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**O:TU INVESTMENT LIMITED  
BLIND RIVER LOOP ROAD, SEDDON  
HAMMOND BLOCK WATER STORAGE DAM  
REMEDICATION OF EXISTING DAM  
DESIGN REPORT**

Prepared for:

3 April 2014

O:TU Investment Limited  
Blind River Loop Road  
Seddon  
**MARLBOROUGH**



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**Marlborough District Council**

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3 April 2014

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**1.0 INTRODUCTION**

Engineering Geology Ltd (EGL) has been requested to undertake the investigation and design of remediation works for an existing water storage dam on the Hammond Block at the Marlborough Vineyard Group Ltd property located off Blind River Loop Road, Seddon.

The dam was originally designed by Davidson Partners Ltd (DPL) in 2008. The design was used to support the application for a Building Consent which was later issued by the Marlborough District Council (MDC) in 2009. We understand that construction of the dam commenced and was completed in 2012. We also understand that the construction work was undertaken and managed by the former manager of the vineyard on an owner-operator basis.

The original design intention was to construct a dam with a storage capacity of up to 60,000m<sup>3</sup>. However, an as-built survey by Gilbert Haymes and Associates Ltd indicates a maximum volume of 31,600m<sup>3</sup> up to the spillway inlet level. There are also other variations of the built dam when compared to the documents approved by the Building Consent as listed below:

- The dam crest is narrower than the design width of 5m
- The distance between the maximum water level and the lowest crest level (i.e. freeboard) is less than the 1.2m shown on the approved Building Consent documents
- The upper part of the upstream and downstream shoulders of the dam are steeper than the design of 1V:3H
- The location of the spillway is on the northwest end of the dam rather than the south east as shown on the drawings
- The materials used for the construction of the top 3m of the dam appear to be significantly different from the type of materials used to construct the bulk of the embankment.

As a result of the M<sub>w</sub>6.6 Lake Grassmere earthquake on 16 August 2013 (located near Seddon) the dam suffered damage, including longitudinal cracking on the downstream face just below the dam crest, longitudinal cracking and slumping along the upstream face some



2m below the crest and a transverse cracks at the eastern and western ends of the dam crest. Cracks appeared along the slopes surrounding the reservoir.

## 2.0 CONSENT CONDITIONS AND DESIGN REQUIREMENTS

A Building Consent has previously been issued for the dam in 2009. However, owing to the earthquake related issues outlined above and following a site visit made by the Marlborough District Council (MDC) Representatives on 3<sup>rd</sup> October 2013, MDC indicated that a Code Compliance Certificate for the dam will not be issued until the damage from the earthquake has been investigated and repaired and all the variations to the current Building Consent design are reviewed and rectified as necessary and certified as complying with the NZSOLD Guidelines.

It was also noted by the MDC that any remediation work for the earthquake related damage will require a new Building Consent application submitted and a complete set of design documents will need to be provided. Given the history of the dam the MDC will require that design changes be subject to a regulatory review.

## 3.0 POST-EARTHQUAKE INVESTIGATION

### 3.1. Initial Assessment

The investigation of the earthquake affected dam commenced in September 2013 with an inspection of cracks that had appeared on the crest of the dam after the Grassmere earthquake. The dam at the time of inspection was at its full operating level so an instruction was given to withdraw water from the reservoir to minimise the risk of piping failure through the cracks. Following lowering of water in the reservoir it was observed that the upper part of the upstream shoulder of the dam was subjected to slumping of about 0.3m.

Following the initial inspections a site meeting was arranged in December 2013. The meeting was held with the presence of the current vineyard manager, Mr Paul McIntyre, and the former construction foreman, Mr Dan Cooper, who was one of the key people during the construction of the dam. The purpose of the meeting was to review the available information and to establish the construction history of the dam. The main items covered included:

- Embankment design and documentation (including Drawings and Specification)
- Construction materials
- Construction stages
- Construction inspections
- Foundation excavation
- Drainage works
- Compaction of fill materials
- Irrigation outlet pipe
- Spillway

A summary of the construction related items discussed in the meeting is outlined below;

- The drawings, including a plan and section, used for the construction of the dam were prepared by Smart Alliances (SA) which is a Blenheim based engineering consultant. We note that these drawings are not exactly the same as those prepared by DPL which were submitted to support the application for the Building Consent
- No inspections were made by the Representatives of either DPL or SA during the course of construction
- Irrigation outlet pipe was installed by a subcontractor (Findlater Construction)
- No pressure testing was undertaken following the installation of the irrigation outlet pipe
- Compaction testing was conducted and recorded by an employee of HEB Construction. A limited number of Nuclear Density Meter (NDM) tests was taken during construction (only 2 visits over the course of construction of 3 months)
- The compaction testing records indicated that from the 16 NDM tests conducted some 50% were non-conforming with respect to the earthwork specification provided (i.e. 98% of NZ Standard Compaction with moisture content within 0% to 3% wetter than optimum)
- Cut-off trench excavated into the underlying mudstone to provide effective seepage control for the dam
- Chimney drain to a thickness of 1m was installed and extended into the cut-off trench and abutments
- The auxiliary spillway was relocated from the eastern abutment to the western side

One of the main comments made by Dan Cooper was that the dam was constructed in 2-stages. The reason for the multi-stage approach was that the initial setout for the dam footprint was incorrect. In particular it was too small, resulting in a dam crest narrower and lower than what was shown in SA's Drawings. At Stage 1 the dam was built to RL79 but subsequently it was raised to RL80.5 (at Stage 2) to provide greater storage volume. The embankment was constructed to RL79 mainly from mudstone and silty clayey materials sourced from the reservoir area. Loess and pit metals sourced from a nearby quarry were used to raise the dam above RL79.

A meeting minute, incorporating the above matters, was prepared and is included in Appendix A.

### 3.2. Site Investigation

Following the initial inspection and site meeting a two stage site investigation was undertaken. Stage 1 included the excavation a number of test pits (five) undertaken in January 2014. The test pits were excavated on the crest and the upstream shoulder of the embankment to a maximum depth of 3m. Three shallow test pits were also excavated within the reservoir area to assess potential borrow material for construction. Stage 2 of the investigation comprised twelve boreholes drilled in March 2014. The holes were drilled over the upstream shoulder of the embankment. The holes were relatively shallow to a maximum depth of about 0.6m. The *in situ*, undrained shear strengths of the subsoils were measured in the holes at 0.3m depth intervals with a hand operated Pilcon shear vane. The locations of the boreholes and test pits are shown in the attached Drawing 7585-01.



- Option 1: Lowering the spillway inlet by about 2.5m to RL77.5m and leaving the marginal fill in place to maintain an access for the vineyard
- Option 2: Removal and replacement of the weak fill with compacted, competent fill to a crest level of about RL80.3m
- Option 3: Removal and replacement of the weak fill with compacted competent fill as well as further raising of the dam to a crest level of about RL81.5m

Cost estimates were also prepared for each of the remedial options and presented to the client. Following the review of the costs associated with each option the client decided to select Option 2 as the preferred remediation option.

## 5.0 DAM REPAIR DESIGN

### 5.1. Existing Embankment Performance

The existing water storage dam has been used for the irrigation of grapes. The reservoir was created by impounding water on an unnamed tributary of the Blind River. Water is stored in the reservoir prior to irrigation, which is normally the months of January to March.

The dam was constructed in 2012 and it was in operation for about a year before it was temporarily decommissioned due to cracking following the August 2013 earthquake. It is understood that the dam performed satisfactorily during its operational period with no report of significant seepage through the dam or its foundation. There was severe erosion along the spillway channel as it was unlined. The dam had minimal freeboard (0.5m) above the spillway inlet. Periodic monitoring of the chimney drain outlet indicated very small discharges.

We also understand the dam was constructed using the materials available on site comprising loess, colluvium and mudstone which were borrowed from the reservoir area and within the footprint of the dam.

The upper 3m portion of the dam was subjected to strength softening, slumping and cracking as a result of the August 2013 earthquake. There were no any signs of any significant movement and deformation of the remaining portion of the dam (i.e. below RL77m) during the August 2013 earthquakes. This section of the embankment has a flatter profile (i.e. shoulder slope of 3:1 (H:V)) and our investigation found the fill to be stronger.

### 5.2. Hazard Classification

It was stated in the DPL's Design Report (Ref. 1), that the dam was designed in accordance with the New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines which was submitted to the MDC to support the application for Building Consent.

The NZSOLD design requirements are dependent on the potential impact classification (PIC) of the dam. The DPL's report stated that the dam has a low PIC, and based on our knowledge of the downstream conditions this seems reasonable. The vineyard manager confirms that there have not been any major changes downstream of the dam that would be expected to affect the PIC. Thus, a low PIC is adopted for the proposed repair design.

### 5.3. Design Standards

NZSOLD (Ref. 2) design standards have been adopted and they are summarised below.

#### 5.3.1. Embankment Stability

For stability under static and rapid drawdown conditions the conventional factors of safety of  $F \geq 1.5$  and  $F \geq 1.3$ , respectively, have been adopted.

For assessing stability under earthquake loads two levels of shaking have been considered. The lower level, known as the Operating Basis Earthquake (OBE) is based on the 150 year return period level of shaking. The higher level known as the Maximum Design Earthquake (MDE), has been taken equal to the median Maximum Credible Earthquake (MCE). This is conservative for a low PIC dam. The dam is designed to be built on rock (mudstone) foundation which corresponds to Site Subsoil Condition Class B in NZS1170.5. The horizontal acceleration response spectra (5% damping) for the 150 year return period and MDE are shown in Figure 1. The estimated peak ground accelerations (PGA) are 0.24g and 0.36g for OBE and MDE respectively.

The 150 year spectrum is based on the peak ground acceleration derived from the probabilistic seismic hazard study undertaken by GNS Science (formerly Institute of Geological and Nuclear Sciences, Ref. 3). This is 0.24g and is for Class B site subsoil condition. The acceleration spectrum is based on the normalised spectra for Class B site subsoil condition contained in NZS1170.5 (Ref. 4).

The MDE is based on a magnitude Mw 7.5 earthquake occurring on the Awatere Fault which is approximately 8km away from the site. The minimum average shear wave in the top 30m of the foundation profile ( $V_{s30}$ ) for Site Subsoil Class B in NZS1170.5 is 360m/s. This has been assumed in the attenuation models for estimating the MDE level of design ground motion. This assumption is conservative. The MDE spectrum is based on a weighted average of the median spectra predicted by three next generation attenuation (NGA) models. They are Abrahamson and Silva (Ref. 5), Chiou and Youngs (Ref. 6) and Campbell and Bozorgnia (Ref. 7).

#### 5.3.2. Flood Design

For low potential impact dams NZSOLD (Ref. 2) recommends the design flood for sizing spillways be taken between the 1 in 100 and then 1 in 1,000 annual exceedance probability (AEP). We have considered a 1 in 1,000 AEP flood event for sizing the spillways.

### 5.4. Proposed Repair Design

The proposed remediation involves removal and replacement of the upper 3m of weak material with compacted, competent fill up to a crest level of RL80.3m as shown on the attached Drawing 7585-02. The crest of the re-constructed embankment will be 4m wide with upstream and downstream shoulder slopes of 2.5:1 (H:V).

The new fill (Zone 1A) forming the upper 3m of the dam will be homogeneous earthfill constructed from mudstone sourced from the reservoir area. Such materials, once properly conditioned and compacted, can provide a low permeability zone to control seepage through the embankment. The new fill will be keyed into the existing embankment and the natural ground on the abutments of the gully. Zone 1A shall be

placed in loose lifts of up to 300mm (dependent on size of compactor) and be compacted to achieve the following requirements. It shall have a permeability of no greater than  $10^{-8}$ m/s. It shall be compacted within -1 percent to + 2 percent of optimum moisture content to a minimum of 95 percent of maximum dry density with respect to the results of NZ heavy compaction test. Average air voids from any 10 consecutive tests shall be no greater than 6 percent while single test results of up to 8 percent will be permitted.

The existing chimney drain will be raised up to RL 79.1m. This corresponds to the reservoir normal operating level.

The volume of Zone 1A fill associated with the proposed repair work is approximately 1,800m<sup>3</sup>. This assumes that the dam is constructed to RL80.3m. Unsuitable materials excavated from the upper portion of the dam, i.e. weak fill, will be used for contouring of the land for vineyard development. Topsoil will be stripped and temporarily stockpiled. It will be used to rehabilitate the downstream shoulder of the dam and for topsoiling the auxiliary spillway channel.

The storage volume of the dam following the remediation work will be about 21,000m<sup>3</sup>. This assumes the reservoir level under normal operating condition at RL79.1m. The height-storage curve for the reservoir is shown in Figure 2.

### 5.5. Embankment Stability

Stability analysis of the upstream shoulder of the embankment has been performed using the information obtained from the site investigation of the crest and upstream shoulder of the embankment. Stability of the embankment has been analysed for both static (end of construction, drawdown and long-term) and earthquake load cases. The assumptions of the soil strength are summarised in Table 5. Analyses have been undertaken using the SLOPE/W (Ref. 8) software using the Spencer (Ref. 9) method. The results from these analyses are presented in Table 6 and the detailed results are presented in Appendix C.

For the stability analyses the phreatic surface through the embankment is taken level with the impounded water level to the chimney drain and assumed hydrostatic. This is a conservative assumption. Note this assumption does not apply to end of construction case as there is no water present in the reservoir.

Static soil strength parameters for Zones 1A and Existing Embankment materials are based on properties determined from laboratory testing on similar materials from other sites in the area. A total of 12 shallow hand auger boreholes with shear vane tests have been performed to confirm the undrained shear strength of the existing material of the upstream embankment. These results are shown in Table 4 and indicate the undrained strength of the Existing Embankment Materials is generally greater than 150kPa. Both end of construction and long term loading condition have been analysed. For end of construction, analyses have been performed using both effective strength parameters and undrained shear strength for Zone 1A and Existing Embankment Materials. A special case which considered complete rapid drawdown of the reservoir has also been analysed. This analysis assumes the embankment material has finite hydraulic conductivity during the drawdown such that the change in pore-water pressure at the base of the slice is instantaneously equal to the change in ponded water head above the slice. This is considered to be a practical assumption as in reality it is impossible to draw down the water instantaneously.

For seismic stability analyses, undrained strengths of 150kPa and 100kPa have been adopted for Zone 1A and Existing Embankment Material respectively. These values are equivalent to the static undrained soil strength determined in conjunction with the compaction criteria specified in Section 5.4.

Under static loading conditions, the stability analyses indicate factors of safety which are considerably greater than the normally accepted minimum value of 1.5. Under rapid drawdown conditions the calculated minimum factor of safety is 1.5. This is greater than the normally accepted minimum value of 1.3.

Seismic stability has been analysed for the Operating Basic Earthquake (OBE) and the Maximum Design Earthquake (MDE). The PGA for the OBE and MDE level of ground motion is 0.24g and 0.36g. This is discussed in more detail in Section 5.3 of the report. The ground motion amplification (crest acceleration over PGA) relationship given by Harder et al. (Ref. 10) has been used to determine the peak motion at the crest of the embankment for stability analyses. This method is based on actual measurements of ground motions recorded at the crest of the embankment relative to those recorded near their base. Crest accelerations are 0.58g and 0.65g for the OBE and MDE levels of ground motion respectively using the Harder *et al.* recommendations.

Stability has been assessed for a range of potential failure surfaces (failure surface located H, 2/3H and 1/3H below the crest). The results are summarised in Table 6. Under both OBE and MDE levels of ground motion, the pseudo-static factors of safety for all slip circles are greater than 1.0. These results show that yielding of both upstream and downstream shoulders of the embankment is unlikely indicating satisfactory performance.

## 5.6. Embankment Drainage

The dam incorporates a vertical chimney drain to intercept seepage and to act as a filter to prevent internal erosion. The extent of the chimney drain is shown in the attached Drawings 7585-02 and 03.

The existing chimney drain was installed at the downstream crest of the embankment with a thickness of about 1m. The chimney drain also included an outlet drain at the base of the dam. The top of the existing chimney drain is about RL77.5m. It is proposed to extend the chimney drain up to RL79.1m in stages by excavating down through the embankment as it is raised.

The chimney drain will consist of Type 2A drainage material. This material shall have a minimum permeability of  $5 \times 10^{-5}$  m/s, and the particle size distribution shall be within the limits summarised in Section 5.9.2. It will be required to be lightly compacted to achieve a relative density of approximately 75 percent, although it is important that it is not over-compacted.

## 5.7. Spillways

### 5.7.1. General Description

The dam existing spillway will be reconstructed as part of the repair works to operate as an auxiliary spillway. In addition, a primary spillway, consisting of an inlet manhole and outlet pipe, will be incorporated into the design and reconstruction of

the dam. The addition of the primarily spillway means the existing open channel spillway will only ever operate in larger flood events and the channel need only be grassed as it will only operate intermittently.

The main objective of the design is to have a spillway with a capacity to safely discharge flood flows from rare events (1 in 1,000 APE flood event). This is achieved by having primary and auxiliary spillways. The primary spillway is to be durable and to have the capacity to safely pass by itself floods of modest return period (1 in 5 AEP flood event). For greater floods excess water will flow over the auxiliary spillway.

Detailed modelling has been undertaken to optimise the design of the spillways and to ensure their performance will meet design objectives. The primary spillway is small, but is capable of passing flood flows with a 1 in 5 AEP. In conjunction with the primary spillway, the auxiliary spillway is capable of passing the 1 in 1,000 AEP flood with approximately 0.20m freeboard at the dam crest. This is discussed in detail in the next section.

The primary and auxiliary spillways are shown on Drawings 7585-01 to 07. The primary spillway is located on the eastern abutment and consists of a 1200mm diameter manhole entry structure with a 450mm diameter polyethylene outlet pipe (450OD, SDR26, PE80 pipe). The outlet pipe will penetrate through the dam and will be concrete encased through the embankment up to the chimney drain zone. Beyond the chimney drain and through the downstream shoulder, it is bedded on and surrounded by Type 2B drainage material. The above details are shown on Drawings 7585-05 and 06.

The auxiliary spillway is located on the western abutment and consists of a 5m wide inlet weir and 3.5m wide channel. It has a reinforced concrete sill at the entry to provide accurate level control and to ensure that flow is spread uniformly across the grassed spillway. Details are shown on Drawing 7585-02 and a longitudinal section and details for concrete sill are shown on Drawing 7585-04.

For dissipation of energy associated with spillway discharges from the reservoir, riprap will be placed at the end of the primary and auxiliary spillways.

### **5.7.2. Modelling of Spillway Performance**

The height storage curve for the proposed impoundment is shown on Figure 2. Storage effects and the discharge capacity of the outlet works can attenuate the inflow to the reservoir so that the outflow discharge is reduced. To take this into account flood routing to determine design spillway flows has been performed with the program HEC-HMS (Hydrologic Modelling System from the Hydrologic Engineering Centre, Ref.11). This has involved routing of floods through the reservoir for 5 year, 100 year, and 1,000 year design rainfall events.

There is no defined temporal rainfall pattern for this project area, hence the design is based on a 72 hour design rainfall event with critical rainfall intensities for all durations between 10 minutes and 72 hours included. This assumption is conservative.

The catchment area for the dam is 68 ha. The rainfall depth duration for the site was produced by NIWA High Intensity Rainfall System (HIRDS) version 3 as shown in Table 3. Note that HIRDS only produce rainfall information for annual return

interval up to 100 years. For the 1,000 year design rainfall event, the rainfall intensities were estimated based on extrapolation of the data in the table, assuming extreme value theory (i.e. Gumbel distribution). Different runoff coefficients were used for different return periods. For the 100 year return period a runoff coefficient C of 0.35 has been used. This was adopted based on information provided by Brin Williman (Marlborough District Council) on his analysis of flow recording sites and relating 100 year floods to mean annual flood for the Marlborough region. The methodology was based on the regional flood estimation procedure by Pearson and McKerchar (Ref. 12). For a less significant rainfall event, it would appear that a large proportion of runoff is absorbed into storage within the catchment area. On that basis, we adopted runoff coefficient C of 0.30 and 0.405 for the 5 year and 1,000 year return period respectively.

It was assumed that at the beginning of the design rainfall events the water level is at the maximum normal operating level, i.e. at the top manhole level (79.1). The pond water level will rise above the top of the manhole and water will flow over the top of the manhole and through the outlet pipe. In a very large storm, the pond water level may rise above the auxiliary spillway inlet level (RL79.7) and water will flow down the auxiliary spillway channel. Two discharge curves are shown on Figure 3: the discharge capacity of the primary spillway entry (i.e. top of manhole), and the discharge capacity of the outlet pipe. The primary spillway discharge increases as the level of ponded water increases. The discharge is limited by the outlet pipe discharge capacity after the pond water level rises above RL79.25m.

The inflow hydrographs were estimated using the SCS unit hydrograph where the time of concentration is estimated to be approximately 30 minutes. The peak inflows for the 5, 100 and 1,000 year floods into the reservoir are estimated to be approximately 1.3, 3.0, and 4.6m<sup>3</sup>/s, as shown on Figure 4. Discharge over the spillway is much less because of the attenuating effects of the storage that is available between the normal maximum water level and the auxiliary spillway. The peak outflow discharge (from both primary and auxiliary spillways) for the 5, 100 and 1,000 year design rainfall events are estimated to be 0.44, 1.9 and 3.4m<sup>3</sup>/sec, respectively, as shown on Figure 4.

The results of the flood routing study for the 5 year design rainfall event, which is shown on Figure 5, indicate that the primary spillway has sufficient capacity to discharge this event by itself. The pond RL and discharge curves for the 100 and 1,000 year storms are shown on Figures 6 and 7 respectively. It is shown in Figures 6 and 7 that the auxiliary spillway will operate for approximately 23 and 47 hours in the 100 and 1,000 year design rainfall events respectively. Figure 7 shows that the auxiliary spillway will discharge a maximum flow of approximately 2.9m<sup>3</sup>/s in the 1,000 year flood with a freeboard of approximately 200mm. The grassed spillway channel has a greater discharge capacity than this.

The performance of the grassed auxiliary spillway has been assessed by comparing the expected flow velocity and duration against acceptable limits that have been proposed for such spillways (Ref. 13). This is shown on Figure 8 where it can be seen that good cover topsoil and plain grass is required to achieve satisfactory performance. It will be necessary to undertake inspections of the spillway channel following a heavy rainfall event to determine if any erosion has developed in the spillway. If erosion is present, it will be necessary to repair any damage.

In the event that the primary spillway is blocked the reservoir water level will rise higher and flow over the auxiliary spillway. This scenario is unlikely, because in large flood events it is expected that the owners would undertake inspections of the dam and clear any obstructions/blockages in the primary spillway.

**5.8. Irrigation Outlet Pipe**

There is an existing irrigation outlet pipe through the dam which was installed during construction. It is a 250 diameter UPVC PN9 pipe encased in concrete to the chimney drain. Downstream of the chimney drain it is encased in 20/40 drainage material. The outlet pipe incorporates a floating inlet structure. A pump, located immediately below the dam, via a pipe, flange and valve is also connected to the irrigation outlet pipe. The DPL report recommended pressure testing be undertaken for the pipe. However, it is understood that no pressure testing was carried out following the installation of the pipe. The outlet pipe has been in operation since 2012 without any significant signs of leakage. However, since it has been subjected to strong earthquake shaking it is recommended that pressure testing be undertaken to ensure the integrity of the outlet pipe.

**5.9. CONSTRUCTION MATERIALS**

**5.9.1. Embankment Fill**

The main source of earthfill for the remediation works (single zone embankment) will be mudstone that will be obtained from the dam reservoir area. The existing chimney drain will be raised and extended up to RL79.1m within the newly constructed embankment. River sand deposit will be imported for construction of the chimney drain.

**5.9.2. Drainage Materials**

Imported drainage materials are required for the chimney drain and primary spillway outlet drain. The chimney drain material (Type 2A drainage material) is a sandy material. The material for the spillway outlet drain (Type 2B drainage material) is clean gravel. The proposed grading limits for Type 2A and 2B drainage materials are summarised below.

**Type 2A Drainage Material**

Sieve Size	Percentage Passing by Dry Weight
2.36mm	100
1.18mm	70 – 100
600 microns	10 - 40
212 microns	0 - 20
150 microns	0 - 20
75 microns	0 - 5

**Type 2B Drainage Material**

Sieve Size	Percentage Passing by Dry Weight
75mm	100
63mm	80 -100
37.5mm	45 -100
19mm	5 -60
13.2mm	0 - 40
9.5mm	0 - 25
4.75mm	0 -10
2.36mm	no greater than 2

**5.1. Rehabilitation and Erosion Protection**

Existing riprap rock on the upper portion of the dam upstream shoulder can be reused for the rehabilitation of the repaired embankment. Riprap (0.10 to 0.3m) will be placed 0.3m thick on the upstream shoulder of the dam between RL80.3 – RL77.5. Larger armour rock (0.6 to 0.9m) will be required to be placed at the outlet of the spillways.

It is proposed to topsoil and grass the downstream shoulder of the repaired embankment. This will provide resistance to surface erosion from surface runoff. The crest of the dam will be metalled as it provides access to different parts of the vineyard.

**6.0 CONSTRUCTION ASPECTS**

It is proposed that construction will be undertaken by an experienced earthworks Contractor. Construction is planned for 2014 and is expected to take approximately 6 weeks.

Construction drawings and a Technical Specification have been prepared. They detail the requirements for construction including standards for foundation preparation, earthfill compaction, drainage materials and spillways as well as quality assurance requirements. The Designer will undertake inspections at times to confirm critical details and to ensure design requirements are being achieved. An inspection schedule is attached (Appendix D). The Contractor will be required to undertake control testing to confirm fill standards have been achieved. The Designer will check that materials used in construction and construction standards meet the specified requirements.

Some volumes of water will be required for conditioning of the fill and for dust control. This will be provided by water stored in the dam reservoir.

The Contractor will be required to provide a sediment control plan prior to construction commencing. This will set out the proposed works and construction methods that will be implemented to minimise and control sediment. Requirements for dust control are included in the Specification.

Following the completion of construction, the Contractor will be required to provide a Producer Statement, confirming the construction works have been undertaken in accordance with the construction drawings and technical specification.

## 7.0 OPERATION, MAINTENANCE AND SURVEILLANCE

An Operation, Maintenance and Surveillance (OMS) Manual will be prepared, that conforms to the NZSOLD requirements. It will set out the operational and maintenance requirements necessary to ensure the on-going safety of the dam. Monitoring and inspections are a fundamental part of the dam safety process. These range from routine regular inspections to more comprehensive reviews at longer periods. Specific requirements for the dam will be prepared.

Operation of the dam will be under the control of the Owners of the reservoir.

Operational activities include clearing debris that may accumulate around the primary spillway inlet manhole removing silt from the pond.

Maintenance activities include any repairs to the decant manhole and outlet pipe, repairing any erosion to the upstream or downstream shoulders of the embankment, repairing any erosion along the spillway and channel and replacing riprap if necessary.

To ensure the safety of the embankment regular inspections are to be undertaken. Standard forms will be provided for this purpose. More detailed inspections will be required on first filling. Thereafter inspections will be undertaken on a monthly basis and during periods of heavy rain when the pond will impound water. The NZSOLD Guidelines also recommend intermediate inspections by a Technical Advisor every 1 - 2 years and a comprehensive review every 10 years for low PIC dams.

The OMS Manual will also include advice on procedures to follow in the unlikely event of an emergency.

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**Table 1. Summary of Compaction Test Results (NZ Standard)**

Sample Location	Depth	Sample Description	Natural Water Content (%)	Maximum Dry Density (t/m <sup>3</sup> )	Optimum Water Content (%)
Barrow Site	0.5m	Mudstone	13.3	1.77	17.0

**Table 2. Summary of Grading Test Results on Filter (sand) Material**

Sample Location	Sample Description	Sieve Analysis									
		37.5 (mm)	19 (mm)	9.5 (mm)	4.75 (mm)	2.36 (mm)	1.18 (mm)	0.6 (mm)	0.3 (mm)	0.15 (mm)	0.075 (mm)
Existing Chimney Drain	Filter Material	100%	96%	77%	60%	48%	38%	28%	17%	11%	8%

**Table 3. Design Rainfall Depth Duration Frequency from HIRDS v3**

ARI (y)	Duration									
	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
10	7	10	13	20	29	50	70	99	115	125
100	11	17	22	35	48	82	115	161	187	204
1000*	14	23	29	46	64	108	151	211	244	266

\*Based on extreme value theory (Gumbel Distribution)

Table 4. Summary of Site Investigations of Shallow Hand Auger Boreholes on the Upstream Shoulder

Section	Borehole No.	Vane Shear Strength (kPa) <sup>1</sup>				Comments
		Uncorrected		Corrected		
		300mm	600mm	300mm	600mm	
1	1A	UTP,55	-	UTP,107	-	Terminated at 500mm (hit gravel/cobble)
	1AA	-	-	-	-	Terminated at 500mm (hit gravel/cobble)
	1B	102,24	UTP,58	198,47	UTP,112	-
	1C	75,24	-	145,47	-	Terminated at 600mm (hit gravel/cobble)
2	2A	-	-	-	-	Terminated at 200mm (hit gravel/cobble), dug a pit
	2AA	75,25	90,30	145,48	174,58	-
	2B	-	-	-	-	Terminated at 200mm (hit gravel/cobble)
	2BB	-	-	-	-	Terminated at 300mm (hit gravel/cobble), dug a pit
	2C	90,35	UTP,52	174,68	UTP,101	-
3	3A	UTP,35	UTP,60	UTP,68	UTP,116	-
	3B	UTP,54	75,28	UTP,105	145,54	-
	3C	UTP,30	UTP,50	UTP,58	UTP,97	-

1. Vane Shear Strength recorded as "Field Vane Strength, Remoulded Vane Strength"
2. Unless specified, all holes drilled to 600mm depth and shear vane test performed at 300mm and 600mm
3. UTP stands for "Unable to Penetrate" which corresponds to strengths greater than 200 kPa

Table 5. Soil Strength Properties

Zone	Material Properties				
	Effective Stress Parameter			Total Stress Parameter	$r_u$ (end of construction)
	$\gamma$ (kN/m <sup>3</sup> )	$c'$ (kPa)	$\phi'$ (°)	Su (kPa)	
Zone 1A	22	10	38	150	0.2
Existing Embankment	21	5	35	100	0.1
Riprap	24	0	45	NA	NA

Table 6. Results of Stability Analysis

Loading Condition	Strength Characteristics	Circle Location	Pore Pressure Condition	$K_h$ (g) <sup>1</sup>	FoS	Figure
End of Construction	TSA	U/S	$r_u$	-	4.8	C1
End of Construction	ESA	U/S	$r_u$	-	2.3	C2
Long Term	ESA	U/S	GWL	-	2.7	C3
Complete Rapid Drawdown	ESA	U/S	GWL	-	1.5	C4
OBE	TSA	U/S_H	GWL	0.24	3.2	C5
OBE	TSA	U/S_2/3H	GWL	0.41	2.7	C6
OBE	TSA	U/S_1/3H	GWL	0.58	3.2	C7
MDE	TSA	U/S_H	GWL	0.35	2.4	C8
MDE	TSA	U/S_2/3H	GWL	0.49	2.3	C9
MDE	TSA	U/S_1/3H	GWL	0.63	2.9	C10

1. Seismic Coefficient of the Potential Sliding Mass