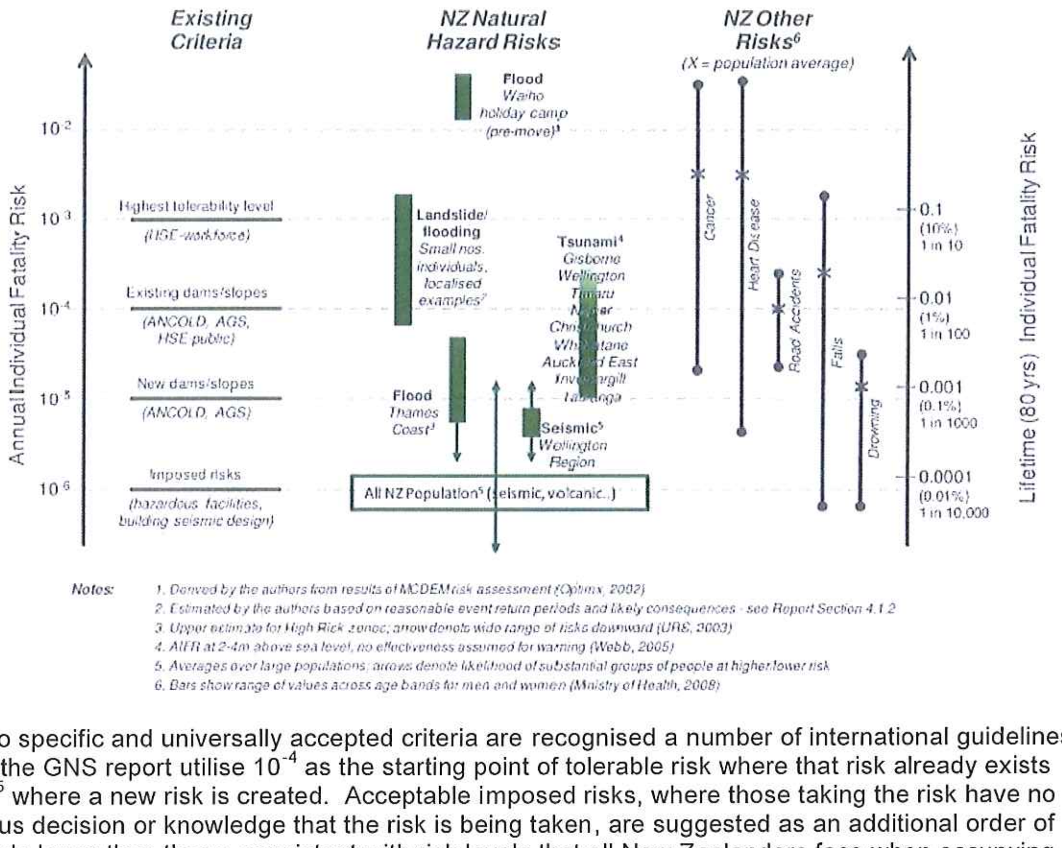
5 Mitigation Measures

5.1 Design Criteria

As part of their risk assessment work GNS has produced a document entitled Principles and Criteria for the Assessment of Risk from Slope Instability in the Port Hills, Christchurch (GNS Science Consultancy Report 2011/319) which outlines international risk criteria that could be applied to the New Zealand context. The following figure presents annual individual fatality risk criteria from a range of international guidelines and illustrates how these compare to common risks such as road accidents that all New Zealanders are exposed to.

Figure 5.1 – Comparison of International Risk Criteria to common New Zealand Risks



Notes:
1. Derived by the authors from results of MCDEM risk assessment (Optimx, 2002)
2. Estimated by the authors based on reasonable event return periods and likely consequences - see Report Section 4.1.2
3. Upper estimate for High Risk zones; arrow denote wide range of risks downward (URS, 2003)
4. AIFR at 2-4m above sea level, no effectiveness assumed for warning (Webb, 2005)
5. Averages over large populations; arrows denote likelihood of substantial groups of people at higher/lower risk
6. Bars show range of values across age bands for men and women (Ministry of Health, 2008)

While no specific and universally accepted criteria are recognised a number of international guidelines cited in the GNS report utilise 10^{-4} as the starting point of tolerable risk where that risk already exists and 10^{-5} where a new risk is created. Acceptable imposed risks, where those taking the risk have no conscious decision or knowledge that the risk is being taken, are suggested as an additional order of magnitude lower than these, consistent with risk levels that all New Zealanders face when occupying buildings designed to current building standards.

GNS suggest that a hierarchy of risk acceptability should be considered such as:

- "intolerable risk - risk greater than say 10^{-4} annual individual fatality risk"
- "not suitable for any development involving a high level of occupancy by people - risk greater than say 10^{-5} annual individual fatality risk"
- "not suitable for particularly sensitive developments such as schools, hospitals or care homes – risk greater than say 10^{-6} annual individual fatality risk"

We understand that tolerable risk levels for existing dwellings in Christchurch has been set at an average of 10^{-4} annual individual fatality risk (GNS, September 2012) and this risk level has been used as the basis of zoning residential properties as acceptable to occupy or not in the Port Hills.

There is no regulated guidance on the level of risk that is acceptable for a school site. Christchurch City Council's Technical Guideline for Rockfall Protection Structures notes that any protection structure must be designed to reduce the annual individual fatality risk below 10^{-4} but is no more specific.

The GNS risk principles report also discusses the concept of societal risk. This recognises that it is more unacceptable to society when large numbers of people are killed in a single event rather than in multiple events. The Australian Geomechanics Society is quoted as saying "Societal risk should be evaluated for buildings having high numbers of occupants, such as schools, hospitals hotels or motels where many lives are at risk. This then addresses society's aversion to loss of many lives from single landslide events". The societal risk concept is that as the number of potential lives that could be lost increases the acceptable risk level should decrease.

It will be necessary for the MoE to agree an acceptable level of risk with the Christchurch City Council (CCC) as the regulating authority. Based on the discussion above it is recommended that the MoE should target an annual individual fatality risk level of no more than 10^{-6} on the site as this level of risk could be justified against international risk guidelines.

The revised school grounds boundary falls outside the extent of the GNS model (the red dashed line in figure 4-1) and are therefore considered by GNS to be at the background annual individual risk level of 10^{-6}. No further calculation of individual or societal risk is possible on the revised school grounds, again because the GNS model does not predict any boulders to reach this distance from the cliffs. Therefore a case could be made that sufficient distance from the cliffs has been achieved such that further mitigation is un-necessary in order to achieve an acceptable level of risk. However, given the aversion to risk on a school site and noting the limitations inherent in any model we believe it would be prudent to provide additional protection on the school boundary in the form of a physical barrier to further reduce residual risk not allowed for in the GNS modelling. As noted in section 4.3.3.2, the potential for individual boulders to reach beyond the risk model limits cannot be discounted.

5.2 Basis of Mitigation Design

Modelling undertaken by both GNS and MWH suggests that large volumes of debris are extremely unlikely to reach a protection structure located on the revised school boundary. Such a barrier would therefore be designed for individual "freak" boulder strikes that may not have been captured by GNS risk modelling. The most appropriate form of such a barrier is considered to be a mechanically stabilised earth bund because of its ability to absorb considerable energy without excessive deformation and damage. Protection bunds utilising this technology have already been constructed in Christchurch as illustrated in photo 5.1 below.

Photo 5.1 – Mechanically Stabilised Earth Rockfall Protection Bund.

It is considered that the most appropriate location for such a barrier is on the southern and western school boundaries as shown by the red line on the following figure. For further detail refer to drawing C001 in Appendix A.

Figure 5.1 – Proposed Extent of Protection Works.

A barrier located in such a position is beyond the maximum rock roll distances identified by GNS modelling and therefore can be viewed as a secondary level of protection. The barrier can be constructed wholly on school land in this location and space is available behind the barrier that is within risk levels acceptable for residential occupation so that long term access behind the barrier for maintenance and clearing should be acceptable in the future.

A design basis has been defined for the barrier as follows:

- A slope at an approximate angle of 2H:1V is formed due to an extremely large scale failure. The volume of rock which falls uses up most of the available storage space behind the barrier.
- Under conditions of extremely high vertical and horizontal acceleration a boulder roll scenario develops down the residual slope.
- A boulder of 5 m^3 (largest size observed to have significantly rolled during previous events) is mobilised during the event and strikes the barrier.
- The design boulder reaches a velocity of 9 m/s at the base of the slope and does not lose any velocity before striking the barrier. A velocity of 9 m/s is the highest velocity modelled at the base of the modelled slopes discussed in section 4.3.3.2, using realistic parameters.

The above scenario is considered a 'serviceability' condition, (i.e. one from which the barrier should be able to be returned to service with minimal works). Additional discussion on an ultimate design case, where the barrier may sustain significant damage is discussed in section 5.3.

CCC's Technical Guideline for Rockfall Protection Structures notes that the design of a rockfall barrier must consider the 95^{th} percentile boulder size as a minimum. With reference to GNS Science Consultancy Report 2011/311 Figure 16 which is based on the database of boulder sizes which fell during the February and June 2011 events, a 5 m^3 boulder equates to approximately a 97% boulder size (the 95^{th} percentile is 3 m^3). The design scenario considered above is compliant with this requirement.

5.3 Barrier Design

The proposed barrier dimensions are shown on Section A of drawing C001 in Appendix A. Calculations justifying the barrier design are attached in Appendix D and discussed in more detail in the following sections.

5.3.1 Service Case

The adequacy of the proposed barrier has been tested against the service design case outlined in section 5.2 using two methods.

1. An assessment of the internal stability of the mechanically stabilised bund to resist the design boulder based on proprietary modelling software operated by Maccaferri New Zealand. This software is based on case histories and full scale tests of Terramesh rockfall bunds.
2. Consideration of the ability of the mass of the barrier acting as an overall mass block to absorb the design energy without excessive deformation.

Appendix D contains calculations produced by Maccaferri New Zealand Limited for this barrier. In undertaking this assessment a nominal bounce height of 0.5 m has been assumed for the design boulder, i.e. although the boulder is essentially considered to be rolling along the ground the potential for a small 'hop' due to any imperfections in the ground surface has been taken into account.

The height of the bund is dictated by:

- the idealised diameter of the boulder,
- the height of any bounce before the bund,
- the recommendation from Maccaferri New Zealand that a safety height of at least one boulder diameter is provided above the centreline of the idealised strike, and
- the need for the bund to be constructed from modular 600mm high units to suit the construction methodology.

The resulting bund has a total height of 4.2 m, 0.3 m of which would be located below existing ground level (refer drawing C001 attached). This would mean the finished barrier was 0.1 m lower than the existing fly rock fence and would essentially replace the function of this fence. Because the barrier is placed at the existing limit that fly rock has been encountered (refer to the red dashed line on Figure 4-1) no additional fly rock protection on top of the bund is envisaged.

The design boulder of 5 m^3 has approximately 560 kJ of energy when it strikes the bund. The bund has been assessed as having a capacity of 2000 kJ under a serviceability condition (i.e. after which repairs should be possible) and in excess of 10,000 kJ of energy in an ultimate capacity condition. There is therefore ample capacity in the design to resist the design boulder.

The Maccaferri design methodology assesses the potential penetration of the boulder into the bund, with this deformation absorbing the energy of the boulder strike. If instead the bund is considered to be a rigid body, the resulting sliding of the overall mass would be the mechanism that absorbs the energy of the boulder strike. Modelled in this way and assuming a 10 m wide length of the bund is mobilised by the boulder strike, the resulting sliding deformation of the bund is approximately 0.36 m. This is considered a very conservative assessment as it ignores internal deformation of the bund and the weight of the bund outside the assumed 10 m wide zone. Nonetheless it provides a secondary assessment that the resulting deformations of the bund are considered to be small and the design is acceptable.

5.3.2 Ultimate Case

The barrier has significant reserve capacity beyond the design case outlined above. Consideration has therefore been given to what boulders the barrier would be able to stop if its ultimate capacity was utilised.

The case of a 95th percentile size 3 m^3 boulder rolling at its maximum possible velocity assuming all of its potential energy from the top of a 70 m cliff was converted to kinetic energy is considered. The resulting velocity is 37 m/s with a resulting energy of approximately 5500 kJ. With reference to page 7 in Appendix D, a boulder with these characteristics would be expected to penetrate approximately 1 m into the bund which is in excess of 4 m wide at its base. While this event would cause sufficient damage to require a partial re-build of the bund, it is still within the ultimate capacity of the bund. It is therefore considered that the bund has sufficient capacity for even the most extreme individual boulder strikes that may occur. It is noted that this event is significantly more adverse than can be modelled and involves considerably more energy even than the extreme case outlined in section 4.3.3.2, which was considered to have adopted unrealistically conservative parameters.

5.3.3 Barrier Layout

The extent and cross section of the bund are shown on drawing C001 attached in Appendix A. The bund has been positioned on land currently owned by the school. It is noted that the bund would clash with some of the existing school buildings on this alignment, however, we understand that these have been, or will be, moved as part of the overall school redevelopment.

The bund has not been extended to protect the existing school carpark on the basis that it would be acceptable to the MoE to accept residential risk criteria in the staff carpark and to minimise the impact on the neighbouring residential properties that are in the green zone and will remain occupied in the future

The bund is located beyond (i.e. further from the cliffs) the point considered acceptable for residential occupation so the risk to workers during construction of the bund would be less than for many homeowners. However, basic health and safety procedures will need to be developed for the construction process. The potential for demolition work to be undertaken nearby the school site at the same time as bund construction will also need to be considered. During operation, it is envisaged that regular inspections would be required on the bund. Given that there is land between the bund and the cliffs that is considered acceptable for residential occupation it is considered that such an inspection system could be put in place while satisfying occupational health and safety requirements.

Continued rockfalls from the cliffs surrounding the school can be expected in the future. Other than confirmation that rockfall behaviour is as expected through a regular inspection programme, no intervention or removal of fallen rock is anticipated. In the event of a large volume falling in one event (most likely from one of the mass movement areas identified) it is anticipated that there would be a period of reassessment of the resulting slope to confirm that it did not represent a more adverse situation than covered in assessments undertaken to date. Provided that the resulting slope configuration did not represent a more adverse situation then no intervention or removal of debris at the base of the cliffs would be anticipated. Given the large scale failures that have already been modelled by GNS it is not anticipated that a more adverse slope configuration would occur.

Following confirmation of the interim concept we understand that the bund arrangement and design will need to be taken through both resource consent and building consent processes. Work associated with these processes is beyond the current scope of this assessment.