Defensive CFRD Embankment Design using Dirty Rockfill in a High Seismicity Environment – Waimea Dam

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Design and construction of CFRDs has evolved for over 100 years. Considered an 'inherently stable' dam type for decades, based on the principle of rockfill's high drainage capacity and strength, CFRDs with poor rockfill challenge this presumption. This paper will present the design challenges posed from dirty, low strength rockfill for a CFRD in a high seismicity environment. Then it illustrates the use of defensive measures to augment CFRD performance to meet current societal safety expectations. For those CFRDs constructed of poor rockfill or in high seismicity environments, the reader will take away an appreciation of vulnerabilities and defensive measures to improve performance.

Keywords: *defensive design, resilience, robustness, reliability, rockfill, dam, CFRD, seismic*

Introduction

Waimea Dam is a 53m high, Concrete Faced Rockfill Dam (CFRD) under construction for the primary purpose of providing irrigation and community water supply. The dam is located in the northwest part of New Zealand's South Island. This region lies in the tectonic transition between the Hikurangi Subduction zone to the north and the Alpine Fault to the south. It is a high seismicity region. Several regional faults lie within 12 km from the site in different directions.

Site Geology

Geologically, the Waimea Dam basin is dominated by metamorphosed sandstone, siltstone and mudstone. The rock mass consists of a sequence of slightly weathered, moderately strong to strong, light grey to grey, jointed fine sandstone (greywacke) and finely laminated dark grey to black siltstone and mudstone (argillite). Argillite comprises the majority of the rock mass beds and is commonly fissile. The tectonic history of the rock formation has resulted in a complex occurrence of both macroscopic and microscopic scale defects. These defects include at least four prominent joint sets; bedding ranging between 10 mm – 1 m and generally spaced at ~ 100 mm; crushed and shear zones varying between 20 mm to ~2 m in thickness characterised as shattered rock containing clay filled seams. The argillite typically splits along micro fractures parallel to bedding. Bedding and joints are generally closely spaced and tight.

Damsite Materials for Embankment Fill

Dam development plans included using site-won alluvium for filter and transition zones, and excavated rock for bulk rockfill.

Alluvial riverbed and terrace deposits consisted of broadly graded gravel with sand and cobbles, with variable silt fractions. The silt fraction of alluvial deposits increased along terraces further from the river. A fines limit of 8% for the filter and transition zone materials limited the quantity of exploitable alluvium. Design and a typical site-won alluvium grading are presented in Figure 1 (additional gradings shown are discussed later in the paper).

Site-won alluvium was processed to produce filter and transition and drain materials. Oversized cobbles and boulders were crushed and screened on the 38 mm size sieve to segregate the filter / transition from the drain materials. The minus 38 mm material was intended for concrete face support (Zone 2B) and foundation filter materials. The 38 mm to 300 mm sized particles were intended for drainage zones.

The service spillway was cut through up to about 30 m of rock of variable weathering and strength. Up to several metres of rock was cut for the diversion culvert. Most extracted rock was moderately strong (Read & Richards, 2008). The rockmass was closely jointed and bedded, producing a fine rockfill once excavated. The rockmass consisted of complexly interbedded, predominant argillite and minor fine sandstone. With minor exception, the sandstone could not effectively be segregated from the argillite. The argillite was fissile resulting in micro-fracturing.

The rockfill specification required highly weathered, weak rock to be excluded from the rockfill zones. The bulk of rockfill material consisted of moderately weathered and moderately strong argillaceous rock (Read & Richards, 2008) that produced a fine, dirty rockfill. Two photos in Figure 2 from a trial embankment of unprocessed rockfill show the compacted surface and particle matrix in a trench cut through the upper 1.5 m of the trial embankment. Close visual inspection of the rockfill's matrix beneath the compacted surface indicates significant particle breakage, and negligible open voids. Water was observed to pond on the compacted surface.

A grading of the post-compacted, unprocessed rockfill trial embankment materials is shown Figure 1. The compacted, unprocessed rockfill is broadly graded with a minus 19 mm fraction typically 25 to 35% taken from construction QC gradings. A rockfill grading specification reflecting current Australasian practice indicates limiting the minus 19 mm fraction to less than 20 or 30% (Fell, MacGregor, Stapledon, Bell, & Foster, 2015). The observations, grading and empirical evidence indicated the compacted fine, dirty rockfill had a low porosity and could not confidently be considered free draining. Thus, dynamic numerical analyses were performed to understand the embankment seismic deformations and required drainage to provide adequate post-earthquake stability.

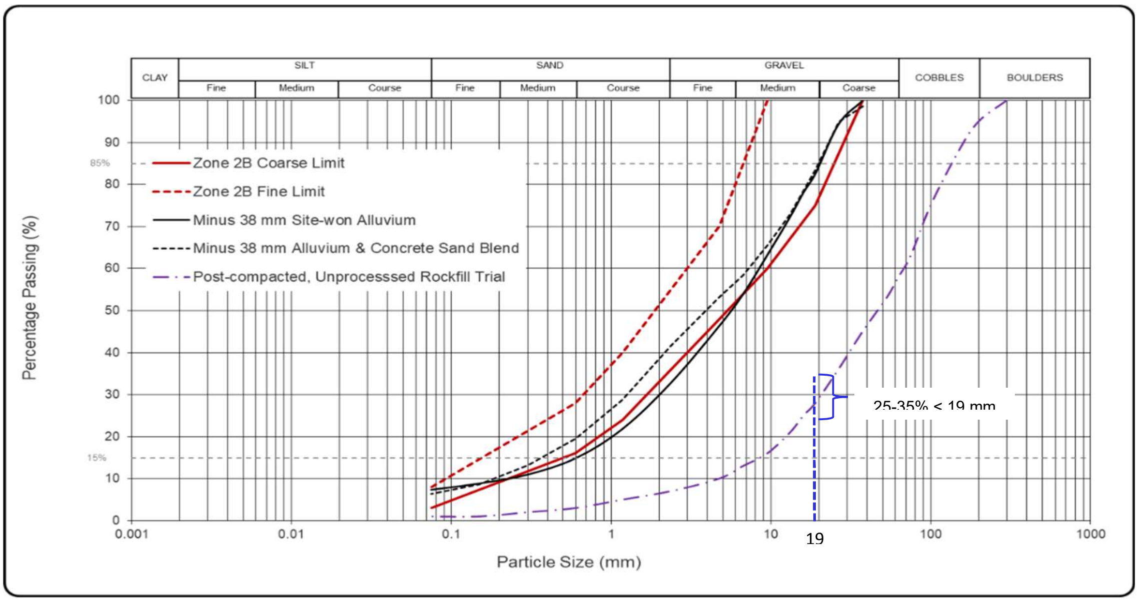


Figure 1. Concrete face support material (Zone 2B) & Fine, Dirty Rockfill Gradings

(a) Compacted surface

(b) Vertical view of material conditions in trench cut

Figure 2. Embankment Trial Conditions using Unprocessed Rockfill

Seismicity & Predicted Seismic Deformations

Waimea Dam has a "High" Potential Impact Classification (PIC) rating. As such, the New Zealand Dam Safety Guidelines (NZSOLD, 2015) recommend the following design ground motions:

- Operating Basis Earthquake, OBE: Typically represented by the probabilistic ground motions at 150 year (yr) average return period (ARP).
- Safety Evaluation Earthquake, SEE: Represented by the 84th percentile level of the Controlling Maximum Earthquake (CME) if developed by a deterministic approach, and need not exceed the probabilistic ground motions at the mean 10,000yr ARP.
- Aftershock: Consider at least one aftershock event at one magnitude less than the CME within one day of the SEE.

GNS (GNS, 2017) developed the design ground motions for Waimea Dam. The ground motions were developed based on the foundation bedrock having an average shear wave velocity over 30m (V_{s30}) of 800m/s. Table 1 lists the Peak Ground Accelerations (PGAs) corresponding to the OBE, SEE and aftershock. The corresponding horizontal response spectra are presented below in Figure 3.

Table 1. Design seismic ground motions - Waimea Dam

Design Ground Motion	PGA (g)	Earthquake Magnitude, M_w	Earthquake Scenario
OBE	0.15		150 ARP
SEE	0.64	7.1	10,000 ARP
Aftershock SEE	0.44	6.8	84th percentile of Alpine Kaniere-Tophouse and Waimea South fault aftershock; Representative of SEE aftershock where mainshock M_w 7.8 response spectra is similar to 10,000 ARP
	0.58	6.5	84th percentile CME (Waimea Central and Southern fault) aftershock

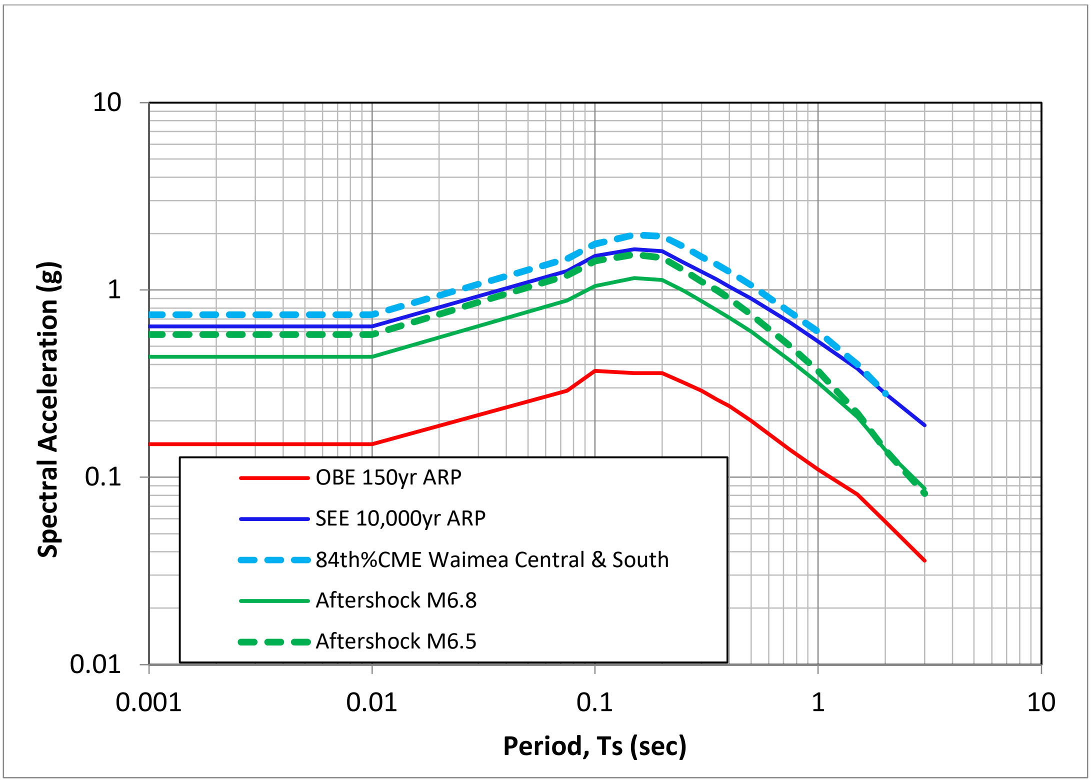


Figure 3. Seismic horizontal spectral accelerations - Waimea Dam

Dynamic numerical analysis was performed to estimate seismic deformations for embankment design. A two-dimensional model of the maximum embankment section was developed for numerical analysis. Because planned operation of the Waimea Reservoir cycled between the normal maximum operating level (Nmax) and normal minimum operating level (Nmin) annually, both conditions were modelled and analysed. Analyses were performed for three sets of time histories for the 10,000yr ARP and three sets for the aftershock.

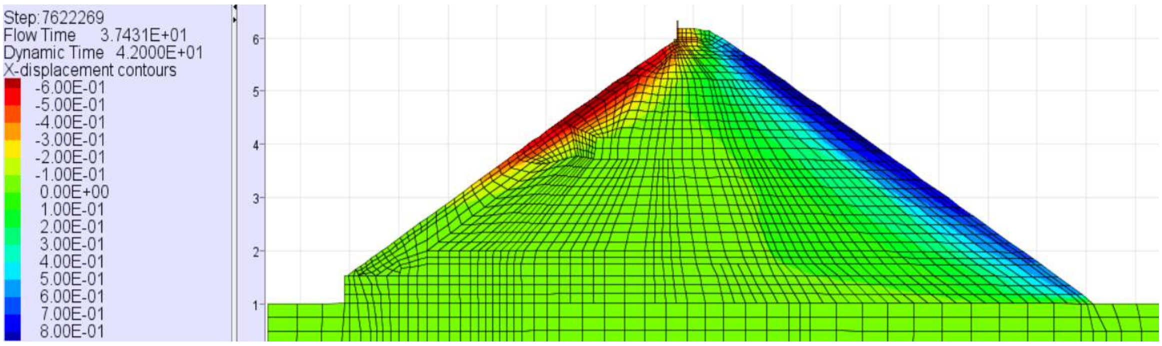
Table 2 summarises the dynamic numerical analysis results. The highest permanent displacements from the three sets of time histories were selected for design and are shown in Table 2. Crest centreline settlement, crest centreline horizontal displacement, downstream side of crest displacement, and upstream and downstream slope displacements for SEE and SEE + aftershock ground motions are provided in Table 2.

Table 2. Predicted SEE deformations from numerical analysis for embankment design

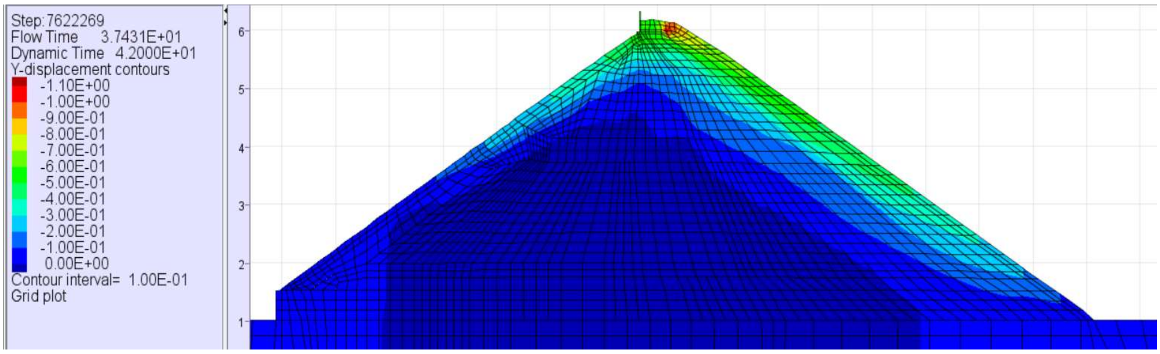
Reservoir Level / Direction	***Mainshock***	**Mainshock plus Aftershock**	**Normalised Crest Settlement (%)**
Crest centreline settlement (mm)			
Nmax	290	370	0.72
Nmin	630	790	1.49
Crest centreline horizontal displacement (mm)			
Nmax	300	430	n/a
Nmin	480	620	n/a
Maximum deformations along downstream slope at Nmax (mm)			
Horizontal	900	1230	n/a
Vertical	750	1000	n/a
Maximum horizontal deformations along embankment slopes at Nmin (mm)			
Upstream slope	700	950	n/a
Downstream slope	800	1100	n/a
Maximum vertical deformations along embankment slopes at Nmin			
Upstream slope	360	480	n/a
Downstream slope	580	780	n/a
Maximum vertical deformations at crest downstream side (mm)			
Nmin	1100	1450	n/a

Understanding the distribution and patterns of predicted seismic deformations is essential for design to meet seismic performance expectations. Figure 4 displays contours of SEE induced displacements, horizontal and vertical separately in the two graphic insets, shown on a two-dimensional cross section of the embankment dam for Nmin conditions. A similar two-dimensional embankment dam cross section plot of SEE displacement vectors is presented in Figure 5. Each vector represents the summed horizontal and vertical displacements at each node in the mesh. A concentration of vectors, visible as high saturation of red colour in Figure 5, generally indicates the concentrations and patterns of displacements. The maximum vector length is approximately 1.1 m in Figure 5.

There is a significant difference in predicted seismic deformation patterns between the reservoir at normal maximum (Nmax) and minimum (Nmin) operating levels. At Nmax, the reservoir water provides a large degree of inertial restraint of the upstream slope during seismic ground shaking; i.e. the water restrains the upstream slope. At Nmax conditions, the crest settles and tends to rotate downstream with the downstream slope displacing downward towards the lower slope. Under Nmin conditions, both the upstream and downstream slope experience seismic displacements, which results in greater crest deformations and settlement compared to that exhibited along the slopes. The upstream slope displacements do not extend as far down the slope because of the reservoir water's restraint below Nmin.



(a) Horizontal displacements [scale in metres]



(b) Vertical displacements [scale in metres]

Figure 4. Contours of seismic displacements for Safety Evaluation Earthquake at normal minimum operating level

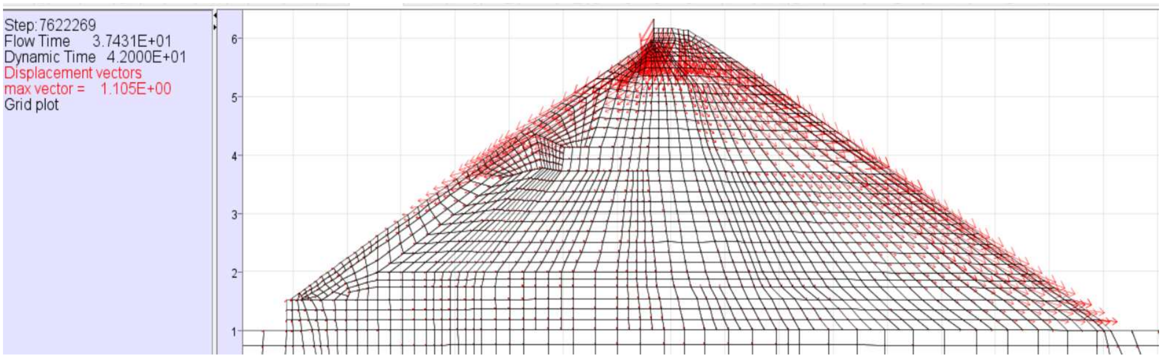


Figure 5. Displacement vectors for Safety Evaluation Earthquake at normal minimum operating level

Seismic Performance Challenge

Seismic deformations could severely damage the crest area and upstream slope. Such deformations may lead to joint opening and cracking of the concrete facing, especially in the uppermost portion of the dam (Wieland, 2009). Leakage through the concrete facing could saturate the poorly drained, fine, dirty rockfill. Partial saturation of the downstream slope does not meet stability design requirements. High level saturation could induce failure of the dam.

Two solutions were initially considered to remedy this seismic performance shortcoming: 1) flatten the downstream slope; or 2) incorporate adequate drainage into the embankment. Flattening the downstream slope was quickly discounted as damsite construction had already begun and the dam envelope was considered fixed, in part because diversion and permanent outlet works limited options for the downstream toe. Thus, the default remedy was to incorporate drainage sufficient to keep the bulk embankment fill unsaturated following the SEE to maintain adequate post-earthquake stability.

Defensive Design Approach

Consequences of Waimea Dam failing include significant population at risk downstream, a state highway bridge and significant adverse societal, environmental, and economic impacts. The dam has the highest New Zealand hazard rating (i.e. High) in accordance with New Zealand Dam Safety Guidelines (NZSOLD, 2015). Thus, a defensive design philosophy is warranted and was adopted. Applied to the Waimea Dam embankment, the defensive design principle resulted in assuming the concrete facing becomes ineffective as a seepage barrier following the SEE.

The three Rs (3Rs) of resilience, robustness and reliability guided defensive design. The 3Rs are outlined in a paper (Davidson, 2019) presented at the 2019 ANCOLD/NZSOLD Conference. This paper summarises the 3Rs as stated by the U.S. Army Corps of Engineers (USACE):

- Resilience: *The ability to avoid, minimise, withstand and recover from the effects of adversity, whether natural or manmade, under all circumstances.*
- Robustness: *The ability of a system to operate correctly across a wide range of operational conditions, with minimal damage, alteration or loss of functionality, and to fail gracefully outside that range.*
- Reliability: *The duplication of critical components of a system with the intention of increasing reliability of the system, usually in the case of a backup or failsafe.*

Considering the 3Rs resulted in two major embankment design features to satisfy post-earthquake performance: 1) the Zone 2B material supporting the upstream concrete face must limit leakage; and 2) internal drainage must safely discharge leakage from the dam while keeping the bulk embankment largely unsaturated. The general arrangement of these features is illustrated by the maximum embankment cross section in Figure 6.

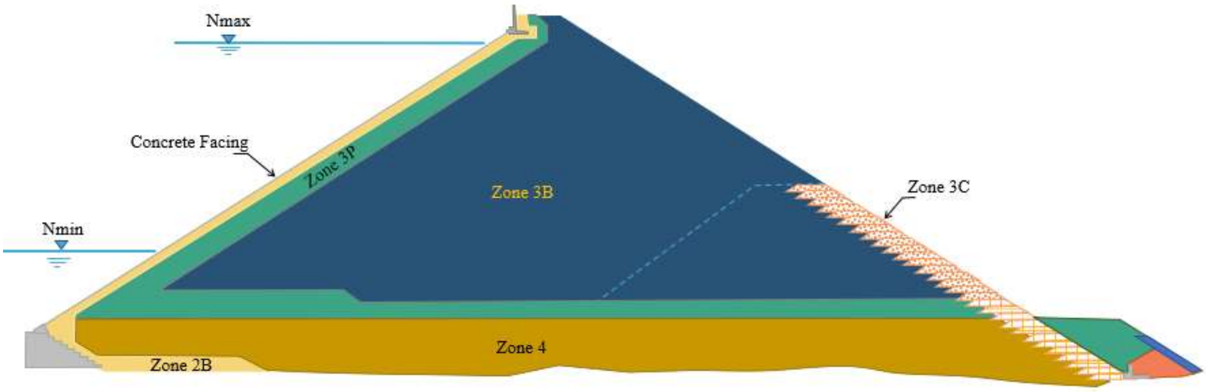


Figure 6. Maximum cross section - Waimea Dam

Zone 2B Concrete Face Support Material

The concrete face support zone is designated Zone 2B at Waimea Dam. In addition to the stiffness required to support the concrete facing, post-earthquake performance dictated it survive the SEE intact to serve as a crack-stopping, flow limiter. Zone 2B design requirements are summarised as:

- Be stiff enough to adequately support the concrete facing.
- In the event of a crack or rupture of the upstream concrete facing, collapse in to limit the flow through the opening.
- Have a permeability low enough to limit post-earthquake through-leakage in the event of major damage to the concrete facing.
- Be fine enough to be a no-erosion filter where Zone 2B lies directly upon foundation rock defects.
- Be internally stable; i.e. non-suffusive, to retain its stiffness, low permeability and filtering characteristics.
- Be coarse enough to be compatible to the adjacent drainage materials.

The first decision faced to design Zone 2B was “one material and zone, or two separate materials and subzones?” Zone 2B is not a filter in its position directly beneath the upstream concrete face, although it must perform several similar functions such as being collapsible for crack stopping, and internally stable, and having adequately low permeability. Additional constraints consisted of maintaining the current design’s geometrical envelope and, of course, cost. Large, predicted crest and slope seismic deformations and damage indicated the narrow widths of two stages for Zone B resulted in one or both stages potentially being truncated. This would result in not meeting the post-earthquake design criteria above. Constructing two narrow stages would also add considerable cost and schedule to construction.

It was decided to trial Zone 2B as a single stage. In conjunction with internal drainage zones, drainage design dictated a target design permeability for Zone 2B of 1 x 10^{-5} m/s.

The base material available for Zone 2B was the site-won processed alluvium. A typical grading is shown on Figure 1. Some of this material had low plasticity. Its typical grading is sand deficient and empirically determined to be potentially susceptible to internal instability. Given post-earthquake hydraulic gradients as high as 20 across Zone 2B, it being internally stable was deemed essential. A large permeameter constant head permeability test of the site-won, minus 38 mm processed alluvium yielded a hydraulic conductivity of 2.3 x 10^{-4} m/s. It was concluded the site-won, processed alluvium would not meet the post-earthquake performance requirements of Zone 2B.

It was believed blending the site-won processed alluvium with clean sand could potentially improve its permeability, collapsibility, and internal stability. After geologic search in the damsite area for exploitable sand sources was fruitless, blending the site-won, processed alluvium with commercially available sand was pursued. Several products were theoretically blended by combining their particle size distributions. Washed concrete sand proved most promising. By adding a minor fraction of concrete sand to the site-won, processed minus 38 mm alluvium, the grading became internally stable and its permeability would likely be lowered.

Next a field trial blending the two materials and constructing an embankment was performed. All efforts were made to alleviate further generation of fines to limit the fines content.. Blending was done using a front-end-loader with a built-in scale and an experienced operator. A sample of the blended stockpile was sent to the lab for grading, lab density and permeability testing. The typical site-won, processed alluvium and concrete sand blend grading is shown in Figure 1. The first permeability test of the theoretically blended Zone 2B material for the coarse limit grading yielded a hydraulic conductivity of 7.7 x 10^{-6} m/s. The embankment trial showed four overlapping passes with a vibratory steel drum compactor yielded adequate density without excessive fines generation.

It was concluded that a single material and zone for Zone 2B could meet all the design performance criteria, and this was carried through construction of the embankment. In the end, shortages of concrete sand and site-won alluvium with low enough fines content required finding materials from additional sources. Numerous theoretical blends using these alternative materials eventuated into field blends and placement in Zone 2B.

Controlling the moisture content of the materials to be blended and an experienced operator performing the blending were essential to success. A single operator did nearly all the blending of materials for Zone 2B. In all eight lab permeability tests were performed with results in the range between 4.4 x 10^{-7} m/s to 7.9 x 10^{-5} m/s. This was considered very good quality control of the blended material.

Zone 2B was designed to nominally be 2 m wide beneath the upstream concrete facing. A custom-made spreader box was built to place the material uniformly and safely. The spreader box included a guide rail that rode along the upstream concrete curbing to maintain alignment. Zone 2B was placed in nominal 400 mm lifts and compacted with four passes of a 4T dual steel drum, vibratory compactor.

Zone 2B wraps around the lower upstream face behind the plinth and extends along the cleaned and treated rock foundation as a filter. It extends for a distance of ½ times the reservoir water height up to the Inflow Design Flood (IDF). Refer to Figure 6.

Internal Drainage of the Embankment Dam

Post-earthquake conditions for internal drainage design determined the concrete facing could be an ineffective water barrier and seepage would be controlled by Zone 2B underlying the concrete facing. Post-earthquake internal drainage design took the reservoir level to be at normal maximum operating level (Nmax). For these conditions, the total post-earthquake leakage was calculated on a per metre width basis across the embankment and summed. The embankment's internal drainage system is required to discharge this summed total leakage without saturating the bulk of the embankment composed of relatively poorly drained, fine, dirty rockfill.

Given the seismic performance requirements, an upstream inclined drain adjoining Zone 2B contiguously connected to a blanket drain was selected as the internal drainage general arrangement. ICOLD CFRD guidance (ICOLD, 2010) includes placing an underdrain of coarse rock within the valley section to enhance drainage (Zone 4). Refer to Figure 6. Design of the inclined drain relies on efficiently using gravity to convey all upstream face leakage laterally and down to the blanket drain located in the original river channel at the base of the dam.

The key design criteria for the internal drainage zones were:

- The combined drainage system must convey upstream face leakage out the dam without significant pressures developing in the embankment above the top of the blanket drain.
- The as-placed drain material needs high permeability.
- The drain material interfacing with Zone 2B needs to be compatible with Zone 2B material.

The grading of the drainage material was designed to meet permeability and material compatibility requirements. The design target permeability was 10^{-1} m/s. To meet compatibility with Zone 2B, the nominal minimum particle size was determined to be 19 mm. To maintain high permeability, a strict limit to undersize particles, i.e. fraction < 19 mm, was set for stockpile gradings (6% ≤ 19mm) and as-placed gradings (12% ≤ 19 mm). Gradings showing the drainage material design envelope; i.e. stockpile gradings, and the as-placed fine limit are presented on Figure 7.

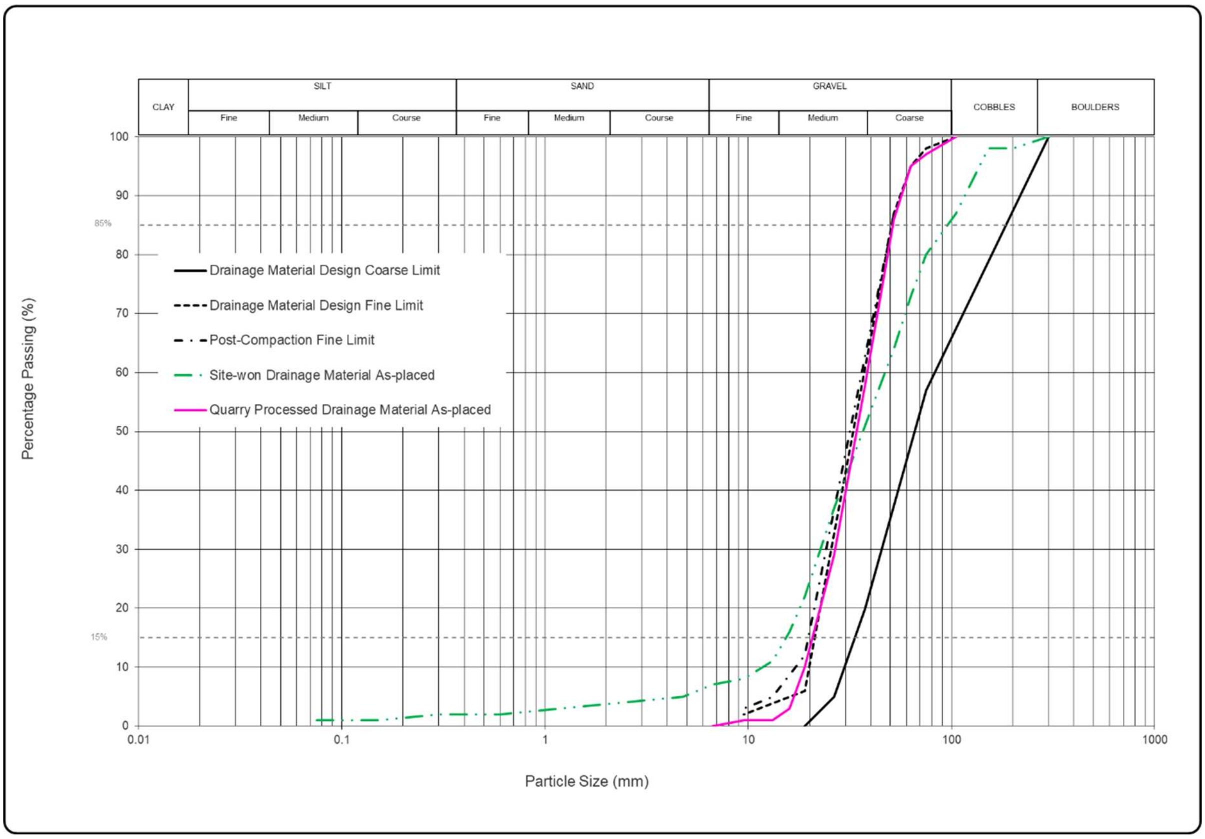


Figure 7. Drainage Material Gradings

A series of drainage material processing and embankment trials was conducted to assure as-placed drainage materials could meet the stringent grading requirements while being adequately compacted. First, processing and embankment trials of site-won rockfill for drainage material were performed. Site-won rockfill was successfully processed. A photograph showing site-won rockfill processed for drainage material being spread for trial in presented in Figure 8. However, repeated trials experienced excessive particle breakdown. A post-compaction grading from the site-won rockfill drainage material trials is presented in Figure 7. After several trials, it was concluded adequate drainage material could not be produced from site-won rockfill. Site-won, processed alluvium produced excellent drainage material, but its quantity was much less than needed.

The drainage materials volume was substantial; on the order of 144,000 as-placed cubic metres. The contractor's stockpile of site-won processed alluvium drainage material was soon exhausted. All additional drain material was obtained from the Contractor's quarry 8 km from the damsite. The quarry consists of hard, massive greywacke and limestone. Both rock types were trialled by crushing, screening and constructing an embankment. Both were deemed acceptable. Subsequently, all remaining drainage material was sourced from the hard greywacke at the contractor's quarry. An as-placed (post-compacted) grading of quarry processed drainage material is presented in Figure 7.

Figure 8. Site-won rockfill processed for drainage material being spread for an embankment trial

It proved difficult to consistently limit as-placed undersized particles. Minimising handling and compaction, and using a 20 mm screening bucket for loading, were done towards this end. Considerable drainage material was rejected that did not meet this requirement due to poor rock quality. On one occasion, an as-placed drainage material run with excessive undersized particles was remediated by removing and replacing a large part of the run in conjunction with re-design resulting in adding additional height to the upstream end of the blanket drain. This additional height of 1.6 m along the upstream part of the blanket drain is shown on Figure 6.

The internal drainage zones needed to be contiguous to collect and convey post-earthquake leakage out of the dam safely. With the position of the inclined drain in the upstream portion of the embankment, the blanket drain needed to extend from the downstream toe contiguously upstream to the filter behind the plinth; nearly the entire base width of the dam. This long seepage path length reduced the hydraulic gradient along the blanket drain. Seepage discharge is directly related to hydraulic gradient according to Darcy's Law. Thus, an adequate hydraulic gradient is required to convey the estimated leakage to the downstream dam toe. The two countermeasures to the long seepage path length were high permeability and increasing the blanket drain's height.

The design permeability of 10^{-1} m/s was set based on what could be consistently achieved in the field. The total post-earthquake leakage collected by the inclined drain will be conveyed to the blanket drain. It was assumed leakage spreads across the base width of the drain, which allows simplistic two-dimensional, analytical design of the blanket drain using Darcy's Law. Using this approach, the drain height was designed to be nominally 9 m thick. The contractor wanted the top surface level. This resulted in the downstream end being 1 m to 2 m thicker than the downstream end.

Conclusions

High seismic loads from the Safety Evaluation Earthquake (SEE) are predicted to induce large deformations and damage to the crest and slopes of the Waimea Dam embankment. Post-earthquake conditions predict the concrete facing would be an ineffective water barrier. The necessity of using site-won, friable, highly fractured rock as the bulk embankment fill presented an unacceptable risk that lack of drainage capacity in the rockfill would lead to potential instability and failure of this high hazard dam.

Potential post-earthquake instability necessitated defensive seismic design measures. As a remedy, internal drainage was incorporated into the embankment design. The internal drainage system consisted of three main components:

1. A flow-limiting, collapsible, internally stable upstream face support zone (Zone 2B).
2. An inclined drain adjoining downstream and compatible with Zone 2B.
3. A high permeability blanket drain at the base of the dam contiguous with the inclined drain.

The internal drainage system enhanced the seismic performance. In terms of the 3Rs:

- Resilience: The internal drainage zones can withstand predicted seismic deformations and damage, alleviating post-earthquake embankment pressures and maintaining embankment stability. This allows for recovery by the embankment surviving and giving opportunity to effect repairs.
- Robustness: The embankment dam can remain an effective water barrier under extreme seismic loading by incorporating the internal drainage system.
- Reliability: The primary water barrier, i.e. the concrete face, could be badly cracked during extreme seismic events. The upstream face support zone (Zone 2B) provides a secondary water barrier. Under post-earthquake conditions, Zone 2B acts as a flow limiter and the drainage zones maintain low embankment pressures while conveying the leakage out the dam.

Manufacture and placement of the internal drainage materials to stringent specifications to meet seismic performance requirements proved challenging. Adherence to original plans to use site-won materials to the extent possible was required. The site-won alluvium (of sandy gravel with silt) was processed and blended with concrete sand to create an efficiently blended material meeting performance requirements. Oversize site-won alluvium made excellent drainage material, but its quantity was limited. Site-won rock excavated for the spillway and diversion culvert produced a fine, dirty rockfill. But the site-won rock was moderately strong, with foliations / micro-fractures resulting in friability, and experienced excessive breakdown during compaction, precluding its use because of poor drainage performance. A significant quantity of additional drainage material was obtained from crushed, processed and imported rock from a nearby quarry. It was good fortune the nearest high quality rock quarry was owned by the Contractor. This facilitated supply on schedule in a severely supply-restrained environment.

Construction of the internal drainage zones required traditional, but strict, procedures to achieve as-placed specifications. Zone 2B was placed using a spreader box with the upstream edge guided along the concrete curbing. It was compacted in 400 mm lifts with strictly four passes of the vibrating roller. In the drainage zones, construction traffic was kept to a bare minimum to alleviate excessive particle breakdown. The material was placed in 800 mm lifts and compacted with four overlapping passes with a large smooth drum, vibratory compactor.

Upon reflection, successful execution of the defensive seismic design measures was only possible because the Contractor was agreeable, conscientious and competent with the earthworks and placed a high degree of attention to consistency in material manufacture. They worked in partnership to a large degree. Without their willingness, meeting all the defensive seismic design criteria would have been far from certain. This is an old lesson, but worth repeating.

Acknowledgements

The authors are grateful to Waimea Water Limited, owner of Waimea Dam, for their support throughout design and construction, and permission to write this paper. The authors are grateful to Iain Lonie and Peter Amos for review of this paper. The authors also wish to thank those dam engineers who blazed the trail before us, some of whom are referenced in this paper.

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