# Claydon Investment Trust

## Claydon Reservoir

## Technical Report in Support of Resource Consent Application

**March 2019**

**MARLBOROUGH MANAGEMENT SERVICES LTD**

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**MARLBOROUGH MANAGEMENT SERVICES LTD**

65 Cob Cottage Road
RD 4
Riverlands
BLENHEIM 7274

Phone: (03) 577 5954
Fax: (03) 578 5481
Mobile: (021) 221 0003
E-mail: tim@smitventures.co.nz

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# 1. Terms of Reference

Marlborough Management Services Ltd has been engaged by Claydon Investment Trust to undertake the concept design and prepare a technical report for the proposed Claydon storage reservoir.

This report has been prepared for inclusion into the Resource Consent Application being prepared by Steve Wilkes of Wilkes Resource Management.

# 2. Introduction/Background

Claydon Investment Trust require this new storage reservoir to provide a secure supply of water for the irrigation of a proposed 20ha vineyard development and 7ha of pasture and crops on their property in the Wairau Valley.

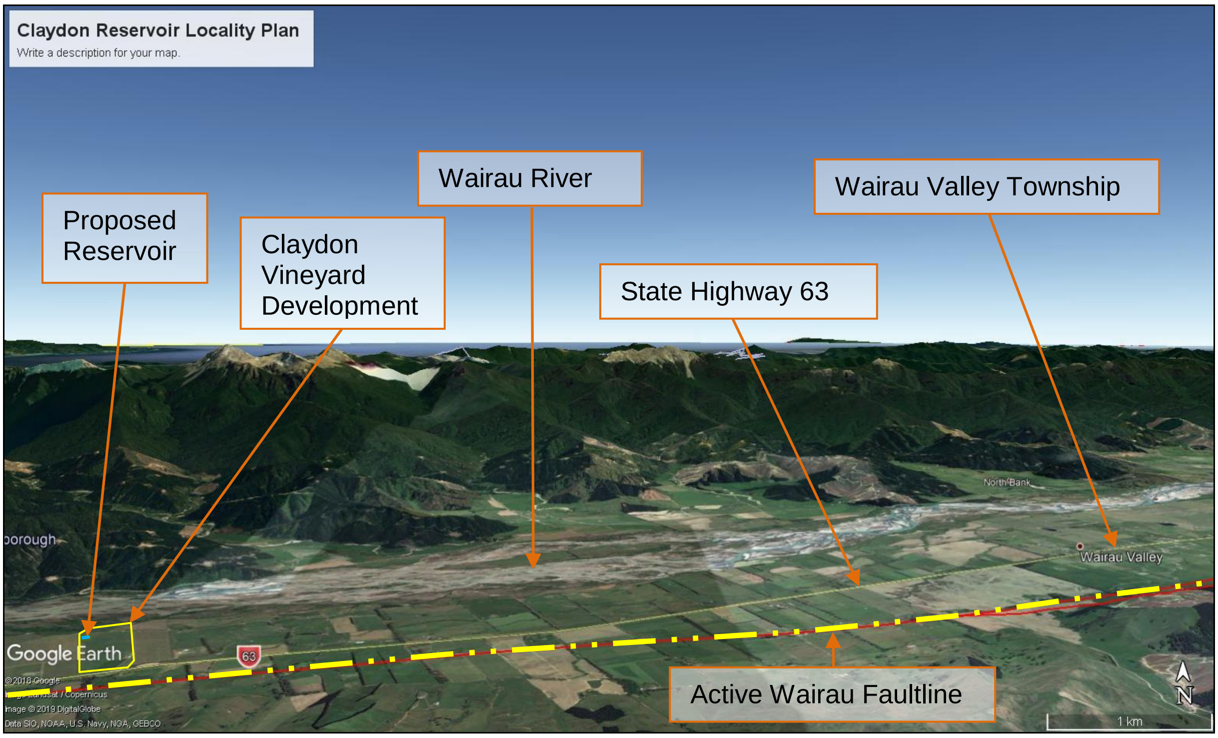
The property has water permit U180766 which provides for the abstraction of up to 748 $m^3$/day of Wairau River Class A FMU water from well 10268 for the irrigation of up to 20 hectares of vineyard and 7 hectares of pasture and crops on Lot 1 DP 419657.

Based on this water permit regime, back-up irrigation water storage is required in the event of the Wairau River Class A FMU water permit being restricted due Wairau River flows dropping below $8m^3/s$ as recorded at Council's Barnett Bank recorder or below $7.32m^3/s$ as recorded at Council's Dip Flat recorder.

# 3. Proposed Reservoir Location

The proposed reservoir is to be constructed some 4.8m west of the Wairau Valley township, at 3516 State Highway 63 Wairau Valley on LOT 1 DP 419657, owned by Adrian Douglas Claydon, Heather Joy Claydon and Hibiscus Independent Trustees 2010 Limited (as trustees of the Claydon Investment Trust)

Refer to the locality plan below, showing the location of the proposed reservoir relative to the vineyard development, State Highway 63, the Wairau River, the active Wairau Faultline and the Wairau Valley township.



*Locality Plan Looking North*

# 4. Description of the Site and Geotechnical Conditions.

The new reservoir is proposed to be constructed in the north-western corner of the vineyard, some 1,040m to the north of the active Wairau Faultline. The landform in the location of the proposed reservoir is flat with a 1 in 150 down-valley gradient to the east.

Just to the north of the proposed reservoir there exists a 4m high natural river terrace.

The land to be occupied by the proposed reservoir is covered in grass pasture and is currently being developed into a vineyard.

*Photo looking north-east towards the site for the proposed reservoir.*

Field investigations were carried out on 11 May 2018 with the help of a 7-tonne excavator and two test pits were dug in the floor of the proposed reservoir, to a depth around 3.5m.

The soils encountered in both test pits were similar and showed the following typical soil profile:

- a 0.25m thick layer of dry, brown, silty gravel (0 to 150mm) topsoil with roots, over;
- a 2.5m thick layer of dry, light brown, sandy gravel (ranging from 0 to 300mm), over;
- dry, grey-brown sandy gravel (ranging from 0 – 300mm).

*Photo of Test Pit A showing soil layers.*

*Photo showing material excavated from Test Pit A.*

Refer to Appendix B for detailed test pit logs.

The proposed reservoir site is relatively close to the active Wairau Fault, which runs some 1,040m to the south of the proposed reservoir site.

An earthquake or fault rupture along this fault line could affect the reservoir by means of seismic instability or liquefaction, although the latter is unlikely due to the nature of the gravels which exist in this area. Liquefaction generally only occurs in areas with thick, uniformly graded, saturated sand and silt layers.

All subsoil materials below the design floor depth of the proposed reservoir were found to be dry.

The impact of these seismic forces will be taken into account in the final reservoir structural design.

# 5. Proposed Reservoir Layout / Configuration

The new 20,000m$^3$ reservoir is proposed to be constructed on flat land some 10m south of the top of the natural terrace edge.

For efficiency and ease of construction, and to fit in with the proposed vineyard rows, the reservoir is designed with the toe of the embankments to be roughly square and parallel with the western boundary and terrace edge to the north.

To assist with draining the reservoir, the floor of the reservoir has been graded with 0.4m fall to a 0.5m deep sump halfway along the eastern embankment.

The reservoir has been designed with an average water depth of 6.0m and a maximum water depth of 6.7m due to this fall across the reservoir floor and the 0.5m deep sump.

The average embankment height is around 2.7m. However, due to the undulating topography of the underlying land, the maximum embankment height halfway along the eastern embankment is 4.4m and the minimum embankment height in the south-western and north-western corners is 1.9m.

The completed reservoir occupies an area of some 0.91ha.

Please refer to the Reservoir Plan, Cross Sections and Details, attached as Appendix A.

# 6. Reservoir Design

## 6.1. Embankment

The reservoir will be designed in accordance with the New Zealand Society on Large Dam (NZSOLD) (2015) Dam Safety Guidelines.

As a lined reservoir, the reservoir embankment will be designed to meet stability requirements only, as the synthetic liner will replace the low permeability core commonly found in other earthfill dams. The embankment will have side slopes of 1m Vertical to 3m Horizontal inside and outside.

The crest is proposed to be 2.5m wide to facilitate access around the reservoir.

The maximum height of the wall, above natural ground level, will be 4.4m.

The earth embankment will consist of reworked, well compacted, sandy and silty gravels, which will all be obtained from the excavation within the proposed reservoir. Based on local experience on similar structures, the design allows for a 10% to 15% reduction of these materials from their natural state to a compacted state and should result in a near balance of cut to fill.

The total earth fill required for the reservoir has been estimated at 11,000m$^3$ compared to a total combined cut volume of topsoil and material to be used as fill from within the reservoir of 6,000m$^3$, allowing for topsoil removal and a bulking/compaction factor.

The sandy and silty gravel fill materials are good for structural embankments, although clearly not good for retaining water. Hence, it is proposed to line the reservoir with a continuously welded, 1.5mm thick, synthetic HDPE liner.

To protect the HDPE liner from possible punctures, from the underlying sandy gravel fill, the finished internal slope and floor of the reservoir will be covered with a minimum 25mm thick layer of blinding material, prior to the placement of the liner. As it is unlikely that this blinding material will be able to be sourced from within the excavation, it will most likely need to be imported.

The blinding layer on the inside faces of the reservoir needs to be a clean, easily worked material to provide a smooth layer, without any sharp stones that may puncture the HDPE liner, which will be placed directly onto it.

The blinding layer could be an imported crusher dust or similar material.

The liner will be anchored into a 0.6m deep, 0.6m wide anchor trench along the crest of the embankment.

Some of the stripped topsoil will be placed on the outside face of the reservoir and grassed on completion and the crest of the reservoir will be capped with a 150mm crushed gravel layer.

## 6.2. Reservoir Spillway / Freeboard

As the reservoir embankments are all above natural ground level the reservoir has no natural upstream catchment area, as one would find with a dam on a river, stream or creek. Hence, the only water entering the reservoir naturally will be any precipitation which falls directly on the internal lined surface.

The reservoir has therefore been designed with a relatively small HDPE lined spillway, just to cope with this minimal rainfall capture, and to discharge any pumped water, if for some reason the pumps filling the reservoir were kept going after it was full.

The size of the spillway is therefore largely dictated by construction practicalities as the 1.5mm HDPE liner is relatively rigid. For these reasons, a 4m wide spillway is proposed with a 1m wide bottom and 1 in 3 sloped sides halfway along the eastern embankment of the reservoir. The water from this spillway will be discharged to a soakpit for disposal, which with the gravelly nature of the subsoil should easily cope with this minimal outflow.

Freeboard is similarly not required for flooding reasons and is purely to cope with wave action. As the water surface when the reservoir is full is only 112m long, in the prevailing north-west/south-east wind diagonal directions, the fetch for wave formation is quite limited.

A total freeboard of 0.5 metres is therefore deemed appropriate.

### 6.3. Reservoir Outlet and Inflow Pipelines

As mentioned earlier, the reservoir floor will be designed with a fall towards a 0.5 metre deep sump, halfway along the eastern embankment of the proposed reservoir. This sump will allow the majority of the water stored to be drawn for irrigation, without a suction vortex occurring at the pipe end.

The size of the irrigation outlet pipe underneath the embankment will be determined as part of the vineyard irrigation design, but is likely to be a 150mm or 200mm diameter pipe which penetrates runs from within the sump in the floor of the reservoir to a proposed pump shed halfway along the eastern embankment.

The pipe will consist of continuously welded HDPE which will be installed within a trench excavated below the bottom of the embankment fill.

Where the pipe enters the reservoir, a concrete slab will be constructed around the pipe to match the internal dam profile and a HDPE E-lock Channel will be cast into the concrete, to allow the liner to be “welded” to the concrete and its integrity to be continuous. In addition, a “boot” made from the liner will be welded to the liner and strapped to the pipe with stainless steel straps.

The reservoir fill pipe is proposed to come over the reservoir wall and discharge into the reservoir above full water level. Hence, the liner penetration will be a boot only.

### 6.4. Reservoir Drainage

As can be seen from the typical cross sections on the concept reservoir drawings, the reservoir sump floor has been designed to be above the natural ground level on the lower terrace to the north-east of the proposed reservoir.

Most of the reservoir floor is also just above the bottom of the test pits within which no groundwater was encountered. As the pit was excavated shortly after a significant rainfall event in the area, we do not expect that this groundwater table will rise much in winter.

Having said that, it may still be possible that minor water seepage into the lower part of the excavated reservoir may occur when the water level rises above what was observed in May 2018. This water seepage could potentially be above the final sump floor level and may therefore create problems during construction and may also result in water pressures lifting the liner when the completed reservoir is empty.

A sub liner drainage network will therefore be installed to avoid these problems, with the drainage outlet discharging by gravity to the lower terrace to the north-east of the reservoir.

This network will consist of perforated PVC pipes laid in drainage metal, wrapped in geotextile in the reservoir. Where the drainage pipes crosses under the reservoir embankment it will consist of a solid pipe with no perforations.

## 7. Dam Break Assessment and Effects

Most historic reservoir failures have resulted from either seismic events causing embankment rupture, overtopping, or piping where a small “leak” progressively scours out till a partial or total failure occurs.

Due to their construction methods, synthetically lined, earthfill embankments have a lower risk profile than normal earth fill dams. This is due to the fact that the HDPE liner is both flexible and strong, and even when ruptured, will not easily tear and because the whole of the embankments remain drained and do not have upstream saturated fill zones.

In addition, in the unlikely event of a small hole in the liner occurring, the gravel embankment fill and gravel subgrade under the proposed embankments have a relatively high permeability, allowing some seepage of the water from a hypothetical liner rupture to occur within the fill structure, without structural failure of the embankment.

As noted above, such seepage will be picked up by the sub liner drainage and a sudden increase in flows from this system would provide an early warning that a small liner failure may have occurred, allowing the reservoir to be drained and repairs to be undertaken. It is therefore important that following construction, monitoring of the sub liner drainage outlet is undertaken on a programmed regular basis.

Well compacted, dry, sandy gravel embankments have a high internal structural strength, and hence the chances of slumping of the fill during seismic events, will be less than saturated upstream zones in finer earthfill dams.

It should however be noted that the reservoir sits relatively close to the active Wairau Fault line.

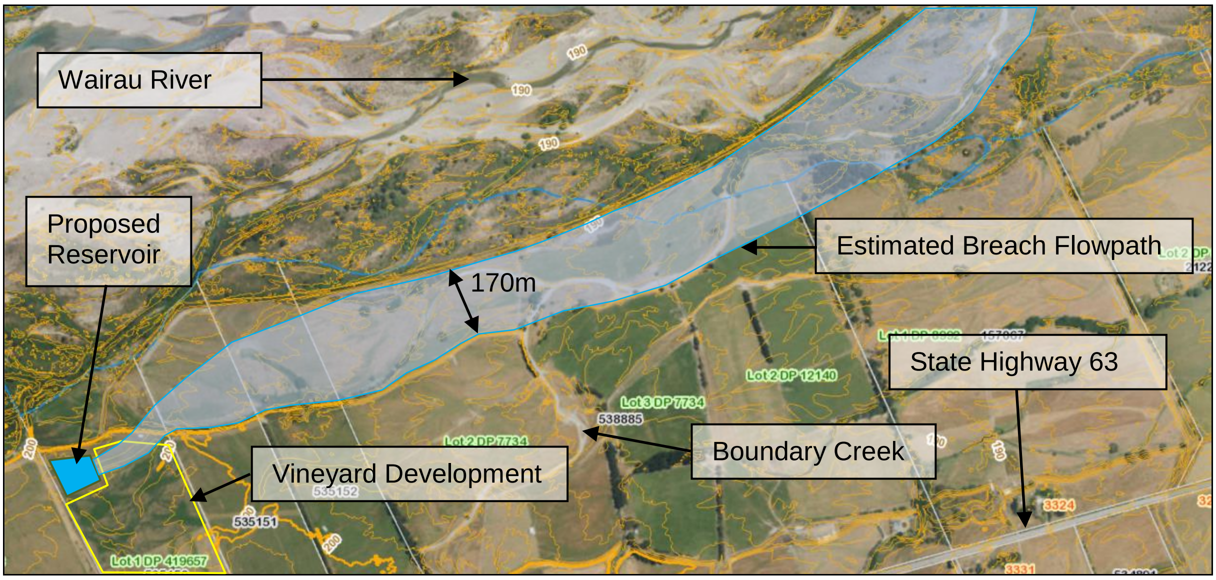
Reservoirs will generally fail during a significant seismic event where the embankment is at its highest, due to increasing water pressure on the embankment with increasing head.

In the case of the proposed reservoir, the highest embankment is along the eastern side, where it is 4.4m high, with the water level behind this embankment being 3.9m above the lowest natural ground around the reservoir.

The water volume impounded within this 3.9m depth between the spillway invert level and the lowest surrounding ground level is some 15,600m$^3$.

In the unlikely event of a failure, a breach is most likely to occur along the reservoir's eastern embankment where the impounded water level is at its highest and water escaping will discharge onto and spread out over the flat land downstream, and to the east, of the reservoir and flow into the Claydon Investment Trust vineyard.

From there the flow will drop down the natural terrace and continue in an easterly direction across downstream properties before discharging back into Boundary Creek and from there into Wairau River, as shown on the marked-up aerial photo below.

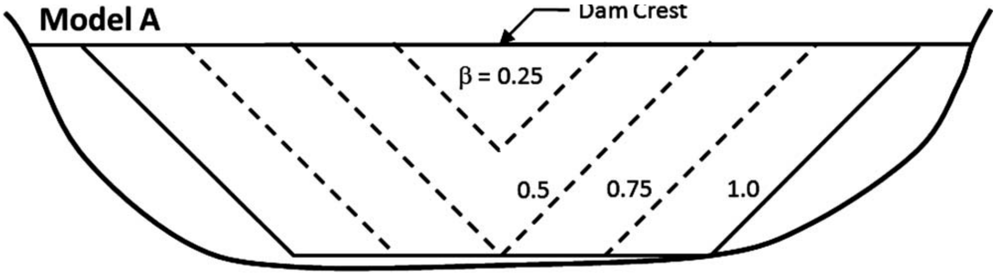


*Plan Showing Estimated Breach Flowpath*

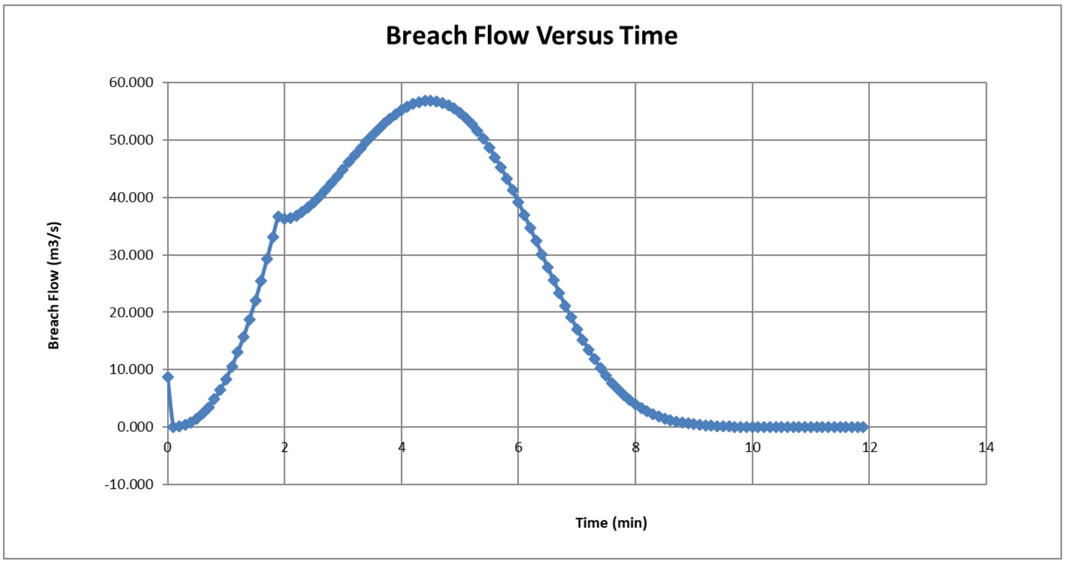
A dam break assessment has been undertaken (using the Froehlich equations) giving:

- a Breach Formation Time, being the time from the beginning of rapid growth of a breach to the time when significant lateral erosion of the embankment has stopped, of 11 minutes, and:
- a Peak Breach Outflow of 56.5m$^3$/s.

The reservoir was assumed to be full at the time of the breach, with the breach occurring as a rapidly growing, triangular failure till it reaches the level of the downstream surrounding ground, after which it widens at the same rate till the breach has been completed and the reservoir has emptied. This is diagrammatically shown on the Model A failure diagram below.



This breach has been modelled on this basis and the estimated breach flow versus time, matching the calculated values using the Froehlich equations, is shown in the graph below.



*Breach Flow Versus Time Curve*

This graph shows how the breach flow steadily increases from nothing to the estimated peak of 56.5m$^3$/s over some 5 minutes, after which it reduces back down over a slightly shorter period, with a total breach being completed in 9 minutes.

The breach flows will ultimately discharge to Boundary Creek and from there to the Wairau River some 2.6km to the north-east. To reach the river, the breach flows will likely spread out and flow as shown on the estimated breach flow path shown on the previous page.

Looking at this estimated breach flow path, and ignoring the reduction in peak breach flow with increasing distance from the reservoir, based on a flow width of 170m (being the narrowest point between the river stopbank and the terrace edge), an average flow depth of some 0.31m would carry the peak breach flow 56.5m$^3$/s, with a velocity of 1.05m/s down the general valley gradient of 1 in 150, based on a Mannings n factor of 0.035.

In reality, the peak flow will reduce due to retention, as the breach flow progresses downstream. However, for the purpose of this exercise, a conservative approach with no reduction has been taken.

It is noted that, as can be seen from the contours, there are local shallow depressions and small watercourses within this breach flow path area, and working on a maximum watercourse depth of 0.5m, the likely flow depth in these small watercourses could be in the order of 0.81, with flow velocities increasing to some 1.5m/s.

## 8. Potential Impact Category (PIC)

New Zealand Society on Large Dam (NZSOLD) (2015) Dam Safety Guidelines (hereinafter referred to as the Guidelines) describes a dam's classification, termed its Potential Impact Classification (PIC), as a function of the consequences of a hypothetical failure breach or other uncontrolled release of the stored contents.

As a result of the height of water retained being only 3.9m and the volume of water retained being only 15,600m$^3$, the dam is not classified as a Large Dam and hence the NZSOLD Guidelines do not strictly apply, and a Building Consent will not be required. The NZSOLD Guidelines only apply to dams that retain more than 3m depth and 20,000m$^3$ volume of water.

Having said that, we have assessed the PIC of the proposed reservoir for completeness.

It should be noted that PIC category has no correlation with the probability of the dam failing or experiencing a dam safety incident.

To determine the reservoir's PIC, we have reviewed the effect of a breach against Tables 2.2 and 3.1 of the Guidelines (copies attached as Appendix C) and note the following:

## 8.1. Residential Houses

Immediately downstream of the reservoir, the land across which this breach water will flow before it reaches the first dwelling, will have been planted in vineyard and the remainder of the breach flowpath affects other downstream vineyards and pasture.

There are no dwellings within the estimated breach flowpath.

Based on the flow depth and velocities detailed above, the comparison of estimated breach flows relative to natural flood flows in these streams (refer Section 8.3 below), as well as the lack of dwellings potentially affected, the Assessed Damage Level to residential houses is assessed to be “Minimal” (i.e. minor damage).

## 8.2. Critical or Major Infrastructure

Critical or major infrastructure is defined in the Table 2.2 of the Guidelines to include lifelines (e.g. power supply, water supply, transportation systems, etc.), emergency facilities (e.g. hospitals, police, etc.) and large industrial, commercial facilities.

The only “critical or major” infrastructure identified in the dam breach flow path is an old section of Wairau River stopbank. However, with the limited flow depth and velocity, the effect of the breach flow would be similar to what the stopbank would be subjected to in minor Wairau River and Boundary Creek floods and the extent of damage, if any, is likely to be localised and able to be repaired in a number of days.

Based on this, the Assessed Damage Level to critical or major infrastructure is assessed to be “Minimal” (i.e. minor damage with a time to restore up to 1 week).

## 8.3. Natural Environment

The downstream environment between the reservoir and the Wairau River is highly modified and the most significant natural environment impact from a breach would be Boundary Creek.

If we consider the effects of the breach flow on the Boundary Creek’s natural environment, we find that the stream flows from larger rain events from its 39.8km$^2$ catchment are much larger than the estimated breach flows.

For comparison, the 1 in 5 and 1 in 10 year flood flows in Boundary Creek have been estimated to be in the order of 76m$^3$/s and 93m$^3$/s respectively and the 1 in 100 year flood is estimated to be 151m$^3$/s (calculated using an average of the 2015 Williman and McKerchar & Pearson Regional Flood Formulae and the Rational Method for the for the 39.8km$^2$ catchment).

Boundary Creek, into which the breach water will flow, runs through pastoral land until it reaches the Wairau River.

Hence, there is unlikely to be any real damage to the natural Boundary Creek environment from such a breach which is estimated to result in a peak flow of only 56.5m$^3$/s.

Based on this, the Assessed Damage Level to the natural environment is assessed to be “Minimal” (i.e. short-term damage).

## 8.4. Community Recovery Time

The impacts of the dam breach flow on the community has the potential to include siltation and erosion of farmland, livestock deaths, damaged fences, vineyards and other damaged farm utilities and Resources.

Based on this, the community recovery time is judged to be “Minimal (i.e. days to weeks).

## 8.5. Population at Risk (PAR)

The population exposed to the potential dam breach flow path area is termed in the Guidelines as the “Population at Risk” (PAR), and defined as “the number of people who would be directly exposed to inundation greater than 0.5m in depth if they took no action to evacuate”.

The PAR includes both permanent populations (e.g. people in houses or workplaces) and temporary populations (e.g. road users, anglers, farm workers, etc).

Note that PAR estimates do not equate to loss-of-life estimates.

Taking into account the increased depth and velocities in the natural watercourses, the Flood severity (DV), (from RCEM – *Reclamation Consequence Estimating Methodology* - U.S. Department of the Interior Bureau of Reclamation), being the maximum depth (D) of flooding multiplied by maximum velocity (V) of the breach flow, is very low and outside the Fatality Rate graphs provided in this publication.

We have therefore considered the U.S. Department of the Interior Bureau of Reclamation *Dam Safety Report No. DSO-99-06 Fatality Rates Table*. Using this and adopting a Low Flood Severity and a No Warning scenario, a suggested Fatality Rate of 0.01 has been adopted.

Having noted that, in a worst-case scenario of a breach occurring at a time when say a couple of people were to be unexpectedly caught in the breach flow, and a Fatality Rate of 0.01, the PAR has been estimated as 0.02 as a worst-case scenario. This is however unlikely as the depth and velocity of the water is such that people would likely be able to escape the flow before it reached its peak.

Considering the 1st four categories, the highest damage level from the above categories is "minimal" and this has been selected.

Considering the Minimal Category with a PAR of 0.02, based on Table 3.1 of the Guidelines, the Potential Impact Category (PIC) for the reservoir falls between the Low and the Low/Medium/High damage level, with the latter referencing notes1, 3 and 4 related to this Table (refer below).

*Note 1. With a PAR of 5 or more people, it is unlikely that the potential impact will be low.*

*Note 3. Use a medium classification if it is highly likely that a life will be lost.*

*Note 4. Use a high classification if it is highly likely that 2 or more lives will be lost.*

Based on the PAR of 0.02 a Potential Impact Category (PIC) of Low has therefore been adopted.

## 9. Construction Methods

The reservoir will be constructed in accordance with the New Zealand Society of Large Dam Guidelines by a contractor experienced with this type of work. The reservoir construction will also be under a standard civil engineering construction contract with adequate supervision by a representative of the design firm, allowing a Construction Producer Statement to be provided.

Construction will largely be undertaken with excavators, trucks and vibrating roller compactors to ensure the earth fill meets recognized compaction criteria.

On completion of earth filling, the inside surface of the reservoir will be "dressed" with a minimum of 25mm of sand or silt blinding layer to provide a smooth profile, prior to the placing of the liner.

The HDPE liner will be installed, and the edges of adjacent sheets welded by a recognized/certified installer using double track fusion welds, and all welds will be air tested prior to the initial filling of the reservoir, to ensure the total integrity of the liner.

Quality Control records will be kept of all earth fill compaction and liner placement and welding.

To minimise the impact of the work on surrounding properties, adequate dust and silt erosion/run-off control will be required to be put in place through the Contract Conditions.

## 10. Conclusion

Synthetically lined gravel and earth fill reservoirs have been successfully constructed and operated in Marlborough, other parts of New Zealand and the world for many years.

They are recognized as safe structures, which also provide high water quality at reasonable costs.

The proposed reservoir is relatively close to the active Wairau Fault line and it is deemed a Low hazard structure as defined in New Zealand Society of Large Dam Guidelines, due to its design, and downstream development affected by the hypothetical breach flows.

In our opinion, there are no technical reasons for Council not to grant resource consent for this proposed reservoir.

We trust this provides the information you require, but if there are any questions relating to this report, do not hesitate to contact the undersigned.

# 11. Applicability

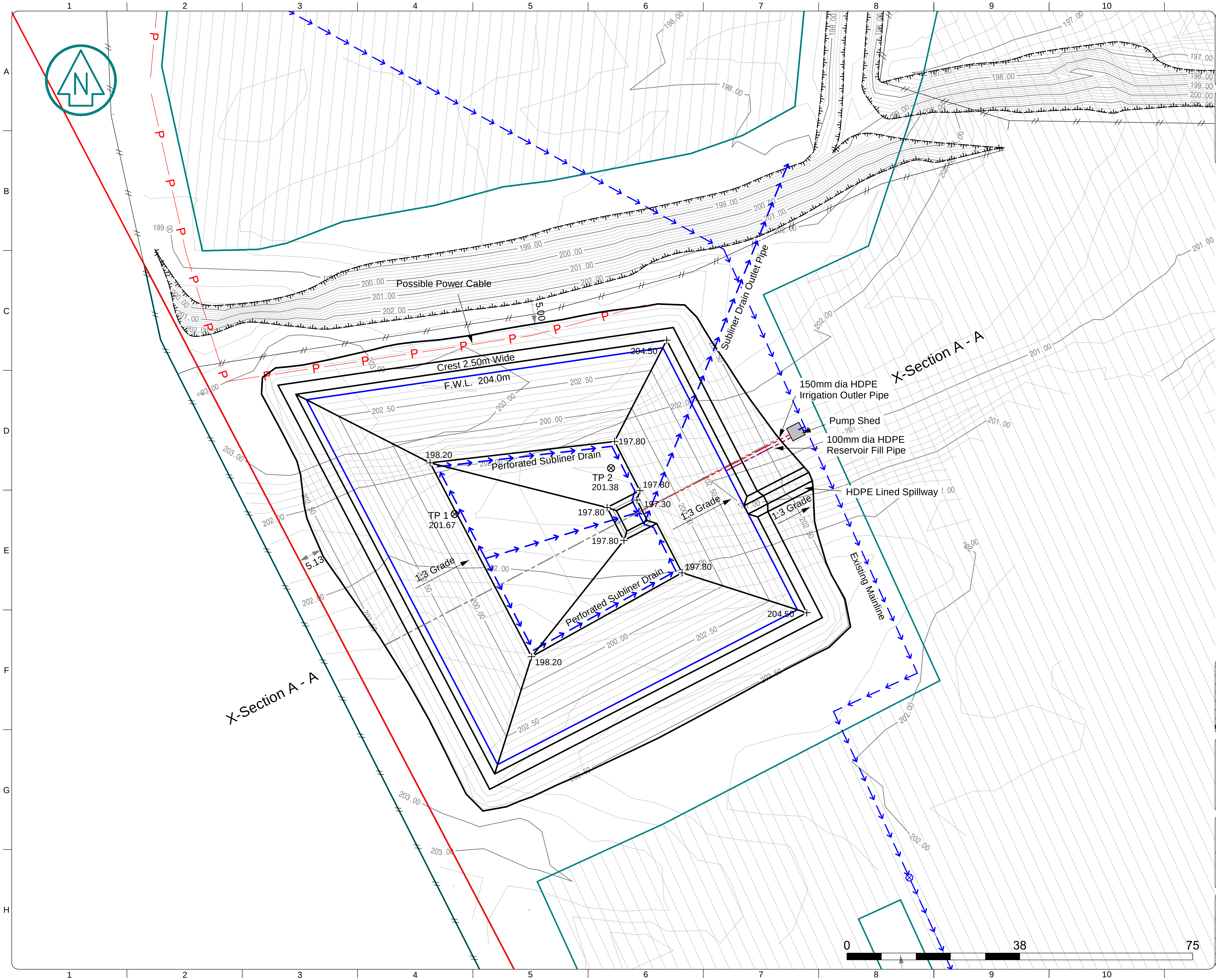
This report has been prepared for the benefit of Claydon Investment Trust with respect to the particular brief given to us and it may not be relied upon by any other party, other than the Marlborough District Council, in other contexts or for any other purpose without our prior review and arrangement.

MARLBOROUGH MANAGEMENT SERVICES LTD

Report prepared by:

T H Smit

# Appendix A – Concept Reservoir Plan and Cross Section



**GILBERT HAYMES AND ASSOCIATES LTD**

REGISTERED SURVEYORS
P.O.BOX 380 - 181 HIGH STREET - BLENHEIM
PHONE (03)5787984 - FAX (03)5787709
E-MAIL office@gilberthaymes.co.nz

PROJECT TITLE:
PROJECT NUMBER:

MARLBOROUGH MANAGEMENT SERVICES
65 Cob Cottage Rd
RD 4, Riverlands
Blenheim, 7274
Phone (03) 5775954
Fax (03) 5785481
Mobile (021) 221 0003
E-mail tim@smitventure.co.nz

| REV | DATE | DESCRIPTION |
|---|---|---|

SITE: Claydon Reservoir
WORK AREA: Wairau Valley

DRAWING TITLE:
Proposed Reservoir

DRAWING NUMBER

REVISION

SCALE: 1:750
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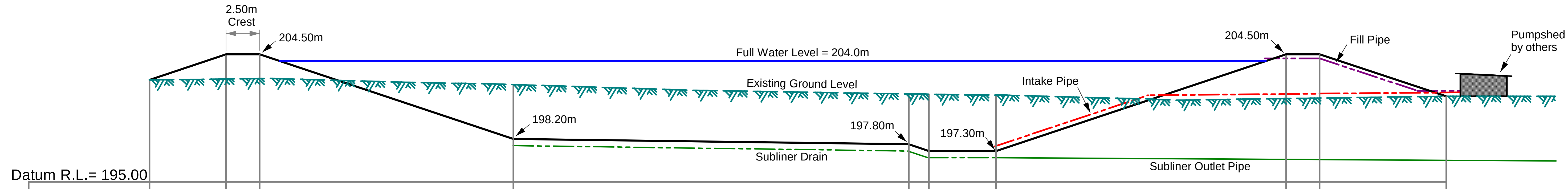

| | | | | | | | | | | |
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| Design | 202.60 | 204.50 | 204.50 | 198.20 | 197.80 | 197.30 | 197.30 | 204.50 | 204.50 | 201.38 |
| Natural | 202.60 | 202.66 | 202.69 | 202.25 | 201.55 | 201.53 | 201.46 | 201.21 | 201.24 | 201.37 |
| Chainage | 0.00 | 5.70 | 8.20 | 27.10 | 56.57 | 58.07 | 63.07 | 84.67 | 87.17 | 96.61 |

Scale Horizontal 1:300 Vertical 1:300

# X-Section A - A

GILBERT HAYMES AND ASSOCIATES LTD

REGISTERED SURVEYORS
P.O.BOX 380 - 181 HIGH STREET - BLENHEIM
PHONE (03)5787984 - FAX (03)5787709
E-MAIL office@gilberthaymes.co.nz

PROJECT TITLE:
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MARLBOROUGH MANAGEMENT SERVICES
65 Cobb Cottage Rd
RD 4, Riverlands
Blenheim, 7274
Phone (03) 5775954
(03) 5785481
Mobile (021) 221 0003
E-mail tim@smitventures.co.nz

| REV | DATE | DESCRIPTION |
|---|---|---|

SITE: Claydon Reservoir
WORK AREA: Wairau Valley
DRAWING TITLE: Proposed Reservoir

DRAWING NUMBER
REVISION
SCALE: As Above
DRG SIZE: A3
CAD FILE NAME: Claydon Reservoir.PDF

# Appendix B – Test Pit Logs

## Claydon Investment Trust
## Claydon Reservoir

## Trial Pit Log No. A

**Date** Friday, 11 May 2018
**Equipment** 7t Excavator

| Depth | Graphic Log | Description |
|---|---|---|
| **0** | | |
| 0.1 | | dry, brown, silty gravel topsoil with roots (ranging from 0 to 150mm) |
| 0.2 | | |
| 0.3 | | dry, light brown, sandy gravel (ranging from 0 to 300mm) |
| 0.4 | | |
| **0.5** | | |
| 0.6 | | |
| 0.7 | | |
| 0.8 | | |
| 0.9 | | |
| **1** | | |
| 1.1 | | |
| 1.2 | | |
| 1.3 | | |
| 1.4 | | |
| **1.5** | | |
| 1.6 | | |
| 1.7 | | |
| 1.8 | | |
| 1.9 | | |
| **2** | | |
| 2.1 | | |
| 2.2 | | |
| 2.3 | | |
| 2.4 | | |
| **2.5** | | |
| 2.6 | | |
| 2.7 | | |
| 2.8 | | |
| 2.9 | | |
| **3** | | |
| 3.1 | | dry, grey-brown, sandy gravel (ranging from 0 to 300mm) |
| 3.2 | | |
| 3.3 | | |
| 3.4 | | |
| **3.5** | | |
| 3.6 | Bottom of Pit | |
| 3.7 | | |
| 3.8 | | |
| 3.9 | | |
| **4** | | |
| 4.1 | | |
| 4.2 | | |
| 4.3 | | |
| 4.4 | | |
| **4.5** | | |
| 4.6 | | |
| 4.7 | | |
| 4.8 | | |
| 4.9 | | |
| **5** | | |

## Claydon Investment Trust
## Claydon Reservoir

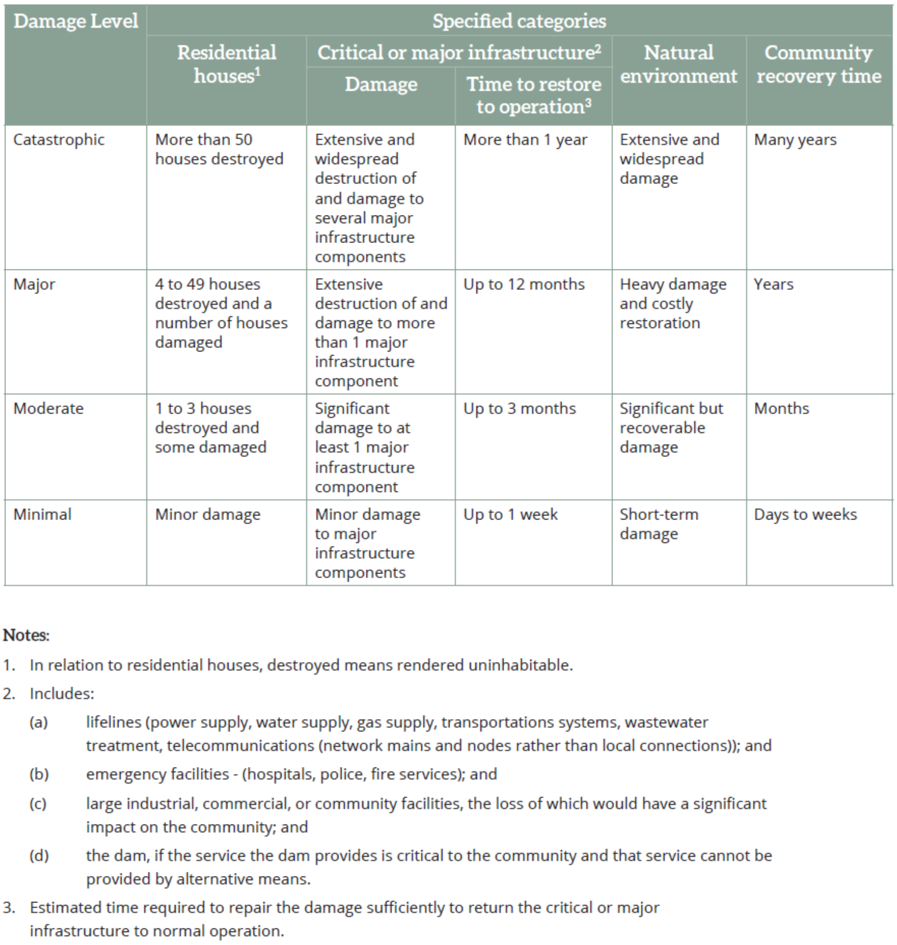
## Trial Pit Log No. B

**Date** Friday, 11 May 2018
**Equipment** 7t Excavator

| Depth | Graphic Log | Description |
|---|---|---|
| **0** | | |
| 0.1 | | dry, brown, silty gravel topsoil with roots (ranging from 0 to 150mm) |
| 0.2 | | |
| 0.3 | | dry, light brown, sandy gravel (ranging from 0 to 300mm) |
| 0.4 | | |
| **0.5** | | |
| 0.6 | | |
| 0.7 | | |
| 0.8 | | |
| 0.9 | | |
| **1** | | |
| 1.1 | | |
| 1.2 | | |
| 1.3 | | |
| 1.4 | | |
| **1.5** | | |
| 1.6 | | |
| 1.7 | | |
| 1.8 | | |
| 1.9 | | |
| **2** | | |
| 2.1 | | |
| 2.2 | | |
| 2.3 | | |
| 2.4 | | |
| **2.5** | | |
| 2.6 | | |
| 2.7 | | |
| 2.8 | | |
| 2.9 | | |
| **3** | | dry, grey-brown, sandy gravel (ranging from 0 to 300mm) |
| 3.1 | | |
| 3.2 | | |
| 3.3 | | |
| 3.4 | Bottom of Pit | |
| **3.5** | | |
| 3.6 | | |
| 3.7 | | |
| 3.8 | | |
| 3.9 | | |
| **4** | | |
| 4.1 | | |
| 4.2 | | |
| 4.3 | | |
| 4.4 | | |
| **4.5** | | |
| 4.6 | | |
| 4.7 | | |
| 4.8 | | |
| 4.9 | | |
| **5** | | |

# Appendix C – New Zealand Society on Large Dam (NZSOLD) (2015) Dam Safety Guidelines - Tables 2.2 and 3.1

*Table 2.2: Determination of Assessed Damage Level*

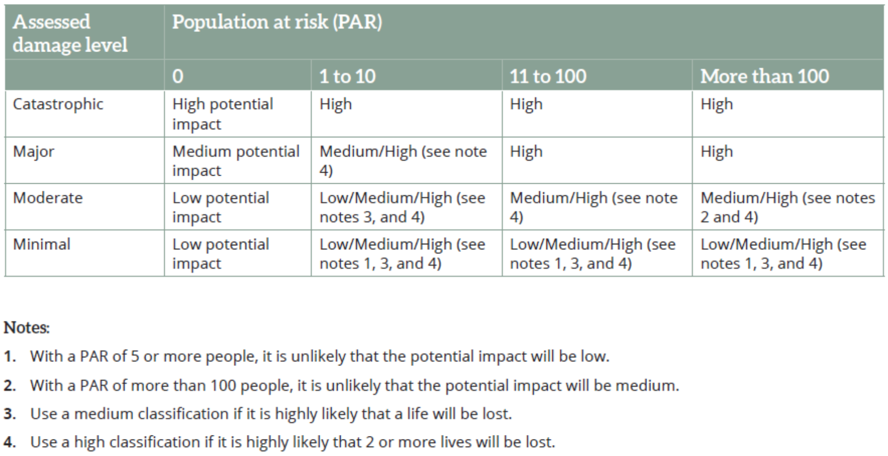


| Damage Level | Specified categories | | | | |
|---|---|---|---|---|---|
| | Residential houses[1] | Critical or major infrastructure[2] | | Natural environment | Community recovery time |
| | | Damage | Time to restore to operation[3] | | |
| Catastrophic | More than 50 houses destroyed | Extensive and widespread destruction of and damage to several major infrastructure components | More than 1 year | Extensive and widespread damage | Many years |
| Major | 4 to 49 houses destroyed and a number of houses damaged | Extensive destruction of and damage to more than 1 major infrastructure component | Up to 12 months | Heavy damage and costly restoration | Years |
| Moderate | 1 to 3 houses destroyed and some damaged | Significant damage to at least 1 major infrastructure component | Up to 3 months | Significant but recoverable damage | Months |
| Minimal | Minor damage | Minor damage to major infrastructure components | Up to 1 week | Short-term damage | Days to weeks |

**Notes:**

1. In relation to residential houses, destroyed means rendered uninhabitable.
2. Includes:
   (a) lifelines (power supply, water supply, gas supply, transportations systems, wastewater treatment, telecommunications (network mains and nodes rather than local connections)); and
   (b) emergency facilities - (hospitals, police, fire services); and
   (c) large industrial, commercial, or community facilities, the loss of which would have a significant impact on the community; and
   (d) the dam, if the service the dam provides is critical to the community and that service cannot be provided by alternative means.
3. Estimated time required to repair the damage sufficiently to return the critical or major infrastructure to normal operation.

*Table 3.1: Determination of Potential Impact Classification (PIC)*



| Assessed damage level | Population at risk (PAR) | | | |
|---|---|---|---|---|
| | 0 | 1 to 10 | 11 to 100 | More than 100 |
| Catastrophic | High potential impact | High | High | High |
| Major | Medium potential impact | Medium/High (see note 4) | High | High |
| Moderate | Low potential impact | Low/Medium/High (see notes 3, and 4) | Medium/High (see note 4) | Medium/High (see notes 2 and 4) |
| Minimal | Low potential impact | Low/Medium/High (see notes 1, 3, and 4) | Low/Medium/High (see notes 1, 3, and 4) | Low/Medium/High (see notes 1, 3, and 4) |

**Notes:**

1. With a PAR of 5 or more people, it is unlikely that the potential impact will be low.
2. With a PAR of more than 100 people, it is unlikely that the potential impact will be medium.
3. Use a medium classification if it is highly likely that a life will be lost.
4. Use a high classification if it is highly likely that 2 or more lives will be lost.