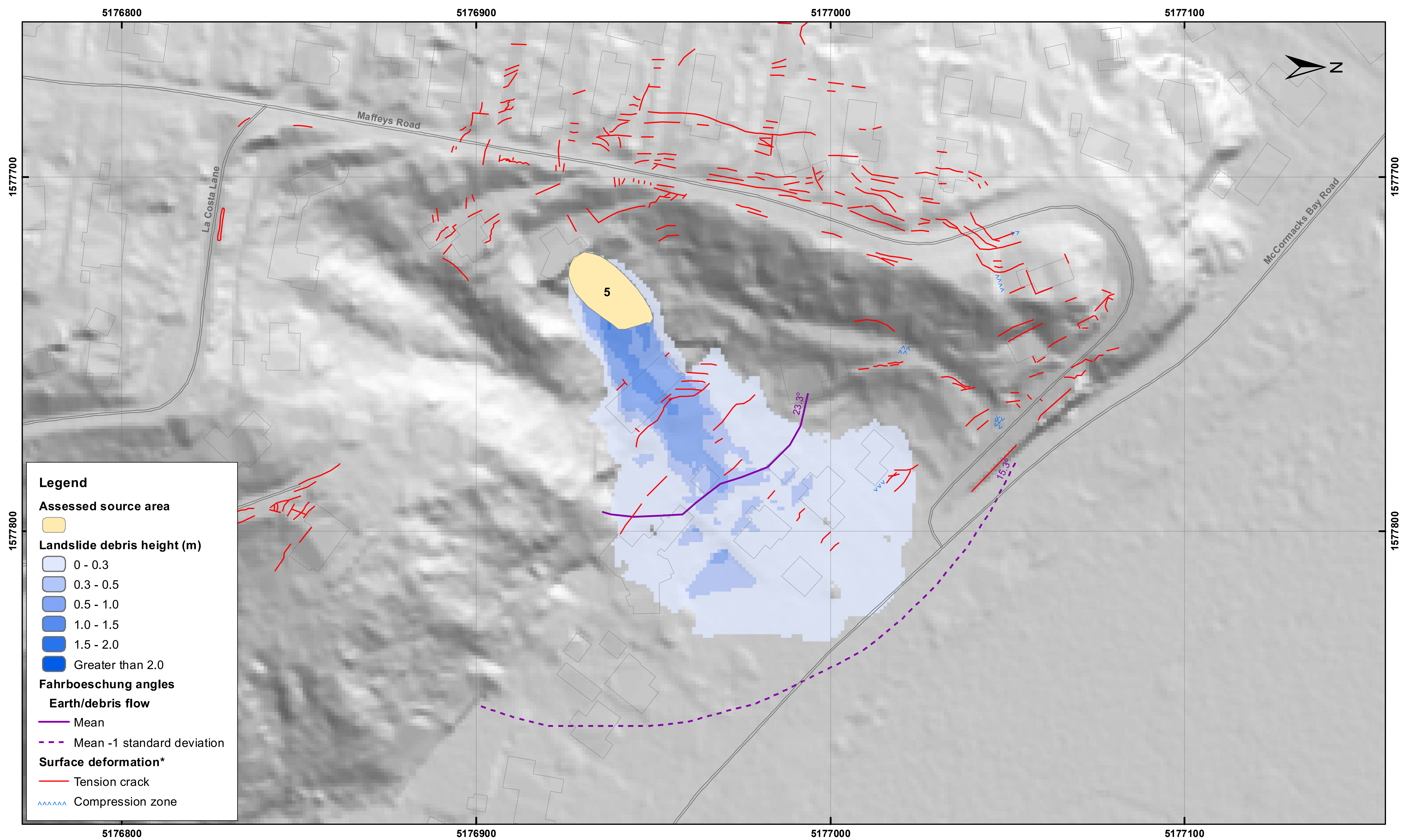
APPENDIX 5

Figure 14

FINAL

REPORT: CR2014/79

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 - Lower Volume (320 m^3)

Maffeys Road - Port Hills
Christchurch

APPENDIX 5

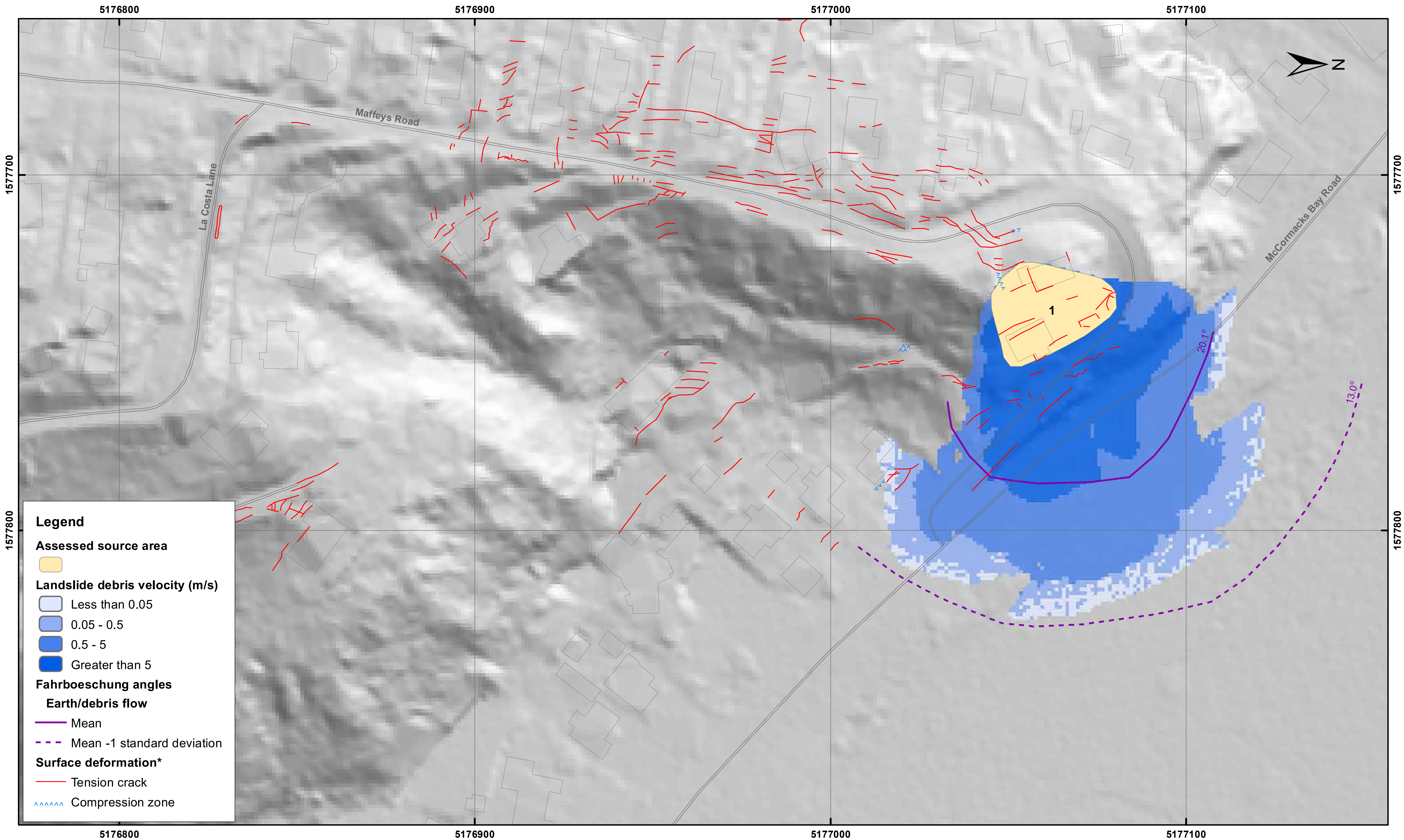
Figure 15

FINAL

REPORT: CR2014/79

DATE: June 2014

A6 APPENDIX 6: RAMMS MODELLING RESULTS FOR SOURCE AREAS 1 AND 2; ESTIMATED LANDSLIDE RUNOUT VELOCITY



SCALE BAR: 0 50 100 m

EXPLANATION:

* 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 VELOCITY
Source 1 - Upper Volume (1,500 m^3)

Maffeys Road - Port Hills
Christchurch

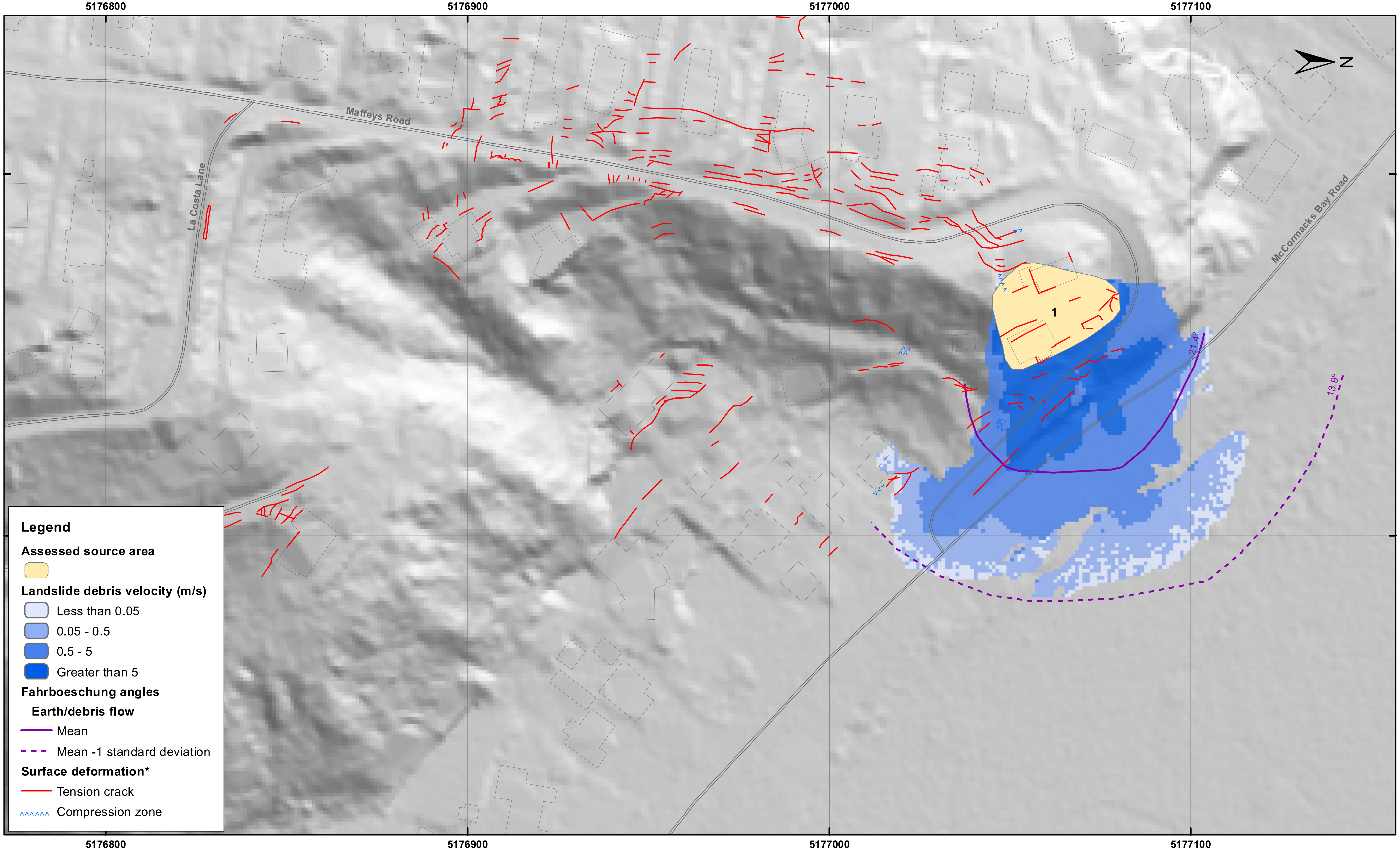
APPENDIX 6

Figure 1

FINAL

REPORT: CR2014/79

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

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 1 - Middle Volume (770 m^3)

Maffeys Road - Port Hills
Christchurch

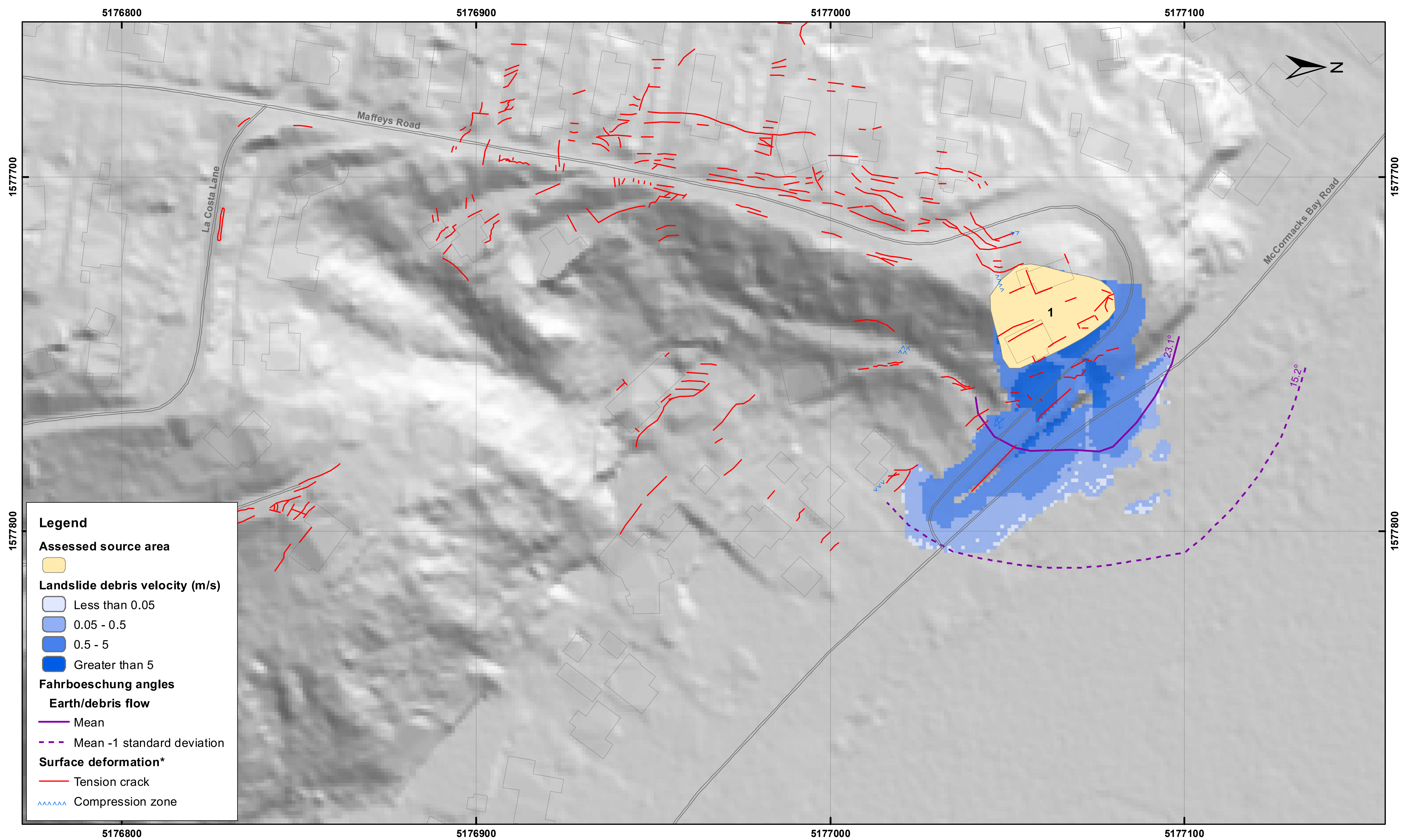
APPENDIX 6

Figure 2

FINAL

REPORT: CR2014/79

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 VELOCITY
Source 1 - Lower Volume (340 m^3)

Maffeys Road - Port Hills
Christchurch

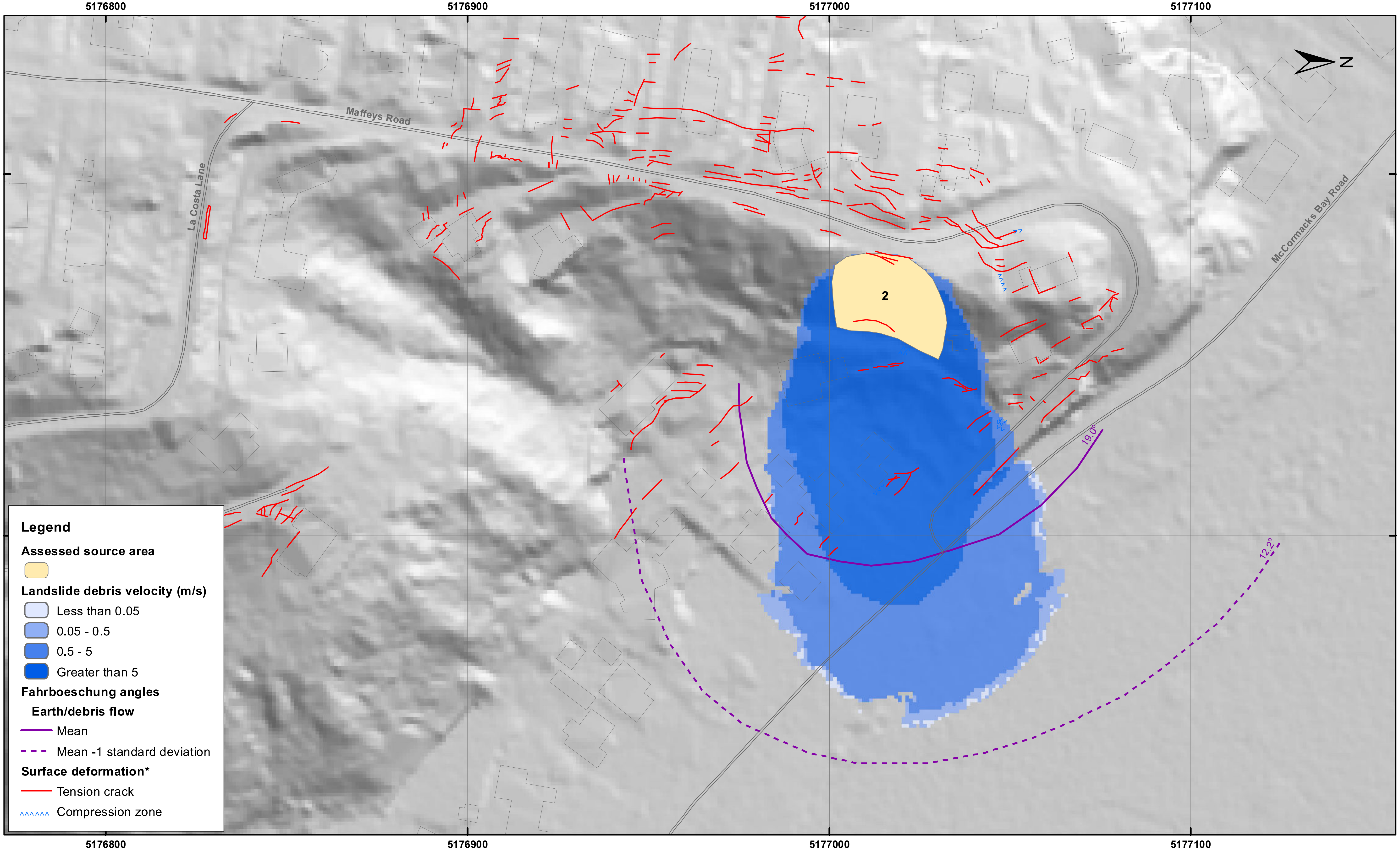
APPENDIX 6

Figure 3

FINAL

REPORT: CR2014/79

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 2 - Upper Volume (2,700 m^3)

Maffeys Road - Port Hills
Christchurch

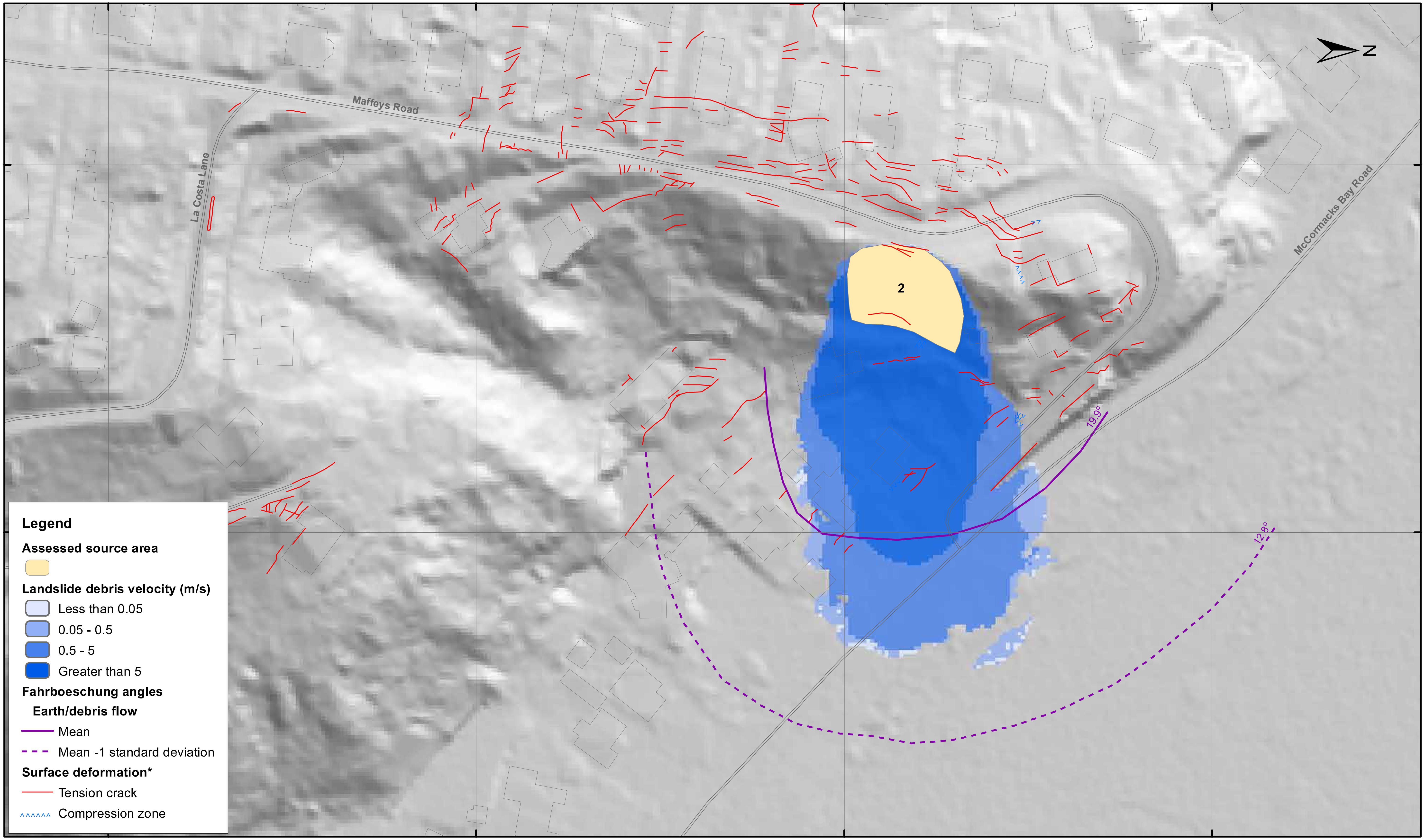
APPENDIX 6

Figure 4

FINAL

REPORT: CR2014/79

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 2 - Middle Volume (1,600 m^3)

Maffeys Road - Port Hills
Christchurch

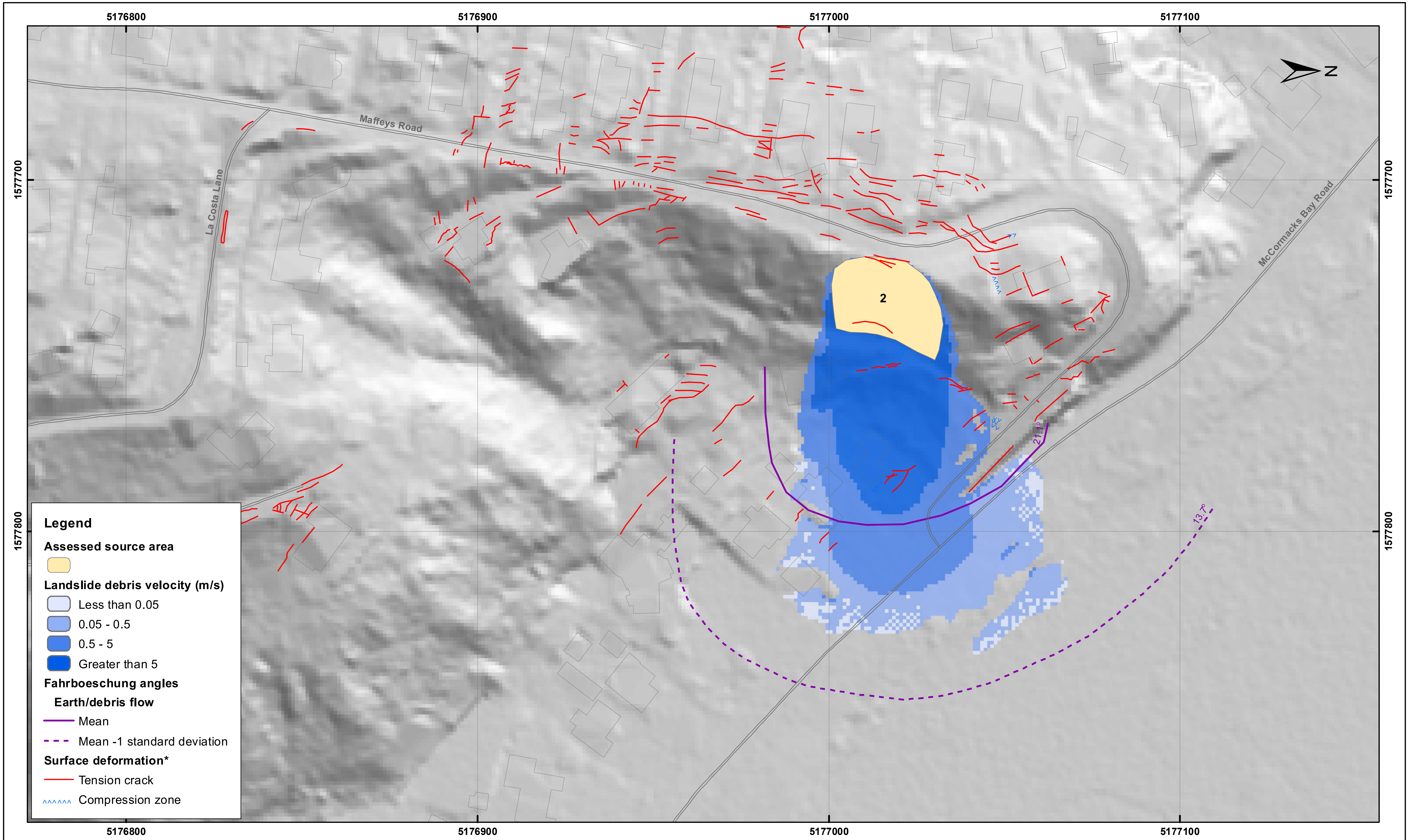
APPENDIX 6

Figure 5

FINAL

REPORT: CR2014/79

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 VELOCITY
Source 2 - Lower Volume (910 m^3)

Maffeys Road - Port Hills
Christchurch

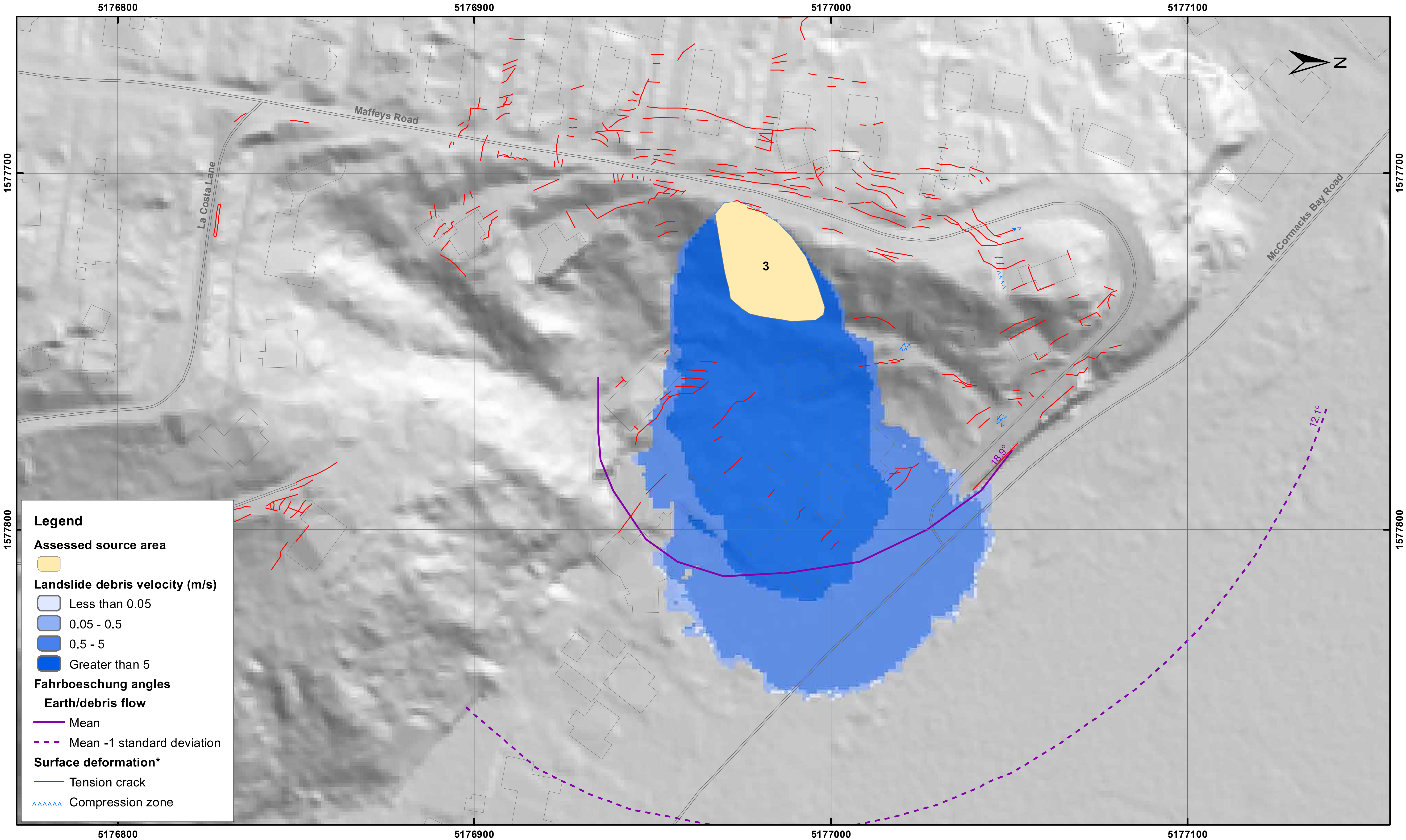
APPENDIX 6

Figure 6

FINAL

REPORT: CR2014/79

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 - Upper Volume (2,800 m^3)

Maffeys Road - Port Hills
Christchurch

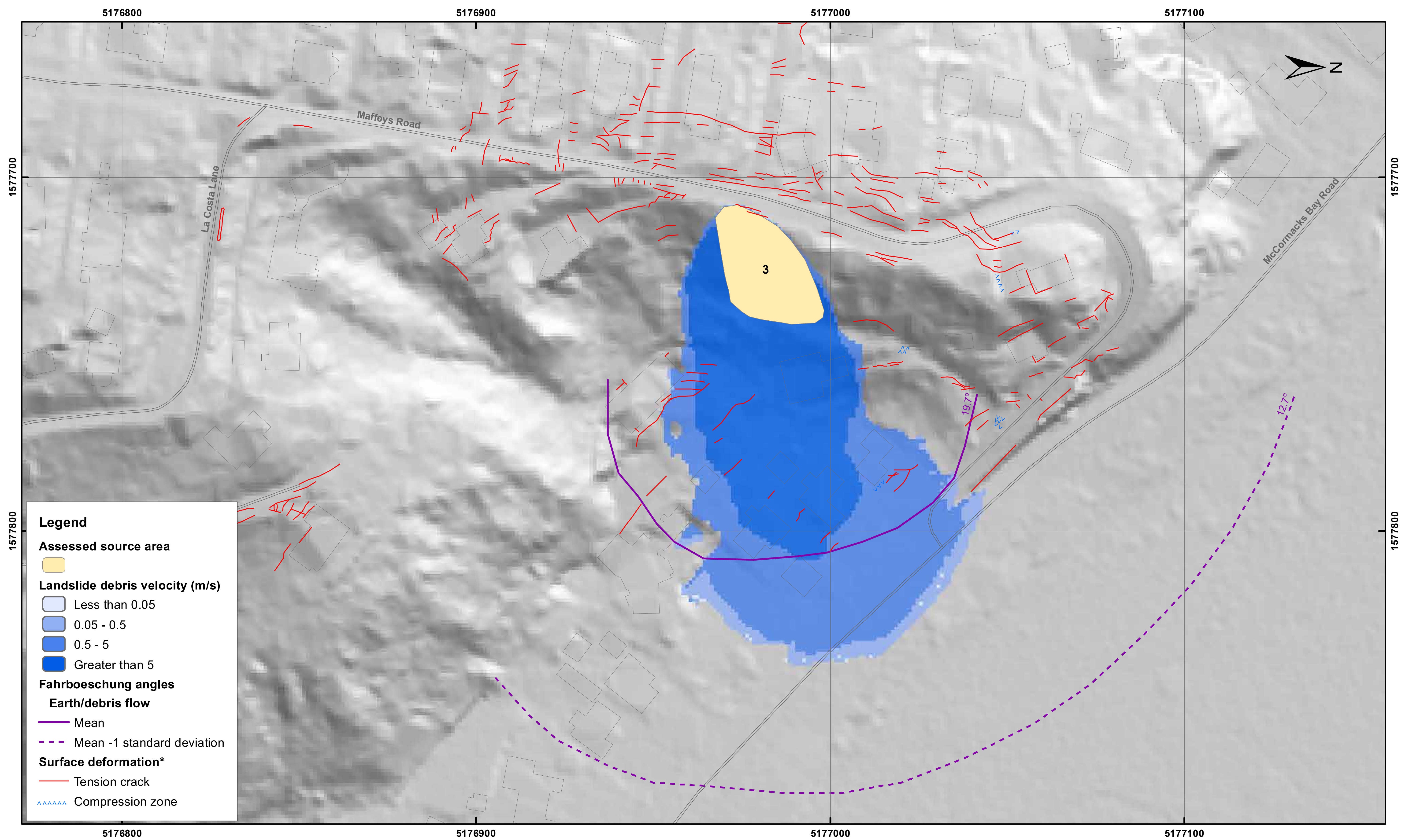
APPENDIX 6

Figure 7

FINAL

REPORT: CR2014/79

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).

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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 3 - Middle Volume (1,800 m^3)

Maffeys Road - Port Hills
Christchurch

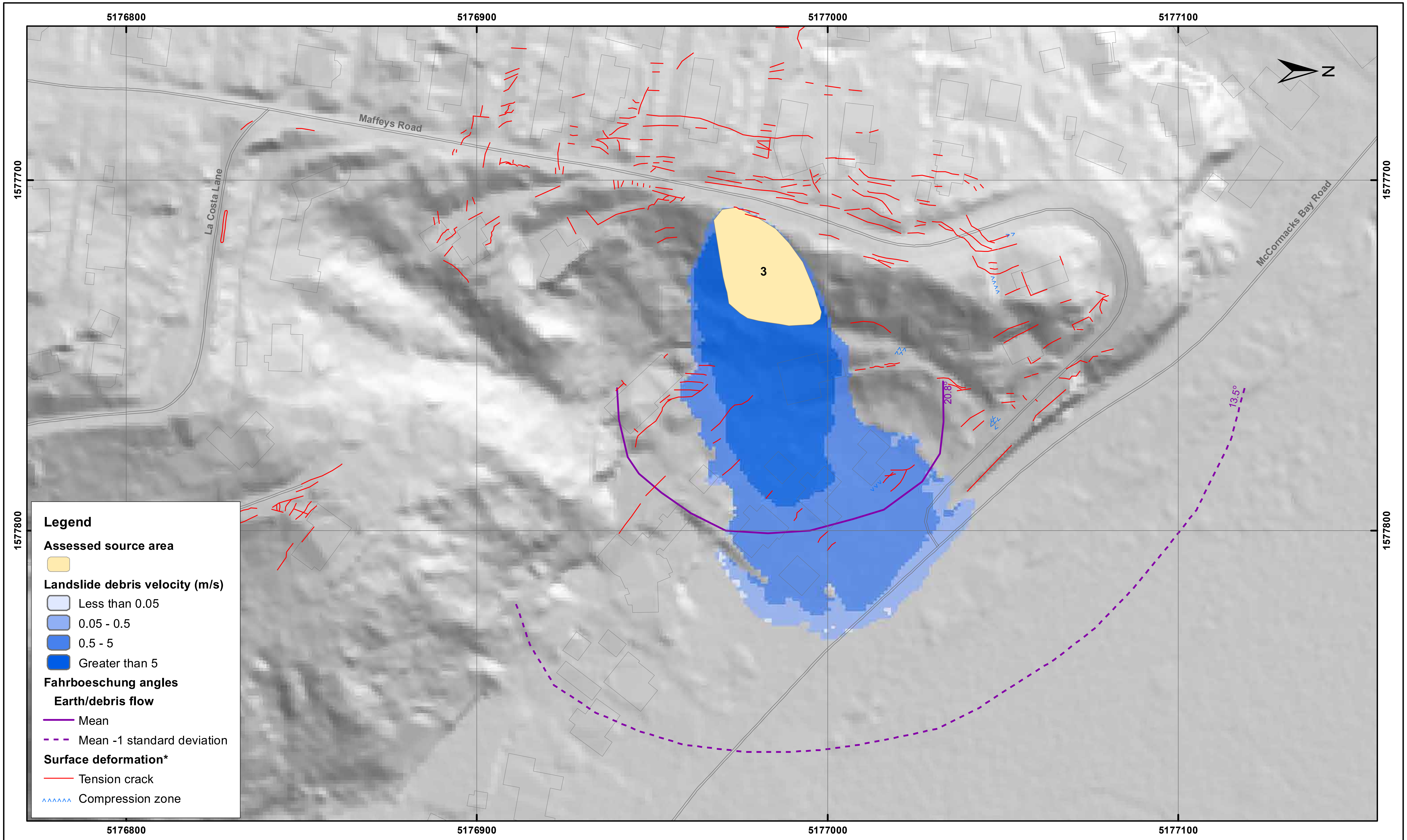
APPENDIX 6

Figure 8

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

EXPLANATION:

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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 3 - Lower Volume (1,000 m^3)

Maffeys Road - Port Hills Christchurch

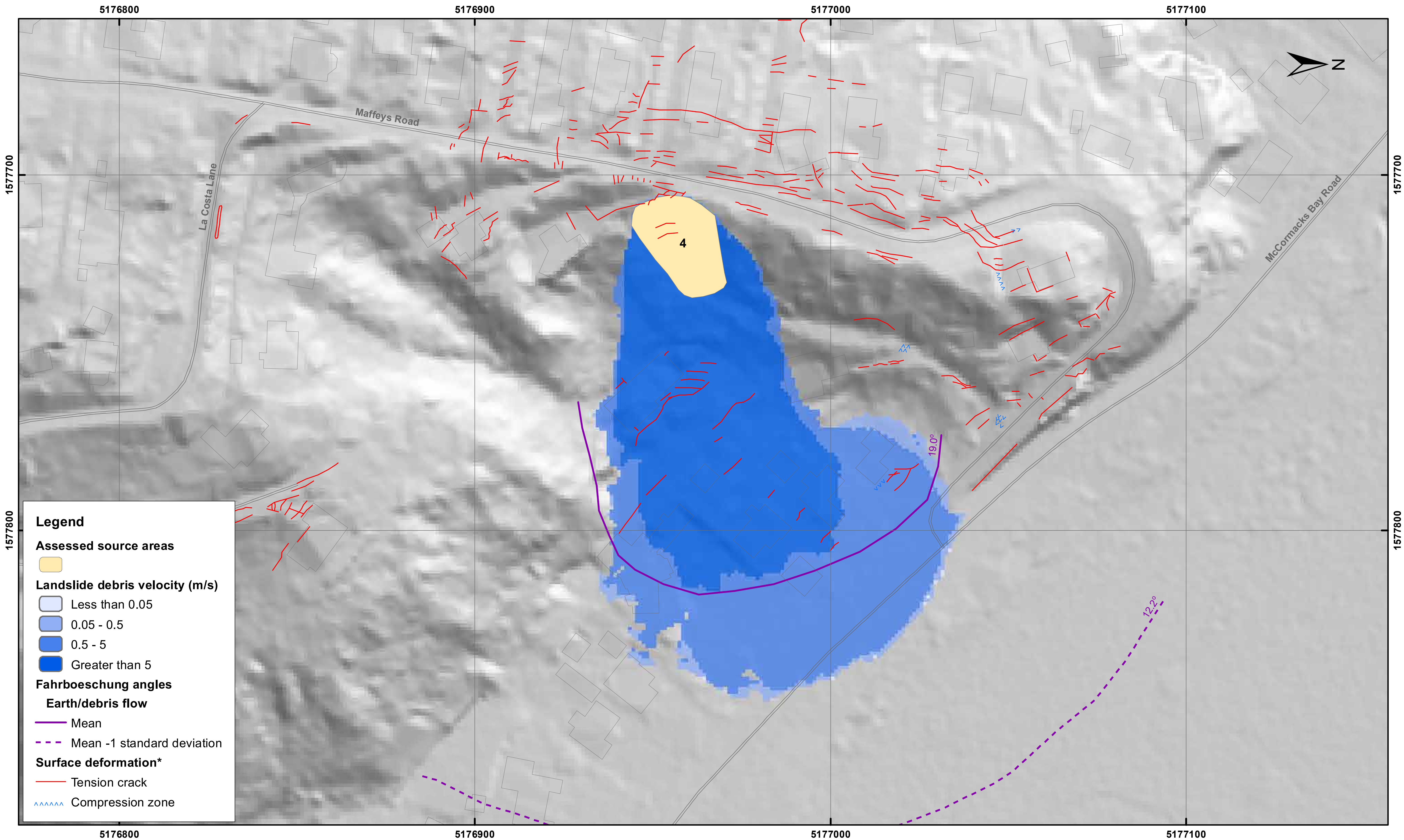
APPENDIX 6

Figure 9

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

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DRW: BL, WR

CHK: CM, FDP

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ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 4 - Upper Volume (2,700 m^3)

Maffeys Road - Port Hills Christchurch

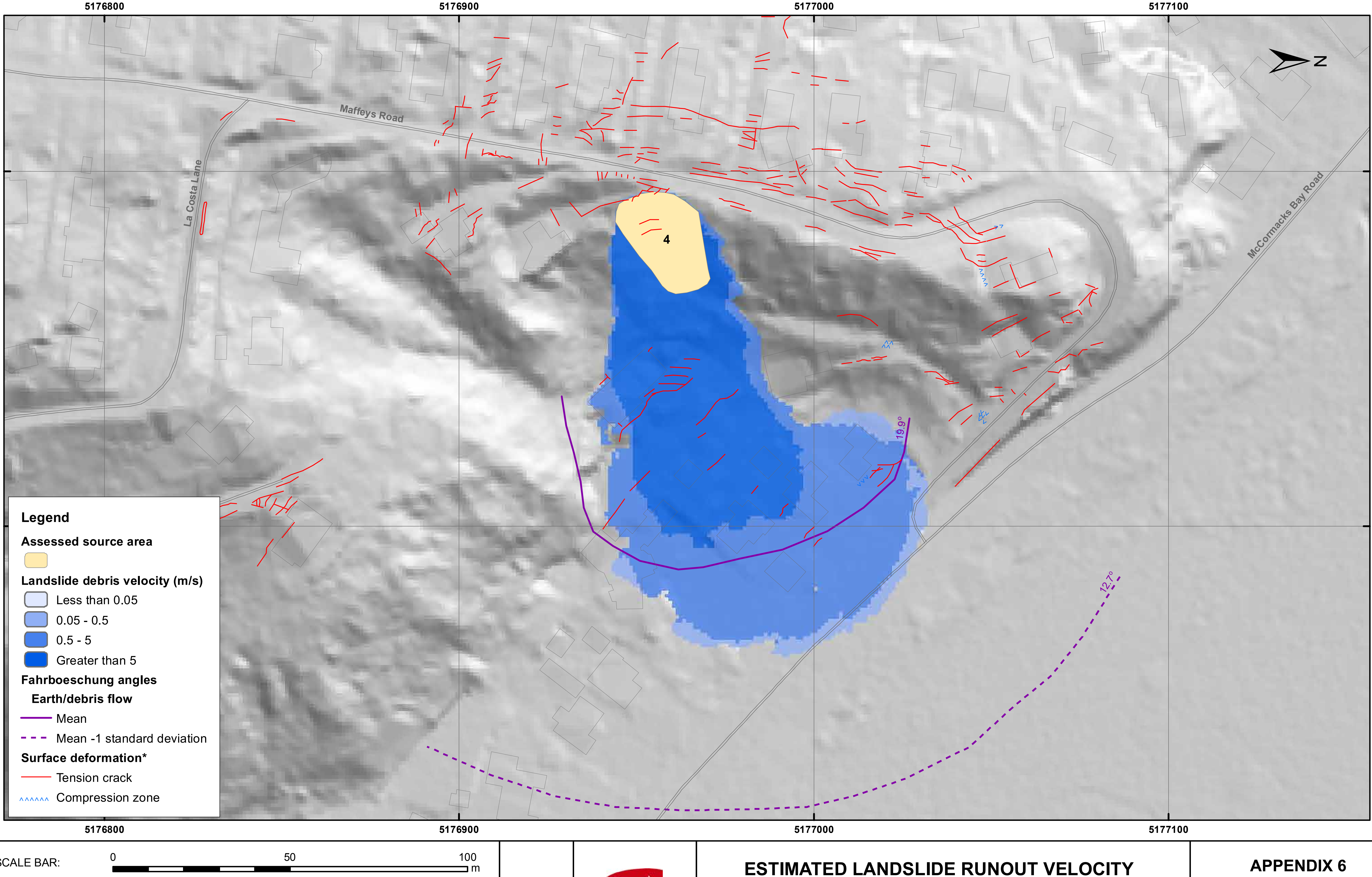
APPENDIX 6

Figure 10

FINAL

REPORT: CR2014/79

DATE: June 2014



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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 4 - Middle Volume (1,700 m^3)

Maffeys Road - Port Hills
Christchurch

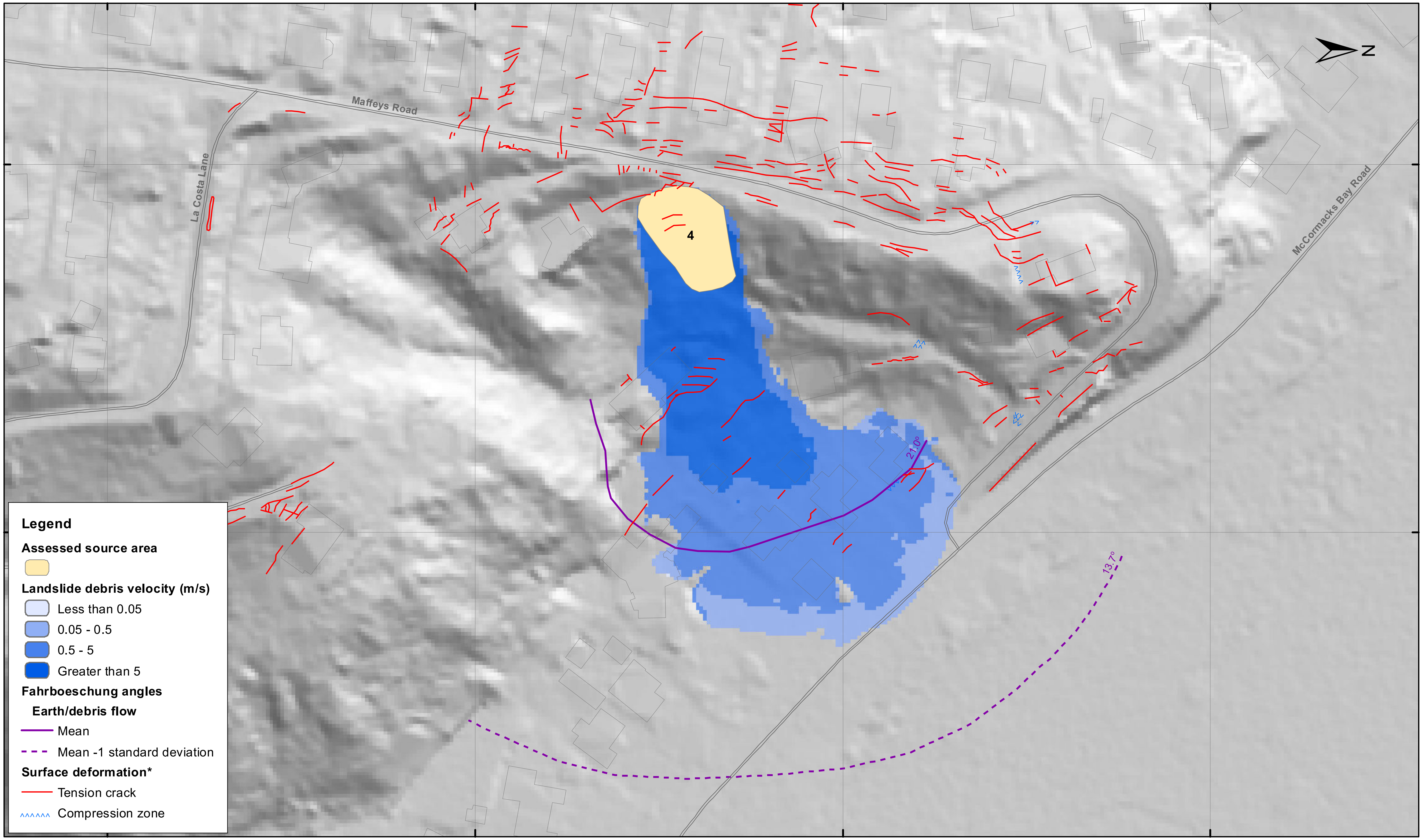
APPENDIX 6

Figure 11

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 4 - Lower Volume (920 m^3)

Maffeys Road - Port Hills
Christchurch

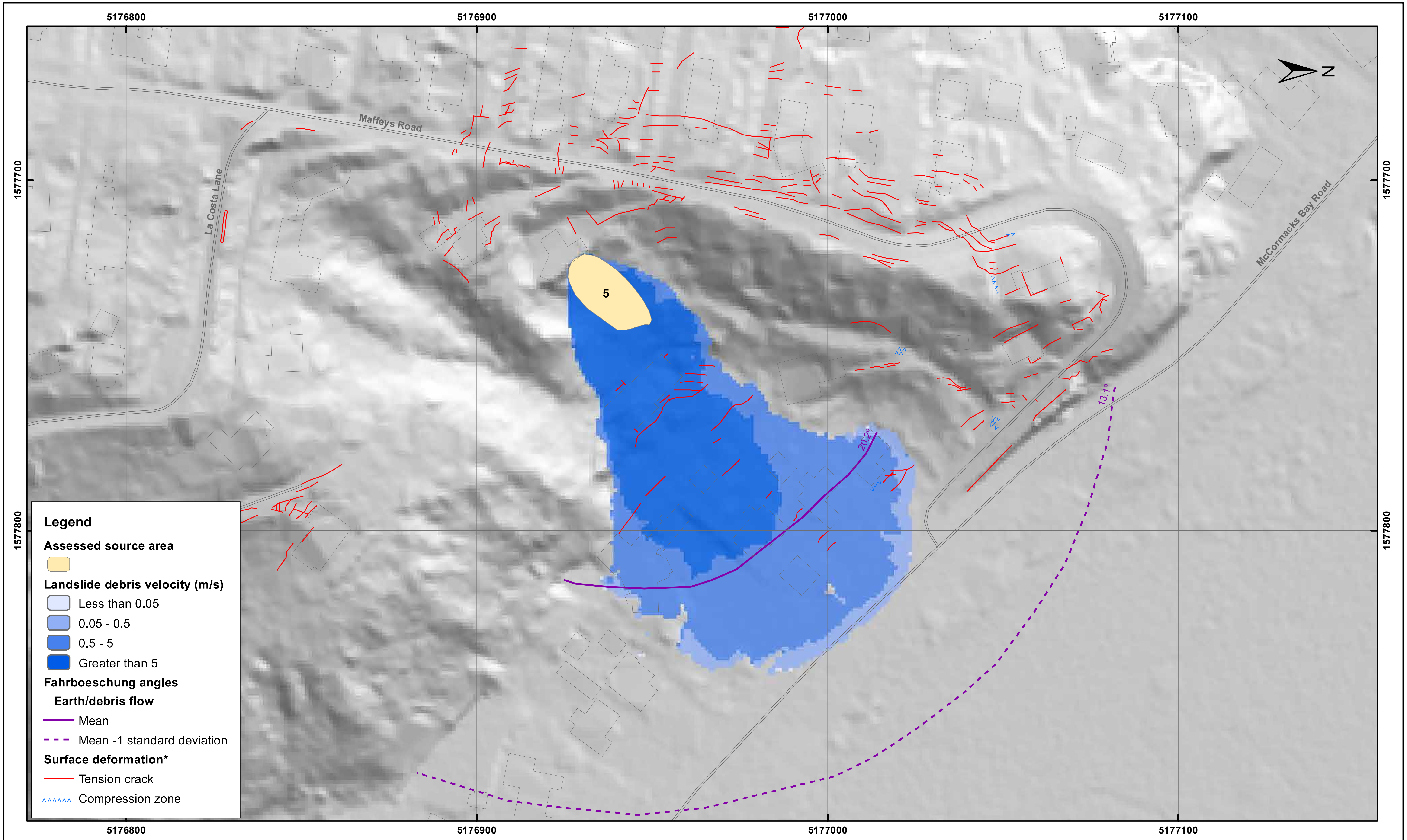
APPENDIX 6

Figure 12

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

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DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 5 - Upper Volume (1,400 m^3)

Maffeys Road - Port Hills Christchurch

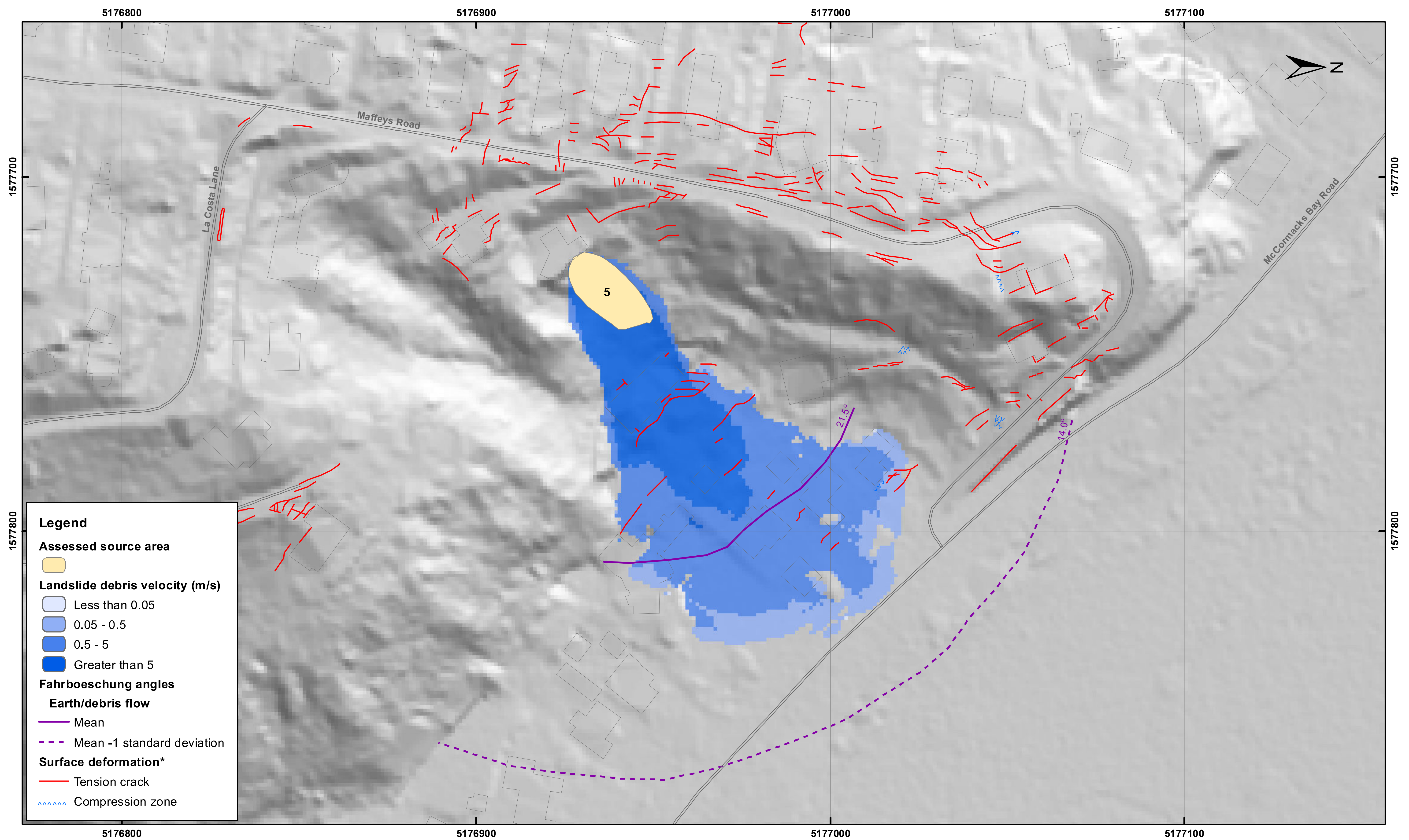
APPENDIX 6

Figure 13

FINAL

REPORT: CR2014/79

DATE: June 2014



SCALE BAR: 0 50 100 m

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PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 5 - Middle Volume (740 m^3)

Maffeys Road - Port Hills
Christchurch

APPENDIX 6

Figure 14

FINAL

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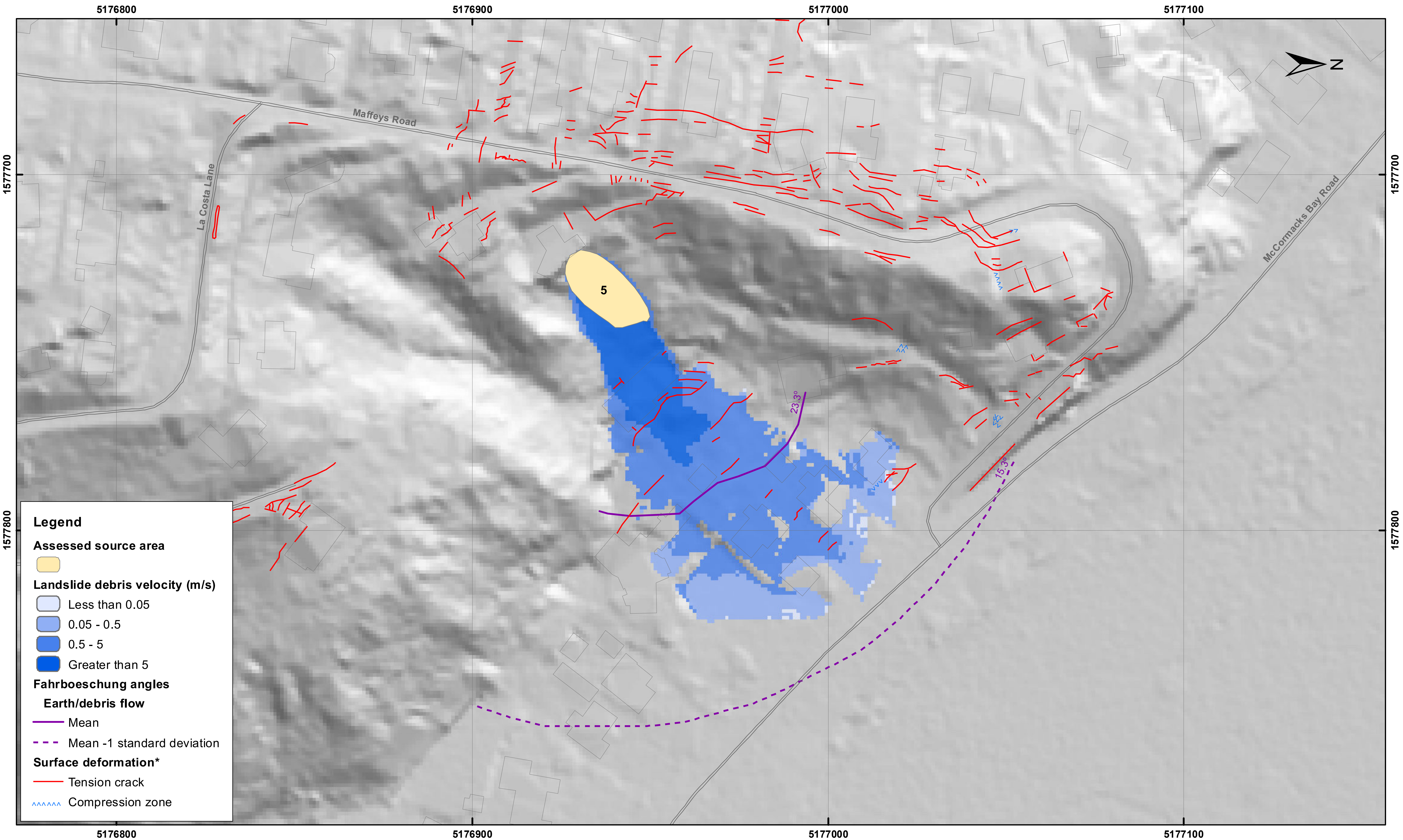
DATE: June 2014



SCALE BAR: 0 50 100 m

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PROJECTION: New Zealand Transverse Mercator 2000

DRW: BL, WR

CHK: CM, FDP

GNS SCIENCE TE PŪ AO

ESTIMATED LANDSLIDE RUNOUT VELOCITY
Source 5 - Lower Volume (320 m^3)

Maffeys Road - Port Hills
Christchurch

APPENDIX 6

Figure 15

FINAL

REPORT: CR2014/79

DATE: June 2014

www.gns.cri.nz

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