APPENDICES

A1 APPENDIX 1: METHODS OF ASSESSMENT

A1.1 HAZARD ASSESSMENT METHOD

A1.1.1 Slope stability modelling

The purpose of the stability assessment was to determine the likelihood of cliff collapse of assessed source areas 1–3, under both static (non-earthquake) and dynamic (earthquake) conditions.

The key output from the static stability assessment is a factor of safety of the given source area and associated volume, while the key output from the dynamic assessment is the magnitude of permanent slope displacement of the given source area expected at given levels of earthquake-induced ground acceleration. These two assessments are then used to determine: 1) the likely local source volumes of material that could be generated under the different conditions; and 2) probability that they will be generated in an earthquake event.

A1.1.1.1 Static slope stability

If a slope has a static factor of safety of 1.0 or less, the slope is assessed as being unstable. Slopes with structures designed for civil engineering purposes are typically designed to achieve a long-term factor of safety of at least 1.5 under drained conditions, as set out in the New Zealand Transport Agency (NZTA) 3rd edition of the bridge manual (NZTA, 2013).

Static assessment of the slope was carried out by limit equilibrium method using the Rocscience SLIDE® software and the general limit equilibrium method (Morgenstern and Price, 1965). The failure surfaces were defined using the path search feature in the SLIDE® software, and a zone of tension cracks was modelled corresponding to mapped crack locations on the surface and in exposures. For the assessment, tension cracks depths were defined: 1) based on the relationship of Craig (1997), where the depth of tensions cracks was determined by the software in order to satisfy the thrust line verification method in the numerical model; and 2) based on field observations of cracks, where the tension cracks were thought to extend from the surface, downwards through the upper basalt lava breccia and into the underlying basalt lava.

Models were run based on geological cross-sections 2, 4 and 6, representing assessed source areas 2, 1 and 3 respectively. The critical slide surface was determined based on the lowest calculated factor of safety. Sensitivity of the slope factor of safety to different geotechnical material strength parameters (models 1–3), was carried out. These strength parameters were derived from in-house laboratory testing on samples of materials taken from the site, and samples of similar materials taken from other sites in the Port Hills and published information on similar materials. Strength parameters were also assessed by back analysis in the limit equilibrium and dynamic analyses.

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

A1.1.1.2 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 Redcliffs 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 history as the input motion.

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

The two steps above are described below in more detail.

1. Dynamic response assessment:
    a. Two-dimensional dynamic site response assessment using Quake/W was carried out adopting synthetic time acceleration histories for the four main earthquakes known to have triggered debris avalanches, cliff-top deformation and cracking in the Port Hills. The modelled versus actual displacements inferred from survey results and crack apertures were compared to calibrate the models.
    b. Synthetic out-of-phase vertical and horizontal free-field rock-outcrop horizontal and vertical time acceleration histories for the site – at 0.02 second intervals for the 22 February, 16 April, 13 June and 23 December 2011 earthquakes – were used as inputs for the assessment (refer to Holden et al. (2014) for details).
    c. The equivalent linear soil behaviour model was used for the assessment, using drained conditions. Strain-dependent shear-modulus reduction and damping functions for the rock materials were based on data from Schanbel et al. (1972) and Choi (2008). For the loess shear modulus and damping ratio functions from Ishibashi and Zhang (1993) were adopted assuming a plasticity index of 5 (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 drillholes BH-MB-01 and BH-MB-02.

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 (models 1–3) for the rock, colluvium and loess as per those used in the static stability assessment. This was done to assess the sensitivity of the modelled permanent deformation of the slope 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 force 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 1.0. 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 (K_y) 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.

Forecasting probability of failure

The probability that the source areas 1–3 would fail during a given earthquake event was based on the estimated amount of permanent displacement of the failure mass, estimated from the decoupled results. For this assessment, the term "fail" refers to a state where the magnitude of permanent displacement causes the given failure mass to break down, forming a mobile debris avalanche.

For this assessment the following assumptions were adopted:

- If the estimated displacement of the source is ≤0.1 m then the probability of catastrophic failure = 0, assuming that the source area is unlikely to fail catastrophically if permanent displacements are ≤0.1 m. This was based on measurements of slopes that underwent permanent displacement (i.e., cracking) but where the displacement magnitudes were <0.1 m and where catastrophic failure did not occur.
- If the estimated permanent displacement of the source ≥1.0 m then the probability of catastrophic failure = 1. Meaning that the source area is likely to fail catastrophically if displacements are ≥1 m. This was based on the magnitudes of displacement inferred from crack apertures at the cliff crests in the Port Hills. Cumulative displacements at the cliff edge, inferred from crack apertures and survey displacements, tended not to exceed 1 m when measured up to the cliff edge. However, in these locations the cliff edge had fallen away, indicating failure at cumulative displacements of greater than 1 m.
- If the estimated permanent displacements are between 0.1 m and 1 m then the probability of failure (P) is calculated based on a linear interpolation between P=0 at displacements of 0.1 m, and P=1, at displacements of 1 m.

A1.1.1.3 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 Redcliffs slope and other similar slopes, during the 2010/11 Canterbury earthquakes.

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

1. Archived newspaper reports (paperspast.natlib.govt.nz). Papers Past contains more than three million pages of digitised New Zealand newspapers and periodicals. The collection covers the years 1839–1945 and includes 84 publications from all regions of New Zealand;
2. The GNS Science landslide database, which is "complete" only since 1996;
3. Insurance claims made to the Earthquake Commission for landslips which are "complete" only since 1996; and
4. Information from local consultants (M. Yetton, Geotechnical Consulting Ltd. and D. Bell, University of Canterbury) which incompletely covers the period from 1968 to present (McSaveney et al., 2014).

A1.1.1.4 Debris runout modelling

The potential runout of debris from the local assessed source areas 1–3 was assessed empirically by the fahrboeschung method and also by numerical modelling. The potential runout of debris from the distributed sources was assessed empirically by the fahrboeschung methods.

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 assumes the slope crest to be the crown of each potential source area.
    b. For distributed source areas, the volume of debris passing a given location within the study area is based on the volumes of material that fell and passed a given fahrboeschung angle, at Redcliffs, during the 22 February and 13 June 2011 earthquakes.
    c. For local assessed source areas 1–3, an empirical relationship established from a compilation of 45 slope sections through discrete debris avalanches that were triggered by the 22 February and 13 June 2011 earthquakes, was used to check the limits of debris runout estimated by the numerical model. This relationship was not used to proportion debris down the slope, as the numerical RAMMS model was used for this.

[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.

2. Numerical methods – RAMMS:

   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).

   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 35,000 m^3) that fell from the slopes at Richmond Hill Road, Shag Rock Reserve and Redcliffs during the 22 February and 13 June 2011 earthquakes.

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

   d. The RAMMS modelling uses a "bare earth" topographic model, and so the runout impedance of buildings and larger trees is not considered (other than incidentally in back analysis).

A1.1.2 Risk assessment

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

A1.1.2.1 Fatality risk to dwelling occupants

The risk is based on the annual individual fatality risk and is assessed for dwelling occupants. The risk includes the assessment of the fatality risk to an individual in a residential home from: 1) debris avalanches (derived from the cliffs); and 2) cliff-top recession. The risk method was similar to the one detailed in Massey et al. (2012a), but now includes the possibility of larger debris avalanches occurring from local assessed source areas 1–3 on the cliff, which because they are larger, could travel further down slope were they to occur.

Annual individual fatality risk is the probability (likelihood) that a particular individual will be killed by a cliff collapse in spending one year at their place of residence. For most localities this probability is a small number. The report therefore makes extensive use of the scientific number format of expressing risk in terms of powers of ten. For example, the number 10^{-4} ("10 to the power of minus 4") is the fraction 1/10,000, and the decimal number 0.0001; it may also be expressed as 0.01%. The units of risk are dimensionless probability per unit of time and the units of annual fatality risk are probability of death per year.

To investigate the influence of uncertainties in the input parameters used in the risk model, three risk-assessment scenarios were examined. These scenarios were based on: 1) an upper, central and lower estimate of the volumes of material that could fall from the slope; and 2) the volume of that debris passing a given distance down the slope. The other parameters represented GNS Science's "best" and "reasonable but more cautious" estimates based on the range of uncertainties identified in the available data at the time of writing. The results for each scenario were modelled using the ArcGIS programme to produce the contoured maps of risk.

For debris avalanches and cliff top recession the risk assessment comprised the following steps:

1. Divide the study area into a series of 2 m by 2 m grid cells.
2. Consider the possible range of triggering events (following the method of Moon et al., 2005) in terms of a set of earthquake triggers and a set of non-seismic (e.g., rain) triggers.
3. Choose a small set of representative events for each type of trigger spanning the range of event severity, from the lowest to the highest.
4. For each representative event, estimate:

   For debris avalanches:

   a. the frequency of the event and the volume of material produced in that event ($P_{(H)}$)
   b. the proportion of debris reaching or passing a given grid cell and the probability of a person at that location being in the path of at least one of the boulders in the debris – the earthquake events include debris from both the randomly distributed sources and the local assessed source areas 1–3 ($P_{(S:H)}$)
   c. the probability that a person is present at a given location in their dwelling as the debris moves through it ($P_{(T:S)}$)
   d. the probability that a person is killed if present and in the path of one or more boulders within the debris ($V_{(D:T)}$)

   For cliff-top recession:

   a. the frequency of the event and the area of cliff top lost ($P_{(H)}$)
   b. the proportion of cliff top lost at a given distance back from the cliff edge and the probability that one of the *N* square metres of cliff top is lost at that location ($P_{(S:H)}$) factoring in both randomly distributed failures and the local assessed source areas 1–3
   c. the probability that a person is present at a given location at the cliff top as the material falls ($P_{(T:S)}$)
   d. the probability that a person if present on an area of cliff top that falls is killed ($V_{(D:T)}$)
5. Multiply 4(a)–(d) for debris avalanche and cliff-top recession to estimate the annual individual fatality risk to individuals at different locations below the cliff or at the cliff crest, contributed by each representative event.
6. Sum the risks from all events (4(a)–(d) separately for debris avalanche and cliff-top recession to estimate the overall risk.

7. Enter the risk value for each grid cell (a 2 m by 2 m grid was used in this study) into a GIS programme and interpolate between the risks estimated in each grid cell to produce contours of equal risk across the GIS map.

A1.1.2.2 Non-seismic events

Rates of debris avalanches and rockfalls triggered without earthquakes, mainly rain, were taken from Massey et al. (2012a). These rates were used to estimate the contribution to total risk from non-seismic triggering events. Four representative event-trigger frequencies were used and the volumes of the debris triggered by events with these frequencies were estimated using a series of steps (frequency was expressed as its inverse, i.e. as return period):

Step 1 – Estimate the trigger frequency of events of a given size that have occurred over a given time period for all sites using the available data. Four event return-period bands were used: 1) 1–14 years; 2) 15–99 years; 3) 100–1,000 years; and 4) >1,000 (nominally 1,000–10,000 years).

Step 2 – Assume a conservative volume of N m^3 per "typical" event in each band, assuming the same volumes per event for all cliffs.

Step 3 – Estimate the annual frequency of a given volume event occurring in each band.

A1.1.2.3 For seismic events

Debris avalanche volumes likely to be generated in an earthquake were determined from the relationship between the volumes of material leaving the cliffs during the 2010/11 Canterbury earthquakes (per square metre of cliff face), and the calculated free field rock outcrop peak ground acceleration at the Redcliffs site (Holden et al., 2014).

Step 1 – Estimate the volumes of material that could be generated at different levels of peak ground acceleration adopting seven event bands, that cover the range of peak ground accelerations from 0.01 to 3 g. For each band adopt a representative event, in terms of the volume generated, by taking the midpoint of each band, and the corresponding volume generated (adopting upper, middle and lower volume estimates based on the statistical range of the data).

Step 2 – for each representative event (volume of debris), calculate the annual frequency of the event occurring. The frequency of a given free field peak ground acceleration band occurring is obtained from the National Seismic Hazard Model. 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 (Stirling et al., 2012), which incorporates the now-increased probabilities of rupture for major faults in the region (Gerstenberger et al., 2011). The risk assessment adopts the year 2016 seismic hazard model results, assuming "aftershocks".

This differs from the previous cliff collapse assessment in Massey et al. (2012a), which used the year 2012 model results (these were the available results at the time of that report). At the instruction of Christchurch City Council, for the risk assessment in this report the year 2016 model results have been adopted to take into account the currently elevated seismic hazard, which is elevated above the 50-year average due to the 2010/11 Canterbury earthquakes.

The model results used in this assessment also include the contributions from all earthquakes, including earthquakes that follow a main earthquake (aftershocks). This differs from the seismic hazard model results adopted by the Canterbury Earthquake Recovery Authority for land zoning purposes, where contributions from aftershocks where removed. Aftershocks were removed because the Canterbury Earthquake Recovery Authority policy makers assumed that people would be evacuated after a large earthquake, and therefore would not be present in their dwelling, and not exposed to cliff collapses triggered by subsequent aftershocks.

GNS Science has assumed the year 2016 seismic hazard model results including contributions from all earthquakes (including aftershocks), as it is not the role of GNS Science to recommend an evacuation policy after a large earthquake.

Step 3 – Take into account the possibility of larger local failure of assessed source areas 1–3. To do this the total volume of debris generated in each band was partitioned between: 1) Random uniformly distributed failures of the cliff face comprising 40% of the total volume, that fall from anywhere on the slope; and 2) Local (non-random) failures comprising 60% of the total volume, corresponding to assessed source areas 1–3.

Step 4 – Calculate the probability of each assessed source area occurring based on the results of the decoupled assessment and the estimated amount of permanent slope displacement (detailed in previous section A1.1.2.3).

Step 5 – Check that the total combined volume of assessed source areas 1–3 is not less than or greater than the 60% of the total volume attributed to these failures per band. At lower event bands the total volume of all the assessed source areas 1–3 significantly exceeds the estimated total debris avalanche volumes produced in the band. For the upper event bands, the total volume of the combined source areas 1–3 is less than the 60% of the total volume produced in the band and attributed to them. Therefore the probability (*P*) of each source occurring is calculated such that *P* x total volume of all assessed sources associated with earthquake events (*V*) = the expected total volume from the sources per given band (Figure A1.1). Thus, the summed volume of the assessed source areas per band cannot exceed 60% of the total volume produced in that band. However, if the total volume of all assessed source areas associated with a given band is less than the total expected volume in that band, the difference in volume is partitioned back to the distributed failures (Figure A1.1).

Expanded calculation of **P**(each localised source occurring)		
Prob of source i occurring given an earthquake =	**Pi**	
Relative probability of source i occurring =	p_i	based on estimated Newmark displacment
Total volume ancitipated, all sources =	**V**	
Volume of source i	V_i	
The requirement is that	$V = \sum Pi.Vi$	(summed over all sources)
But	$\mathbf{Pi} = C.p_i$	where C is a constant
So	$V = C\ .\ \sum p_i.Vi$	

Figure A1.1 Expanded calculation of the probability of each local source area "scoop" occurring.