IN THE MATTER OF the Resource Management Act 1991

AND

IN THE MATTER OF applications by Central Plains Water Trust to:

Canterbury Regional Council for resource consents to take and use water from the Waimakariri and Rakaia Rivers and for all associated consents required for the construction and operation of the Central Plains Water Enhancement Scheme

Selwyn District Council for resource consents to construct and operate the Central Plains Water Enhancement Scheme

AND

IN THE MATTER OF a notice of requirement by Central Plains Water Limited to:

Selwyn District Council for the designation of land for works associated with the construction and operation of the Central Plains Water Enhancement Scheme

---

BRIEF OF EVIDENCE OF TIMOTHY JOHN MCMORRAN

31/01/08

BUDDLE FINDLAY
Barristers and Solicitors
Christchurch

Solicitor Acting: **Rachel Dunningham**
Counsel: **Dr E D Wylie Q C**
Tel 64-3-379 1747 Fax 64-3-379 5659 PO Box 322 DX WP20307 Christchurch

Qualifications and experience

1. My full name is Timothy John McMorran.

2. My professional qualifications include a BSc (geology and chemistry) and MSc (Hons) in engineering geology from University of Canterbury in 1991. Since then I have worked as a consulting engineering geologist, mainly based in Christchurch. I am familiar with Canterbury geology having studied and worked in the region for twenty years. I have been involved in geotechnical investigations for many dams and have experience in seismic hazard and active fault evaluation, construction materials assessment and slope stability evaluation. I have undertaken geotechnical investigations and seismic hazard assessments of many dams at concept or preliminary design stage and assisted in analysis, remedial design and construction supervision. I was involved throughout investigations, design and construction of remedial works of two large dams in New Zealand with high seismic risk. I supervised extensive design-stage geotechnical investigations for Meridian Energy’s Project Aqua in the lower Waitaki valley. Along with a colleague at URS I am Contact Energy’s Geological Consultant advising on management of reservoir landslides around Lakes Dunstan and Roxburgh.

3. I have read the code of conduct for expert witnesses set out in Environment Court practice note, and confirm that I have complied with the code in the preparation of my evidence.

Scope of Evidence

4. I have been asked to present evidence on the geology of the scheme area and how it will affect the proposed scheme. My evidence will cover the geology affecting the proposed dam in the Wainiwaniwa valley, the main distribution canal and the intake tunnel. Issues of dam design will be covered within evidence prepared by Mr Richard Davidson.

Site Description

5. The Central Plains Water Enhancement Scheme proposes to distribute irrigation water to 60,000 ha of central Canterbury. A nearly horizontal headrace canal will link the Rakaia and Waimakariri Rivers at approximately RL 235. This canal will have an intake on the Rakaia River downstream from the Rakaia Gorge and on the Waimakariri River. A tunnel will transfer water to the reservoir from an intake between the Kowai River and Waimakariri Gorge. The proposed scheme layout is shown in Figure 1.

6. The proposed Waianiwaniwa dam site is at the mouth of the Waianiwaniwa valley where the Waianiwaniwa River emerges from the Malvern Hills (Figure 1). The valley has a 300 m wide floor at approximately 235 m elevation in the vicinity of the proposed dam site. The Waianiwaniwa River follows a 6 km long valley starting near where the Selwyn River emerges from its gorge upstream from Whitecliffs. Near its headwaters the Waianiwaniwa River flows east for about 3 km, then turns abruptly to flow south for a further 3 km down to the proposed dam site. It then flows across the Canterbury Plains to join the Selwyn River about 16 km to the southeast.

Regional Geology

7. Gregg (1964) and Wilson (1988) have undertaken regional geological studies presented as geological maps with scales of 1:250 000 and 1:100 000 respectively. Detailed geological studies of the Malvern Hills were carried out by Speight (1929) and new geological mapping has been undertaken for the QMap 1:250 000 geological map compiled at draft stage by GNS (in prep). The geological setting for the proposed scheme is presented in Figure 2. Figure 3 is a more detailed map of the geology of the reservoir area.

8. The eastern Southern Alps largely consists of indurated sandstone and mudstone usually referred to as ‘Torlesse greywacke’. In Canterbury, this ‘basement’ rock is overlain by a sequence of younger sedimentary and volcanic rocks all of early to mid Tertiary age (between 60 and 10 million years old). Table 1 summarises the sequence of geological units (‘stratigraphy’) found in the Malvern Hills. ‘Mount Somers Volcanics’, comprising 100 million year old volcanic rhyolite and andesite (Cretaceous age), form Mount Misery to the south of the Selwyn River, but do not outcrop in the Waianiwaniwa valley.

9. Fluvioglacial gravels of Quaternary age overlie the basement and Tertiary age rocks as extensive outwash terraces. Wilson (1988) defines these as a series of formations of similar age: the Hororata, Woodlands, Windwhistle, Burnham and Springston Formations. These units underlie the reservoir, dam site and proposed headrace corridor. The gravels generally become younger to the east, away from the mountain source.

10. The proposed reservoir is crossed by an anticlinal fold structure affecting the Tertiary rocks. The structure describes an asymmetric anticline plunging to the northeast. The older outwash gravels appear to be tilted in a manner suggesting that the folding is a response to ongoing tectonic deformation.

Table 1: Stratigraphy of the Malvern Hills area

Unit name	**Description**	**Age**	
Harper Hills Basalt	Porphyritic and vesicular basalt	Miocene (10M years)	**Youngest (upper) units**
Homebush Sandstone	Quartz rich and glauconite poor sandstone	Eocene	
View Hill Volcanics	Basaltic flows, pillow lavas, marine tuff, sandstone, volcanogenic breccia	Early Eocene	
Waipara Greensand	Glauconitic sandstone	Paleocene	
Conway Formation	Massive, micaceous concretionary marine sandstone	Late Cretaceous to early Paleocene	
Broken River Formation	Quartz-rich sandstone, mudstone, thick coal seams	Late Cretaceous	
Monro Conglomerate	Sandstone, mudstone and well-cemented conglomerate containing rhyolitic pebbles, thin coal lenses	Late Cretaceous	
Wakaepa Plant Beds	Mudstone, sandstone and conglomerate, containing plant fossils	Jurassic	**Oldest (lower) units**
Torlesse Terrane “greywacke”	Indurated sandstone and mudstone	Triassic	

Regional Seismicity

11. In the last two decades considerable research effort in New Zealand has been directed to evaluate the hazard associated with geological structures capable of generating earthquakes (eg. Pettinga et al. 1998, Stirling et al., 2002, 2007). More than 100 active faults that affect Canterbury are identified by Stirling et al. 2007.

12. The activity of a fault can be defined according to its recurrence interval (i.e. the average interval between fault ruptures). A fault that has ruptured during the last 125,000 years is considered ‘active’ (Kerr et al., 2003), while faults can have recurrence intervals of as little as 300 years.

Table 2: Recurrence interval classes for active faults (following Kerr et al., 2003)

Recurrence interval class	Fault recurrence interval
I	≤2000 years
II	>2000 years to ≤3500 years
III	>3500 years to ≤5000 years
IV	>5000 years to ≤10,000 years
V	>10,000 years to ≤20,000 years
VI	>20,000 years to ≤125,000 years

Note: Faults with average recurrence intervals >125,000 years are not considered active.

13. I have reviewed web-based databases maintained by GNS (New Zealand Active Faults Database[1]) and Environment Canterbury along with current literature on active faulting in Canterbury. Several faults have been identified that pass within 30 km of the proposed dam site, as shown on Figure 4, and discussed below.

Hororata Fault

14. The Hororata Fault was discovered by recent seismic reflection surveys carried out for oil exploration (GNS, in prep). The fault is postulated to pass about 2.5 km south of the dam site (Figure 4). No young fault scarp has been identified to define this structure at the surface but the following evidence suggests deformation:

    (a) Windwhistle age (45,000 year old) outwash terraces in the vicinity of Racecourse Hill are backtilted to the northwest.

    (b) In groundwater wells Tertiary sediments were encountered at 11 m depth underlying Quaternary gravel on the north side of the fault and more than 255 m depth of Quaternary gravel was encountered immediately south of the fault (GNS in prep).

    (c) Interpreted fault deformation of Quaternary gravels in a seismic reflection profile completed for petroleum exploration by Indopacific along Plantation Road.

---

[1] NZAFDB is a web based database run by GNS Science of active faults throughout New Zealand. (http://data.gns.cri.nz/af/index.jsp)

15. The latest version of the New Zealand National Seismic Hazard Model (NZNSHM) (Stirling et al 2007) characterises the Hororata Fault with a return period of 4800 years and M_{max}[2] of M_w7.1[3].

16. Based on this evidence, I believe this fault defines the source for the controlling maximum earthquake (CME) for the dam.

17. The headrace canal will cross the Hororata Fault between the Selwyn River and Hororata.

Porters Pass Fault

18. The Porters Pass Fault is the expression of the southwestern part of the Porters Pass-Amberley Fault Zone. It crosses the southeastern end of Lake Coleridge and strikes northeast to cross Lake Lyndon and Porters Pass. The fault is the most active structure known in the vicinity of the CPWE scheme. Recent research (Howard et al. 2005) indicates that the Porters Pass Fault generates earthquakes of Magnitude M_w7.2+ every 1500 years. Associated with this earthquake is up to 8 m of lateral ground displacement. The Porters Pass Fault does not directly cross any part of the CPWE scheme, but passes approximately 27 km north of the proposed Waianiwaniwa dam site.

19. The NZNSHM includes the Porters Pass Fault and characterises it with a maximum magnitude of M_w7.4 and a return period of 1400 years (Stirling et al. 2007).

Springfield Fault

20. The recently discovered Springfield Fault strikes approximately northeast near to Springfield. Stirling et al. 2007 characterise this fault with a return period of 5200 years and magnitude of M_w7.2.

21. The Springfield fault passes approximately 14 km north of the proposed Waianiwaniwa dam site. It may cross the proposed intake tunnel for the scheme.

Kowai Fault

22. The Kowai Fault is indicated in the NZAFD and ECan database to pass along the Kowai River valley west of Springfield, approximately 20 km from the dam site. Magnitude and recurrence intervals have not been defined and this fault is not currently included in the NZNSHM.

---

[2] M_{max} (maximum magnitude) refers to the expected earthquake magnitude if a fault ruptures over its full area.

Western Gully Fault

23. The sequence of Cretaceous to Tertiary aged sediments described in Paragraph 8 dip to the south east in the area of the proposed dam site as a result of tectonic deformation. Speight (1928) inferred an east-west striking fault at the contact between a small outcrop of Tertiary sediments in the upper Waianiwaniwa Valley and adjacent Torlesse greywacke outcrop.

24. During investigations for a proposed regional landfill 5 km north of the proposed dam site, a west-dipping thrust fault (referred to as the Western Gully Fault) was found at the contact between Cretaceous sedimentary rocks and basement Torlesse sandstones (Dr Mark Yetton, personal communication). East-dipping reverse faults were also found outcropping south of the Western Gully Fault. These faults were inferred to be active on the basis of displaced Late Pleistocene and Holocene colluvium. Sheared surfaces were also found in drill core during the landfill investigation and these were interpreted to represent small amounts of displacement along bedding within the Tertiary sediments. Stream gradient and sinuosity analysis undertaken by URS suggests that ongoing tectonic deformation affects the Waianiwaniwa River where it crosses the Western Gully Fault.

25. GNS does not consider the Western Gully Fault large or active enough to be included on the draft QMap (GNS in prep) and it is not included on the NZAFD, ECan faults database or in the NZNSHM. However, the information presented above suggests that a fault underlying the upper reservoir should be treated as active. If this proves to be a significantly large fault that is active, its shaking hazard to the proposed dam is expected to be similar to, or lower than, other known active faults shown in Figure 4. In addition the possibility of a seiche wave generated by this fault needs to be considered in the dam design.

Regional tilting

26. Remnant outwash surfaces exist around the Malvern Hills at up to 100 m above the floor of the Waianiwaniwa Valley. The highest surface has been mapped as Hororata Formation (Wilson 1988) and is characterised as a highly dissected morphology, thick loess cover (up to 15 m thickness) and gravel comprising weathered greywacke clasts. Several square kilometres of dissected terraces east of the proposed reservoir have been mapped as Hororata Formation by Wilson (1988). The Hororata Formation terrace

---

[3] M_w (moment magnitude) is an expression for earthquake size based on the area of fault rupture and the average fault displacement.

surface is tilted toward the southeast at about 2 degrees, which is consistent with on-going tectonic deformation. No fault offset of this surface has been observed.

Preliminary seismic hazard assessment

27. A review of the New Zealand Society on Large Dams (NZSOLD) dam design guidelines and equivalent international standards (Mejia et al. 2001) indicates that many agencies recommend dual performance criteria for dams:

    (a) The Operating Basis Earthquake (OBE) should be withstood with only minor damage. The OBE is used to address dam serviceability and should have an annual exceedance probability (AEP) of 1/150 years.

    (b) The Maximum Design Earthquake (MDE) is used to address dam safety in dam design. The dam must be designed to withstand the MDE without severe damage (i.e. without catastrophic release of the reservoir). In the case of a large dam such as the Waianiwaniwa Dam, the MDE is usually evaluated deterministically (i.e. based on a realistic and usually demanding earthquake scenario).

28. The Controlling Maximum Earthquake (CME) is the maximum earthquake on the seismic source that is capable of inducing the largest seismic demand on a dam. In the case of the Waianiwaniwa dam, the seismic source most likely to generate the CME is the Hororata Fault due to its close proximity to the dam.

29. Assuming the dam is designed as a high potential impact category dam in accordance with NZSOLD guidelines (URS, 2006), Mejia et al. (2001) suggest the dam should be designed to accommodate 84th percentile ground motions for the controlling maximum earthquake (CME) but need not exceed the 10,000 year annual exceedance probability ground motions. This means that the dam must be designed to accommodate a conservative estimate of the ground shaking intensity that will occur during the most severe earthquake expected once every ten thousand years at the dam site.

Probabilistic hazard assessment based on the NZNSHM

30. The New Zealand National Seismic Hazard Model predicts the seismic hazard throughout New Zealand using a probabilistic seismic hazard assessment method (Stirling et al. 2002). The model predicts future seismic hazard by considering historical seismicity as well as geological data characterising active faults. The method yields a description of how likely it is

that different levels of ground motion will be exceeded at a site within a given time period. Estimates of peak ground acceleration (pga) predicted for the Waianiwaniwa dam site by the most recent version of the NZNSHM for Canterbury (Stirling et al. 2007) are presented in Table 3. This data is adequate to undertake a preliminary estimate of the OBE for the dam. A more thorough site-specific assessment will be required as part of the detailed dam design. The 150 year return period peak ground acceleration that constitutes the preliminary OBE is 0.25 g [where "g" is the acceleration due to gravity 9.8 m/s^2].

Table 3 Estimates of peak ground acceleration (pga) for the Waianiwaniwa dam site

Return Period (years)	**Stirling et al. (2007) pga (g)**
150	0.25
475	0.41
1000	0.54

Preliminary deterministic assessment of the MDE

31. In accordance with the guidelines suggested by Mejia et al (2001) the MDE for the Waianiwaniwa dam should be based on a deterministic assessment of the controlling maximum earthquake (CME) which I believe to be a maximum earthquake on the Hororata Fault.

32. I have estimated the peak ground acceleration that will be experienced at the dam site during maximum earthquakes on the Hororata Fault, Springfield Fault and Porters Pass Fault. The values presented in Table 4 correspond to the mean and 84th percentile peak ground accelerations predicted by the attenuation relationship of McVerry et al. (2006). This is considered to be appropriate for an assessment of ground motions for preliminary dam design. A more detailed assessment will be carried out during detailed dam design to refine the values.

33. For preliminary design purposes the MDE is a maximum earthquake on the Hororata Fault generating peak ground acceleration in the range 0.64 g to 1.0 g.

Table 4 Active faults within 30 km of the proposed dam site

Fault	Distance to site	M_{max}	Peak Ground Acceleration (g)	
			Mean	84th Percentile
Hororata Fault	2.5 km	7.1	0.66	1.0
Porter's Pass Fault	27 km	7.4	0.28	0.44
Springfield Fault	14 km	7.2	0.39	0.62
Springbank Fault	15 km	6.8	0.33	0.53
Cust Fault	18 km	6.9	0.30	0.49

Notes: M_{Max} estimates from Stirling et al. 2007.
Peak ground acceleration calculated using the attenuation relationship of McVerry et al. 2006 assuming a Class C (Shallow Soil) site.

Foundation faulting

34. My assessment of the reservoir and dam site geomorphology and available subsurface investigations has not identified any fault through the dam site. However, given the relatively close proximity of the Hororata Fault and Western Gully Fault, there remains a possibility of a fault passing directly through the dam foundation site being discovered during detailed site investigations. While faults that have short return periods (less than a few thousand years) typically advertise themselves in the local geomorphology, faults with less frequent rupture history can be hidden in the landscape. I have utilised aerial photographs and digital terrain models to identify lineaments that could advertise faults. Many of the lineaments evident in the landscape are parallel to bedding strike in the Tertiary age units. Several lineaments cross the proposed embankment footprint though none are associated with identified deformation of postglacial surfaces. Figure 4 shows identified lineaments and stream gradient or sinuosity anomalies. One trench excavated during preliminary site investigations (TR1 refer Paragraph 64) specifically targeted a lineament and found no evidence of tilting or displacement of the near surface gravels.

35. Mejia et al. (2001) present guidelines for dam design accommodating active faults within the dam foundation. The guidelines treat foundation faults in the same way as earthquake ground motions and floods and suggest that a foundation fault with a return period of 10,000 years or less requires specific design features, to allow the dam to accommodate the predicted fault movement without uncontrolled release of the reservoir. Foundation faults that have return periods significantly greater than 10,000 years represent an

acceptable risk and require no specific design features. This approach to faults in dam foundations is used by owners of large dams in New Zealand including Meridian Energy and Mighty River Power.

36. Additional site investigations will be undertaken as the project proceeds through design and construction to identify and quantify the level of activity of any faults through the dam foundation. Such investigations will target lineaments shown in Figure 4 and are expected to include additional drilling, geophysics and exploratory trenching. Subsurface investigations for foundation faults will need to cover a large proportion of the dam footprint as reverse, or thrust, faults are often discontinuous or sinuous and do not necessarily follow lineaments. This approach is consistent with the normal course of dam design.

Seismic hazard affecting the headrace canal and intake tunnel

37. The headrace canal will incorporate sections in cut and sections constructed on fill embankments. The design of the embankments will need to take into account the seismic hazard as with dam design. Design ground motions will need to be selected that are appropriate for the stored volume and population at risk from the embankment. Appropriate design features will need to be incorporated to limit the embankment deformations under earthquake shaking to protect embankment freeboard and limit to potential volume of discharged water.

38. The canal alignment crosses the Hororata Fault at least three times: twice near to Racecourse Hill where the canal is in cut and once north of Hororata where the canal is partly in cut and partly on embankments. The Hororata Fault may also underlie the canal alignment on the left bank terraces of the Rakaia River. In these areas the canal design may need to take account of deformation that could occur during rupture of the Hororata Fault. Movement of the Hororata Fault is expected to result in warping or folding at the ground surface rather than direct ground rupture forming a fault scarp.

39. Tunnels are not usually significantly affected by earthquake shaking but the shear waves could cause flexure, and possibly cracking in the lining, where the tunnel is shallow and in readily deformable materials. An assessment of the location and level of activity of the Kowai or Springfield Faults should be undertaken as part of the detailed design, but the fault rupture hazard is unlikely to constrain the design as the return period of these faults is thought to be relatively long.

Site Investigations

40. Geological investigations for the Central Plains Water Scheme have included aerial photograph interpretation, geological mapping and subsurface investigations (URS 2001, 2002a, 2002b).

41. Aerial photograph interpretation has been undertaken using six sets of aerial photographs dating between 1943 and 1996. Digital terrain models have also been evaluated. The digital terrain models are based on 5 m contour topographic data from the dam and reservoir footprint, supplemented by 20 m contour topographic data elsewhere.

42. A preliminary investigation (URS, 2002a) was undertaken to establish the depth of fine-grained sediments in the Waianiwaniwa Valley. A total of 6 Cone Penetrometer Test (CPT) probes were completed at two possible dam sites. Locations of the tests are shown in Figure 5 and the results of those tests are presented in Appendix A. CPTs 1, 2 & 3 were undertaken along a possible embankment alignment approximately 500 m upstream from the current alignment. These CPTs identified at least 15 m of silt and sand beneath the valley floor. These materials represent significant adverse foundation conditions for an embankment dam and the decision was made to abandon the upstream alignment and investigate the current alignment instead.

43. Subsequently, a total of 12 test pits (including Trench TR1) were excavated to investigate the near surface geology of the current dam site and the locations of these are presented in Figure 5. These were excavated using a 20 Tonne tracked excavator operated by W. A. Boyes Contracting Limited. All excavations were logged and photographed. The logs are presented in Appendix B. On completion of logging and photographing the excavations were backfilled.

44. Three exploratory drillholes were carried out along the proposed dam alignment. These were drilled using a UDR 650 drilling rig operated by McNeill Drilling. The holes were mainly advanced using PQ triple tube techniques, but non-cored “Tubex” techniques were used in some of the near surface gravels. Very good core recovery was achieved, even in loess and gravels. The drill holes were logged and photographed and are presented in Appendix C. The locations of the drillholes are presented in Figure 5.

Stratigraphic Sequence and Distribution

45. The geology of the reservoir is summarised in Figures 2, 4 and 5, which summarise my observations along with information from Speight (1928), Gregg (1964), Wilson (1988) and GNS (in prep). Cross sections of the reservoir and dam site are presented in Figures 6 and 7. Geological mapping undertaken by URS includes extensive walkover of the dam site and abutments, Homebush Ridge and access on public land throughout the reservoir allowing observation of outcrops by binoculars.

46. The complete sequence of Cretaceous and Tertiary age units and Torlesse greywacke basement described in Table 1 are present within the area of the reservoir.

Quaternary Stratigraphy

47. Pleistocene glacial outwash gravels form remnant aggradation surfaces overlying the older rocks. Aggradation surfaces are extensive, nearly horizontal geomorphic surfaces built up by aggrading rivers as a result of excessive sediment supply, typically during periods of glacial advance. Wilson (1988) has studied the glacial stratigraphy of the Canterbury Plains, and Rains (1966) has studied the glacial stratigraphy of the upper Selwyn River catchment.

48. Many of these outwash gravels have been grouped together as “Hororata Formation” (Wilson 1988) which includes a wide range of ages but is thought to have been deposited at least several hundred thousand years before present. The Hororata Formation includes a covering of up to 15 m thickness of loess that has accumulated since deposition of the gravels. The highest outwash surfaces, which Wilson includes in the Hororata Formation, are approximately 80 m above current Waianiwaniwa River level.

49. A prominent terrace at about RL 270 (about 30 m above valley floor level) is interpreted to be an aggradation surface formed by gravels of the Woodlands Formation. This terrace is at a similar elevation to Woodlands surfaces recognised by Wilson (1988), but has previously been mapped as Hororata Formation. Extensive terraces southeast of Homebush Ridge are also inferred to be Woodlands Formation aggradation surfaces.

50. The Waianiwaniwa Valley contains sand and silt dominated alluvium encountered by cone penetrometer tests CPT1, CPT2 and CPT3. The valley floor grades toward a Burnham age aggradation surface of the Selwyn River

at about RL 235. Colluvium and landslide deposits mantle the valley sides and fill tributary gullies. No peat deposits were encountered during investigations at the proposed dam site.

Drainage Pattern Changes

51. A complex history of river aggradation and downcutting, has resulted from distributary glaciers of the Rakaia Valley directing meltwater into the Selwyn River catchment (Rains, 1966). Drainage pattern changes have probably resulted from aggradation, downcutting and possibly tectonic adjustment, and I infer that the Waianiwaniwa Valley may be an abandoned paleochannel of the Selwyn River. After abandoning the Waianiwaniwa Valley, and occupying the modern Selwyn River channel, aggradation during the last glacial advance appears to have blocked tributaries including the Waianiwaniwa Valley and the Wairiri Valley. This has resulted in accumulation of silt and sand dominated lake sediments or alluvium.

Dam site Geology

52. The proposed embankment alignment runs approximately east-west for most of its length and curves to the northeast near the left abutment. The total length of the proposed dam is about 2000 m with a crest elevation of about 283 m, giving a maximum height of about 50 m above existing ground level. The dam footprint geology is summarised in plan on Figure 5, and in cross section on Figures 6 and 7. In the following sections I have described the dam site in four distinct areas with similar geological conditions:

    (a) Left abutment;
    (b) Valley mouth;
    (c) 270 m terrace; and,
    (d) Right abutment.

Left Abutment

53. The left abutment is founded on the end of Homebush Ridge. The geological conditions underlying the left abutment are expected to comprise Tertiary-age sedimentary and volcanic rocks in a sequence dipping about 30° to the southeast (i.e. downstream). These rocks are expected to be overlain by a thin layer of colluvium including bouldery volcanic rocks and clay-rich volcanic ash-derived material. A series of terrace remnants is also evident on the left abutment, the most prominent of which is at approximately RL270, the elevation of the terrace that occupies most of the dam footprint. The

terraces are expected to be underlain by a small deposit of greywacke gravel with a loess cap up to about 10 m in thickness.

54. The proposed dam spillway will most likely be located on the left abutment of the dam, separated from the embankment to protect the embankment from erosion during very large flood events or failure of the spillway. Excavation for the spillway is expected to encounter basalt or volcanic sediments which are expected to be suitable as a foundation for the spillway. Hummocky ground and rounded scarp-like features on the south side of Homebush Ridge could indicate old landslides and it is possible that the proposed spillway location could be affected by this. Additional subsurface investigations will be required prior to detailed design to confirm the foundation conditions for the spillway. It is unlikely that ground conditions for this spillway location will prove impractical for spillway construction.

Waianiwaniwa Valley Mouth

55. The valley mouth is approximately 300 m wide and has a flat floor at about RL 235 m. This elevation corresponds to a Burnham-age aggradation terrace formed by the Selwyn River (Wilson, 1988). Test pits 5, 6 & 7, and drillhole WN3 were carried out in this area. The test pits investigated to a maximum depth of 6 m, and between 6 m and 70 m below ground level the geological materials have been described based on the results of WN3. Test pits 5 and 7 encountered interbedded gravels, sands and silts, containing mainly greywacke pebbles and cobbles with rare volcanic pebbles. Test pit 6 encountered greywacke gravels to a depth of 4 m.

56. Drillhole WN3 encountered greywacke-dominated gravels to a depth of 18 m. Gravel clasts were typically slightly weathered, brown or black stained on the outside. This sequence included a sandy silt layer at about 5.8 to 8 m depth (the drillhole log is presented in Appendix C).

57. Between 17 and 18 m depth, volcanic clasts were noted within the gravels. Between 18 and 27 m depth grey, very stiff, silt and very fine sand was encountered. This was laminated in part with typical lamination thickness of less than a few millimetres. Pocket penetrometer strength measurements were undertaken with most results falling in the range of very stiff to hard. Between 27 m and 38 m depth further gravels were encountered. These were typically coarse greywacke gravels with a slightly cohesive silty sand matrix, and the clasts were typically relatively weathered. At the base of the

gravel sequence an angular gravel layer of a few metres thickness directly overlies rock.

58. Tertiary sandstone was encountered from 38 m depth (~RL200m) and recovered to a maximum depth of 70 m. The Tertiary sandstone comprised silty fine grained slightly glauconitic sands, greenish grey in colour. This material could be described as a very weak rock or a very dense sand[4], typical of Tertiary marine sandstones in New Zealand. Bedding typically dips at about 15° but dip direction of bedding could not be measured as the core was not orientated. Joints were absent in the core, and the core was unweathered.

59. The statigraphic and depositional relationship between the different Quaternary units encountered during this investigation (such as the terrace gravels, valley infill and valley mouth infill) is unknown. I infer that the most likely geological scenario is that the lower part of the channel fill is relatively old (possibly Woodlands age), and that the upper channel fill is of relatively young Burnham age.

Terrace

60. A terrace, inferred to be a Woodlands Formation aggradational deposit, occupies the majority of the dam footprint with a relatively level surface at about RL 270 m elevation. The terrace forms a spur protruding from the west side of the valley. The terrace is between about 200 m and 400 m wide in a north-south direction. Malvern Hills Road follows a gully that crosses the terrace, falling to the northeast.

61. Test pits 1, 2, 3, 4, 8 and 9 and drillholes WN1 and WN2 are within this area of the dam footprint. The top surface of the terrace is covered by about 9 m of loess consisting of very stiff, light yellow, fine sandy silt. WN1 is located below the proposed dam crest line approximately 400 m from the right abutment. Surface elevation at WN1 is about RL264 m and gravel was encountered to about 30 m depth underlain by Tertiary sediments (i.e. rockhead is at about 233 m elevation). WN2 is located on the north side of the terrace near the dam centreline, with a ground surface elevation of about RL232 m. In WN2, silt and gravel-dominated colluvium and alluvium was encountered to a depth of 12 m, underlain by Tertiary sediments.

---

[4] Strength descriptions are in accordance with the guideline for the field classification and description of soil and rock for engineering purposes published by the New Zealand Geotechnical Society Inc. Dec 2005.

62. A gravel quarry at the eastern end of the terrace exposes horizontally bedded brown silty gravel in an exposure about 10 m high. Approximately 1 m of loess is exposed in the quarry, with the majority of the 9 m thickness apparently having been eroded off at that location.

Right Abutment

63. The right abutment of the proposed dam will be formed against a terrace with a surface elevation of about RL320 m. Test pit 10 was excavated in the upper terrace surface and encountered 3 m of weathered, very stiff to hard, yellow brown sandy silty loess overlying highly weathered brown greywacke gravels. Test pit 11 was excavated on a spring line at about elevation RL280 m and encountered gravelly colluvium and in situ weathered gravel to a depth of 2 m overlying very weak sandstone. Seepage was observed entering the pit at the base of the colluvium. The spring line is inferred to result from the permeability contrast between outwash gravels and underlying Tertiary sediments.

Trench TR1

64. Trench TR1 was excavated on a steep south-facing slope west of Malvern Hills Road to look for evidence of fault-related deformation within the near surface materials. The location of Trench TR1 is shown in Figure 5 and the log is presented in Appendix B. The location was chosen to avoid thick fan deposits which have formed at gully mouths along the terrace edge. Subhorizontal brown gravels were encountered in the trench underlying about 1.5 m thickness of loess and gravel colluvium. No evidence of faulting or tectonic deformation was identified in the trench.

Hydrogeology

65. Piezometers were installed in the three drillholes WN1-WN3 (inclusive), and water level and permeability information gathered from them is presented in Table 5. Groundwater was encountered in several of the test pits excavated in the valley floor. The Waianiwaniwa River water level at the time of the investigation was about 4 m below valley floor level. Water was encountered in test pits at shallower depths, particularly in the vicinity of TP1 and WN2 where standing water lies on the swampy paddocks.

Table 5 Depth to Groundwater and Mass Permeability Test Results

Borehole and Depth	Depth to Groundwater	Permeability range (m/s)	Material Description
WN1	>30 m	NA	Outwash Gravels
WN2	1.97 m	1×10^{-4} to 7×10^{-5}	Tertiary Sandstone
WN3	3.69	6×10^{-5} to 4×10^{-6}	Outwash Gravels

Note:
1. WN1 contained no standing water. Rising and falling head tests not undertaken
2. Depth to groundwater measured on 17 Sept 2002

66. The gravel-dominated units within the dam footprint are expected to have variable mass permeability. The gravels that form the RL270 m outwash deposit are generally intermediate age greywacke gravels that are expected to have a mass permeability in the range 10^{-4} to 10^{-5} ms^{-1}. The valley fill alluvium encountered in WN3 (between ground surface and about 17 m depth) within the valley mouth includes Burnham-age sandy gravels that are expected to be relatively permeable. Permeability tests were not carried out on these materials during this study but would be expected to be in the range of 10^{-3} to 10^{-4} ms^{-1}. The gravel encountered between 27 and 38 m depth is a relatively silty gravel with a relatively low permeability (see Table 5) as tested by rising and falling head tests.

67. The general permeability of the Tertiary sediments is expected to be in the range of 10^{-7} to 10^{-9} ms^{-1}, based on previous permeability testing carried out in similar materials at the proposed Wairiri dam site (URS 2001) and elsewhere (Dr Mark Yetton, Geotech Consulting Ltd, personal communication). Testing carried out in WN2 indicates a permeability of 1×10^{-4} ms^{-1}, which is much higher than expected in these materials. The higher permeability is likely to be due to a locally fractured zone or possibly a malfunctioning piezometer.

68. More extensive permeability testing will be carried out at design stage to establish if any areas of the dam foundation require treatment such as grouting. (Grouting means injection of cement and water into the ground to fill fractures and reduce the permeability of the ground.) Additional investigations are expected to include packer testing[5], rising and falling head tests, and installation of piezometers and temporary pump wells for pump testing, to more accurately assess permeability of the embankment foundation.

---

[5] Packer testing is a method of assessing the permeability of rock strata by measuring the rate at which water is pumped into an interval of a borehole at a known pressure. The test is sometimes called a ‘Lugeon test’.

Suitability of the proposed dam site

69. Site investigations undertaken to date provide sufficient information about subsurface conditions at the dam site to confirm that it is a suitable site to construct an embankment dam. Additional site investigations will be required to confirm the distribution of materials at the dam site and to further characterise the geotechnical properties of the materials.

Reservoir margin stability

70. Existing slope instability has been observed along the reservoir margins. This includes mainly shallow slides and debris flows on the steep western slope of Homebush Ridge, slumping on dip slopes within the coal measures, and shallow failures of loess colluvium around the edges of the outwash terraces. To significantly affect the dam, a landslide would need to be large enough, and rapid enough, to generate a wave that would exceed the dam freeboard. None of the landslides evident around the reservoir are capable of doing this.

71. Saturation of colluvium and existing landslide debris following lake filling will probably cause increased small-scale landslide activity around the reservoir, particularly due to the fluctuation in water level caused by water demand. Increased landslide activity outside the reservoir is not expected, due to the long flow paths for groundwater leakage from the reservoir and the low permeability of the Tertiary sediments. A specific memo about reservoir slope stability is presented in Appendix D.

Risk of leakage from reservoir

72. Coal mines were operated in the Malvern Hills throughout the late 1800’s and early 1900’s. The majority of mines were located in Bush Gully and in Surveyors Gully, though other mines were located where coal seams were found to outcrop. Figure 2 shows the locations of coal mines throughout the Waianiwaniwa Valley.

73. The underground mining was extensive in places, particularly in the case of the Klondyke mine, which followed seams up to “1500 feet down dip” (approximately 300 m below ground level), and up to about 600 m along strike (personal communication, Ken Shearer, Canterbury Coal Ltd). Klondyke Mine was unusually extensive; many mines were discontinued because they were overwhelmed by groundwater, or because the coal seams were faulted out. In some circumstances the coal seam was found again and mining continued. The faults responsible for displacing the coal do

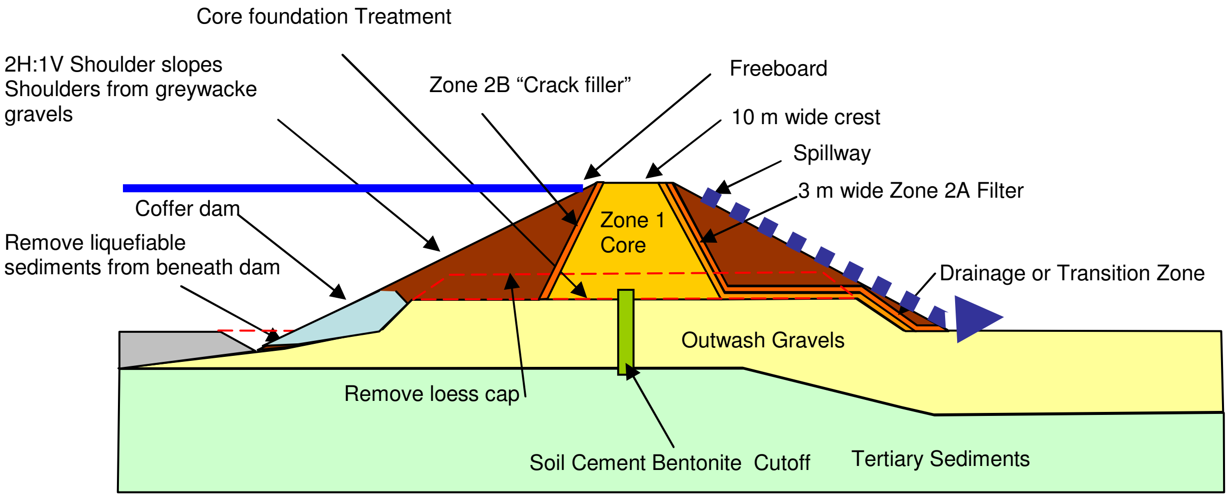
not have large throws as the coal seams generally follow a line of strike that is not radically offset.

74. The proposed dam is outside the area of coal measures outcrop. It is therefore extremely unlikely that underground mining has been carried out in the vicinity of the dam footprint. Coal mining has been undertaken within the reservoir footprint, but there is a low likelihood of underground coal mines forming a continuous conduit out of the reservoir. If evidence is found indicating that mining breached the reservoir integrity, engineering works may need to be designed to address this. Such measures would probably involve blocking the mines with low permeability fill barriers. All other mines, if not already flooded will simply fill with water when the lake fills. Collapse of old workings propagating to the surface is possible, but the likelihood of property damage resulting from this risk is very small. To mitigate the likelihood of damage from collapsing underground workings a detailed survey of structures near to the reservoir edge, and underlain by coal measures, should be undertaken so that appropriate measures can be implemented if required; such as strengthening or relocation.

Conceptual Embankment Design

75. The conceptual design of the embankment dam is described in URS (2006). The conceptual dam design will be described in the evidence of Mr Davidson. In summary, the proposed dam cross-section incorporates a zoned earthfill embankment comprising locally borrowed gravel dominated materials. The core material comprises compacted weathered gravels of Hororata Formation with slopes of 0.5 horizontal to 1 vertical. A 3 m wide sand filter drain is required against the downstream face of the dam core to provide filter protection. Shoulder material could comprise younger greywacke-derived gravels with an outer construction slope of 2 horizontal to 1 vertical. A typical cross section of the conceptual embankment design is given in Figure 8.

Figure -8 Conceptual embankment design



76. The conceptual design requires that the loess cap is removed from the 270 m terrace (a depth of about 9 to 10 m). Elsewhere the design adopts a foundation level three metres below existing ground level. A cut-off system is included to prevent excessive flow through the gravels under the embankment. This cutoff comprises a 5 m deep cutoff trench with a ten metre deep slurry wall constructed in the base of the core trench, and extending to the Tertiary bedrock. At the valley mouth a deeper cutoff may be required if the lower gravels prove to be too permeable.

77. The reservoir volume has been calculated from the DTM and indicates that 280Mm^3 of storage will require a maximum water level of RL280 m.

Foundation Suitability

78. Most of the embankment foundation footprint rests on the 270 m terrace. The layer of loess that caps the terrace will need to be removed prior to embankment construction. Loess can suffer from collapse when saturated, and can undergo piping failure under hydraulic gradients, which could cause stability problems for the embankment. Additional investigations into the material properties and distribution of the loess will be needed during detailed design to assess its suitability for use in the embankment.

79. Springs on the right abutment, at about RL280, will need to be controlled during construction. The springs indicate that there may be relatively

permeable zones within the gravels that cap the abutment. Some form of seepage control will be required in this area to prevent leakage around the dam abutment. This will likely be a cutoff trench or upstream low permeability blanket.

80. The gravels form a suitable foundation for supporting the embankment. The permeability of the gravels is relatively high, and a cutoff beneath the embankment, or a low permeability blanket, will probably be required to limit leakage to an acceptable level. Where the embankment footprint sits on young valley fill sediments, further investigation will be required to ensure that weak materials are not present, as this could cause differential settlement. Removal of these materials may be required. The embankment will be positioned so that a minimum amount of the dam footprint overlies the valley fill materials.

81. Small terrace remnants evident on the left abutment are interpreted to be outwash gravels. No subsurface investigations have yet been carried out on these terrace remnants, and local gravel deposits may need to be removed or treated. Investigations to map the extent of such deposits should be carried out in this area for detailed design.

Embankment Construction Materials

82. At the current stage of the project, construction materials have been identified based on experience and knowledge of the geological setting of the dam. Full feasibility and design stages of the project will include detailed site and laboratory investigations to confirm the engineering properties of the various materials. This information will be used to confirm the most efficient embankment cross section and appropriate borrow sites. The identified materials are discussed below and summarised in a specific memo on construction materials presented in Appendix E.

83. Table 1 in Appendix E summarises which geological units could be used to source the materials required for construction. Figure 2 of Appendix E shows the locations considered most likely to form practical borrow areas referred to in Table 1.

84. As part of URS's geological studies for the project a number of geological units have been identified that could be developed as borrow areas for the materials required to construct a zoned embankment. With the options available I am confident that sufficient material can be obtained to build the dam as proposed in the Dam Safety Assurance Report (URS 2006). Detailed

studies of the distribution and engineering properties of these materials will be required to choose the optimum combination of sources for dam construction. These studies will include extensive subsurface investigations, sampling, laboratory testing and internal stability and compatibility analysis. Such studies are normally carried out in a staged manner throughout the feasibility, design and construction phases of a project.

Zone 1 core

85. Well-graded silty sandy gravel or silty sand could be used to form the Zone 1 core material of the embankment. Well graded silty gravels are a relatively easy material to construct a low permeability fill from. Old greywacke gravels (locally referred to as Hororata Formation) occur extensively around the reservoir. In outcrop these gravels are judged to be sufficiently weathered to form a strong low-permeability fill when compacted. Greywacke gravels have previously been used as core material on large embankment dams in New Zealand (for example, Benmore and Ruataniwha). The viability of this option will depend on finding material that is sufficiently and consistently weathered with sufficiently high fines content.

86. An alternative material that could be used as Zone 1 core are the Tertiary age sediments that outcrop around the reservoir. These are mainly silty fine sand. Similar material forms the embankment core of Aviemore dam. Detailed investigations will be required to identify the preferred borrow site based on grading and engineering properties of the material.

Zone 2A and Zone 2B filter and transition

87. Embankment filter zones typically comprise sand or gravelly sand with few fines. It is usually difficult to meet the grading requirements with naturally occurring material. As a result the materials for these zones are most commonly manufactured by crushing, screening and washing. Such an operation usually requires a robust parent material. Greywacke gravels are considered suitable if they are unweathered.

88. Potential sources for the Zone 2A and 2B include the sandy alluvium deposited within the reservoir by the Waianiwaniwa River. These deposits are very likely to require washing to remove fines and could prove unsuitable if excessive fines are present, or if the sand is too fine.

89. Unweathered greywacke gravels, as a parent material for processing, can be sourced from outside the reservoir including the terrace immediately

downstream from the dam, river beds or existing commercially operated gravel quarries.

90. It is also possible that relatively unweathered parts of the Hororata Formation could be suitable for processing.

Zone 3A shoulder

91. Embankment shoulder material usually comprises a strong, permeable material. Unweathered greywacke gravels will form a suitable Zone 3A material. Potential borrow sites for suitable greywacke gravels include the terraces immediately downstream from the dam, or extraction from modern river beds. Tunnel spoil is also expected to be suitable as Zone 3A material provided the fines content of the spoil is not too high.

92. An alternative shoulder material borrow could be quarried from greywacke rock exposed in the upper reservoir.

Zone 4 wave protection

93. Wave protection on the upstream shoulder will require a coarse rockfill material that is sufficiently dense and robust to prevent breakdown by wave action. Suitable sources for this material could include the basalt and volcanigenic sediments exposed on Homebush Ridge and the Harper Hills. Part of this material could be sourced from the spillway excavation which is expected to encounter these materials. Outcrops suggest that material of suitable size is present at these locations but quantities have yet to be proven.

94. An alternative source for Zone 4 is greywacke from a new quarry in the upper reservoir area. Greywacke is known to outcrop in the upper reservoir, but the resource has not been evaluated as a Zone 4 source. Careful site selection is required to get suitable sized material from greywacke.

Geological conditions and geotechnical issues affecting the Headrace Canal

95. The anticipated geological conditions along the headrace canal are:

    (a) Almost the entire headrace canal is expected to be constructed within, or on top of, Quaternary age greywacke gravels of various ages. The gravels will typically have an overlying loess layer of variable thickness.

    (b) The canal alignment appears to avoid the landslide complex on the southern slopes of the Harper Hills. A preliminary walkover of

Homebush Ridge suggests that the alignment will not be affected by deep-seated landslides.

(c) The canal will sidle around terrace risers of the Rakaia and Waimakariri Rivers and in these areas it will encounter thick colluvium.

(d) Basement rocks may be encountered in canal excavations in the vicinity of the Waimakariri River intakes.

(e) Tertiary rocks may be encountered at the Rakaia River intake site but are not expected to be encountered elsewhere.

96. Geotechnical issues that will need to be assessed as part of the canal design include:

(a) Suitability of the gravels for embankment construction

(b) Seismic stability of the embankments, i.e. what slope angles are required for adequate stability?

(c) Requirement for lining

(d) Internal stability and filter compatibility of the materials

(e) Extensive site investigations will be required to characterise materials and confirm distributions

97. Based on a preliminary assessment, the geological conditions along the headrace canal are suitable for construction of the proposed canal. Site investigations along the canal corridor will be required as part of the design process to characterise the construction materials. This information will be required to design cut and fill slopes and lining requirements.

Geological conditions and geotechnical issues affecting the Intake Tunnel

98. There is very little geological data available to predict the geological conditions that will be encountered by the intake tunnel. The following summarises the anticipated geological conditions, which are also summarised in Figure 8:

(a) Quaternary-age greywacke gravels are exposed at surface along most of the tunnel alignment. Groundwater wells indicate a minimum of 30 m thickness of gravel in the vicinity of the tunnel.

(b) The Quaternary gravels are expected to mainly comprise greywacke gravels up to 100 mm diameter, with occasional deposits of boulders up to about 300 mm, in a silty or sandy matrix. Weathering is expected to be variable with older gravels moderately or highly weathered and post-glacial gravels unweathered.

(c) The gravels could be underlain by basement Torlesse greywacke, or locally could be underlain by Tertiary sediments.

(d) Local basement highs exist in the vicinity and considerable relief is anticipated on the contact between the base of the gravels and the underlying Tertiary or greywacke.

(e) Based on the regional geological data approximately 75% of the tunnel will be within gravel formations and 25% will be in greywacke, assuming the tunnel is at 280 m to 300 m elevation.

(f) Groundwater levels are close to the proposed tunnel elevation.

99. The proposed tunnel is very likely to be constructed using a tunnel boring machine. The range of possible geological conditions that the intake tunnel could encounter have been discussed with a tunnel boring machine manufacturer (Herrenknecht) and I believe the conditions are suitable for construction of the proposed tunnel.

100. Geotechnical issues that will need to be addressed as part of the tunnel design include:

(a) Material properties and distribution along the alignment

(b) Choice of vertical alignment

(c) Geotechnical investigations for tunnel boring machine performance prediction

(d) Groundwater levels

(e) Ground conditions at portal sites

(f) Design criteria for the tunnel boring machine

Conclusions

101. Based on the results of the investigations to date, I believe it is possible to construct a safe and functional dam in the Waianiwaniwa Valley with a storage volume of 280M m^3. Maximum reservoir level to accommodate 280M m^3 will be RL280 m. I consider the proposed site at the mouth of the valley to

have suitable foundation conditions for construction of an earthfill embankment dam.

102. The proposed reservoir and dam site are underlain by a sequence of Tertiary age sedimentary and volcanic rocks overlain by Quaternary age alluvium. Basement Torlesse greywacke outcrops in the upper reservoir.

103. Gravels underlying the dam footprint are expected to be relatively permeable and will require specific design features to maintain foundation leakage at an acceptable level.

104. Several active faults that pass within 30 km of the dam site contribute to the seismic hazard at the dam site. The Hororata Fault passes about 2.5 km from the dam site and constitutes the controlling maximum earthquake for dam design. Peak ground acceleration at the dam site during a maximum earthquake on the Hororata Fault is expected to be in the range 0.65 to 1.0 g. This event will probably constitute the maximum design earthquake (MDE) for dam design. Based on a preliminary probabilistic seismic hazard assessment, the operating basis earthquake (OBE) will have a peak ground acceleration of about 0.28 g, which equates to a 150 year return period.

105. A fault crossing the upper reservoir about 5 km west of the dam site (the Western Gully Fault) is also inferred to be active, but will not generate ground motions at the dam site exceeding those generated by the Hororata Fault.

106. While no active fault has been found within the dam footprint, there remains the possibility that an active or secondary fault is found during future site investigations. This would require specific design details to be incorporated to address possible fault rupture hazard.

107. Construction materials are expected to include local gravels and processed materials derived from local gravels. Core material is expected to comprise weathered greywacke gravels but Tertiary sediments could be utilised. All of the necessary materials are available within a few kilometres of the dam site. Investigations to characterise potential construction materials will be required during the next phase of the project.

108. Other issues that need to be assessed as part of the dam design are spillway capacity and design, outlet works design, downstream hazard classification, and a review of possible alternative dam section configurations.

109. Additional investigations for detailed dam design are expected to include:

(a) drilling and in-situ testing, test pitting and geophysical investigations of the dam footprint

(b) drilling, test pitting, sampling and laboratory testing of potential borrow areas

(c) detailed site specific seismic hazard assessment

Timothy McMorran

References

Gregg, D. R. (1964): *Geological Map of New Zealand 1:250 000 scale (Hurunui Sheet)* New Zealand Department of Scientific and Industrial Research.

GNS Science (in prep): Draft version of QMap geological map of the Central Canterbury Plains area (provided to URS in June 2007).

Howard, M. E, Nicol, A. , Campbell, J. K., & Pettinga, J. R., (2002): Prehistoric earthquakes on the strike-slip Porters Pass Fault, Canterbury, New Zealand. *New Zealand Journal of Geology and Geophysics, 2002.*

Kerr, J., Nathan, S., Van Dissen, R., Webb, P. Brunsdon, D. and King, A. (2003): *A guideline to assist resource management planners in New Zealand.* Institute of Geological & Nuclear Sciences client report 2002/124

Mejia, L., Gillon, M., Walker, J. and Newson, T., 2001: Criteria for developing seismic loads for safety evaluation of dams of two New Zealand owners. NZSOLD/ANCOLD 2001 Conference on dams.

McVerry, G., Zhao, J. Abrahamson, N and Somerville, P., 2006: New Zealand acceleration response spectrum attenuation relations for crustal and subduction zone earthquakes. *Bulletin of the New Zealand Society for Earthquake Engineering 39, 1*

New Zealand Society on Large Dams (2000): *New Zealand Dam Safety Guidelines* November 2000

Pettinga, J. R., Chamberlain, C. G., Yetton, M. D., Van Dissen, R. J. and Downes, G., 1998: *Canterbury Earthquake Hazard and Risk Assessment Study, Stage 1(a) – Earthquake Source Identification and Characterisation.* Canterbury Regional Council Publication No. U98/10.

Rains, R. B., 1966: The Late Pleistocene Glacial Sequence of the High Peak Valley, Canterbury. *NZ Journal of Geology and Geophysics, Vol 10, No. 4.*

Speight, R. (1928): *The Geology of the Malvern Hills.* Department of Scientific and Industrial Research Memoir No 1.

Stirling, M. W., McVerry, G. H., Berryman, K. R., 2002: A new seismic hazard model for New Zealand. *Bulletin of the Seismological Society of America 92.*

Stirling M., Gerstenberger M., Litchfield, N. McVerry, G., Smith, W., Pettinga, J. and Barnes, P., 2007: *Updated probabilistic seismic hazard assessment for the Canterbury Region.* GNS Science Consultancy Report 2007/232 to Environment Canterbury.

URS New Zealand Limited (2001): *Geotechnical Investigations for the Proposed Central Plains Water Enhancement Scheme.* Technical Memorandum (48685-002\1200\R278a) to CPW 17 September 2001.

URS New Zealand Limited (2002a): *Preliminary Feasibility Assessment of the Waianiwaniwa Storage Dam*. Report (48685-002\2000\R940b) to CPW 14 May 2002.

URS New Zealand Limited (2002b): *Geotechnical investigation for the proposed Waianiwaniwa water storage dam.* Report (48685-002\R1009C) to CPW 11 October 2002.

URS New Zealand Limited (2006) *Central Plains Water Enhancement Scheme Dam Safety Assurance Report.* Unpublished report to CPW, 30 March 2006.

Wilson, D. D. 1989: *Quaternary geology of the northwestern Canterbury Plains 1:100 000.* New Zealand Geological Survey miscellaneous series map 14. DSIR